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Seismic Performance of Highway Bridges

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ABSTRACT

The 921 Taiwan Chi-Chi earthquake incurred tremendous disaster to the central region of the island, particularly to Taichung and Nantou counties. Most of the bridges on the provincial and county routes in Taichung, Nantou, Changhua, and Yunlin counties escaped from serious damage, while approximately 20% of them suffered minor-to-major damage. The construction completion dates of those bridges range from 1960 through 1999. Damage to these bridges include fault rupturing, collapsed spans, landslides, soil settlement, slope failures, flexural and/or shear failures, and liquefaction. This paper begins with a brief description on the evolution of seismic design code for bridges in Taiwan, followed by a general depiction of the performance of highway bridges located in the four counties during the earthquake. Several major damaged bridges with typical damage modes are illustrated and explored. Lessons learned from the field observations and suggestions made on research needs are also discussed.

INTRODUCTION

A devastating earthquake with the magnitude of M_L = 7.3 struck the central region of Taiwan in the early morning on September 21, 1999. It was known as the 921 or Chi-Chi earthquake. As exhibited by the statistics [1], over 2,350 people were killed, 10,000 were injured, and 35 were missing in the earthquake. Moreover, approximately 10,000 buildings or houses collapsed and 7,500 were damaged. About 20% of the bridge inventory on the provincial and county routes in the disaster area suffered minor-to-major damage.

There are approximately 1,000 highway bridges spread on the provincial and county routes in Taichung, Nantou, Changhua, and Yunlin counties, which were the regions with major catastrophe, especially Taichung and Nantou counties.

Most of the bridges escaped from serious damage, while approximately 20% of them suffered minor-to-major damage due to fault rupturing, collapsed spans, landslides, soil settlement, slope failures, flexural and/or shear failures, and liquefaction. The construction completion dates of those bridges range from 1960 through 1999.

This paper begins with a brief description on the evolution of seismic design code for bridges in Taiwan, followed by a general depiction of the performance of highway bridges located in the four counties during the earthquake. Several major damaged bridges with typical damage modes are illustrated and explored. Lessons learned from the field observations and suggestions made on research needs are also discussed.

EVOLUTION OF SEISMIC DESIGN CODES

In Taiwan, consideration of seismic demand on bridge structures may be traced back to the "Pocket-Size Engineering Handbook" issued by Taipei Branch of the Chinese Engineer Association in 1954. It was suggested that there were two different seismic zones in Taiwan, corresponding to two horizontal seismic design coefficients of 1.0 and 1.5, respectively. In 1960, the "Engineering Design Code for Highway Bridges" was announced by the Ministry of Transportation [2]. However, there was no specific guideline established for seismic design in that design code. Therefore, the suggestion provided by the "Pocket- Size Engineering Handbook" was adopted in that era.

Since the simple seismic coefficient design approach adopted in 1960s was out-of-date and did not conform to the progress made in bridge engineering, the Ministry of Transportation of Taiwan issued the "Design Code for Highway Bridges" in 1987 [3]. Seismic resistant conception, geological characteristics of local sites, and dynamic amplification factors were included in that design code announced in 1987. Complying with more understanding of structural dynamic behavior and development on bridge engineering techniques, a need to revise the seismic resistant design approach described in the "Design Code for Highway Bridges" was realized. Based on the research on earthquake engineer- ing in Taiwan with reference to the seismic design methodologies for bridges in Japan and United State, the current "Seismic Resistant Design Code for Highway Bridges" was elaborated and announced by the Ministry of Transportation in 1995 [4].

HIGHWAY BRIDGE PERFORMANCE

General Observation

The primary disaster area in the 921 Chi-Chi earthquake is composed of four counties, Taichung, Nantou, Changhua and Yunlin. According to the preliminary bridge-disaster reconnaissance reports [5,6], there are approximately 1,000 highway bridges spread on the main provincial and county routes in the disaster area. The construction completion dates of those bridges range from 1960 through 1999, and hence are involved in the evolutionary history of seismic design codes. As shown in Table 1, most of the damaged highway bridges are located in Taichung and Nantou because of being close to the epicenter. There were 52 and 13 bridges that experienced minor-to-moderate and major damage, respectively, in Taichung, as well as 82 and 13 bridges, respectively, in Nantou. In fact, these two counties are the disaster zones with the most casualties during the Chi-Chi earthquake. In addition, there were 17 highway bridges in Changhua and 18 in Yunlin that suffered moderate damage, while no severely damaged bridge was found. It should be noted that the damage degree was evaluated by somewhat subjective observation in the preliminary damage survey. Most of those damaged highway bridges have simply-supported, reinforced or prestressed concrete slab-and- girder superstructures, although continuous steel plate-girders and long-span cable-stayed girders are also included.

Table 2 lists all the major damaged highway bridges in the inventory, the associated routes, and their primary damage modes. Approximate locations of these bridges and their proximity to the Chelungpu fault are shown in Fig. 1,

Table 1 An aggregation of highway bridges by located counties and damaged extent

County \ Number	Total inspected bridges	Minor-to-moderate damaged	Major damaged
Taichung	196	52 (26.5%)*	13 (6.6%)
Changhua	199	17 (8.5%)	0
Nantou	410	82 (20.0%)	13 (3.2%)
Yunlin	176	18 (10.2%)	0

'*': the percentage is based on the total inspected bridges in each county.

