Damage Analysis of Xiaoyudong Bridge Affected by Wenchuan Earthquake, China

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1. INTRODUCTION

The Wenchuan earthquake occurred in Sichuan Province, China, at 2:28 p.m. (Beijing time) on May 12th, 2008. Magnitude was 8.0 by CEA and 7.9 by USGS. Authors conducted a field-damage survey of Xiaoyudong Bridge on September 27th, 2009, (as shown in **Photo.1** and **Fig.1**) which crossed Baishui River in Xiaoyudong Town on Peng-Bai Road. This bridge is a 189m long, 13.6m wide, 4 spans, rigid-frame arch bridge that was built in 1998 [1]).Rigid-frame arch bridge is a composite structural type of arch bridge and inclined rigid-frame. This type of bridge is a higher-order hyperstatic structure with horizontal thrust and has been abundantly built in China since 1980s. According to a statistical investigation, the accumulative total spans of this light type bridge are more than 15 thousands kilometers [2]). However, the research for the behaviors of rigid-frame arch bridges under natural disasters, for instance earthquake, is still of great insufficiency.

The detailed result of investigation for Xiaoyudong Bridge is presented in the following chapters. As well, pushover analysis was performed to determine the possible mechanism and evaluate the behaviors of this type of bridge.

Photo.1 Overall Photograph of Xiaoyudong Bridge

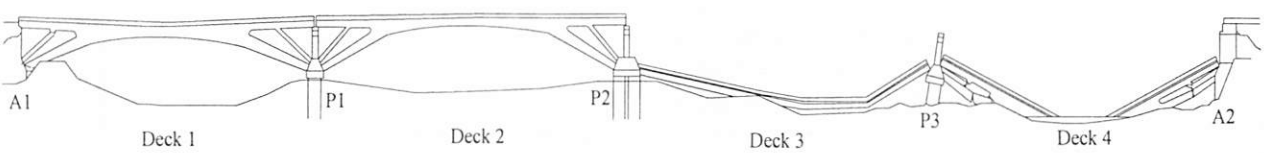


Fig.1 Xiaoyudong Bridge after the Earthquake (according to the field survey)

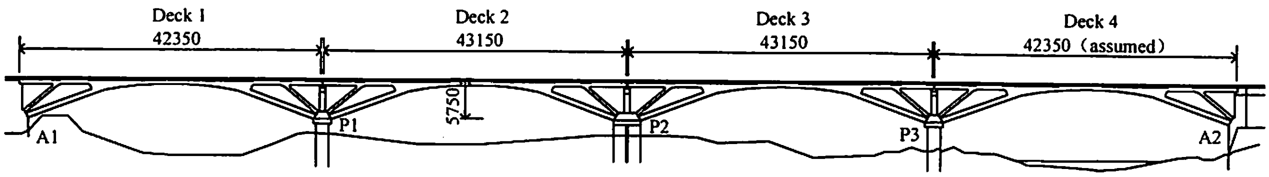


Fig.2 Assumed Length of Span before the Earthquake Based on the Survey (unit: mm)

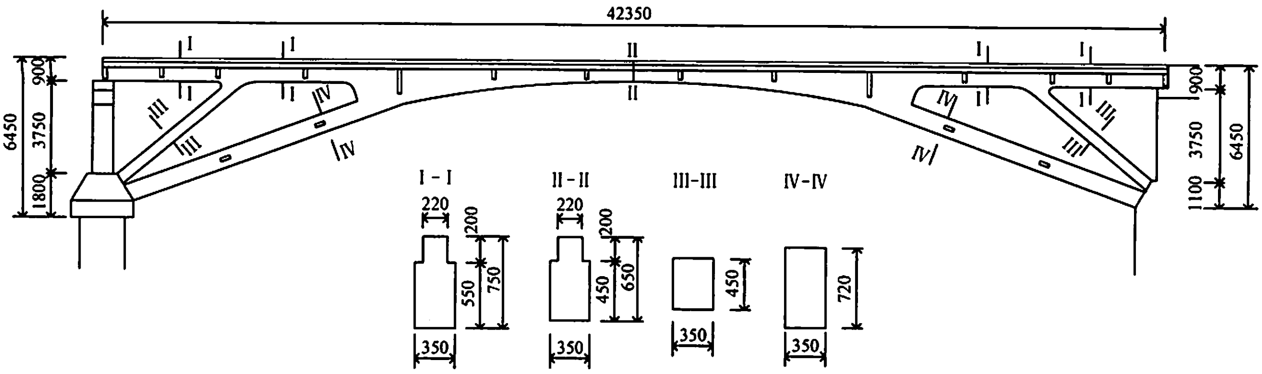
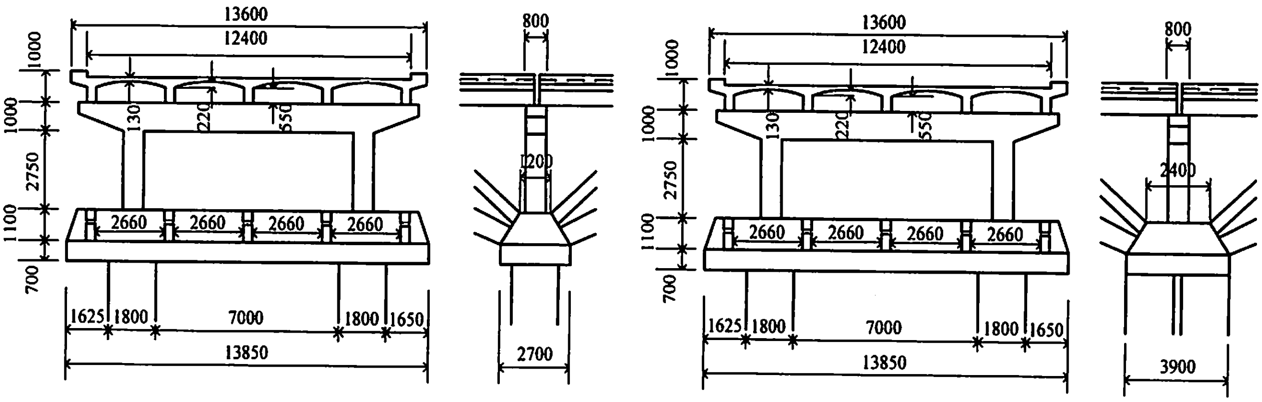


