3. SEISMIC DAMAGE IN THE PAST EARTHQUAKES

3.1 Loma Prieta and Northridge, USA, Earthquakes

3.2 Pre-Kobe and Kobe, Japan, Earthquakes

1) Pre -Kobe earthquakes

Importance of taking account of the seismic effects in design of engineering structures was first recognized when the destructive damage occurred in 1923 Kanto Earthquake. At those days, bridges were constructed based on the technologies imported from the USA, UK, German and French with no consideration for the effects of seismic disturbances. Photo 3.1 shows a typical damage in the earthquake. The foundations settled, laterally moved and tilted. This resulted in the collapse of an entire bridge system. This was a typical damage when seismic effects were not considered, or when the seismic design, in particular for foundations, was insufficient. This type damage occurred in 1920s-1950s.

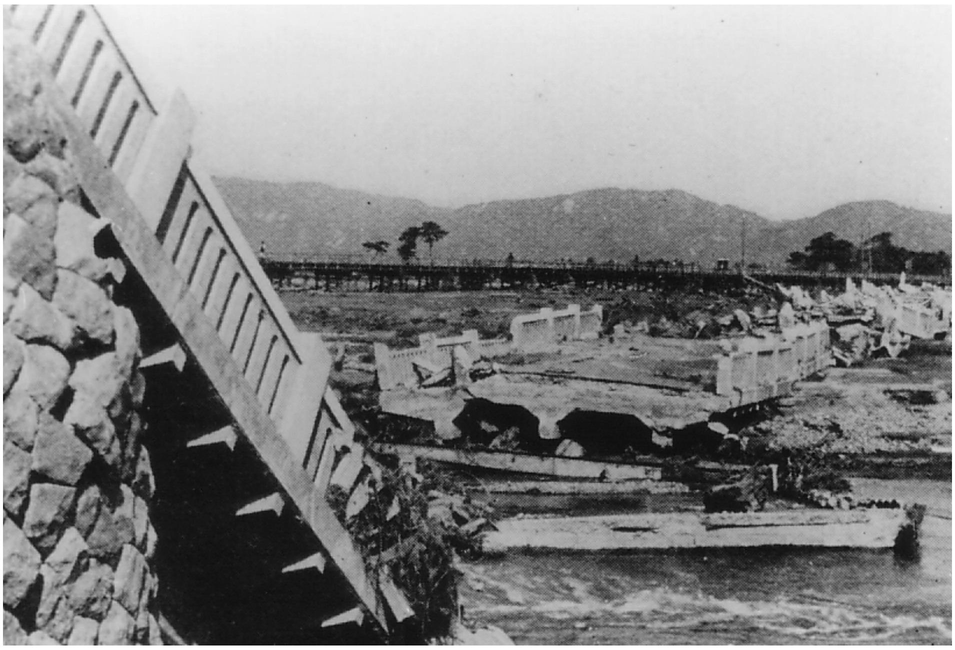


Photo 3.1 Collapse of Sakawa-gawa Bridge, 1923 Kanto Earthquake

Seismic countermeasures were initiated after the 1923 Kanto Earthquake. The equivalent static lateral force method using a seismic coefficient of 0.1-0.3 based on the allowable stress design approach, which is often called Seismic Coefficient Method, was first incorporated in design of highway bridges in 1927 (MI 1927). Since that time, the seismic design practice has been improved and extended based on the seismic damage and the progress of researches. However it was still preliminary stage for assuring the seismic performance. For example, in the 1964 Design Specifications (JRA 1964), seismic related requirement was only the seismic coefficients. It should be 0.2 in horizontal and 0.1 in vertical directions. No other important requirements such as realistic near-field ground motions, ductility, dynamic response, liquefaction and unseating prevention devices were not included. As will be shown later, most bridges which suffered damage in the 1995 Kobe Earthquake were designed and constructed in accordance with the 1964 Design Specifications.

Extensive damage of bridges occurred in the 1964 Niigata Earthquake resulted from soil liquefaction. Showa Bridge supported by flexible bent-piles collapse as shown in Photo 3.2. Soil liquefaction occurred extensively around the bridge. This caused lateral spreading of surface ground as long as 10m along the Shinano River.