Table 2 Major damaged highway bridge

Name	Route	Year	Span (m)	Length (m)	Damaged mode
Shi-wei	Provincial 3	1994	25	75	collapse
Chang-geng	local	1987	25	300	collapse
Dong-feng	Provincial 3	1962/1988	26	572	girder/column
Pi-feng	local	1991	25	300	collapse
E-jian	County 129	1972	11	264	collapse
Wu-shi	Provincial 3	1981/1983	34.7	624	collapse/column
Mao-loh-shi	Provincial 3	1999	40 ~ 70	500	column
Ming-tsu	Provincial 3	1990	25	700	collapse
Ji-lu	local	1999	150	300	pylon/bearing
Tong-tou	County 149	1980	40	160	collapse
Guang-long	local	1986	28	56	deck/abutment
Guan-de	local	1977	20	60	collapse
Bei-keng	County 129	1959	5.7	5.7	deck/abutment
Long-an	County 129	1986	35	280	column
Cheng-feng	County 136	1986	25.6	184	column/abutment
Yan-feng	Provincial 14	1984	35	455	column
Pu-ji	Provincial 16	1979	35	105	pier cap
Hsing-shi-nan	County 127	1994	50	500	column/bearing
Yan-ping	Provincial 3	1986	13	78	abutment
Hsin-yi	Provincial 21	1981	29	180	column
Long-men	Tou 53	1982	40	480	collapse
Li-yu	Tou 53	1988	39	546	bearing
Ping-lin	Tou 6	1969	25	500	collapse/column

Mo-keng No.1	Provincial 16	1996	14.6	14.6	abutment
Mo-keng No.2	Provincial 16	1996	40	40	abutment
Da-feng	Chung 105	1992			deck

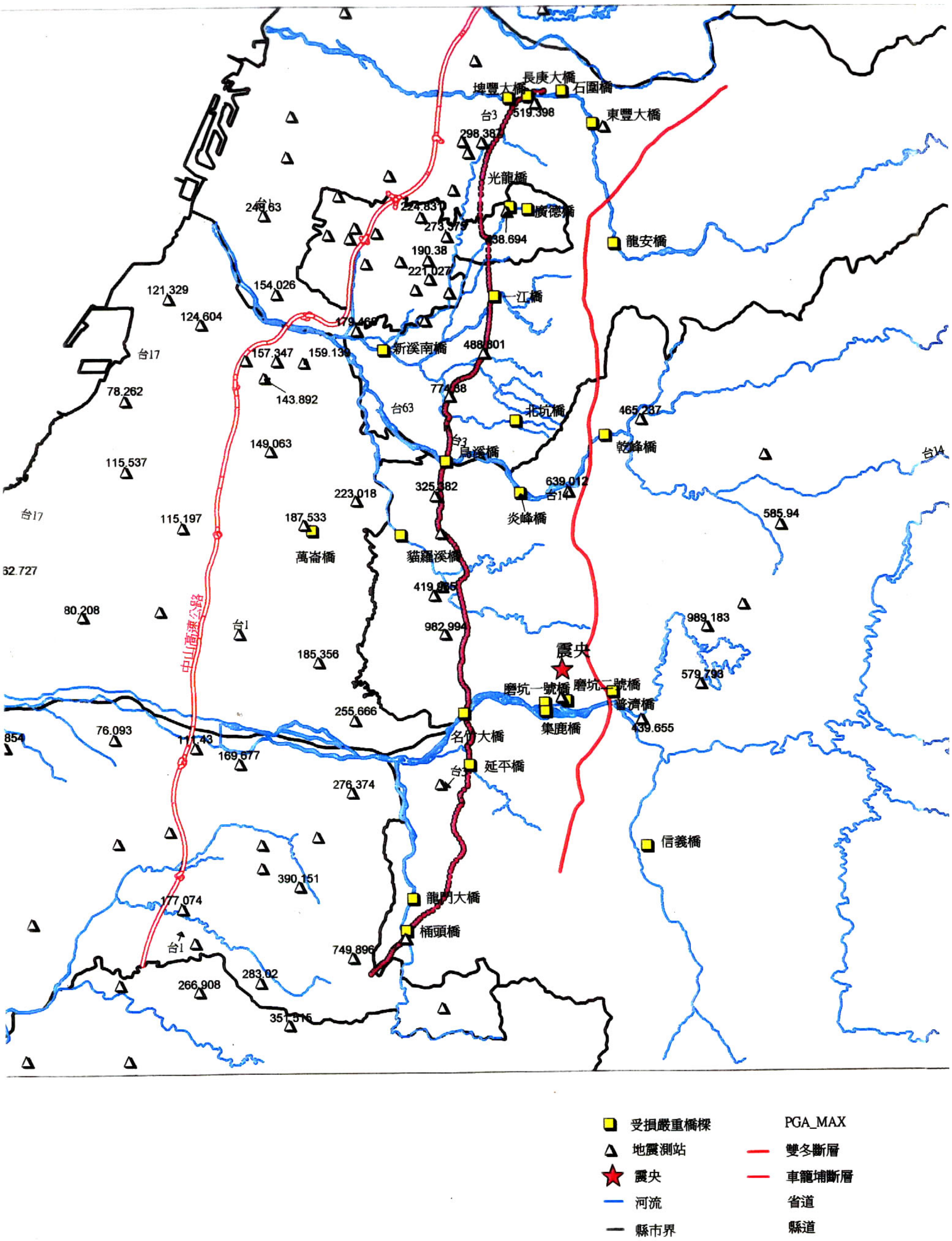

Fig. 1 Location of the major damaged bridges and the Chelungpu fault [5]