Fig.3 Assumed Dimensions of 4th Span Based on the Survey (unit: mm)



(a) Pier 1 and Pier 3

(b) Pier 2

Fig.4 Assumed Dimensions of Piers Based on the Survey (unit: mm)

2. RESULT OF FIELD SURVEY

(1) Bridge Structure

Due to the lack of design drawings and the other necessary information of Xiaoyudong Bridge, the detailed dimensions have been assumed Based on the results of field survey. Here, as shown in **Fig.2,** number the abutments, piers and decks from the left bank. The Deck 1 has a length of about 42.35m, while the Deck 2 and Deck 3 have the same length of 43.15m. By noticing the Deck 2 and Deck 3 have a same length, we assumed the Deck 4 has a same span length with Deck 1 of 42.35m. The detailed dimensions of 4th span are illustrated in **Fig.3** for instance, and the piers are shown in **Fig.4** for each. We can see that, there are two piles under Pier 1 or Pier 3, and four piles under Pier 2, which causes that the capacity of Pier 2 is significantly greater than that of Pier 1 and Pier 3. Besides, because whether there are piles under the abutments or not is still not able to know, here set there is no pile under abutments for the analysis.

For Xiaoyudong Bridge, a pier consists of a reinforced concrete moment resisting frame with two columns and a cap on which two decks were simply supported. 10 inclined legs and 10 arch legs of two adjacent decks were connected to a pile cap supported by two reinforced concrete piles. The arch leg and the inclined leg have about 21° and 40° slopes for each. The arch frame is formed by one arch leg from left pile cap and the corresponding one from the right cap. This arch frame together with two inclined legs, and the chords at the end of deck, composes one single rigid-frame arch. One span consists of five rigid-frame arches connected by several crossing beams, micro-bending slabs, extending slabs. Pile caps and the piers connect every two spans to form the entire bridge, refer to **Fig.2.**

(2) Observed Damage

According to the field survey, shown in **Photo.1** and

Fig.1, Deck 1 moved about 75cm downwards at mid-span while Deck 2 moved about 10cm upwards in the middle and almost fell down from Pier 1. Besides, Pier 3 tilted extensively toward A2, and Deck 3 and Deck 4 collapsed entirely. Furthermore, shear failure happened to the arch legs and inclined legs at A1, and the deck collided into the A1 approximately 90cm. As well, a permanent displacement about 20cm in the backsoil side due to the collision of Deck 4 and A2 was found for the pavement. Thus, extensive shear cracks were developed in the side walls resulting from this. In addition, a great amount of visible cracks were observed at the joints between legs and pile caps, and between legs and girders.

Based on the survey by total station, the settlements of piers and abutments have been determined as illustrated in **Fig.5**. Compared with the left abutment A1, three piers and A2 moves upwards about 838mm, 818mm, 748mm and 547mm respectively, which means that all the three piers have noticeable upwards vertical displacements.

The Pier 3, as shown in Photo.2, titled averagely 7.5° more or less towards A2 (about 8.08° at the upstream side and 6.85° at the downstream side of the bridge). In order to judge the changes of span lengths, backwards rotation has been applied to Pier 3 by using the average angle, as illustrated in **Fig.5**.

After the backwards rotation of Pier 3, the length of every span from Deck 1 to Deck 4 becomes 41.203m, 42.440m, 42.298m and 41.208m respectively, as shown in **Fig.5**. According to the field survey, because the space between decks is relatively very small (not greater than 5cm), here set the length of each support equals 400mm, 1/2 of the width of the beam upon the pier. Thus, add by two lengths of support, the length of each deck becomes 42.003m, 43.240m, 43.098m and 42.008m respectively. Comparison of these lengths by total station with the lengths by measuring tape mentioned in 2.1 has been conducted to judge the horizontal displacements of piers and abutments. Here, the length of span by total station stands for the available value of the deck length between two supports after the earthquake, while the span by measuring tape refers to the actual length of decks before the earthquake. As a consequence, it is obvious to us that Deck 2 and Deck 3 has an approximate displacement of 9cm (the difference between the span length by measuring tape of 43.15m and that by total station of 43.24m) and 5cm for each (about 0.16% and 0.12% changes), which is relatively small and thus can be ignored. On the other hand, Deck 1 and Deck4 have similar assignable displacement of 34cm (about 0.81% change), which suggests that earthquake enormously influence these two spans.