Photo 3.2 Collapse of Showa Bridge, 1923 Niigata Earthquake

Although the phenomena that water with sand brew up from underground during an earthquake was known from old days, it was the Niigata Earthquake when the phenomena was first named "liquefaction," and scientific researches started worldwide. In several technical documents that described the damage of Niigata Earthquake, there was a description that soil moved laterally during the earthquake. However, this movement was not clearly distinguished with the slippage of soft clayey soils. Therefore, attention was limited only to soil liquefaction, not to the lateral spreading. It was 1990s when the lateral spreading was first recognized. Soil liquefaction and lateral spreading were re-evaluated since 1980s.

Countermeasures for soil liquefaction started after the Niigata Earthquake. An assessment of liquefaction potential based on N-value of standard penetration test depending on depth was first incorporated in the 1971 Guide specifications (JRA 1971). However, a design procedure for liquefaction was not included in the 1971 Guide Specifications, because the mechanism of liquefaction was not known. An improved assessment of soil liquefaction based on F_L-value (ratio of lateral force and soil strength), and a procedure that decreases the stiffness of soil springs which connect soils and foundations depending on F_L-value was first included in the 1980 Design Specifications (1980 JRA). To date various improvements have been developed and incorporated in the design codes.

In addition to the countermeasures to soil liquefaction, restrainers, or unseating prevention devices in a broader sense, were first developed and implemented in bridges after the Niigata Earthquake. Extensive damage of bridges that resulted from excessive relative displacements between superstructures and substructures created an inspiration for developing unseating prevention devices. They include steel plate connectors, bar connectors, cable restrainers and chains that tie two decks or a deck and a substructure. Providing sufficient seat length is an important unseating-prevention measure. Today unseating prevention devices have become an important component of bridges worldwide.

In 1970s, design procedures that were independently developed for each type foundations started to be unified. Since Japan is located in the monsoon area, soft and unstable soils sediment. Major cities are located on such thick sedimentation. Since bridges are constructed at those sites, failure of unstable soils as well as scouring always resulted in damage of foundations. Consequently, it has been a basic principle to construct rigid foundations with large concrete sections. This developed various new foundations, and they contributed to reduce damage of foundations.

The same design practice was extended to reinforced concrete piers. Shear was not critical in such reinforced concrete piers. Consequently, no matter how tie reinforcements in reinforced concrete piers were insufficient, damage was limited in the past earthquakes. Although reinforced concrete columns with smaller sections as well as steel columns started to be constructed from 1970s for urban highway viaducts, the design practice was practically unchanged. As will be described later, this resulted in the extensive damage in the 1995 Kobe Earthquake.

In 1980s and 1990s, destructive earthquakes occurred in and around Japan more than 10 times. However, since strength of foundations has been improved, the damage of foundations was limited. Damage occurred at reinforced concrete piers and steel bearings, but they were generally limited. Extensive researches on the ductility capacity of reinforced concrete started since mid 1980s. The capacity and demand seismic design of reinforced concrete columns depending on ductilities was first incorporated in 1990 Design Specifications (JRA 1990, Kawashima and Hasegawa 1994). Response accelerations with 2g as shown in Fig. 5.? was first incorporated. Researches on the ground motion characteristics including the attenuation of response spectra made it possible to develop the design response spectra. It was a turning point changing from the seismic design method to the ductility deign.

2) 1996 Kobe Earthquake

Kobe Earthquake (Hyogo-ken Nanbu Earthquake) occurred on January 17, 1995, exactly one year later the Northridge, USA, Earthquake. It resulted in destructive damage to bridges. Reinforced concrete columns suffered failure in shear. Premature shear failure at terminations of longitudinal bars with insufficient development lengths resulted in collapse of many bridges. Extensive failure of steel columns occurred first in the world. Extensive soil liquefaction occurred, and this resulted in settlements and tilting of foundations and substructures. Lateral spreading of ground associated with soil liquefaction caused movements of foundations. Unseating prevention device suffered damage. In the following, some major damages are presented:

a) Premature shear failure of reinforced concrete columns

Eighteen spans of Fukae Viaduct, Hanshin Expressway collapsed as shown in Photos 3.3 and 3.4. The viaduct was designed in accordance with the 1964 Design Specifications using 0.2 horizontal and 0.1 vertical seismic coefficients based on the allowable stress design approach. It was completed in 1969. Prestressed concrete decks with 22 m span were simply supported by internal hinges at both ends. The reinforced concrete columns were 9.9-12.4 m tall with a diameter of 3.1-3.3 m. Longitudinal and tie bars were deformed bars with a diameter of 35 mm (SD30, D35) and 16 mm (SD30, D16), respectively. Number of the longitudinal bars was 180 at the bottom of columns, and it reduced 120 at 2.5 m from the bottom by terminating 60 bars.

