Structural Inspection and Assessment of Yutengping Broken Bridge —— A study of masonry historic architecture in Taiwan

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Abstract

In this study, the fundamental data for assessment are built through damage inspection, ambient vibration measurement, brick harmer test and Phenolphthalein PH indicator test. Besides, the compressive and shear strength test for bricks are also conducted. Based on these results, the stresses of each pier are analyzed under the wind and earthquake load, which are required in Taiwan's building code.

The average compressive strength of the tested bricks is about 16.7MPa, which still exceeded the compressive strength of the first grade brick required in CNS. The shear strength of the mortar is 0.94MPa.

The fundamental natural periods of measured piers are about 0.17 to 0.22 sec. in E-W direction, and 0.31 to 0.37 sec. in N-S direction. The E-W stiffness of the piers is greater than that in N-S direction.

In according to the present design code in Taiwan, the analyzed maximum flexural stress for the brick pier is 2.28MPa that also associated with a shear stress 0.15MPa at the same cross-section plane. Comparing to the strength obtained in previous studies, these stresses are greater than that can be resisted by the mortar. This also implies that the broken bridge piers are not safe enough under potential great earthquake excitation.

For improving the structural safety of the left piers of the broken bridge, a steel arch between adjacent piers is suggested. The arch shape also needs to be designed following the original arch form.

Keywords: Yutengping broken bridge, masonry, earthquake, cultural heritage, assessment, inspection

1. Introduction

The Yutengping broken bridge is located between the Taian and Shengsing station of Old Mountain Line of Taiwan Railway. In 1908, the bridge was designed to cross the Yutengping River. The total length of the bridge is about 170 m, which contains three major portions. The two end portions were masonry arcade (Fig. 1), and the middle portion was composed of steel truss and steel beams. In 1935, 27 years after the accomplishment of the bridge construction, the bridge was seriously damaged by a strong earthquake occurred in middle Taiwan area. After the earthquake, the masonry arcade of the two end portions were seriously damaged (Fig. 2), however, the brick piers still exist. In the past seventy years, due to part of the railway in Miaoli area changing line and less maintenance has been put by the railway administration bureau, the damage situation of the bridge gets worse gradually. Thus in the 1999 Chi-Chi earthquake the spandrel of Pier 4 collapsed.[3] After the earthquake damage, the safety of the broken bridge caused attention widely, and the conservation is decided by the local government (Fig. 3).

The Yutengping Broken Bridge, in addition to its special historical background, also presents the technology achievement of bridge construction of Taiwan in early twentieth century. Even now, from the brick bond of the damaged arcade and piers, we still can see the delicate masonry craft of that time. Thus from the point of cultural heritage, the broken bridge, not only an important evidence of Taiwan's railway development, is also valuable in the research of Taiwan's technology history.

For the conservation of this cultural property, the fundamental work is to avoid the left arcade and piers further damaged and deteriorated. Furthermore, properly strengthening for improving the structural safety is also necessary to be considered. For these purposes, in this paper, we carry out a series of basic study related to the existing structure. They are structural inspection, non-destructive test, masonry strength test and ambient vibration measurement. Through these processes, we do the structural analysis and assessment. Based upon these, finally we provide several suggestions for future conservation execution.