(3) Assumption of Reinforcements and Properties of Materials

To determine the M-Φ relationships of cross sections and process the pushover analysis, it is of necessity to get the detailed condition of reinforcement and the properties of materials. Although the dimensions have been surveyed and assumed, the reinforcement condition is still not known due to the lack of design drawings and other necessary information. Thanks to that there is a large amount of this type of bridge in China, the design drawings and construction instructions of another rigid-frame arch bridge which has almost same characteristics, as span length, rise, width-girder ratio and the design seismic fortification, with Xiaoyudong Bridge has been found on the internet. That bridge named Jinzhai No.6 Bridge is located at Meishan Town, Jinzhai City, Jiansu Province, Chin. Furthermore, it is

Photo.2 Tilt of Pier 3 (from upstream)

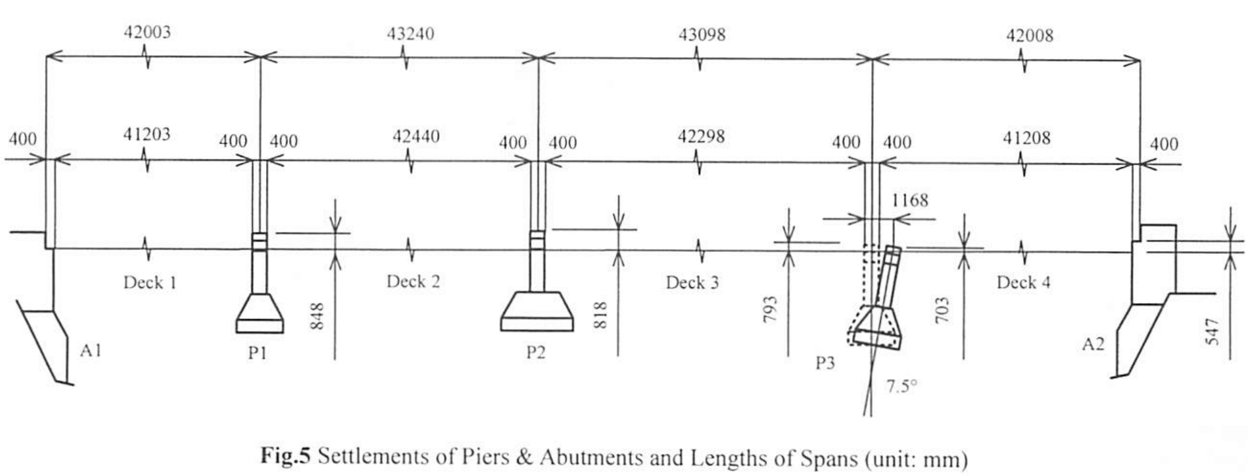


Fig.5 Settlements of Piers & Abutments and Lengths of Spans (unit: mm)

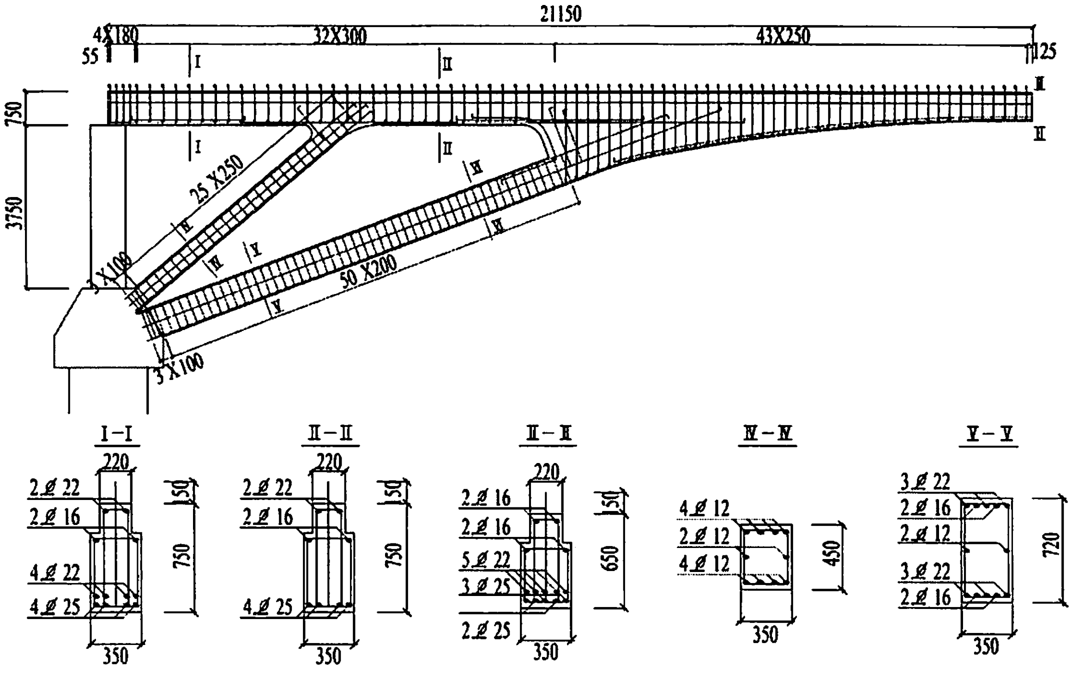


Fig.6 Reinforcements of Half Span (unit: mm)

found that most of the dimensions of this bridge are same with Xiaoyudong Bridge according to the field survey. Therefore, the author assumed the reinforcements of Xiaoyudong Bridge based on that, and modified it according to the results of field survey. As following, the condition of half-span reinforcements is shown in **Fig.6**, and the overall reinforcement ratios for some important cross sections are shown in **Table.1**. It became obvious to us that the inclined has the lowest reinforcement ratio of about 0.72%, while the mid-span has the highest one of about 2.56%. The reinforcement ratios of the other cross sections are all between 1.0% and 2.0%.