Photo 3.3 Collapse of Fukae Viaduct, 1995 Kobe Earthquake

Photo 3.4 Premature Shear Failure of Reinforced Concrete Column, Fukae Viaduct

The footings were supported by 10-15 m long cast-in-place reinforced concrete piles with diameter of 1m. The soil is of sand and gravels, and is classified Type II (moderate) soil site.

There were three important problems in the design of this viaduct. First is the overestimated allowable shear stress. The allowable shear stress that is required in the current codes is less than 60 % of the value used in design of this viaduct. Second is the insufficient development length of longitudinal bars terminated at mid-height. Only 20 times bar diameter was the development length. Based on the current requirements, it has to be further extended an effective width of the column, i.e., 3.1-3.3m. Third is the insufficient amount of tie bars. Tie reinforcement ratio by Eq. (4.?) was only ?.

The failure mechanism of the viaduct is as shown in Fig. 3.1. Subjecting to a strong ground motion, the columns suffered extensive flexural and diagonal cracks at 2.5m above the footing where 1/3 longitudinal bars were terminated with insufficient development length. Since the amount of tie bars was insufficient, this caused premature shear failure in the columns.

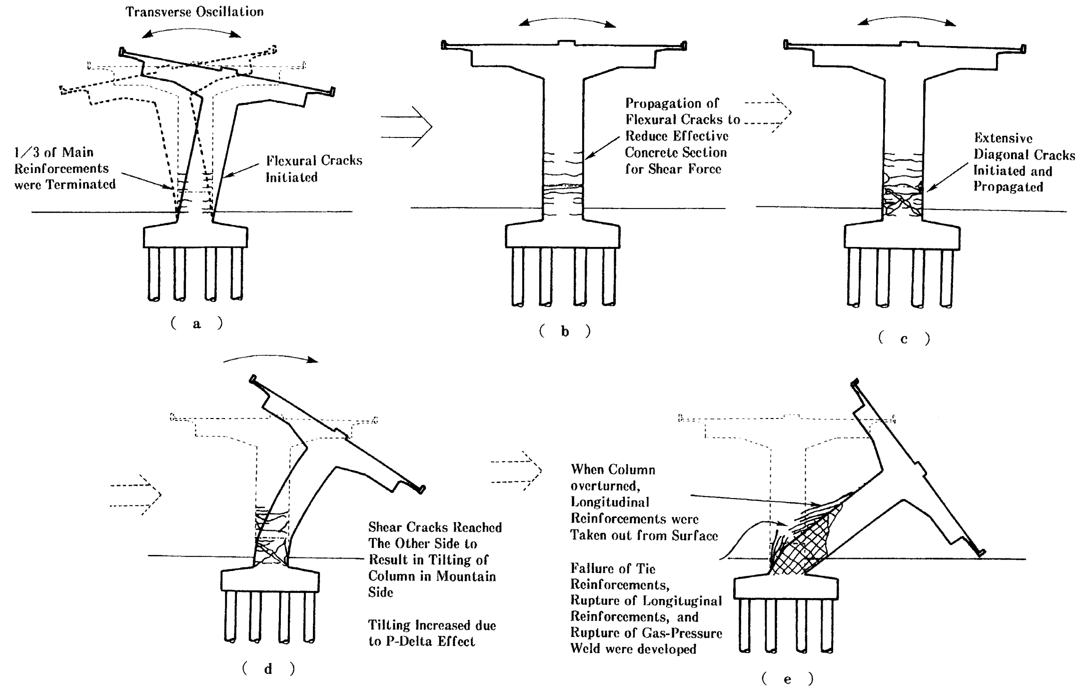


Fig. 3.1 Failure Mechanism of Fukae Viaduct

Similar failures occurred extensively. In fact, it was the major reason for causing the destructive damage of bridges in the Kobe Earthquake. For example, two simply supported steel girder bridges collapsed at Takashio, Hanshin Expressway as shown in Photo 3.5. It was completed in 1979 based on the 1971 Guide Specifications using 0.23 horizontal and ±0.11 vertical seismic coefficients. Tie bars with a diameter of 16mm was provided 300mm interval. Hence the tie reinforcement ratio by Eq. (4.?)was only ? %. The number of longitudinal bars was 150 at the bottom of columns, and it reduced 120 and 60 at 3.3 m and 5.7 m from the bottom, respectively. As was the case of Fukae Viaduct, the insufficient development length of terminated longitudinal bars and the insufficient amount of tie bars resulted in the premature shear failure of columns. Fig. 3.2 shows the estimated failure mechanism.