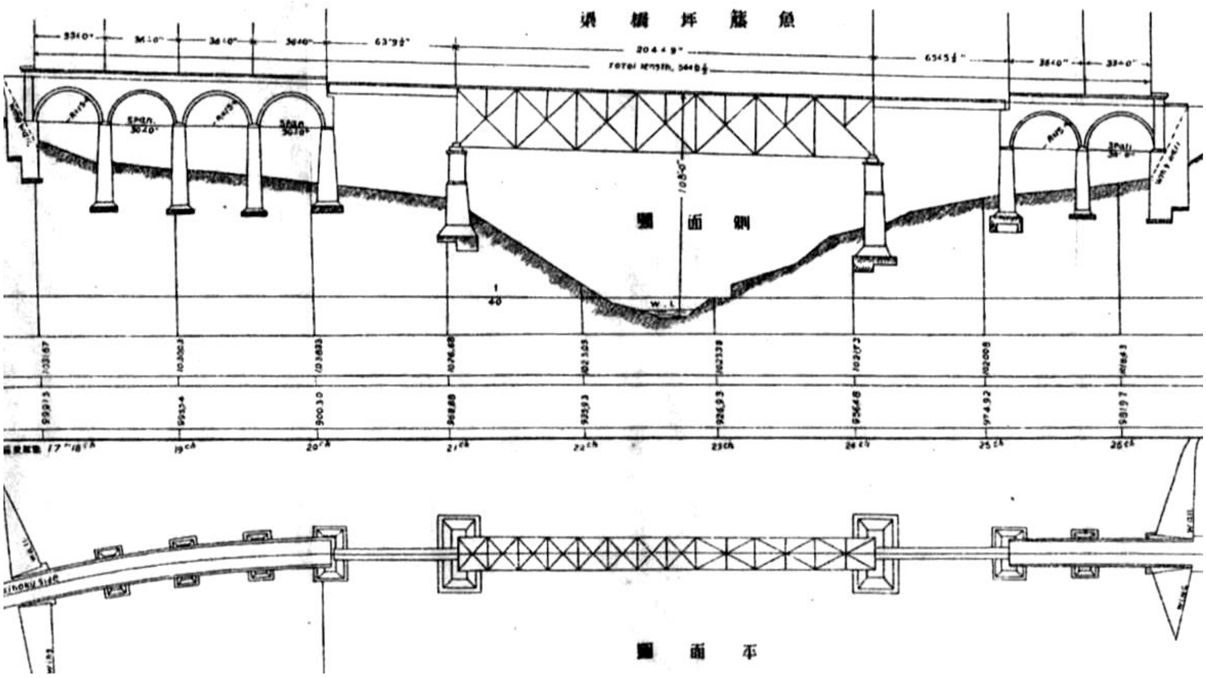

Fig. 1 - The design elevation and plan of Yutengping Bridge [1]

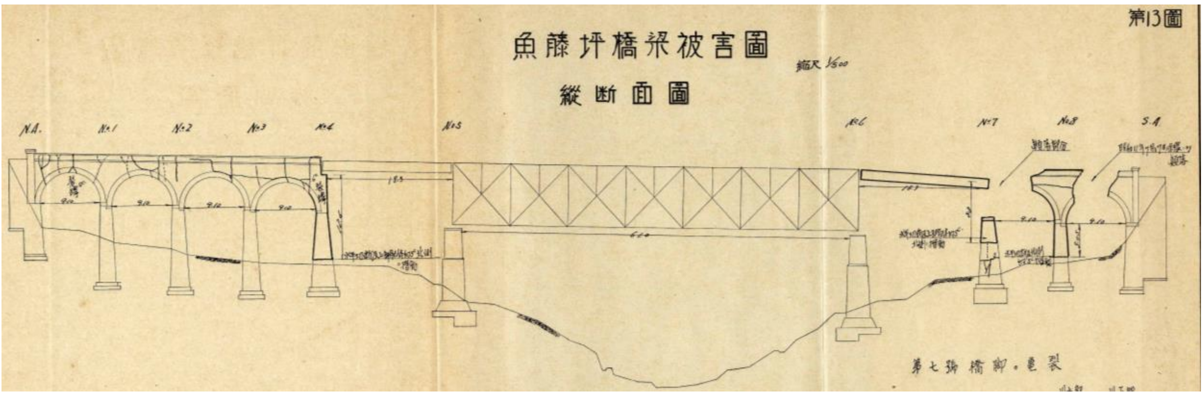

Fig. 2 - The damage of Yutengping Broken Bridge in 1935 earthquake [2]

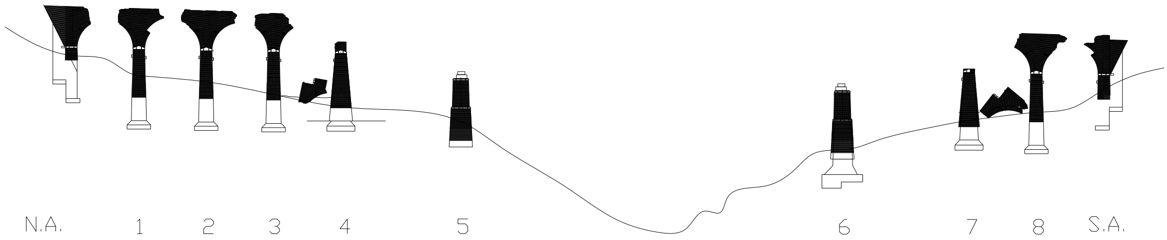

Fig. 3 - Existing Yutengping Broken Bridge [4]