For the materials of construction, set the main rebars as HRB335, the stirrups as HPB235, and the concrete as C30. Their properties are shown in the **Table.2**. (based on *the Code for Design of Concrete Structure, GB50010- 2002*) [3].

Table.1 Reinforcement Ratios of Important Cross Section

	I	II	III	IV	V
Reinforcement Ratio	1.96%	1.32%	2.56%	0.72%	1.53%

Table.2 Properties of Material

	Type	Design Strength (N/mm^2)		Modulus of Elasticity E_S (N/mm^2)
		Tensile	Compressive	
Rebar	HPB235	210	210	210000
	HRB335	300	300	200000
Concrete	C30	1.43	14.3	30000

(4) Possible Mechanism of Failure

Based on the field damage survey, the author considers the possible mechanism of failure as illustrated in **Fig.7** for the 1st and 2nd spans, and in **Fig.8** for 3rd and 4th spans. They will be explained for each two spans as following.

At first, when the earthquake occurred, movement of Deck 1 and Deck 2 in the longitudinal direction due to the huge force of earthquake caused the collisions of A1 and Deck1. The surface fault might cause A1 moved towards the decks, which also contributed to the collision and the consequent shortening of Span 1. Thus, extensive shear cracks were developed in the side wall at left abutment A1. After the cracks occurred to the arch legs and inclined legs, and the shear failure happened to the legs at A1, Deck 1 dropt about 75cm while the Deck 2 raised about 10cm.

On the other hand, Deck 3 and Deck 4 moved

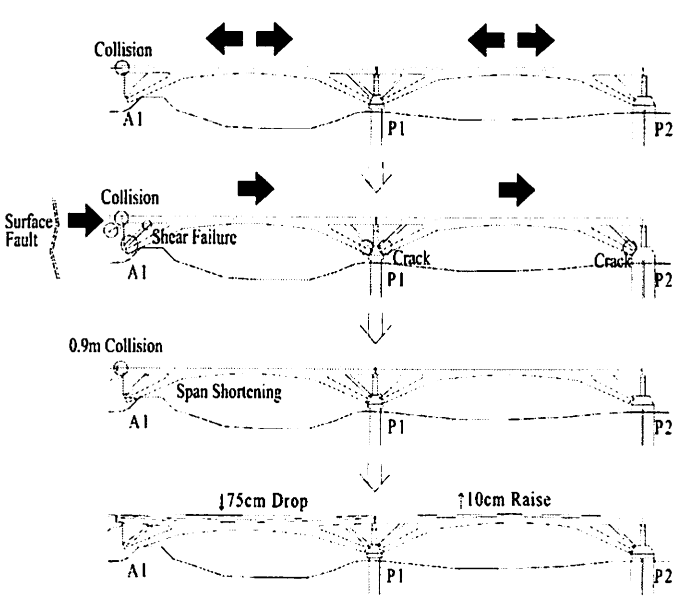


Fig.7 Possible Mechanism of Failure of Span 1&2

longitudinal as well due to the earthquake effect. This led to the collision of Deck 4 and A2, and the movement of A2 towards decks. Consequently, at right abutment A2, huge shear crack about 0.2m permanent displacement in the backsoil occurred. Then, Deck4 dislodged from A2 support due to excessive response placement by

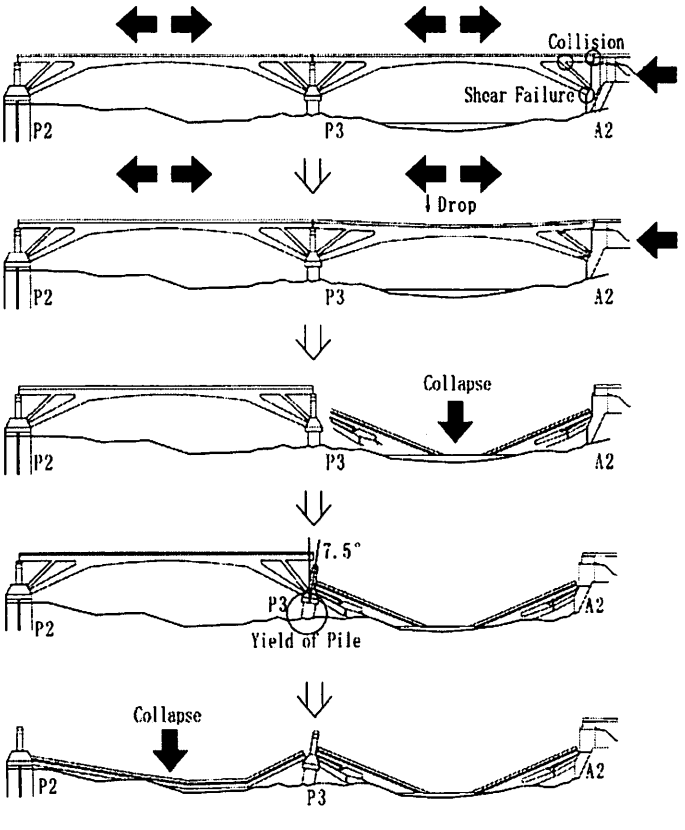


Fig.8 Possible Mechanism of Failure of Span 3&4

Table.3 Rigidities of Spring

Spring	Stiffness (kN/m)
Horizontal Base Spring	1.0×10^8
Rotational Base Spring	9.0×10^8
Shear Resisting Spring	1.0×10^2
Support Spring	1.0×10^8