where some peak ground accelerations sensing stations by the Central Weather Bureau, the main national freeway (Chung-shan Freeway), and some provincial highways are shown as well. As observed from Table 2 and Fig. 1, most of the major damaged highway bridges are located on or very close to the Chelungpu fault, or are in the vicinity of the epicenter at Chichi. It was noticed that for those bridge sites where the Chelungpu fault passes through, the highway bridges experienced collapsed spans in most cases, such as the Chang-geng bridge, Pi-feng bridge, E-jian bridge, Wu-shi bridge, Ming-tsu bridge, and Tong-tou bridge. Most of these collapsed bridges were installed with unseating prevention devices. Moreover, it is seen that the PGAs measured in the east of or close to the Chelungpu fault are in the range of 400 gal to 980 gal, and are generally larger than those in the west of it. Thus, most of the damaged highway bridges are close to or in the east of the Chelungpu fault.

It should be noted that the main north-south lengthwise freeway running from Taipei in the north to Kaohsiung in the south is far away from the Chelungpu fault, so that it experienced only little damage. Bridges of the north-south lengthwise railway running from Taipei to Pingtung also sustained only minor injury compared to those highway bridges. Only the Ta-Chia-shi bridge suffered some damage to its bridge columns (Fig. 2(a)) [7]. Those damaged bridge columns have been repaired by steel jacket (Fig. 2(b)). However, the north-south lengthwise railway was closed for two weeks because of the damage to the San-i tunnel and some buckled rails. Moreover, the east-west Chi-Chi Branch of the railway, which only runs for tourism nowadays, suffered devastating (PGA) measured from ground motion fault rupture and land dislocation. The distorted rail and a vertical drop of about 3.0 meters are shown in Fig. 2(c) and Fig. 2(d). This branch of the railway still remains closed today.

Fig. 2(a) Damage to the column of the Ta-chia-shi bridge

Fig. 2(b) The repaired column of the Ta-chia-shi bridge

Fig. 2(c) Distorted railway of the Chi-Chi Branch

Fig. 2(d) A vertical drop at the Chi-Chi Branch

Major Damage Modes

Most of the highway bridges in Taichung, Nantou, Changhua, and Yunlin counties escaped from severe damage and experienced only minor distress such as the settlement of approach fills behind abutment back-walls. Approximately 20% of the bridge invento- ry suffered minor-to-major damage. Damage to these bridges include collapse of superstructures, displaced bearings, unseated girders from bearing supports, shear failures in columns, pier walls, and caissons, abutment back-wall failure, settlement of approach slab, foundation failures due to slope instabilities, joint failures in column-to-girder connections, cable fracture, fault rupture, and liquefaction. However, only Lyu-mei and Wan-lun bridges located in Yuen-lin town were observed to have liquefied appearance since it was not easy to distinguish when it occurred in a flowing river.

Figure 3(a) shows a summary of com- parison among 11 different damage modes. It should be noted that an injured bridge often has more than one damaged mode, so the total amount of all terms exceeds 195, which is the number of all injured bridges in the inventory. It

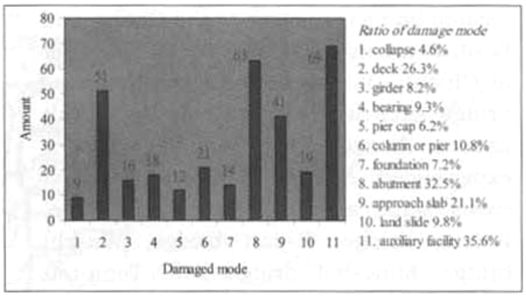

Fig. 3(a) A summary of different damaged modes

is seen that in these 195 bridges, the ratios of damage to auxiliary facility, abutment, deck, and approach slab are 35.6%, 32.5%, 26.3%, and 21.1%, respectively, significantly larger than those of other damage modes. From the field observation, it was found that pounding on abutments due to excessive longitudi- nal vibration of the superstructure usually occurred with damage to the approach slabs and/or bridge decks. The deck damage defined in this paper includes the expansion-joint failure. In addition, the ratio of column or pier damage (10.8%) is greater than those of bearing (9.3%), girder (8.2%), and foundation (7.2%). From the reconnaissance results [5], it was observed that half of those bridges with injured bearings were also with damage to the corresponding columns or piers. Some damaged bearings were due to crack of their pedestals, and some were due to dislodg- ment by the supported girder.

Damaged Bridges Related to Construction Era

Table 3 shows the classification of damaged bridges based on their complet- ed years, extent of damage, and structural types. The percentage in the parenthesis is obtained by dividing the value by the total number of the damaged bridges with same structural type. It is seen that over half of those damaged bridges were constructed before 1989, generally before the issue of "Design Code for Highway Bridges" in 1987. Moreover, over 95% of those injured bridges before 1989 were simply supported. The decreasing percentage of injured, simply-supported bridges after 1989 may be attributed to the decreasing of simply-supported bridges and better seismic design methods as well as construction technologies.

