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ON SEISMIC RETROFIT STRATEGIES OF HIGHWAY BRIDGES - EXPERIENCES LEARNED FROM CHI-CHI EARTHQUAKE

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SUMMARY

This paper simulates the performance of Yen-Feng Bridge during the September 21 Taiwan Chi-Chi Earthquake. In this earthquake, very high peak ground accelerations, near fault velocity pulses, as well as permanent ground displacements were observed in a very wide area. However, the extent of bridge damage is relatively minor when compared to those observed in the 1994 Northridge earthquake and 1995 Kobe earthquake. Most of the bridge damage appeared to be the movement of superstructure and separation of thermal expansion joints due to sliding or failure at the bearings, with the exception of seven bridges collapsed due to large fault displacements directly crossed the bridges. It was also observed that the number of bridge column damage was surprisingly small. Since most of the bridges in the damaged area of Chi-Chi earthquake were designed without ductile detailing, it may be in contrast to the current seismic design concept emphasizing the steel in the plastic hinge zone of the bridge columns. Yen-Feng Bridge is near the epicenter of the earthquake with only moderate damage. A non-linear finite element model of this bridge is first established to simulate the type of damage and verified by the observed conditions. Parametric studies of seismic performances controlled by the bearing with strong or weak strengths are then carried out to discuss why similar bridges experienced with minor to moderate damage during Taiwan Chi-Chi earthquake.

INTRODUCTION

The 921 Taiwan Chi-Chi Earthquake with the magnitude of ML=7.3 incurred tremendous disaster to the central region of the island, particularly to Taichung and Nantou countries. In this earthquake, very high peak ground accelerations, near fault velocity pulses as well as permanent ground displacements were observed. However, the extent of bridge damage is relatively moderate when compared to those observed in the 1994 Northridge Earthquake [1] and 1995 Kobe Earthquake [2]. Most of the bridge damage appeared to be the movement of superstructure and separation of thermal expansion joints due to sliding or failure at the bearings, with the exception of seven bridges collapsed due to large fault displacements directly crossed the bridges [3]. It was also observed that the number of bridge column damage was

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surprisingly small. Yeng-Fang Bridge is located in between the Chu-Lon-Pu and Shuang Tung fault, as shown in Figure 1. It is near the epicenter of the earthquake with only moderate damage. In this study, a non-linear finite element model was established to predict the seismic performance of the Yen-Feng Bridge during the 921 Taiwan Chi-Chi Earthquake. Results of analyses are compared with the actual damage of the bridge under this earthquake. Parametric studies of seismic performances controlled by the bearing with strong or weak strengths are then carried out to discuss why similar bridges experienced with minor to moderate damage during Taiwan Chi-Chi earthquake.

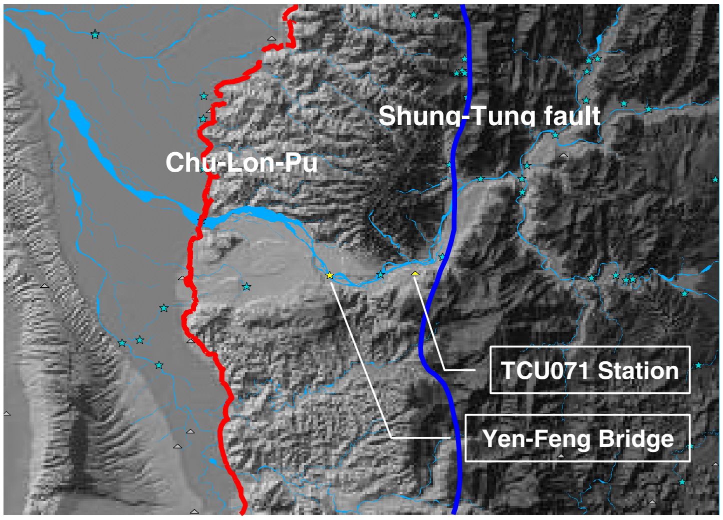


Figure1. Geographical position of Yen-Feng Bridge

THE YENG-FENG BRIDGE

Brief Introduction

Yen-Feng Bridge, located at the milepost of 26km+937m on Provincial Route 14, is one of the main links connecting Tsaotun and Poli town in Nantou country. The bridge is in between the Chu-Lon-Pu and Shung-Tung fault (approximately 5 km from Chu-Lon-Pu fault and 7km from Shung-Tung fault), as shown in Figure 1. It is actually 13-span bridges with a constant span length of 35m and 16m in width. All spans are supported by neoprene bearings at the cap beams. The superstructure is composed of six simple supported, prestressed reinforced concrete girders, and the deck slab has additional 5 cm A.C. on top of the girder. The substructure comprises multicolumn bents (portal frame) on spread footings. At both ends, the bridge girders are supported on the abutments. The schematic drawing and cross section of the whole bridge system are shown in Figure 2 and 3, respectively. Two concrete shear keys are installed inside on each cap beam to prevent the girders from unseating in the transversal direction. The Yeng-Feng Bridge is a typical type highway bridges in Taiwan, constructed before 1990's [3].