earthquake force in the opposite direction. Insufficient seat length of about 0.3m at A2 is another remarkable reason of the drop of Deck4. Once Deck 4 collapsed, the equilibrium of lateral forces between Deck 3 and Deck 4 was lost. Therefore, this enormous lateral force of Deck 3 made the piles under Pier 3 yield and pushed Pier 3 tilted extensively toward A2. The Deck 3 itself fell down as a result of the tilt of Pier 3. In turn, the tilt of Pier 3 resulted in Deck 3 being dislodged from its reliance on Pier 2 and Pier 3. As a consequence, there should have been a lost of balance at Pier 2. However, probably thank to relatively more piles and lager cross section of pile cap, the Pier 2 had got a much better resistance of tilt. Thus there were only some small cracks on the piles of Pier 2, while Deck 2 did not fall down due to the lost of balance.

3. PUSHOVER ANALYSIS

(1) Analysis Model

The model has been made for the 2nd Span considering no serious supports movement happened here. As shown in **Fig.9**, on the right angle direction of the axis of the bridge, considering the five arch frames which have been arranged together to form one span of Xiaoyudong Bridge, here select one single arch frame, included the micro-bending slab, to establish the model. For resisting the positive moment, use the section that included girder and slab, while use only the girder for the negative moment. Because there are only two columns for every pier, the properties of the column have been multiplied by 2/5 to fit the single frame. Besides, for the beam at the top of the pier and the footing, here directly use 1/5 value of the characteristics. Due to the insufficiency of the piles' information, as illustrated in **Table.3**, a horizontal spring and a rotational spring with high rigidities have been set at the bottom of each footing, ignoring the vertical displacement. On the other hand, for the springs between the girder and pier, one shear resisting spring which is assumed to be comparatively weak, and one vertical spring which is only be able to support the compression are in use.

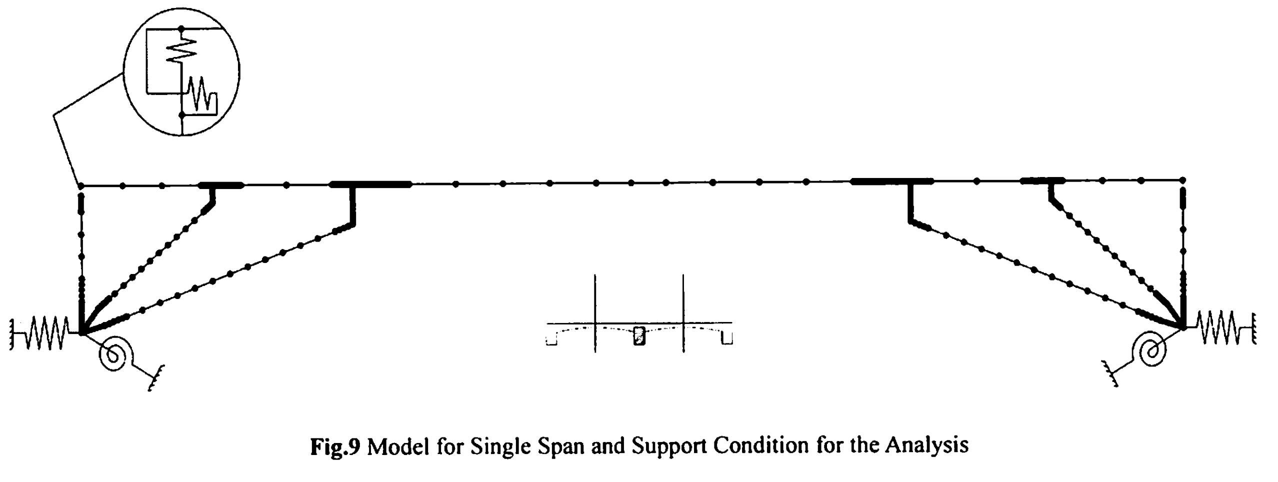


Fig.9 Model for Single Span and Support Condition for the Analysis

Furthermore, at the positions of footing, the beam on the top of the piers and the joints between legs and girder, noticing the relatively greater cross section area and greater amount of reinforcement, rigid bodies have been modeled.

(2) Process of Pushover Analysis

As a rigid-frame arch bridge, a special type of arch bridge, the axial force is of significant importance for the workability of the entire bridge. Therefore, as shown in **Table.4** after the calculation of M-Φ relationship of the members, analysis has been conducted to determine the axial force of each cross section under dead load. Besides, pushover analysis has been done by using the M-Φ relationships under 0 axial force (Case 1). Then, M-Φ relationships were recalculated based on the axial force condition under dead load, and used in the Case 2 of pushover analysis.

