# RETROFIT/REPAIRS OF A DAMAGED BRIDGE FOR SEISMIC LOADING

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## Abstract

This paper described how a damaged river bridge caused by the Chi-Chi earthquake was inspected and retrofited, with consideration on the impact of the new revised code to the existing bridge structure in the light that the structure was actually designed and built in 1984. Comparison was also made between the old design approach and the new revised code requirements.

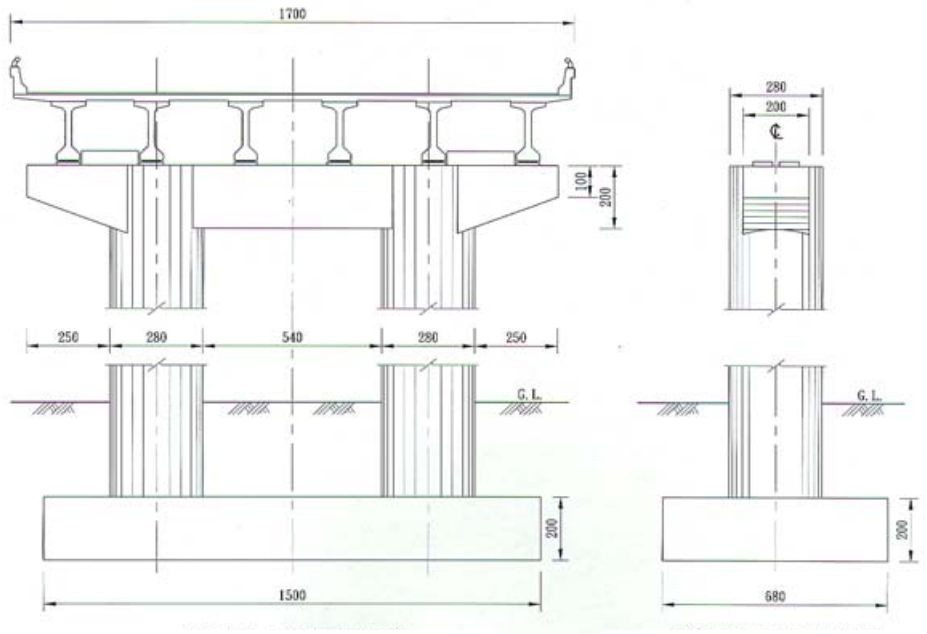
Keywords:Chi-Chi earthquake, Bridge Retrofitting, Seismic Loading, Seismic Design

### 1. Introduction

On September 21th 1999, the Chi-Chi earthquake strike central Taiwan, claimed many lifes and damaged thounsands of building and bridge structures. The affected area cover almost the whole Taiwan island. This is the largest earthquake in a century in Taiwan.

The bridge design code used in Taiwan was revised several time during the past 20 years. The first design code catered for the earthquake force design was first published in 1987[1]. Before that, either AASHTO or informal Japanese design code was adopted in the bridge design works. The 1987 code basically followed the 1980 edition of Japanese code. In year 1995, a new seismic design code [2] was adopted for all the national bridge structures with emphasis on the ductility consideration. Soon after the 1999 Chi-Chi earthquake, the 1995 edition seismic code was reviewed based on data collected during the Chi-Chi earthquake. Seismic zones were rearranged and the peak ground acceleration were revised from the four zones of 0.18,0.23,0.28,0.33 to only two zones of 0.23 and 0.33 respectively in the latest revised code released in April 7, 2000[3]. Vertical component of the seismic force was also emphasised in the design, although most designer in Taiwan had considered this in their work soon after the 1995 Kobe earthquake.

One of the damaged bridge,Yen-Fon bridge, built in 1984, is described in this paper. The bridge has a length and width of 455 meters (13@35m) and 17 meters, respectively. The bridge was design well before the 1987 code. Located at Nan-Tun Town of Nan-Tao county and crossing Wu-Si river, is the main bridge connecting Tsao-Tun town and Pu-Li town along Taiwan Provincial Highway No. 14. The bridge is located between Cher-Lung-Pu fault and Shuang-Dung fault. The superstructure of the bridge is composed of six prestressed I girders at each span. The substructure is of two-column frame type on a foundation of combined footing found on rock (Fig. 1). The height of the pier is 11 meters. The Chi-Chi earthquake caused damage to Yen-Fon bridge in the following manner: (i) superstructures of span No. 4~No. 6 and span No. 12 were displaced transversely; and, (ii) except pier No. 6, all pier caps dislocated transversely to some extent and part of their main reinforcing bars were exposed or completely cut (Fig. 2~4)[4]. The damage is not catastrophic, however, two out of four lanes remained open to traffic in order to maintain the heavy daily traffic. The Yen Fon bridge was inspected and repaired. The retrofit design was based on the new revised code. The retroffiting works were completed and the bridge was opened for traffic in April 30, 2001.