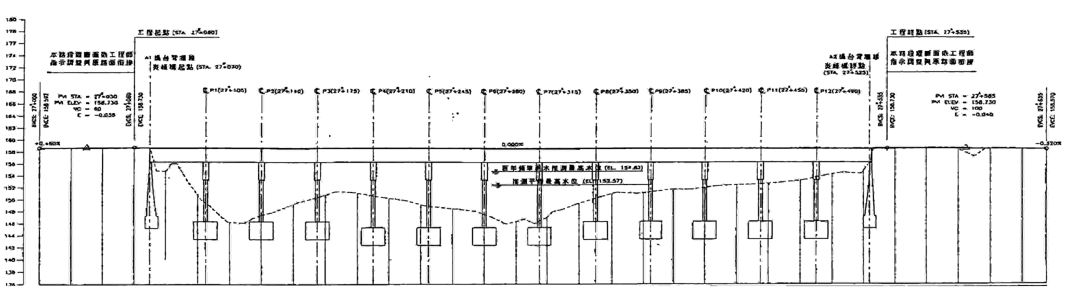
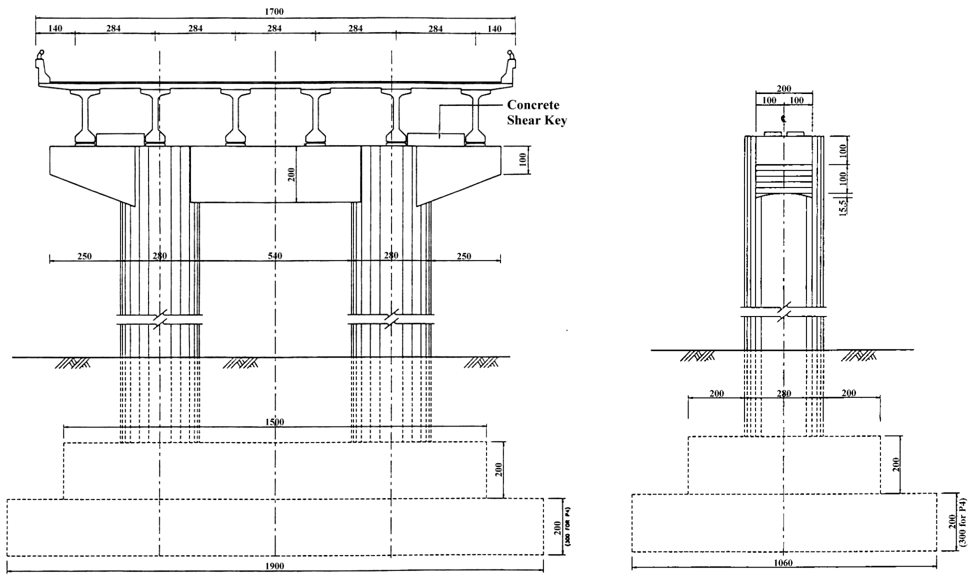
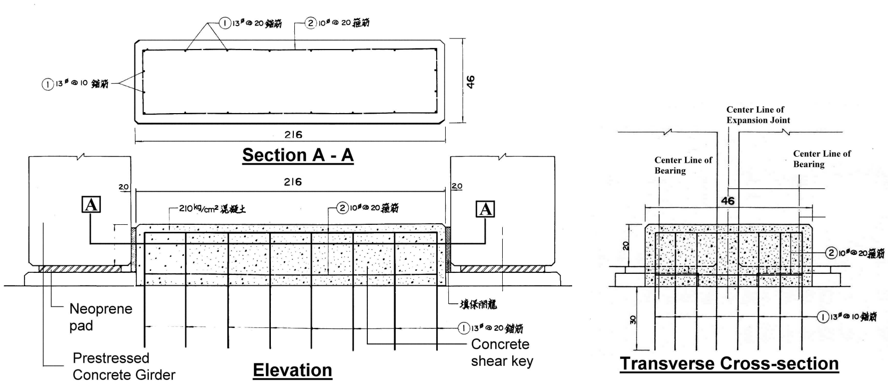