According to the result of Case 1 and Case 2, it becomes obvious to us that the axial force acts an extraordinarily important part to the workability of the bridge. If under only dead load, the axial force can reaches at 1061kN in the arch leg and 361kN in the inclined leg, and the compressive stress will consequently reach at about 4.4 N/mm^2 and 2.3 N/mm^2 for them. And the the axial forces change significantly due to the variety of horizontal load, as illustrated in **Fig.10** (here set the horizontal load acts from the left pier to the right one). For instance, until 0.6g, before when the yield is about to occur to the middle span in Case 2, the axial force drops 34% more or less from 361kN to 237kN in the right inclined leg (Point A), while becomes greater by a same value to 485kN in the left inclined leg (Point B). On the contrary, it increases about 48% from 1061kN to 1574kN in the right arch leg (Point D), and decreases 48% to 549kN in the left one (Point C). On the other hand, the axial forces in the section of girder and piers only have ignorable varieties as the horizontal load changes from 0 to 1.0g. Considering that the main failure, yield of middle span might occur at 0.69g horizontal load, the properties of members just before the failure are of extreme importance. Thus, axial force condition under dead load together with 0.6g horizontal load are used in Case 3 to do the pushover analysis. Although the girder and piers have the almost same resisting moment with Case 2, there are obvious increases of resisting moment in right arch leg and left inclined leg, while decreases in left arch leg and right inclined leg. From the M-N relationship curves in **Fig.11** and **Fig.12**, we can see that some of the member can behave at their strengths near the peak value. As well, there are noticeable growths of the strength for the other members.

(3) Result of Pushover Analysis

According to the results of these three cases, we found the deformation shapes are similar to each other. Due to the horizontal load, the piers rotate clockwise slightly, which causes the downward movement of left half span and the raise of the right. As a result, the maximum of positive moment occurs to a little left of the

Table.4 Three Cases of Pushover Analysis

No.	Axial Force Condition for M-Φ Relationship
Case 1	0 Axial Force
Case 2	Axial Force Under Dead Load
Case 3	Modified Axial Force at 0.6g Horizontal Load Shown in Fig. 12