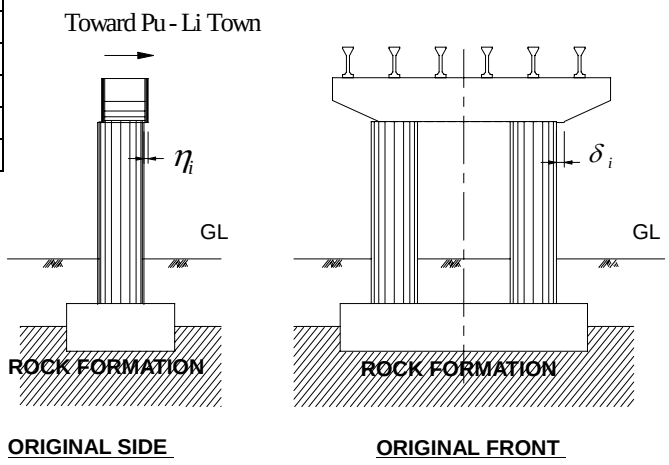

ORIGINAL FRONT ELEVATION

ORIGINAL SIDE ELEVATION

( unit: cm )

**Fig 1: Original Elevation**

| Pier # | $\delta_i$ (cm) | $\eta_i$ (cm) Upstream | Downstream |
|---|---|---|---|
| 8 | 10 | | |
| 9 | 30 | 5 | 5 |
| 10 | 35 | 1 | 5 |
| 11 | 25 | | |




notes

$\delta_i$ :Transverse Displacement of Cap Beam at pier #*i*

$\eta_i$ :Longitudinal Displacement of Cap Beam at pier #*i*

**Fig 2: Dislocation of Cap Beam**

**Fig 3: Conditions of Deck Displacement and Opening to Traffic**

**Fig 4: Damage at Pier Head**

After evaluation of various schemes available, and in considering shorten the construction time and reducing the construction cost, Moh and Associates, Inc. and Highway Bureau decided to retrofit the bridge instead of building a new one. By considering the damaged pattern, the following design principles are adopted:

(i) increase design horizontal acceleration coefficient to 0.33; and,
(ii) increase ductility and shear capacity of piers.
(iii) In addition, in order to accommodate the growing traffic in the future, the bridge width is increased to 21 meters. The design changes the pier type from two-column bents to four-column bents. The original two columns are used only to support part of vertical loading in the design and the new added two columns are designed to resist horizontal seismic forces and to resist the rest part of the loading.

In the following sections, the cause of the earthquake and the reasons why the bridge was damaged are first explained, details of retrofitting design of the bridge is then described.

## 2. Cause of Earthquake

The island of Taiwan was formed during pliocene, due to the collision of Eurasian Plate and Phillipine Sea Plate (Fig. 5)(subdivision plate of Pacific Plate, HO 1986 [4]). The latter is drifting from south-east at a rate of 7-8 cm/yr. And its Northern tip subducted underneath the Eurasian Plate along the east of Central Mountain Range of Taiwan. This thrust force crushed the Tertiary sedimetary rocks of 10 km in thickness to form numerous faults and folds generally in the N-S direction, roughly the same as the axis of the island, and turns toward N-E at northern part of the Taiwan. The release of strain energy stored during the tectonic movement resulting in numerous inevitable earthquakes in Taiwan .

The Chi-Chi Earthquake was result of reactivation of Cher-Lung-Pu Fault with epicentre near town of Chi-Chi and at a depth of about 8 km. The Fault has been known as a thrust fault and to run in N-S direction, from Fung-Yan at north, toward Chu-Shan at south with a total length of 60km , dipping eastward at an angle of 30 degree or less.

GPS survey results indicated that an area east of the fault with a width of about 15km was displaced north-westward for a maximum horizontal displacement of 9.06m at Ta-Kung of Taichung, (10m horizontal displacement was measured on the ground at Fung-Shih Road of Fung-Yan City) and uplifted vertically about 9.8m at Shih-Kang Dam near the north tip of the fault line.