Figure2. Schematic drawing of the bridge



(a) Multicolumn bent



(b) Concrete Shear key

Figure3. Elevation and sectional drawing of Yen-Feng Bridge

Bridge damage

The Chu-Lon-Pu and Shuang-Tung fault did not directly across the Yeng-Feng Bridge, but evoked strong ground motion in vicinity of the bridge with a horizontal PGA of 500 gal in primary north-south, 400 gal in primary east-west and vertical PGA of 300 gal. During such large ground excitations, concrete spalling and cracking occurred at the junction between column top and the cap beam, and several concrete shear keys on the cap beam were crushed. According to the bridge-disaster reconnaissance reports, the damage of Yeng-Feng Bridge are summarized as follows [4~5]:

1. Concrete spalling and cracking on the weak construction joint between cap beam and column top at bents P1 to P12, as shown in Picture (1).
2. Relative movement of cap beams at bents P9, P10 and P11 the in transverse direction, as shown in Picture (2).
3. Several concrete shear keys were cracked.
4. Residual relative displacements occurred at several expansion joints, a typical damage shown in Picture (3). Among many others, at the fourth span, the largest displacement resulted from one outside broken girder moved about 40 cm in transverse direction, as shown in Picture (4).
5. Parts of neoprene bearings had permanent displacement.

(1)

(2)

(3)

(4)

Picture(1) Concrete spalling and cracking at pier top; (2) Relative displacement of cap beams at pier top; (3) Residual displacement at expansion joint; (4) Residual movement between two adjacent spans (at the fourth span)

ANALYTICAL MODELS

Super structure

Yen-Feng Bridge consists of the prestressed reinforced concrete girders, decks and bearings, as shown in Figure 3. In this study, the superstructure is assumed to remain essentially elastic, limiting the nonlinear modeling considerations to the bearings, concrete shear key, expansion joint, and connection between cap beam and bridge columns. Figure 4 shows the FEM model by SAP2000N [6].

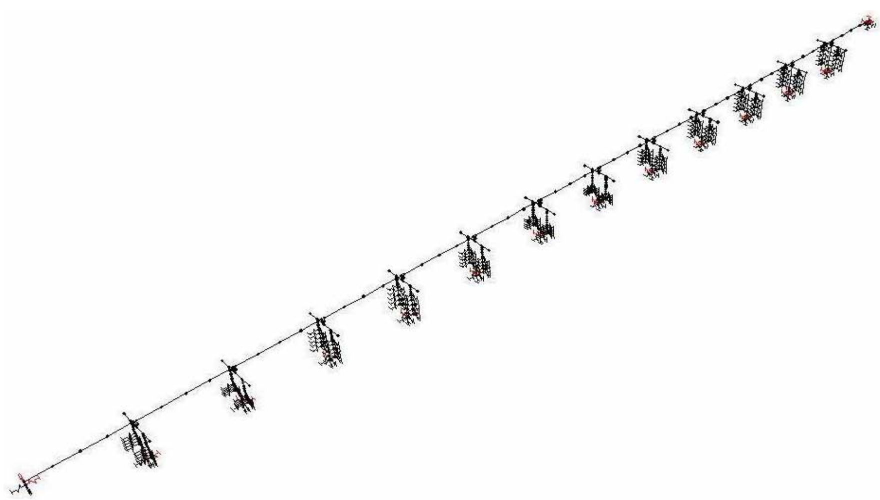


Figure4. Analytical model of the bridge system

Sub structure

The multi-column bent is modeled as a portal frame along the bent axis, consisting of beam and column elements with effective member properties [7]. Except for properly modeling the columns, cap beams and footings, the effect of soil-structure interaction at the abutment and footing was also considered by equivalent sets of linear springs [8~9]. Furthermore, considering the change of riverbed may affect the effective length, the soil-covered part of the column is also supported by spring elements [7]. The construction joint between the cap beam and column top has been found as weak face with lower shear strength, as observed during this earthquake. In order to understand this phenomenon, the weak construction joint is modeled as a bi-linear element and the shear strength is determined according to the shear friction approach [10], as shown in Figure 5.