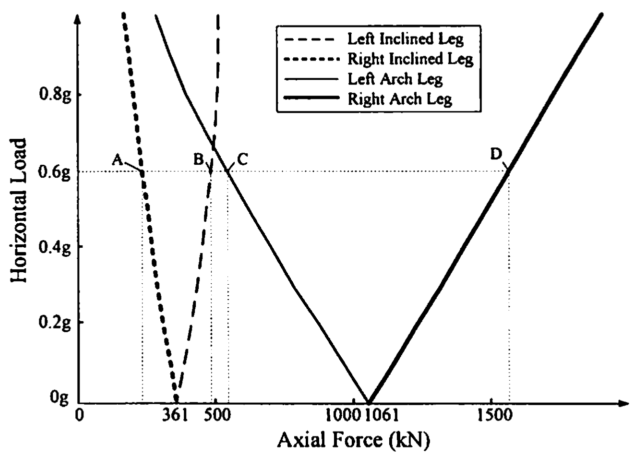


Fig.10 Changes of Axial Forces in the Legs under Dead Load and Horizontal Force

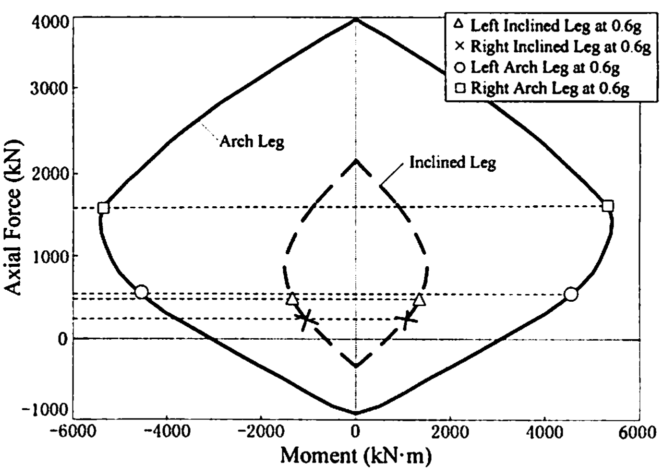


Fig.11 M-N Relationship of Legs

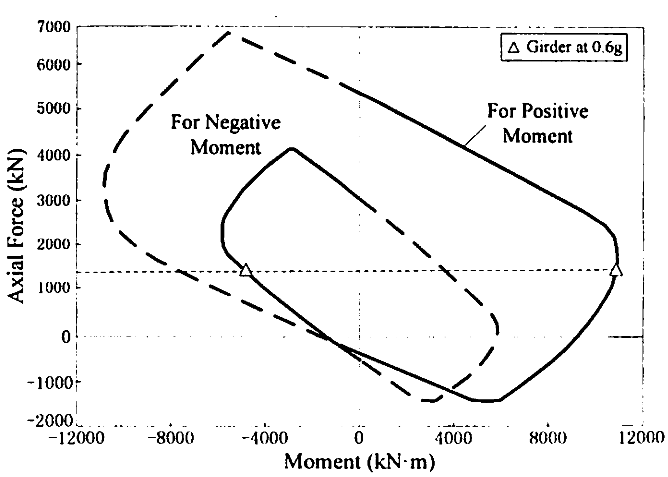
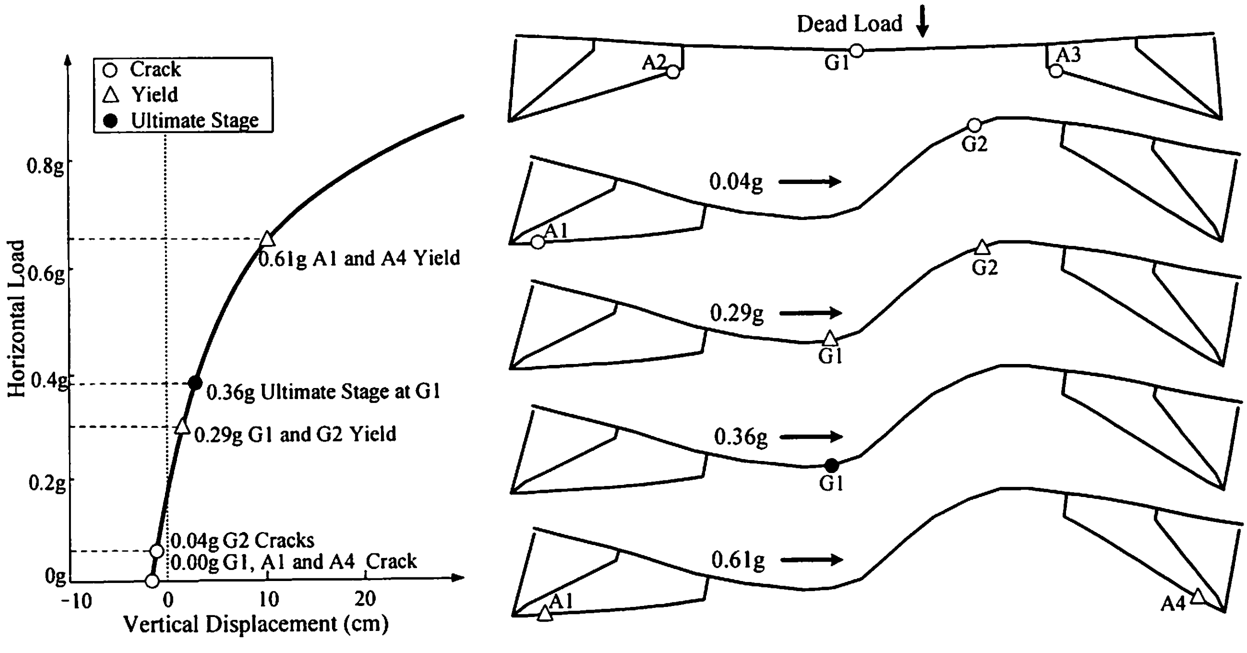
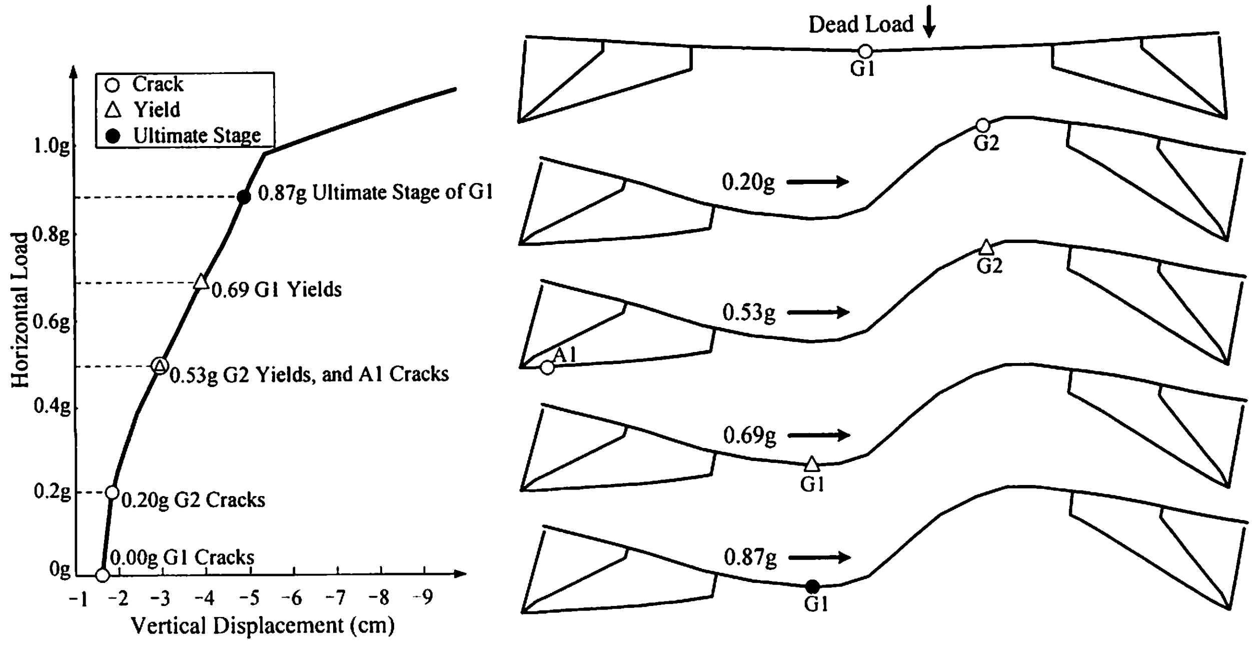