Table 4 is the result of an aggregation of some damaged modes by the completed year of bridges. It is observed that the amount of damaged abutment and deck are usually in the first two ranks for any construction period. Figure 3(b) exhibits the relative percentage of damaged modes for a specific construction period. It is seen that for any construction period considered, the abutment and deck are more liable to damage than other components under the earthquake. The susceptibility of column or pier damage decreases in the latest three construction periods, while it is not the case for the bearing systems. A decreasing trend of

Table 3 An aggregation of damaged bridges by their completed year, damaged extent, and structural types

Classification of damaged bridges by completed year and damaged extent						
	1975 earlier	1975 ~ 1982	1983 ~ 1989	1990 ~ 1996	1996 later	total
minor-to-moderate	33	44	48	19	25	169
major	3	6	7	7	2	25
total	36	50	55	26	27	194
Classification of damaged bridges by completed year and structural type						
single span	19 (17%)	31 (28%)	33 (29%)	14 (12%)	16 (14%)	113

simply supported multi-span	15 (23%)	17 (27%)	20 (31%)	8 (13%)	4 (6%)	64
continuous multi-span	2 (22%)	1 (11%)	0 (0%)	3 (33%)	4 (44%)	9
other	0	1	2	1	3	6

Table 4 An aggregation of damaged bridges by their completed year and damaged modes

Classification of damaged bridges by completed year and damaged mode						
	1975 earlier	1975 ~ 1982	1983 ~ 1989	1990 ~ 1996	1996 later	total
column or pier	2	5	9	3	2	21
girder	2	7	3	2	2	16
abutment	8	10	26	9	10	63
bearing	2	4	4	5	3	18
deck	16	12	9	7	7	51
foundation	2	7	2	3	0	14

the damage ratio of column or pier in the latest three construction periods may be attributed to the improvement on seismic design approaches.

Figure 3(c) shows the variation of the six damaged modes with the construction periods. It is observed that most of the six damaged modes occur in terms of higher damage percentage in a certain construction period than in others, except for the bearing-damaged mode. Generally speaking, over 70% of the damaged bridges for each mode other than the bearing-damaged mode were completed before 1989. Those bridges with damaged bearing systems disperse in each construction period with a nearly uniform way.

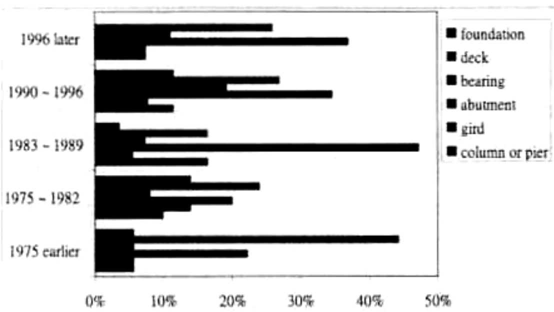

Fig. 3(b) Relative percentage of damaged modes for a specific construction period

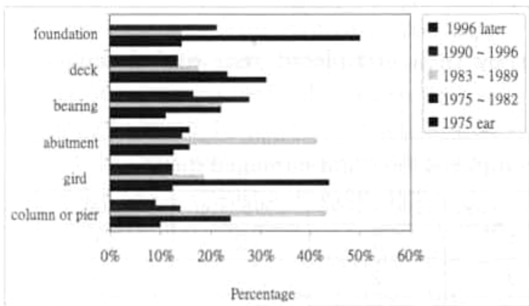

Fig. 3(c) Variation of the six damaged modes with the constructed periods

DETAILED DESCRIPTIONS OF SEVERELY DAMAGED HIGHWAY BRIDGES

Since the Chi-Chi earthquake was mainly induced by rupture of the Chelungpu fault passing through Taichung and Nantou counties, those bridges located on the highways near the fault experienced more severe damage. Provincial Route 3 running from Taipei in the north to Pingtung in the south is one of the main provincial routes. Since the section of Provincial Route 3 in Taichung and Nantou counties is close to the Chelungpu fault, some highway bridges on that route were damaged seriously. As for those bridges located across the Chelungpu fault, it is probable to result in collapse under the action of both fault rupture and strong earthquake ground motion. Insufficient unseating prevention device, on the other hand, may be responsible for collapsed spans when the bridge is not located on or very close to the ruptured fault.

Some typical major damaged bridges as mentioned above, as well as an almost completed, cable-stayed bridge as well as a viaduct newly completed in 1999, are described as follows.

Shi-wei Bridge

The Shi-wei bridge is located at the milepost of 163km + 278m on Provincial Route 3 is the main line for connecting Dong-shih and Drao-lan. It was reconstructed in 1994 as two individual, 3- span curved bridges on single column bents supported by caissons. It has an identical span length of 25m and each span comprises five simply-supported prestressed concrete girders and deck slab. Those beam girders were supported on elastomeric pads with shear keys for restricting transverse movement. The Chelungpu fault crossed the bridge in the vicinity of the southern abutments. Closing to the southern abutments, two consecutive spans of the southbound bridge and one of the other collapsed, as shown in Fig. 4(a). The retaining wall near the southern abutment collapsed, as shown in Fig. 4(b). The first pier including caisson in both bridges tilted away from the southern abutment northeastward, and the abutment tilted north- wards (Fig. 5). Figure 6(a) shows that some shear cracks occurred on the first column bent extending from the northern abutment of the northbound bridge. Moreover, almost all the concrete stoppers on the pier cap of the second pier were crushed (Fig. 6(b)) and transverse movement of the middle span, which escaped from collapse, was observed.