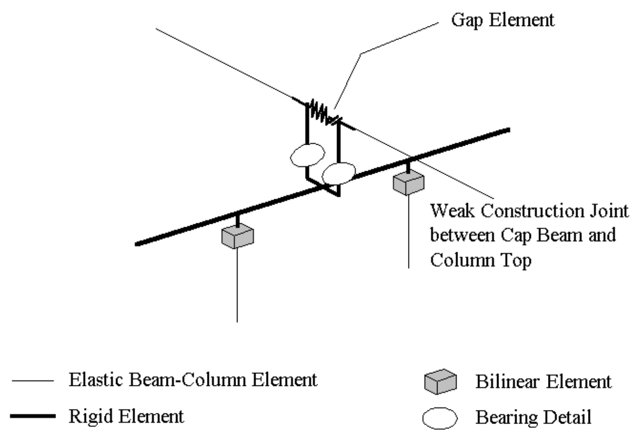


Figure5. Non-linear elements of sub-structure

Expansion joint

From design drawing [5] and seismic investigation after earthquake, the gap length between two adjacent spans is 10cm and the allowable movement for each expansion joint is ± 5cm. Displacement between various separated by the expansion joints can result in impact under the seismic excitation. Therefore, a gap element (as shown in Figure 5) is adopted to estimate the expected differential displacement across the expansion joints. The axial stiffness of expansion joint is equal to 2 times axial stiffness of adjacent girder.

Bearing system

The analytical model of the bearing, including the neoprene stiffness, friction between the beam and concrete surface, and the steel rod are shown in figure 6. The friction behavior between girder and neoprene pad are modeled as friction-pendulum element with an initial stiffness equal to that of the neoprene bearing and zero stiffness after slipping occurs. The friction force is equal to bearing axial reaction force N times friction coefficient μ. The friction coefficient is about 0.15~0.2 [11]. The function of the steel rod is used to limit the displacement of superstructure during small to middle-earthquake. The rod is modeled as a bending short bar with stiffness and yielding shear force equals to 3EI/L3 and 0.57Avfy, respectively. A bi-linear element is used to model the gap elements are considered as the gaps between PCI girder and steel rod. The ratio of gap stiffness to rod stiffness is 1000.

Concrete shear keys

Concrete shear keys between the superstructure and cap beam critically affect the seismic response of the bridge, especially the distribution of inertia forces and displacement between bents in the transverse direction. In this study, the concrete shear key is modeled as a bi-linear element. The stiffness and yielding strength of concrete shear key are determined according to reference [12]. Gap elements are used to consider the gaps between PCI girders and the shear keys. The ratio of gap stiffness to concrete shear key stiffness is 1000.

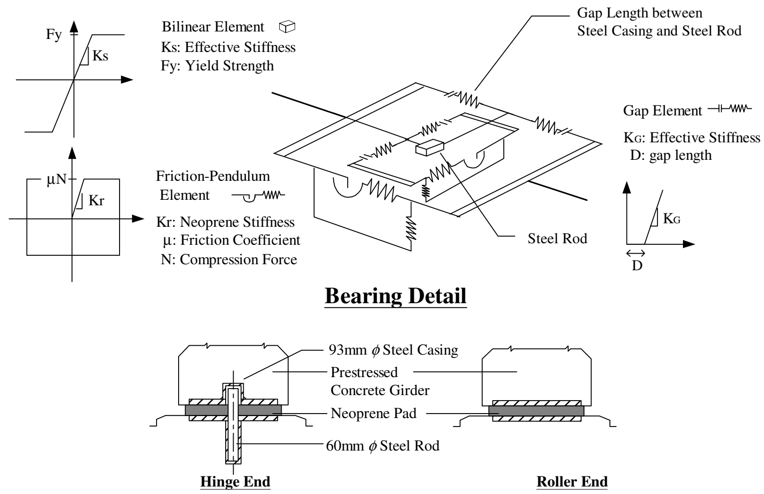


Figure6. Non-linear elements of super-structure

Characteristics of ground acceleration

The ground motion used in this study was selected to represent as closely as possible the actual excitation at the bridge site under the 921 Taiwan Chi-Chi Earthquake. The nearest strong motion accelerometer, TCU071 is located approximately 4 km away from this site. Figure 7 is the acceleration records in the east-west, north-south and vertical directions, respectively. The peak values of the ground acceleration (PGA) measured in those recordings are 518 gal, 640gal and 416 gal, respectively, with east-west, north-south and vertical direction. The response spectrum of this earthquake shows that the design response spectrum is not conservative in a few period ranges, such as the short period of 0.16 ~ 0.4 second and the median period of 0.5~0.82 second in east-west, and in short period of 0.15 ~ 0.35 second in north-south directions.