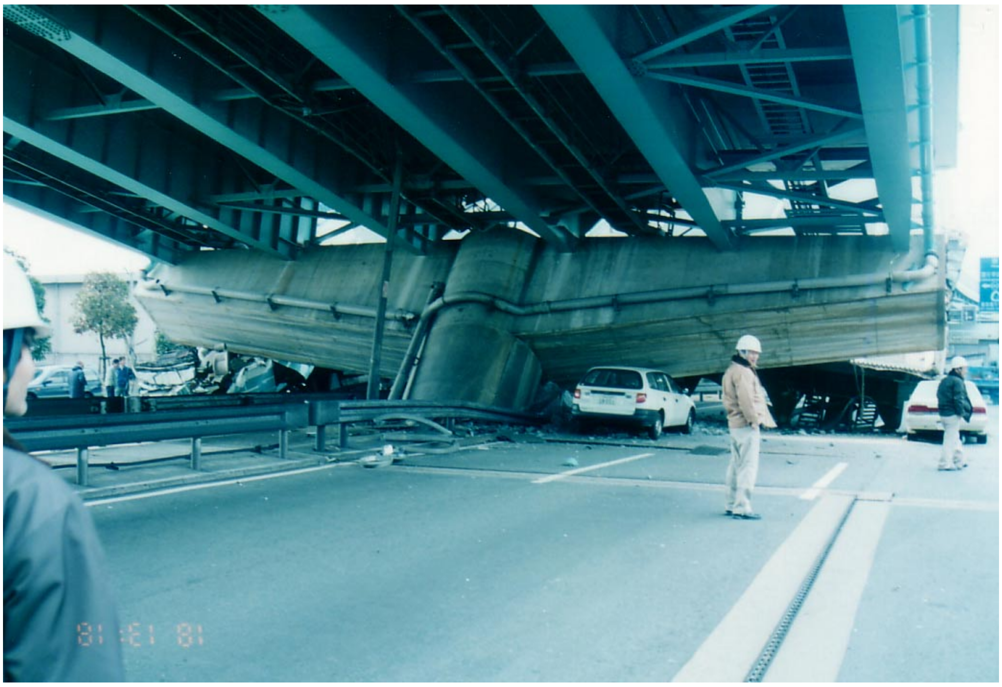


Photo 3.5 Collapse of Takashio Viaduct, 1995 Kobe Earthquake

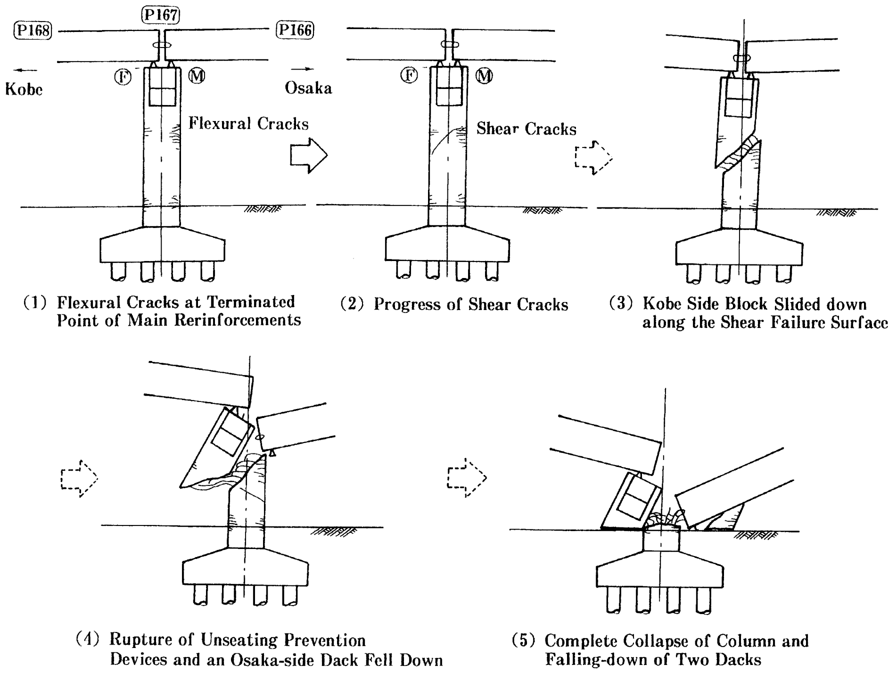


Fig. 3.2 Failure Mechanism of Tateishi Viaduct

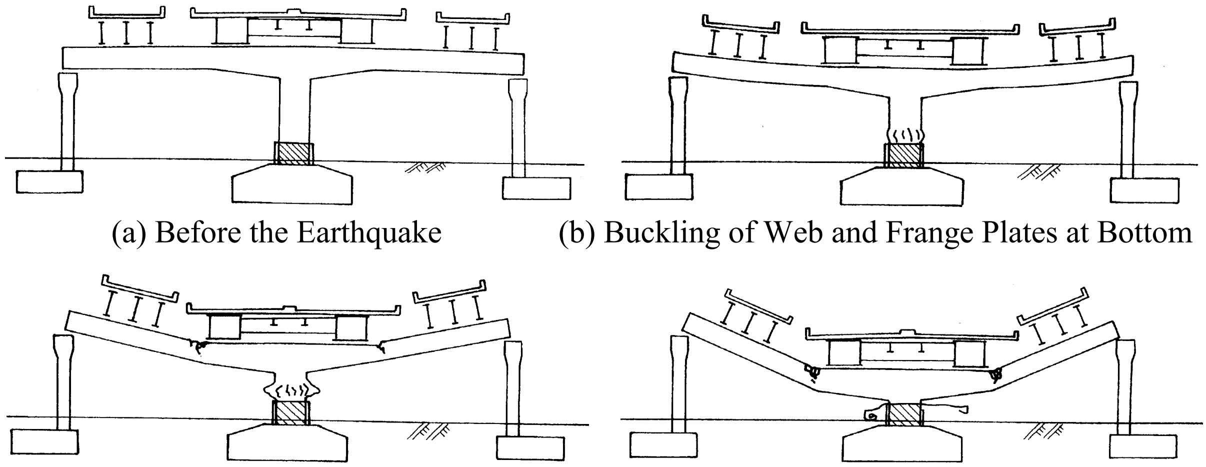
b) Collapse of steel columns

Steel columns collapsed at a number of sites. Photo 3.6 shows collapse of a steel column at the Tateishi Crossing, Hanshin Expressway. The bridge was completed in 1969 based on the allowable stress design approach using 0.2 horizontal and ±0.1 vertical seismic coefficients. The steel column at the center was first constructed, and two reinforced concrete columns at the side and the extended lateral beams were subsequently added when two side decks were constructed.

The steel column failed as if it was crushed in vertical direction. The lateral beam buckled and settled down about 6m. Weak concrete was filled inside the column from bottom to 2.3 m high for protection to automobile collision. The thickness of flange and web plates varied from 28 mm at the column-lateral beam joint to 18 mm at the bottom. They were stiffened by vertical stiffeners and diaphragms. The buckling strength of the column is 65,000 kN, while the compression due to the dead weight of three decks is only 14,900 kN. Consequently, more than 4.3 g acceleration is required to result in the damage of column by compression. It is obvious that the failure was not resulted from the vertical excitation.

Photo 3.6 Collapse of a Steel Column, Tateishi Viaduct, 1995 Kobe Earthquake