Since there is no significant flexural or shear crack observed on the tilted column bents, it seems that fault rupture near the southern abutment may be responsible to those tilted piers. It thus appears that those two collapsed spans close to the abutment were induced by the tiltingcolumn bents. In addition, the measured horizontal PGA is as high as 500 gal in the transverse direction and 350 gal in the other. Supported by one tilted column bent, collapse of the middle span of the southbound bridge may be due to excessive in-plane displacement under the strong ground motion. However, the middle span of the northbound bridge may have been experienced less displacement and thus escaped from collapse.

Fig. 4(a) Three spans collapsed (Shi-wei bridge)

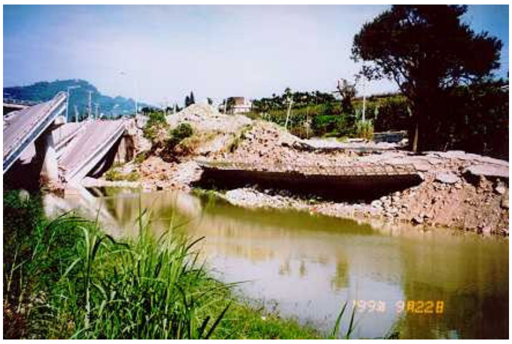

Fig. 4(b) The southern retaining wall collapsed (Shi-wei bridge)

Fig. 5 The southern abutment tilted northwards

Fig. 6(a) Shear cracks at the first column bent of the northbound bridge

Fig. 6(b) Crushed concrete stoppers on the pier caps

Dong-feng Bridge

The 22-span Tong-feng bridge is located at the milepost of 168km + 337m on Provincial Route 3, and is actually three parallel bridges with a length of 572m for each. The central bridge is the original one constructed in 1962, while the other two were widened in 1982. Each span of the original bridge and of the other two is consisted of 5 and 4 prestressed concrete girders, respectively. All spans have an identical length of 26m and are supported by bearings at each pier cap. The substructures of the original and widened bridges comprise wall-type piers and single column bents, respectively, on spread footings. Under the strike of the Chi-Chi earthquake, strong transverse shaking moved the superstructures southward and some bearings were dislodged (Fig. 7). Even more, as shown in Fig. 7, the prestressed concrete girder was fractured near the beam end. Figure 8 shows settlement of the bridge deck due to loss of support by bearings. Significant flexural/shear cracking and spalling occurred in both the sixth column bents extending from the eastern abutments of those widened bridges (Fig. 9(a) and 9(b)).

Fig. 7 Southward movement of the superstructure and the fractured girder

Fig. 8 Settlement of the bridge deck (Dong-feng bridge)

Fig. 9(a) Flexural/shear cracks at the 6th column bent of the westbound bridge

Fig. 9(b) Flexural/shear cracks at the 6th column bent of the eastbound bridge

The Chelungpu fault did not pass through the Tong-feng bridge, but evoked strong ground motion in the vicinity of the bridge with a horizontal PGA of 500 gal in the longitudinal direction and of 350 gal in the other. Such large shaking made the bridge girders move southwards and even fall down from their bearing supports. As a consequence, sinking of the bridge deck resulted from the southward movement of girders. Furthermore, since the column bents of the widened bridges are not as wide as the wall-type piers of the original one, they reserve less shear and flexural capacities than the old one does. It is thus plausible that the column bents experienced much more severe damage than the wall pier.

Pi-feng Bridge

The 13-span Pi-feng bridge with a span length of 30m is located downstream of the Shih-kang dam on Ta-chia river and completed in 1991. It is consisted of 4 simply-supported, pre- stressed concrete girders for each span as the superstructure and of single column bents on caissons as the substructure. During the earthquake, 3 spans ex- tending from the southern abutment collapsed, and the second column bent along with its caisson was uprooted and lay down on the riverbed, as shown in Fig. 10 and Fig. 11. The collapsed second and third spans rotated clockwise as they fell down on the riverbed. Except those collapsed spans, there is no evident damage to the rest components of the bridge.

Since there is no significant flexural or shear crack observed in the lain down pier, it is believable that ground rupture through the pier toe led to its failure. A permanent drop of the riverbed, as shown in Fig. 12, verifies the pass of the Chelungpu fault through the bridge.

Fig. 10 Three spans of the Pi-feng

bridge collapsed

Fig. 11 The third collapsed span of the Pi-feng bridge

Fig. 12 A permanent drop of the riverbed induced by fault rupturing

E-jian Bridge

Located at the milepost of 25km + 195m on County Route 129 and constructed in 1972, the 24-span E-jian bridge with an identical span length of 11m is an important passageway for the Tai-ping city in Taichung county. Its superstructure is composed of a pair of simply-supported, reinforced concrete, double-T girders and deck slab. The substructure comprises unreinforced concrete walls on spread footings. No unseating prevention device was provided between adjacent spans. The bridge was being widened as the earthquake occurred and new piers with a different span length from the original ones had been completed. Distinct damage to this bridge is the "seesaw-like" collapse of nine sequential spans extending from the northern abutment (Fig. 13). The Chelungpu fault crossed the northern abutment with uplift and pressed the soil layers to move along the longitudinal direction towards the other abutment. From the northern abutment to the central pier of the bridge, some piers near the abutment were crushed or snapped, while the remaining piers moved towards the other abutment as rigid bodies or rotated with their spread footings due to ground movement (Fig. 14). Relative movements between adjacent spans were observed for some of those spans remaining supported by their piers, as shown in Fig. 15.