2. Deterioration and damage inspection

According to the site inspection, the damage and deterioration of the Yutengping Broken Bridge are serious. Although the major damages are caused by earthquake initially, the extension of the damage is contributed by man and poor maintenance. In spite of these, the rainwater and the growth of the attached trees and plants also increase the existed damages and accelerate the construction deterioration. In Table 1, the listed damages for each pier are recorded from structural inspection in site.

Presently, the structural system of the existing piers just like several individually cantilever system with a very weighty lumped mass located in top of the pier. This cantilever system will induce a maximum shear and moment at the base of the pier under lateral load action such as wind or seismic load. While the maximum stress may not occur in base also, due to its non-uniform cross section. The maximum stress may occur in the minimum section at middle height position. This may cause horizontal crack or slip near the impost.

The eccentricity of Pier 4 and Pier 7 is quite visible, and will induce an additional eccentricity moment with the negative effect for the system. So the spandrel of pier 7 was collapsed in the 1935 earthquake. In 1999, the spandrel of Pier 4 pier also collapsed due to the Chi-Chi strong earthquake (Fig. 4).

The masonry was constructed in English bond, and the mortar joints are full of mortar with high quality of cement. The archivolt was laid by 6 layers of bricks in header bond. The shape of arch is semicircle with a 9 m diameter. The archivolt was designed to support its upper weight and induce compressive stress, but after the removal of the top of the arch the archivolts lose confined stress, and are easy to drop out (Fig. 5).

On the top of the spandrel there is a concrete slab for rails and two masonry parapets on each side. Due to this change in shape, the parapet is easy to generate a horizontal crack at its bottom area. Because the scuppers are obstructed, some rainwater will stay on top of the arch and cause accumulation of soluble salts seriously (Fig. 6). Mortar joints will be weather out with time, if the drainage did not be treated properly, the deterioration will be worse.

Plants attached also are harmful from the point of structural safety, the root of the plant will penetrate the masonry, and the weights of the plants also induce much great loading (Fig. 7).

Table 1 – Observed pier damage

Damage Situation	Main causes	Pier
Horizontal slip between arch spring and the impost	Horizontal seismic load	Pier 1, Pier 4
Spandrel collapse	Horizontal seismic load	Pier 4, Pier 7
Archivolt drop out	Seismic load and self weight	Pier 1, Pier 2
Horizontal crack at parapet bottom area	Out of plan seismic load	Pier 2, Pier 3, Pier 8
Crack at the surface	Seismic load and decay of material strength	Pier 6, Pier 7
Accumulation of soluble salts and joints weather out	Direct penetration of rainwater	Pier 1, Pier 2, Pier 3
Plants attached	Natural growth	Pier 6, Pier 7, Pier 8, S.A.
Surface broken	Hit and scratched by vehicles	Pier 4

Fig. 4 - The broken brick arcade in north side, the arch spandrel of pier 4 was collapsed by 1999 Chi-Chi earthquake.

Fig. 5 - The archivolt of brick arcade was drop out.

Fig. 6 - Accumulation of soluble salts of Pier 1 and Pier 2

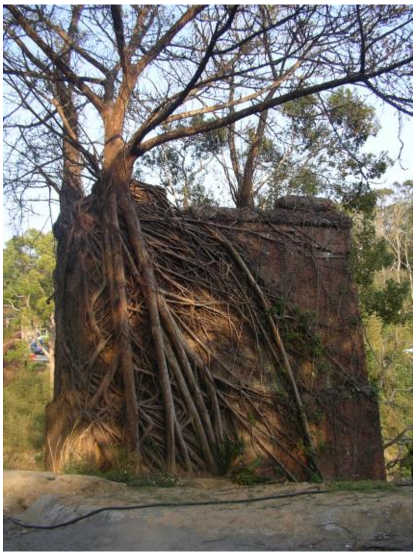

Fig. 7 – Tree attached to Pier 6.