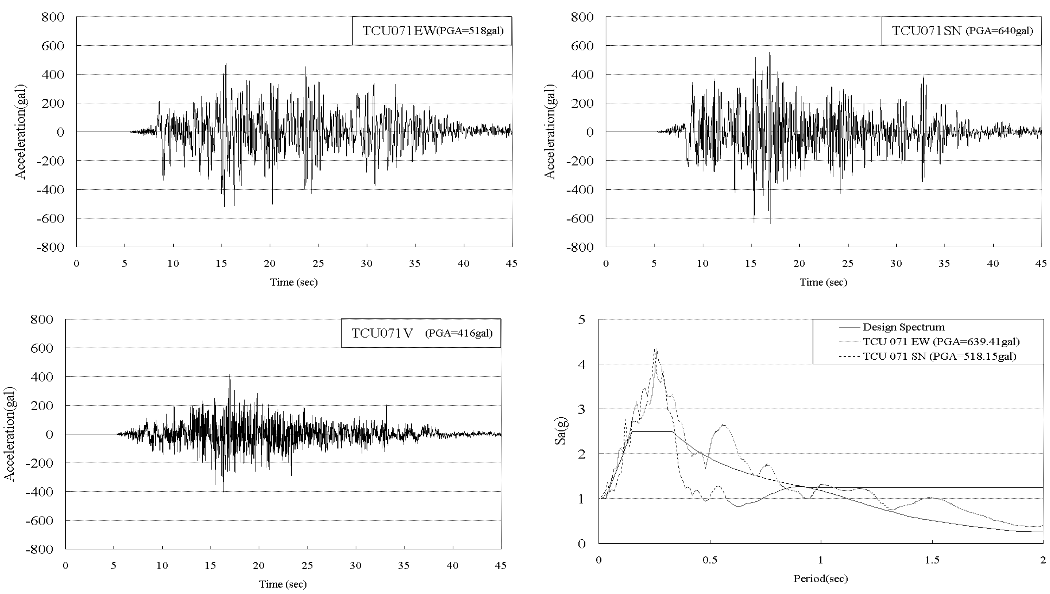


Figure7. Ground Motion Recordings and Response Spectrum (TCU071)

NONLINEAR TIME HISTORY ANALYSES

The damage conditions of Yen-Feng Bridge characterize an interesting failure mode, small residue displacement occurs on the superstructure and minor damage on the column due to sliding at the neoprene bearings, different from plastic hinge at the columns. The major reason may arise from the difference of the bridge systems between Taiwan (the simple supported PCI-girder bridges) and USA (the continued rigid connection bridges). In order to simulate the dynamic response of Yen-Feng Bridge during the 921 Taiwan Chi-Chi Earthquake, all records of TCU071 station in the east-west, north-south and vertical direction are used as input ground motions in the numerical model. The overall responses associated with strong or weak bearing strength, including the bearings, expansion joints, and pier forces are summarized as follows. Here, the strong bearings can take plastic shear forces induced by plastic moment on the column. On the other hand, the weak bearings allow sliding when friction forces are reached.

Weak construction joint between the cap beam and pier top

Figure 8 and 9 show the maximum shear force and bending moment of the construction joint between the cap beam and column top, respectively. The dotted line means the analytical results of the bearing strength being reinforced. From shear friction approach in design code [10], the shear capacity of weak joint is about 4708.8kN (480tonf). The weak joints at piers P8 ~ P11 had been yielded, and ones at piers P2, P5 and P8 are almost yielded. While superstructure moved 20 mm and pounded the concrete shear keys, it transferred impact forces to the bridge columns in the transversal direction. In the longitudinal direction, the shear force transferred from the bearing friction and steel rod was minor. The results represent reasonable damages of the bridge. Apparently, after strengthens the bearing shear capacity, the shear force of the construction joint between the cap beam and column top increased and led to larger damage.