Fig.12 M-N Relationship of Middle Span



(a) P-δ Curve (b) Sequence of Failures

Fig.13 Analysis Result of Case 1

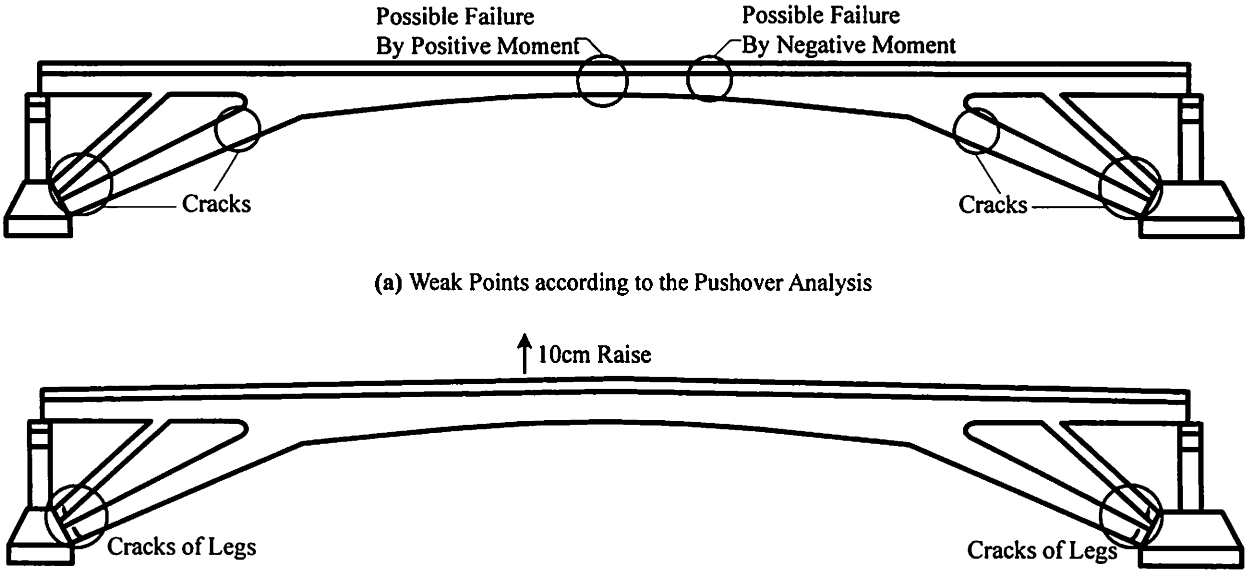


(a) P-δ Curve (b) Sequence of Failures

Fig.14 Analysis Result of Case 3

exact middle point, and the negative moment happens to the right. However, the position of contraflexure point differs from case to case.

For the Case 1, illustrated in **Fig.13** on the next page, which is using the M-Φ relationships under no axial force, cracks will happen at the middle span and the top points of both arch legs only under dead load. Crack may occur due to the negative moment to the right of middle span at 0.04g horizontal load, as well as the bottom of left arch leg. Then as the horizontal load growing, the middle span will yield due to positive moment at 0.29g, while the girder on the right will yield because of negative moment at almost the same time. After that, the middle span will arrive at ultimate stage at 0.36g. Furthermore, the first yield of arch leg will happen at 0.61g to the bottom of both arch legs, and the first ultimate stage of theirs will be reached at 0.89g to the bottom of left arch leg. Besides, because the point of contraflexure is at the left of the point of middle span, the point under observed in Case 1 moves upwards.



(a) Weak Points according to the Pushover Analysis

(b) Actual Failures of 2nd Span

Fig.15 Comparison between the Actual Failures and Results of Analysis

Together with the relatively small capacity to the horizontal load, are two notable differences between Case 1 and the others.

Case 2 uses the axial force condition under dead load for the M-Φ relationships. Although the middle span will still crack only under dead load, the other failures will happen very late. The middle span yields at 0.69g, and reaches the ultimate stage at 0.87g. The first crack of arch leg is found at about 0.69g at the top of right arch leg. Even until 1.2g, when the calculation ended, there were still two top points of the inclined leg had not cracked.

Compared with Case 2, the failures of middle span and the corresponding horizontal load of Case 3 are almost the same, (as illustrated in **Fig.14**) that it cracks under only dead load, yields at about 0.69g and reaches the ultimate stage at 0.87g. Because the M-Φ relationships have been modified based on latest axial forces condition in the legs, the sequence of failures for the legs is distinguished from the other cases. The bottom of left arch leg cracks at first among all the arch legs and no leg will yield till the end of calculation at 1.2g.

(4) Comparison between the Actual Failures and the Results of Analysis

Compared the results from pushover analysis with the actual failure condition of the 2nd span from field survey, we can see that the failures occurred to most of the weak points base on the analysis. According to the analysis, the middle might move upwards by about 40cm at most until 1.0g horizontal load, which can explain the 10cm raise in the middle span. Besides, cracks have been found at the joints of legs and pile caps as the result from the analysis.

4. CONCLUSIONS

From the field survey and the pushover analysis, the following conclusions have been drawn:

(1) According to the field survey, both of the abutments have a displacement about 34cm towards the middle. However, there is no obvious movement for the other structures. The tilt of Pier 3, as a chain failure of the fall of Deck 4, caused the collapse of Deck 3. Consequently, although the surface fault has a notable effect on the bridge, the seismic force is the critical reason for the serious failure.

(2) The pushover analysis shows that the middle span might fail due to positive moment or negative moment at over 0.53g horizontal load. The resisting moments of arch legs and inclined legs have a noticeable enhancement due to the redistribution of axial force under horizontal load. Therefore, the middle span is the most critical point for the whole bridge.

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