3. Ambient vibration measurement

To understand the dynamic properties of the existing piers, ambient vibration measurement was processed. The piers measured are Pier 1, Pier 2 and Pier 8. After the measurement, the recorded data in time domain were converted to frequency domain by Fast Fourier Transform. Using the obtained frequency spectrum, we can calculate the transfer function between top of the pier and ground for the ambient vibration. The identified fundamental natural frequencies of measured piers are about 4.6Hz to 6.0Hz in E-W direction and 2.8Hz to 3.2Hz in N-S direction. Obviously, the pier stiffness in E-W direction is greater than that in N-S direction. Although these 3 piers were built in the same shape and size, their height is different due to the slope of the ground. The height of Pier 2 is higher than the two others, so its vibration period is longer than that of the other 2 piers. For Pier 8, some large trees attached, and some roots grows in the cracks of the pier. These situations may induce greater wind loading and a longer vibration period for the pier. So although the height of Pier 1 and Pier 8 is close near, Pier 8 got a longer vibration period in N-S dir. During measuring Pier 8 in site, we did experience the wind effect on pier vibration. The recorded ambient vibration time history of Pier 1 and Pier 8 are shown in Fig. 8 and Fig. 9. From Fig 9, we can see the longer period for pier 8, which is influenced by wind on the trees.

In Fig. 10 and Fig. 11 the transfer function of Pier 1 in E-W and N-S directions are also shown. The identified period in E-W direction is shorter than that in N-S direction; this is because the section module corresponding to N-S axis is much greater.

The dominant vibration period will govern the response magnitude under dynamic ground motion, and is very important in seismic assessment. The comparison of the vibration period of these 3 piers and those calculated from code are shown in Table 2. Obviously, for each pier, the code vibration period is longer than that identified from ambient vibration in E-W direction, but shorter in N-S direction. The dominant vibration period will be useful in the reservation design or related study.

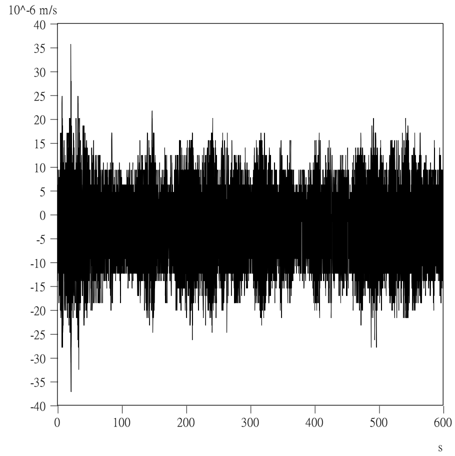

Fig. 8 - The recorded ambient vibration time history of Pier 1 for N-S dir.

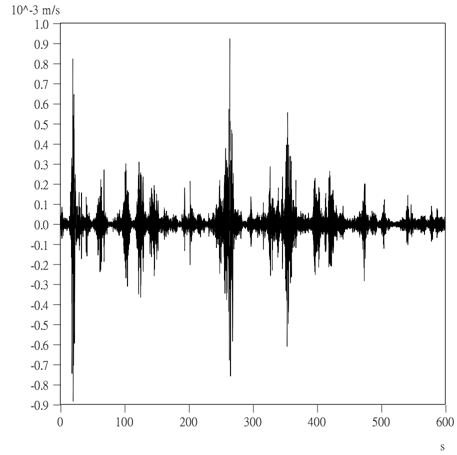

Fig. 9 - The recorded ambient vibration time history of Pier 8 for N-S dir.

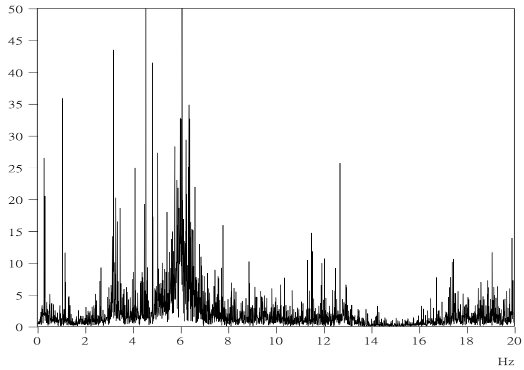

Fig. 10 – Transfer function of Pier 1 pier in E-W direction, $f_{EW} \approx 6.0Hz$