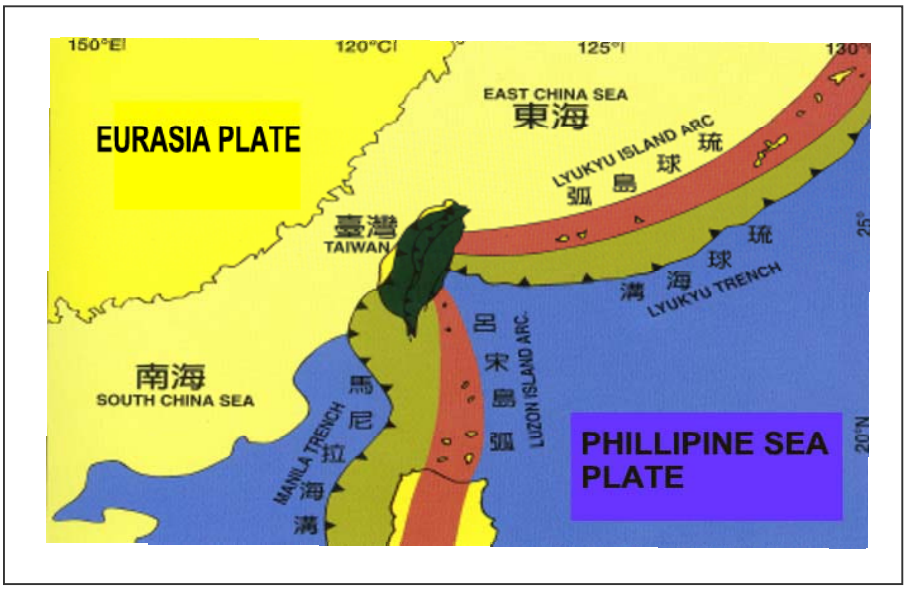



**Fig 5: Tectonic Structure around TAIWAN (After Ho.1986)**

## 3. Cause of Bridge Damage

The upper part of formation near bridge is alluvium and the formation below is Pleistocene alternation. The formation displaced from east by south toward west by north during earthquake, which can be judged from dislocation and cracking pattern of damaged cap beams (pier No. 7~ No. 12). The ground acceleration measured at the nearest seismometer station is about 500 gal, which is much larger than the original design value. However, because the direction of formation displacement is perpendicular to the longitudinal direction of the bridge and two-column bent piers provides redundancy, the earthquake only caused minor damage such as dislocation of pier caps and spalling of concrete. Two lanes remains open to traffic after the earthquake.

## 4. Structural Analysis and Retrofitting Design

The basic principle of the 1995 edition of Highway Bridge Seismic Design Code issused by the Ministry of Transportation and Communications is to allow structures to response only elastically in medium earthquake, and to allow formation of plastic hinges but without collapse when large earthquake strikes. Thus, the increase of shear capacity and ductility of pier is the main consideration in the retrofitting design.

### 4.2 Structural system

The original superstructure of the bridge is composed of six prestressed I girders of 2 meter height. The substructure is of two-column frame type on a foundation of combined footing founded on rock(Fig. 1). The diameter of pier column is 2.8 meters. All together there are 13 spans of which each span length is 35 meter long. All girders are simply supported. The bridge deck width is 17 meters. In order to accommodate the growing traffic in the future, the bridge width is increased to 21 meters in retrofitting work (widening 2 meters of bridge deck at both sides and adding one extra girder at both sides). Two new columns of 2.5m diameter are added to the outside of original two column bents to form four-column bent piers. Cap beams and foundations are enlarged at the same time (Fig. 6~9). Even though all height of the piers are the same for this river bridge, however, due to high river bank at both side of the river, some piers on the river bank are in fact buried deep in the river bank and hence apparently become a rather short column as compare to the other part.

### 4.3 Design code and loading

#### 4.3.1 Design code and live load

The live load HS20-44 as per specification of AASHTO is adopted in the design (Ministy of Transportation

and Communications, 1987)  ( AASHTO, 1996[5]).