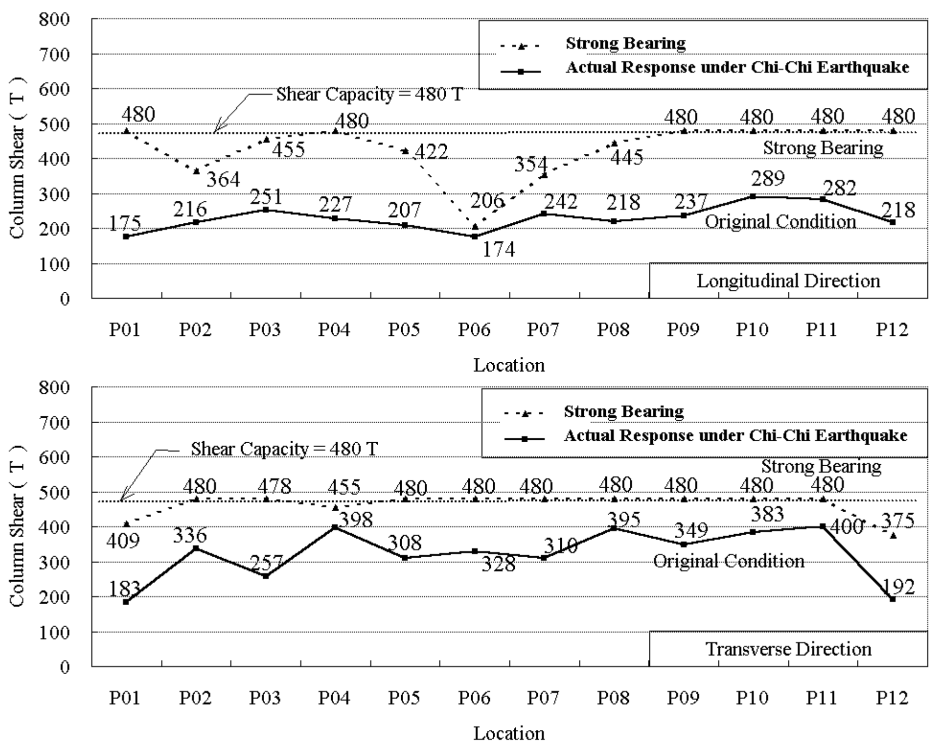


Figure 8. Maximum shear force of the construction joints between the cap beam and column top

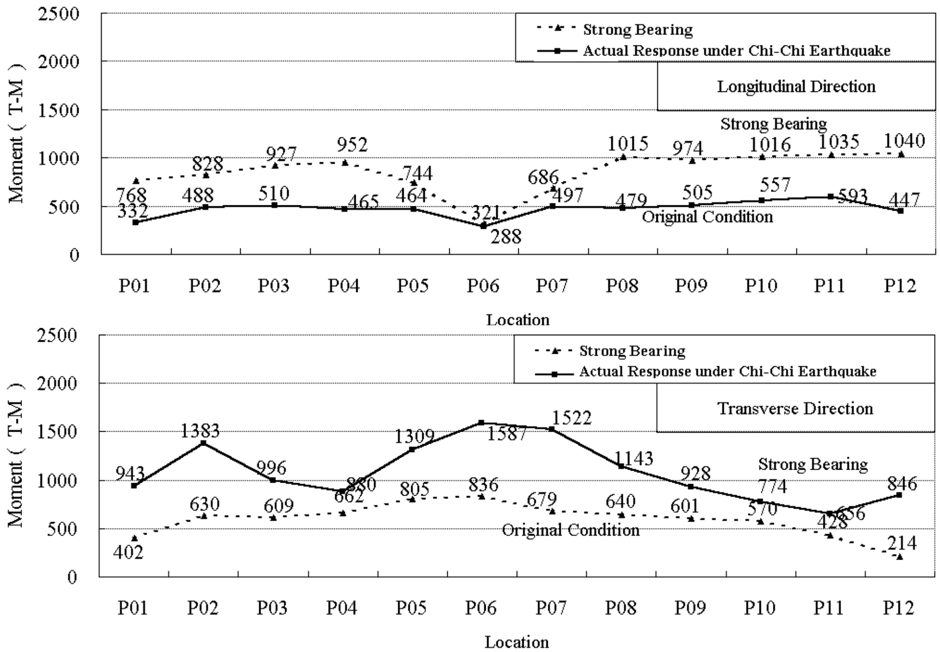
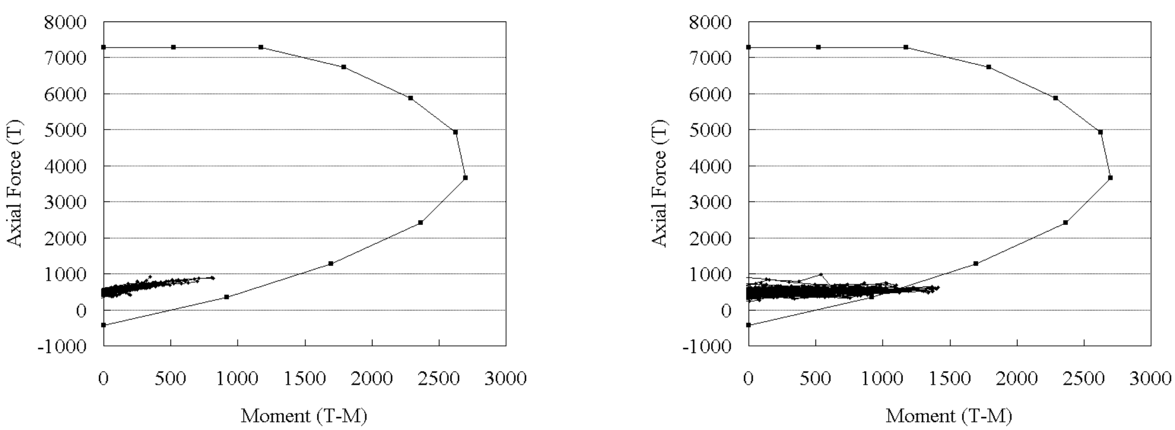