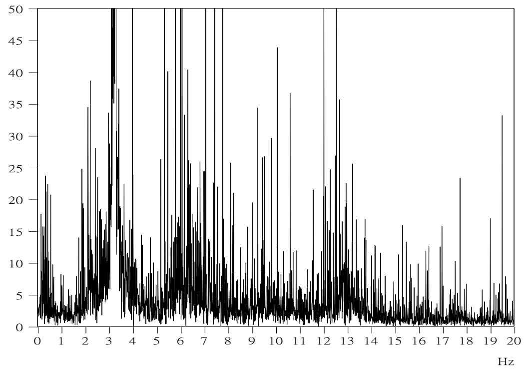

Fig. 11 - Transfer function of Pier 1 pier in N-S direction, $f_{NS} \approx 3.2Hz$

Table 2 – Comparison of identified dominant vibration period and calculated from code

	Height m	Identified dominant natural period, sec		Period calculated from code, sec
		E-W dir.	N-S dir.	
Pier 1	10.75	0.17	0.31	0.297
Pier 2	11.6	0.22	0.37	0.314
Pier 8	10.84	0.18	0.36	0.299

4. Non-destructive tests

There are two non-destructive tests are adopted in this investigation. In order to compare the brick surface hardness with that produced in the same period, in site, we use hammer test to check the hardness of pier brick. The tested values referred to the brick compressive strength are shown in table 3, and the estimated average compressive strength is 33.4MPa. The other non-destructive test is Phenolphthalein PH indicator test, which was used to check the activity of the cement in the mortar joints. In site, the test showed the color was changed to purple very soon, as the Phenolphthalein liquid was dropped on the test mortar. This indicates the cement still keeps its alkalinity.

Table 3 – Estimated brick compressive strength

	Minimum	Maximum	Average
R value	36	50 (58)	45.91
Compressive strength, MPa	15.6	45.4 (76.9)	33.4

5. Compressive and shear test of brick

The compressive and shear test are conducted for the brick masonry collected in site. The obtained ultimate loading of the compressive test is 200.6 KN, and the brick average area is

$$\frac{10.8\times10.8+10.5\times12.8}{2}=124\ cm^2$$

So, the brick compressive strength is

$$\frac{200.6KN}{124cm^2}=16.2\text{MPa} > 14.8\text{MPa (CNS 382 first class)}$$

Although this value is greater than the minimum strength of CNS 382 first class brick, it is still lower than the compressive strength obtained from the brick produced at the same period. According to a test report in reference 5, the average compressive strength of brick produced in Taiwan at that time is 59.3 MPa for first class brick, and is 34.1 MPa for second Class brick. The reason may due to these specimens were fallen to the ground for many years, the damp and the soil could decay the micro structure of the brick. However, the strength of the brick still in pier should be much better.

The test results of mortar shear strength are shown in Table 4. In these two specimens, the shear strength is still quite well, it means the bridge was constructed by high quality mortar.

Table 4 – Shear strength of the masonry mortar

Specimen Pier	Area dimension of the specimens	Ultimate load KN	Shear strength MPa
1	$11.0\times11.0=121\,cm^2$	15.5	1.28
2	$11.4\times11.4=129.96\,cm^2$	12.2	0.94

6. Structural assessment

For the structural system of the piers discussed in Sec. 2, the most important point of structural assessment is to analyze and check the safety of the piers under lateral loadings such as wind and seismic loading. As discussed previously, under present situation, the pier may be treated as a cantilever system. A cantilever system is statically determined. Thus if the loading is defined clearly, we can calculate the corresponding internal forces and stresses simply. In this study, we apply the present building design code for wind and seismic loading in an equivalent static analysis.

The wind pressure and seismic base shear are calculated to assess the safety of the existing piers. The calculation follows the present building code used in Taiwan, The maximum wind base shear of Pier 3 is 82.7KN with 570.6KN-m moment in N-S dir. The internal force of Pier 3 in different height is shown in Fig. 12.

To determine the design seismic load required by present design code, the dominant vibration period identified in ambient vibration measurement is applied. Because the site of the bridge is near two faults, the near fault effect also has to be involved. The maximum seismic base shear of Pier 3 is 1348.9KN with a 12665KN-m moment as shown in Fig. 13.