4.3.2 **Design seismic load:**

Earthquake force: $\mathbf{V=ZICW/1.2\alpha_y F_u}$ (1)

(Ministy of Transportation and Communications, 1995)

Where

**Z** : site horizontal acceleration coefficient, 0.33 (Seismic zone 1A,1995 or zone 1, 2000)

**C**: normalized acceleration response spectrum

**W**: total dead weight

$\alpha_y$ : the ratio of design ground acceleration to ground acceleration expected to initiate yielding in the structure, 1.65

**I**: importance factor, 1.2

$\mathbf{F_u}$: seismic force reduction factor

**4.3.3 Design stress**

Adopt ACI 318-95 design code (ACI, 1995[6]),

Where

(a) concrete:

| | |
|---|---|
| reinforced concrete | fc'=20 MPa |
| prestressed concrete | fc'=35 MPa |

(b) reinforcement:

| | |
|---|---|
| 16mm and smaller size | fy=275 MPa |
| 19mm and larger size | fy=415 MPa |

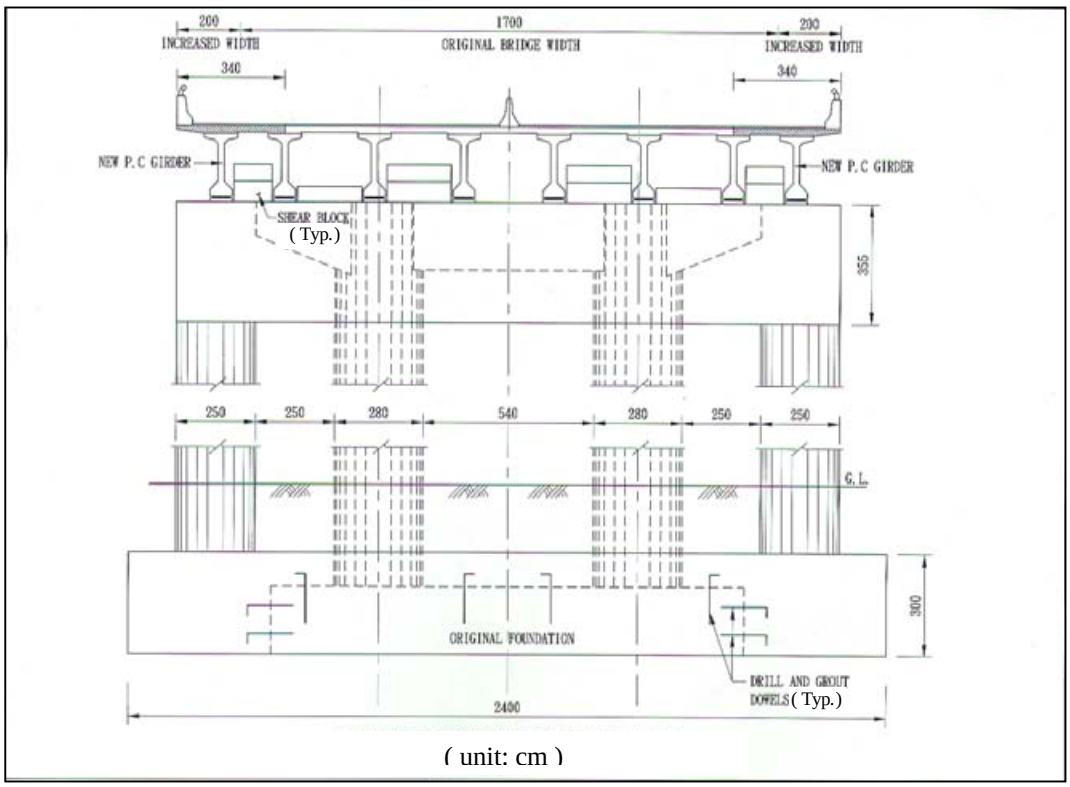



**Fig 5: Front Elevation of the Retrofitted Bridge**

### 4.4 Structural analysis

SAP 2000 is used for the analysis. Three-dimensional structural model is constructed using frame elements. Loading combinations consisted of dead load, live load, and earthquake force are applied to the structures. Dynamic analyses are performed to study the effect of earthquake force using response spectrum of TYPE I soil profile. The design ground horizontal acceleration coefficient is increased from 0.23 of seismic zone 2 to 0.33 of seismic zone 1A as per the local seismic design code.Before carrying out the retrofitting design, a study on the bridge design based on parameters and approaches permitted in the old code and the new code were performed to understand the damage condition. In the old approach, only single span is considered in the analysis, however, in the new code approach, the spans comprising tall pier at river course and short pier at river bank are modeled as one analysis unit. It is found that based on the old code (1987), $K_h$ =0.15, the horizontal shear force at short pier column base and column head are $104^T$ and $76^T$, respectively. However, based on the latest seismic code, the corresponding shear forces are $146^T$ and $125^T$, respectively. For comparison purpuses, the latter if conversed to old approach equivalent, $K_h$=0.21. This is significantly higher than the value used in original design.