Figure 9. Maximum bending moment of the construction joints between the cap beam and column top

Flexural strength

In order to evaluate whether the flexure failure of column occurs or not, the axial load-moment interaction diagram is calculated. Comparing the axial force-moment time history response with axial load-moment interaction diagram, as shown in Figure 10(a), the maximum moment and axial force for P6 did not excess the axial load-moment interaction diagram that indicates the pier did not have flexure failure. Virtually only damage occurred at the weak construction joint between the cap beam and pier top, no flexure failure is found. Figure10 (b) shows the results of the bearing strength being reinforced. Apparently the column moment increases and caused damage.



(a) Original condition (b) Strong Bearings

Figure10. Comparison the P-M time history response with P-M interaction diagram

Bearing system

Figure 11 shows the maximum relative displacement distribution of the bearing and displacement time history at bent P1. With little shear capacity force of steel rod, smaller than concrete shear key in the

transversal direction, the longitudinal displacement move larger than one in the transversal direction, and cause a permanent displacement on the superstructure, when bearing moving and steel rod yielding.
Figure 12 shows the maximum pounding force distribution of the concrete shear keys and pounding force time history at bent P3. When the PCI girders pounded the concrete shear keys, the displacements of superstructure were limited to 20 mm, and the forces were transferred to the substructure. The results of numerical analysis represent that all of the steel rods had been yielded during this earthquake.

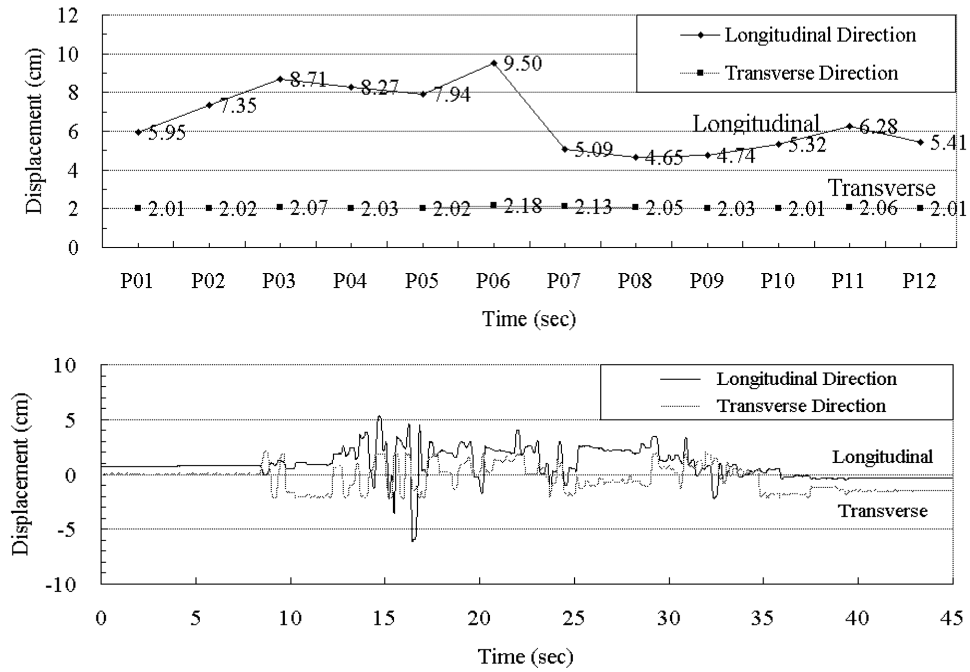


Figure11. Maximum relative displacement distribution at bearings and Relative displacement time history at bent P1