Comparing these 2 loadings, obviously, the seismic load governs the safety of the structure, so only the seismic stress is calculated and discussed. The seismic stress of Pier 3 is shown in Table 5. Although maximum shear force and moment will be at the base of a cantilever system, the maximum shear stress is found near the impost not at the base part. This is because the cross-sectional area near the impost is the smallest in the pier.

It is necessary to be mentioned that the calculation described above is based on the assumption that the existing cracks or construction deterioration of the piers have been improved. Thus the damaged conditions existed in the bridge must be retrofitted first.

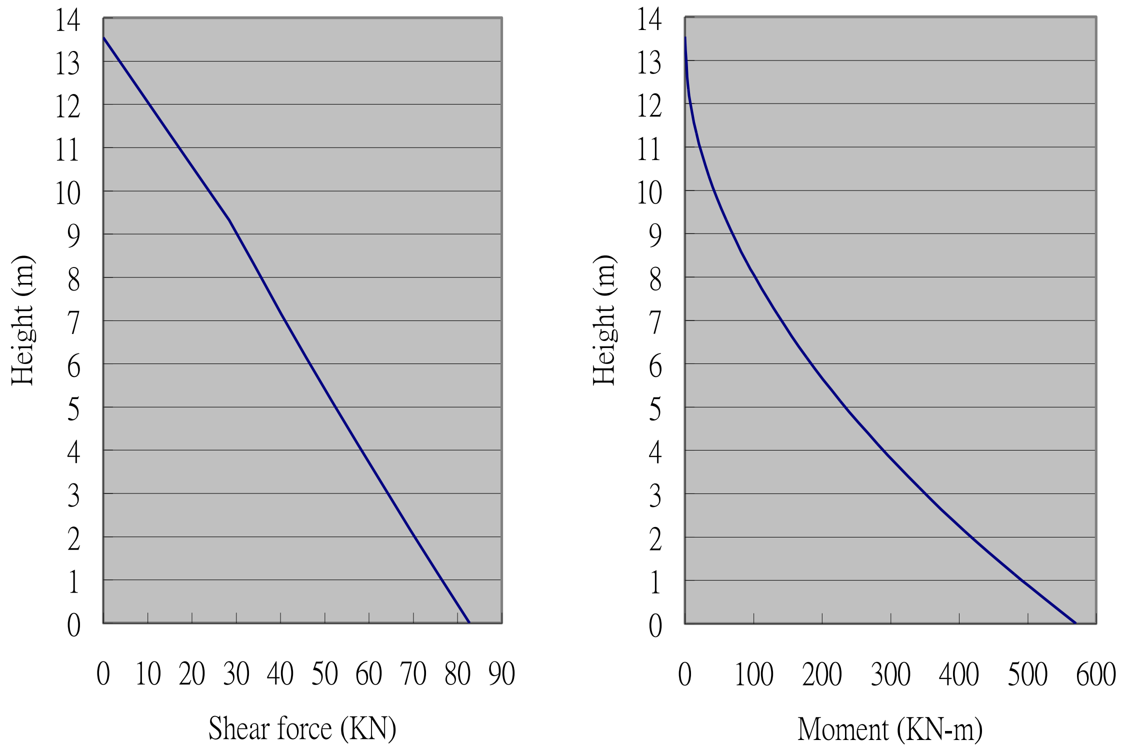

Fig. 12 – Shear force and moment of Pier 3 pier due to design wind pressure

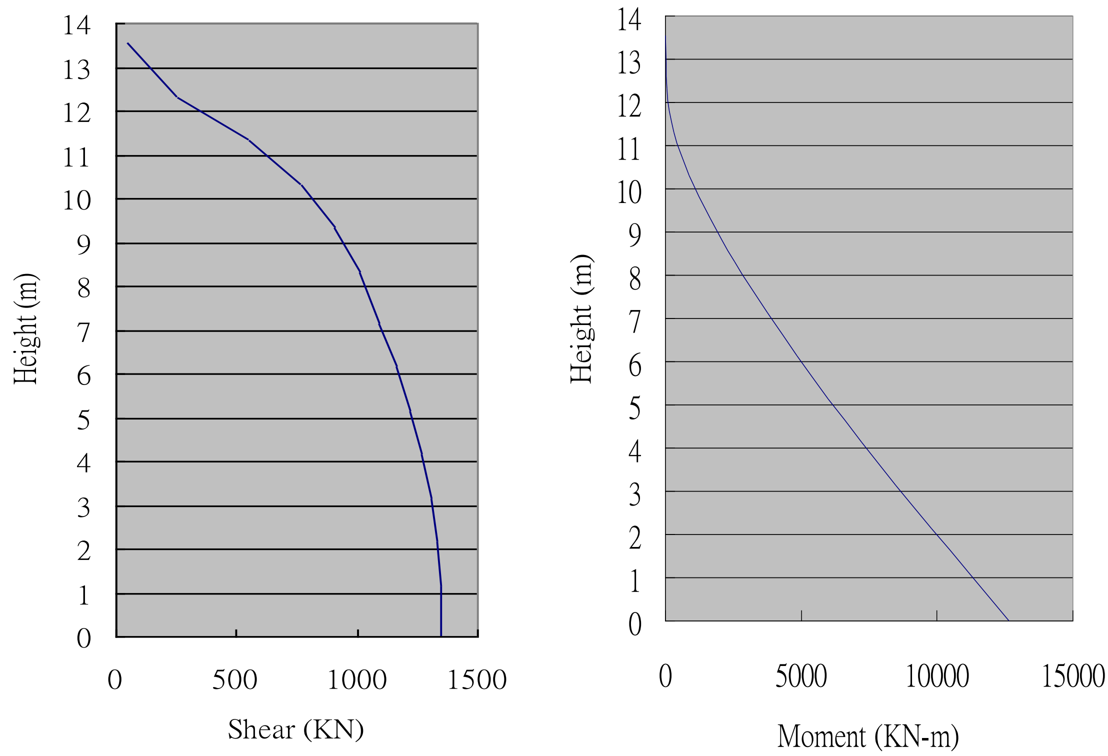

Fig. 13 – Shear force and moment of Pier 3 due to design earthquake base shear

Table 5 - The stress of Pier 3, MPa

Height m	N-S dir.			E-W dir.		
	Tension stress	Compression stress	Shear stress	Tension stress	Compression stress	Shear stress
13.54	0	0	0.004	0	0	0.004
12.32	-0.003	0.008	0.016	-0.002	0.009	0.016
11.32	-0.011	0.033	0.03	-0.007	0.037	0.03
10.32	0.008	0.115	0.058	0.004	0.111	0.058
9.32	0.149	0.353	0.101	0.059	0.264	0.101
8.32	0.548	0.868	0.153	0.177	0.497	0.153
7.14	0.992	1.355	0.164	0.25	0.614	0.164
6.14	1.225	1.61	0.167	0.335	0.719	0.167
5.14	1.442	1.848	**0.168**	0.414	0.82	**0.168**
4.14	1.642	2.069	0.168	0.489	0.916	0.168
3.14	1.823	2.27	0.166	0.558	1.004	0.166
2.14	1.988	2.456	0.164	0.622	1.088	0.164
1.14	2.134	2.621	0.16	0.68	1.166	0.16
0	2.276	2.786	0.154	0.74	1.25	0.154

7. Conservation suggestions

For the protection of the existing Yutengping Broken Bridge proper retrofit and strengthening are necessary. Followings are the suggestion for the conservation.

1. All cracks should be filled by injected cement or epoxy mortar.
2. Joints weather out should be re-pointed.
3. Archivolt should be tied by stainless bars embedded to spandrel.
4. In order to prevent further spandrel collapse, a steel arch between adjacent piers is suggested. The arch shape also needs to be designed following the original arch form.
5. Removal of salts and drainage improvement of the arch should be well considered for avoiding accumulation of soluble salts.

8. Conclusion

The Yutengping Broken Bridge is a cultural property with high value in Taiwan's railway history, architectural craft, and bridge construction technology. For the conservation of this cultural property, the present damage and deterioration should be improved properly. In this study, we investigate the primarily structure related problems, the results will help conservation architect to understand the present safety situation of the existing structure. Other information related to the broken bridge, which also required for conservation design, such as construction deterioration mechanism and its treatment under Taiwan's high temperature and high humidity, also needs to be established before starting the execution of repair and conservation work.

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