A field examination indicated little damage to those collapsed girders and to the southern half of the bridge. It appears that those collapses were caused by large ground movements of their foundations towards the southern abutment rather than caused by excessive structural vibration.

Fig. 13 The "seesaw-like" collapse of the E-jian bridge

Fig. 14 Movement or rotation of the pier walls (E-jian bridge)

Fig. 15 Relative movement of adjacent spans (E-jian bridge)

Wu-shi Bridge

The Wu-shi bridge, located at the milepost of 210km + 371m on Provincial Route 3, is the main bridge connecting Wu-feng town in Taichung county and Tsou-tun town in Nantou county. It is actually two parallel 18-span bridges with a uniform span length of 34.7m. The northbound bridge was completed in 1981 and the southbound one in 1983. Both bridges have similar superstructures as 5 simply-supported, prestressed reinforced concrete girders and similar substructures as wall-type piers on caissons. However, the wall piers of the southbound bridge were not as wide as those of the northbound one. Hence, different damaged characteristics were exhibited although fault rupturing occurred under the third span of both bridges, as shown in Fig. 16, and similar ground motions was experienced.

Extending from the northern abutment of the southbound bridge, severe shear failure occurred at the first five pier walls and/or caissons under the strong ground motion and fault rupture, as seen in Fig. 17. Perhaps because of the higher shear capacity of its pier walls, the northbound bridge did not experience such massive shear failure, but flexural cracks with fractured reinforcements were observed in the third pier wall from the northern abutment (Fig. 18). Moreo- ver, the first two spans from the northern abutment of the northbound bridge were unseated (Fig. 19) due to the ground movement. The bearings at these pier caps were heavily damaged due to strong transverse shaking, as shown in Fig. 20. The third span especially exhibited a westward permanent offset. In addition, the super-structure of the southbound bridge showed westward settlement (Fig. 21) due to sliding down of those injured piers and caissons along their diagonal shear cracks.

Fig. 16 Evidence of fault crossing the Wu-shi bridge

Fig. 17 Shear failure of the pier walls and/or caissons

Fig. 18 Flexural crack at the third pier wall of the northbound bridge

Fig. 19 The first two spans of the northbound bridge collapsed

Fig. 20 Crushed bearings and diaphragms at the pier cap

Fig. 21 Westward settlement of the superstructure

Mao-loh-shi Bridge

Just finished in 1999, the Mao-loh-shi bridge is a multi-span viaduct connecting the Provincial Route 3 to the Provincial Route 63. It is consisted of a series of continuous segments, each of which was constructed from 4 I-type steel girders and steel cap girders as its super-structure supported on reinforced concrete, single column bents. The steel cap girders were rigidly anchored into the corresponding single column bents. Although the viaduct is not close to either the Chelungpu fault or the epicenter as compared to other major damaged bridges, some joint injuries were observed in the column-to-girder connections.

In an 8-span, curved segment, 4 single column bents are eccentrically connected to their steel cap girders, as shown in Fig. 22, in order to keep traffic flow fluent for the highway below it. During the earthquake, diagonal shear cracks occurred at the upper portion of those columns with eccentric connections (Fig. 23). Even more, the confinement hoops are exposed for some cases (Fig. 24). The steel cap girders were temporarily shored to prevent sinking of the superstructures. Since there is no damage to those single column bents with symmetrical connections, it appears that the eccentric connection may be one of the reasons for the damage.

Fig. 22 The eccentric connections of the Mao-loh-shi bridge

Fig. 23 Diagonal shear cracks at the column with eccentric connection

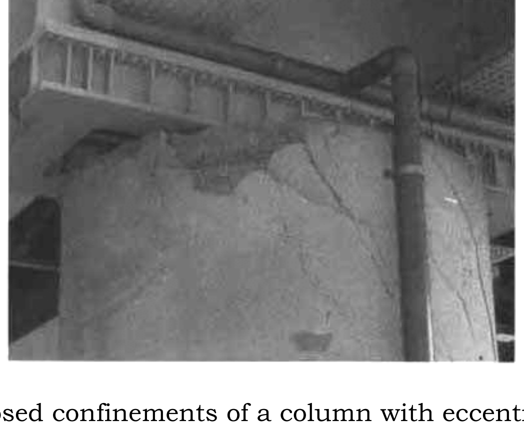

Fig. 24 Exposed confinements of a column with eccentric connection

Ming-tsu Bridge

The 28-span, simply-supported Ming-tsu bridge is located at the milepost of 233km+564m on Provincial Route 3 and was constructed in 1990. It is an important highway bridge connecting Ming-jian and Zhu-shan towns in Nantou county. Each span has 4 prestressed, reinforced concrete girders for both the southbound and northbound lines, and those girders were made continuous by cast-in-place reinforced concrete deck slab over intermediate bents. Each line has its own single column bents supported by caissons. During the earthquake, fault rupturing occurred near the southern abutment and nine spans collapsed as shown in Fig. 25. The first six piers extending from the southern abutment were tilted or fractured (Fig. 26). Moreo- ver, the back-wall of the southern abutment was cracked by the superstructure and thus was driven back into the back-fill (Fig. 27) under the strong longitudinal ground shaking.

From the field observation, ground rupture may be responsible for those tilted piers and collapse of the northern segments. However, the collapse of the spans in the southern segment appears to be induced by the enormous impact of the superstructure on the back-wall.