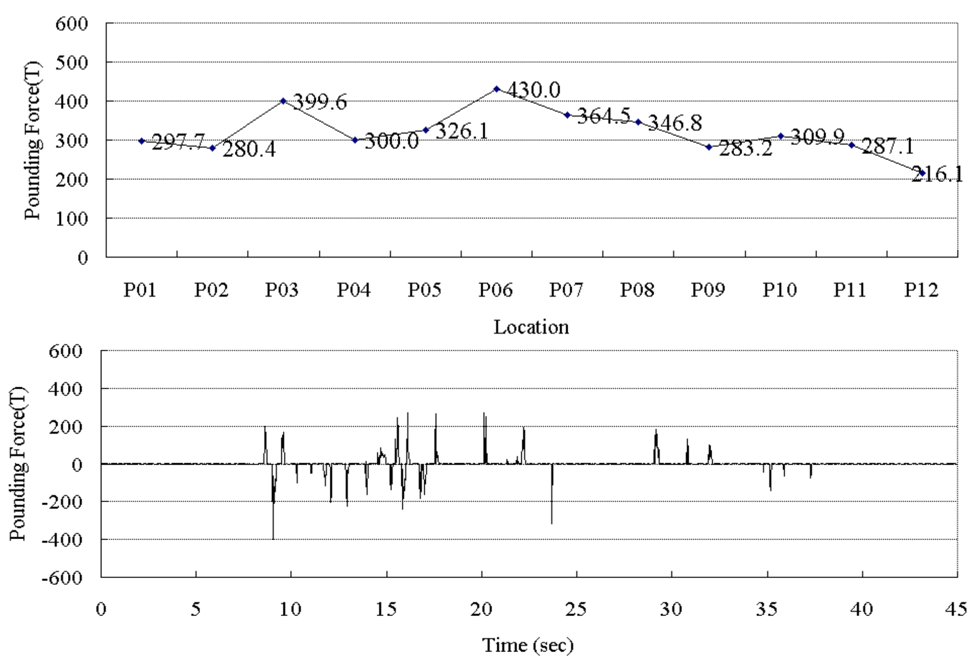


Figure12. Maximum pounding force distribution of the concrete shear keys and Pounding force time history at bent P3.

Expansion joint

The maximum longitudinal displacement for the expansion joint can be obtained using the nonlinear time history analysis, as shown in Figure 13. Except for the span S1 ~ S2, S12 ~ S13 deck slabs had more than 5cm relative displacement, the rest parts of bridge have less relative movement; therefore, no pounding occurred during this earthquake.

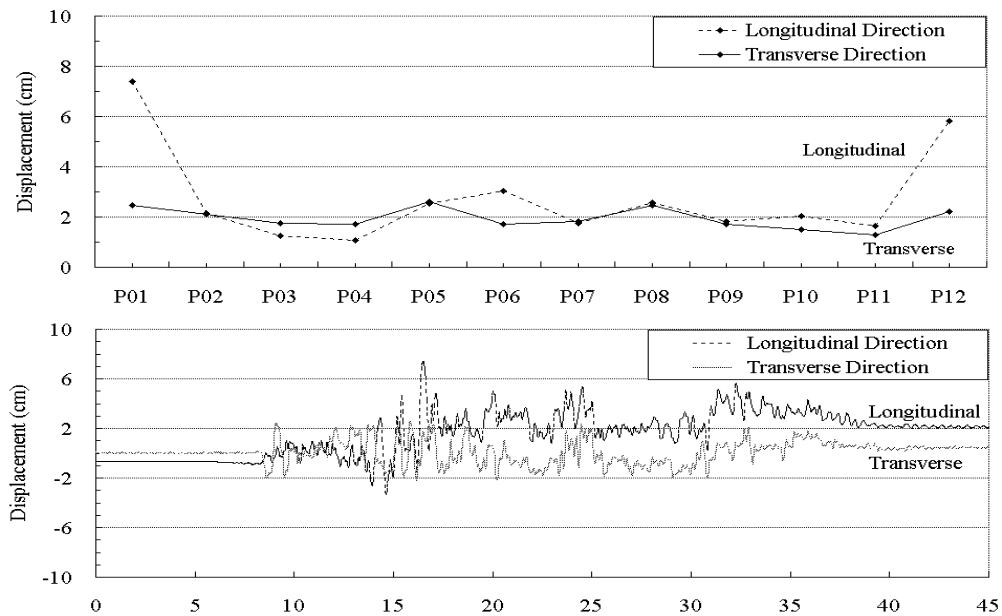


Figure13. Maximum deformation distribution of the expansion joints and Deformation time history at bent P1

CONCLUSION

Based on the analytical results presented in this study, the following conclusions can be made:

1. The analytical model consists of bearings, concrete shear keys and other hysteric behaviors, predicted the actual seismic performance of the bridge and lead to results that were generally in agreement with the bridge-disaster reconnaissance reports.
2. The bearing strength plays an important role to design substructures. The analytical results explain the fact that the bridge columns suffer minor damage when the movement of superstructure occurred due to sliding at the bearings. For a bridge column without detailed design, the analysis results show that the damages are increased when increasing the bearing strength. The major reason may arise from the difference of the bridge systems between Taiwan (the simple supported PCI-girder bridges) and USA (the continued rigid connection bridges).
3. As the superstructure moved forward to the transverse in 2 centimeters and pounded the concrete shear keys, it transferred forces to the bridge column and led the substructure damage to a serious extent in this direction. In the longitudinal direction, the substructure damage is minor without shear keys.

ACKNOWLEDGEMENTS

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