Based on the old design approach, the column diameter of 2.8m and the reinforcing steel are more than sufficient to resist the shear forces though the reinforcement ratio is less than 1%. Since the new code take the effect of the adjacent piers into account, the short pier column do attract more horizontal forces and the existing reinforcing details is not sufficient to withstand the induced forces. Moreover, in normal construction practice the interface between the soffit of the cap beam and the pier head most likely a construction joint which is not a monolithic connection, and hence this interface became a week point. Bearing in mind the ground acceleration measured at the nearest seismometer station is about 500 gal, the damage at the piers head is unevitable.

### 4.5 Retrofitting design

The original two columns are used only to support part of vertical loading in the design and the new added two columns are designed to resist horizontal seismic forces and to resist the rest part of the loading. Based on dynamic earthquake analyses, shear reinforcement is designed by the loading case which causes plastic hinges or by the earthquake force **ZIg** corresponding to a return period of 475 years. In addition, confining reinforcement at plastic zone is checked to meet ductility requirement described below:

$$\rho_S = 0.45\left[\frac{A_g}{A_c} - 1\right]\frac{f_c'}{f_{yh}}\left[0.5 + 1.25\frac{P_e}{f_c' A_g}\right] \qquad (2)$$

or

$$\rho_S = 0.12\frac{f_c'}{f_{yh}}\left[0.5 + 1.25\frac{P_e}{f_c' A_g}\right] \qquad (3)$$

whichever is greater , but is not less than:

$$\rho_S = 0.45\left[\frac{A_g}{A_c} - 1\right]\frac{f_c'}{f_{yh}} \qquad (4)$$

Where

Ac: core concrete area

Ag: gross concrete area

Pe: factorized axial force

fc': compressive strength of concrete

fyh: yielding strength of hoop or spiral reinforcement

$\rho_S$: volume ratio of hoop reinforcement to core concrete (calculating to the outside perimeter of spiral reinforcement).

The reactions at pier base transfered to foundation are used as foundation design forces. The dimension of retrofitted foundations is increased from 15mx6.8mx2.0m to 24mx14.8mx3.0m (LxWxD) to provide appropriate bearing capacity(Fig.6).

Width of the cap beam is increased from 200 cm to 370 cm to provides enough space for stoppers, which also increases seating length at movement joints.

The superstructures of span No. 4~6 and span No. 12 are dismantled and reconstructed. Earthquake stoppers are added and expansion joints are renewed.

### 4.6 Construction procedure

By considering the safety during construction, the following retrofitting construction procedure is adopted:

1. Construct a temporary bridge to divert the traffic.
2. Construct temporary supports for original superstructures.
3. Excavate soil and build temporary retaining structures for foundation construction.
4. Construct foundation slab and pier columns.
5. Backfill soil and construct pier caps.
6. Remove parapets on both sides and part of damaged slab; cut main reinforcement in original pier columns.
7. Remove temporary supports for superstructures.
8. Erect prestressed girders.
9. Construct bridge deck.
10. Construct parapet, railing, and asphalt pavement.Cutting the reinforcing steel at pier head of existing columns.

## 5. Construction Stage

The Yen-Fon Bridge retrofitting contract was awarded soon after the design completed. Based on the original design concept, a temporary bridge will be constructed for detouring the traffic during the retrofitting works, so that the repairing and concreting work will not be affected by the existing traffic. However, due to the difficulties in accessing private land and other restrictions, construction of the temporary bridge was delayed. The construction sequences were then reviewed. The enlargement of the foundations and pier strengthening works were carried out while the daily traffic was maintained. The enlargement of foundation, construction of additional columns and cap beam are shown in Fig.6 to Fig.9.

**Fig 6 Enlargement of Foundation**

**Fig. 7 Enlarged Footing and Additional Columns**

**Fig. 8 Embedding Rebar to Existing Pier Cap**

**Fig. 9 Setting Up Formwork for Cap Beam**

## 6. Conclusions

Yen-Fon Bridge is located between two active faults in western Taiwan and is 4 kilometers to the east of the closer Shuang-Dung fault. The original bridge construction was started on October 29th 1982 and completed on March 18th 1984. The original design used horizontal seismic coefficient $K_h$=0.15 and angle of repose of 30 degree. The ground acceleration near the bridge site in Chi-Chi earthquake is much higher than the original design value. Because of the redundancy provided by two-column bent piers, the bridge didn't suffer severe damage. In order to ensure the bridge response elastically in medium earthquake and allow formation of plastic hinges without collapse, the increase of ductility and shear capacity of pier is the main consideration in the retrofitted design.

## 7. Acknowledgments

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