Fig. 25 Collapsed spans of the Ming-tsu bridge

Fig. 26 Tilted or fractured column bents of the Ming-tsu bridge

Fig. 27 Cracked back-wall of the southern abutment

Ji-lu Bridge

The Ji-lu bridge is a cable-stayed bridge with two 120m spans and a single tower (Fig. 28). The tower, constructed with reinforced concrete, is 58m high and sustains the prestressed concrete superstructure by 17 pairs of parallel steel cables from each side of it. Each end of the cable-stayed bridge is connected to a simply-supported approach span. At the time of the earthquake, this bridge was almost completed except for the guard rails and a cantilevered section of the superstructure at the base of the tower. The tower, superstructure, and some cables experienced significant damage. As shown in Fig. 29, concrete at the base of the tower was spalled off and some horizontal cracks occurred. At the same face of the tower and extending from where concrete was spalled off, vertical splits along the tower height were observed (Fig. 30). Nevertheless, damage to the opposite face of the tower was not as severe. Furthermore, the connection between the tower and the superstructure at its base was smashed (Fig. 31). Similar damage was observed at the end of the superstructure on the pier of the approach span, as shown in Fig. 32. The approach span was unseated from its elastomeric pads and the associated pedestals under the strong transverse ground motion (Fig. 33), and relative movement between certain adjacent spans was observed (Fig. 34). In addition, one of those 68 cables was torn off from its restrainer, as shown in Fig. 35.

This bridge is very close to the epicenter, and the horizontal PGA measured nearby is about 600 gal and 400 gal in the transverse and longitudinal directions, respectively. Such enormous ground shaking may have been responsible for the relative movement of certain adjacent spans and for unseating of the approach span. Moreover, since there is an uninstalled cantilevered section at the base of the tower, the transverse vibration of the cable-stayed bridge may have been unsymmetrical about the longitudinal centerline of it. Thus, damage to both the parallel faces of the tower to the longitudinal direction is unsymmetrical and the connection at the base of the tower to the superstructure was crushed due to stress discontinuity.

Fig. 28 The cable-stayed Ji-lu bridge

Fig. 29 Concrete spalled off and horizontal cracks at the tower base

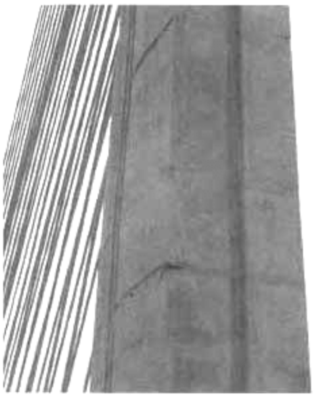

Fig. 30 Vertical split along the tower height

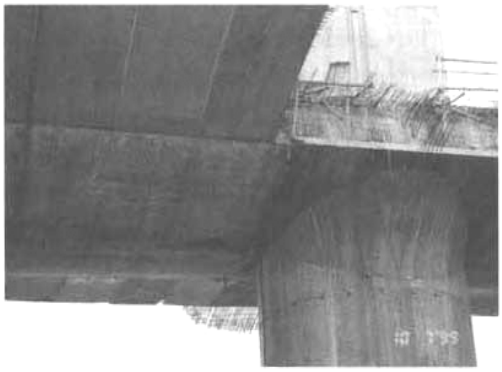

Fig. 31 Crush at the connection between the tower and the superstructure

Fig. 32 Crush at the connection between the cable-stayed and approach spans

Fig. 33 Unseated approach span from its bearing

Fig. 34 Relative transverse movement between adjacent spans

Fig. 35 The torn-off cable from its anchorage

Tong-tou Bridge

The 4 span Tong-tou bridge is located at the milepost of 13km+633m on County Route 149 and connects Zhu-shan and Tsou-ling towns in Nantou county. It was constructed with 3 simply-supported, prestressed reinforced concrete girders as the superstructure and with single circular columns on caissons as the substructure. Each span is 40 meters long. Most of the structural components were destroyed under both the strong ground shaking and fault rupturing (Fig. 36). According to the measured PGAs nearby, the PGA is approximately 750 gal in the north-south direction, which is generally the longitudinal direction, and 360 gal in the other direction. During the earthquake, all three single circular columns were cut off by shear failure (Fig. 37) and the spans collapsed. It was noticed that those large-diameter caissons extended well above the riverbed and thus reduced the relative height of each circular column to the overall height of both the column and its caisson. Both abutments were damaged, with the northern one impacted into its back-fill and crushed severely, as shown in Fig. 38. Furthermore, significant settlement of the northern approach slab occurred (Fig. 39).

Fig. 36 A view on the collapsed Tong-tou bridge

Fig. 37 Shear failure of the single circular column

Fig. 38 The crushed northern abutment of the Tong-tou bridge

Fig. 39 Settlement of the approach slab of the northern abutment

Since the columns and spans collapsed in an unsymmetrical scenario, it appears that they did not fail at the same instant. Extending from the northern abutment, the first column was snapped to the east, while the other two were snapped to the west. The north end of the second span rotated clockwise about its south end, while the whole third span was tilted to the west and partially lay on the caps of both fractured columns. It seems that the northernmost column was destroyed one moment before the others. Moreover, because those two spans close to the abutments still stayed at the longitudinal centerline after their collapse, it seems that they failed a moment before the columns did.

Long-men Bridge

The 12-span, simply-supported Long-men bridge was completed in 1982 and is located on Tou-53 Route. It comprises 2 prestressed, reinforced concrete girders with a span length of 40 meters as the superstructure and single column bents on caissons as the substructure. Under the earthquake excitation, the second and third spans extending from the western abutment collapsed (Fig. 40). Since there is no significant damage to other structural components, it appears that no unseated prevention device (Fig. 41) may have been the reason for its collapse.

Fig. 40 Collapse of two spans of the Long-men bridge

Fig. 41 No unseat prevention device between adjacent spans

DAMAGE TO BRIDGES UNDER CONSTRUCTION

In order to facilitate the east-to-west transportation of Taiwan, several east-to-west express highways or freeways are under construction at present. Some of the bridges on these new express highways or freeways experienced minor-to- moderate damage. Common damage to those bridges is the fracture or tumbling of the uninstalled, cast-in-place, reinforced concrete girders, as shown in Fig. 42(a) and Fig. 42(b). The Feng-yuan viaduct, which is located at the Taichung Branch of the Second Freeway, suffered moderate damage to the elastomeric bearing systems. Many bearing pedestals of steel pot bearings were crushed during the earthquake (Fig.

43) and the box girders supported by them exhibited a permanent transverse dislocation (Fig. 44).

Fig. 42(a) Fracture of uninstalled, cast-in-place, concrete girders

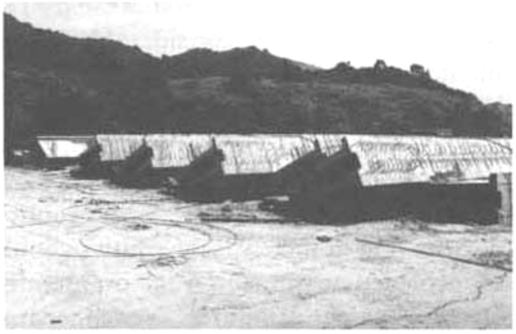

Fig. 42(b) Tumbling of uninstalled, cast-in-place, concrete girders

Fig. 43 Damage to the pot bearing and its pedestal

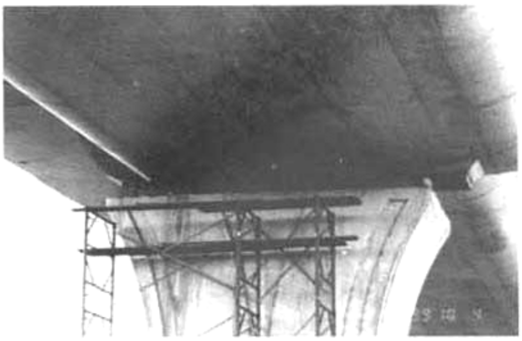

Fig. 44 Transverse movement of the superstructure

LESSONS LEARNED AND SUGGESTIONS

From the bridge-disaster reconnaissance and field observations, several impressive lessons were learned. First of all, fault rupture under or across bridge foundations is catastrophic to the structure and usually leads to collapse of spans, especially when the ground dislocations are large. The Pi-feng bridge is a good example of such a case. However, since faults with different magnitudes are widespread in Taiwan, there may sometimes be no alternative but to construct bridges across or near active faults because of finite serviceable area. Under such a situation, "easy to repair or reconstruct" should be a priority in the bridge design and construction, and alternative routes should be outlined in advance. Shorter spans may be installed in bridges to facilitate urgent restoration.

Moreover, the unusual characteristics of near-fault earthquakes may lead to unexpected damage to structures, particularly to those older bridges. Even long-span bridges are vulnerable to near-fault earthquakes. Seismic input energy of near-fault earthquakes is usually released in a very short instant, *e.g.,* 1~2 seconds, and thus considerable incremental velocity is brought forth. It may be necessary to conduct site specific hazard analyses to have thorough comprehension on the near-fault effect.

Furthermore, unseating prevention devices and sufficient seat widths should be provided to protect from unintended strikes. Ground failure, skewed spans, and liquefaction, *etc.*, may induce addition structural movements, such as the Shi-wei bridge. Well-designed shear keys, restrainers, and concrete stoppers are excellent equipment for unseating prevention. Also, as noticed in the field observation, the span next to either abutment collapsed in all those bridges with collapsed spans. Thus, adequate design and construction of abutment back-walls and fill-backs are essential even for continuous spans.

In addition, column-to-girder joints should be accurately designed and constructed, especially for eccentric connections, such that the seismic force can be transferred by the expected load path. Also, sufficient and well-constructed hoop bars or confinements should be provided to prevent from shear failure in piers.

SUMMARY AND DISCUSSION

The 921 Chi-Chi earthquake brought an intense impact on the seismic resistant design and technical development of bridge engineering. From the bridge- disaster reconnaissance, some valuable lessons were learned to review and improve the current seismic design code. Also, as indicated by those observed appearances, the realistic structural behavior of bridges under real earthquake ground motions is more complex than analytical prediction. Some design details, such as the near-fault effect, confinement, and unseating prevention device, *etc.*, which may be roughly estimated or overlooked, should be comprehended thoroughly. Further investigations on those major damaged bridges and the confirmation of fault locations may be needed.

It is necessary to proceed with both the improvement of seismic design code and retrofit of old bridges. When agreement on the opinions on major damaged modes and modified bridge design methodologies is reached, the seismic design code of bridges can be revised for future construction. In addition, many verified seismic retrofit techniques [8,9] should be applied to repair or strengthen those minor damaged or old bridges.

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