Failure mechanism is shown in Fig. 3.3. Under a strong excitation, local buckling of web and flange plates as well as rupture of welded corner occurred at the bottom of column. This decreased the bearing capacity of the column in both lateral and vertical directions, which progressed the settle down of the column due to dead weight of the decks. This caused buckling to occur at both sides of the center deck, and this further progressed the buckling of the column.



(a) Before the Earthquake

(b) Buckling of Web and Frange Plates at Bottom

(c) Progress of Buckling at Bottom and Buckling of Lateral Beam

(d) Complete Failure of Column and Settlement of Lateral Beam

Fig. 3.3 Failure Mechanism of Tateishi Viaduct

c) Damage of unseating prevention devices and damage resulted from forces transfer through unseating prevention device

Unseating prevention devices are important to prevent collapse of a bridge system even when destructive damage occurs. They have fail-safety function. However, failure of various types unseating prevention devices occurred. For example, Photos 3.7 and 3.8 show a failure of plate-type unseating prevention device and a cable restrainer, respectively. They were designed using design forces multiplying 0.3-0.4 seismic coefficient by a static reaction force. It was obvious that the design force was too small.

Pounding occurred between bridge decks at many sites. Although pounding between bridge decks caused only local damage at the contact face, it transfers large seismic lateral forces from one deck to another, which results in a significant change in the seismic response of the entire bridge system.

Photo 3.7 Failure of Unseating Prevention Device, Kobe Earthquake

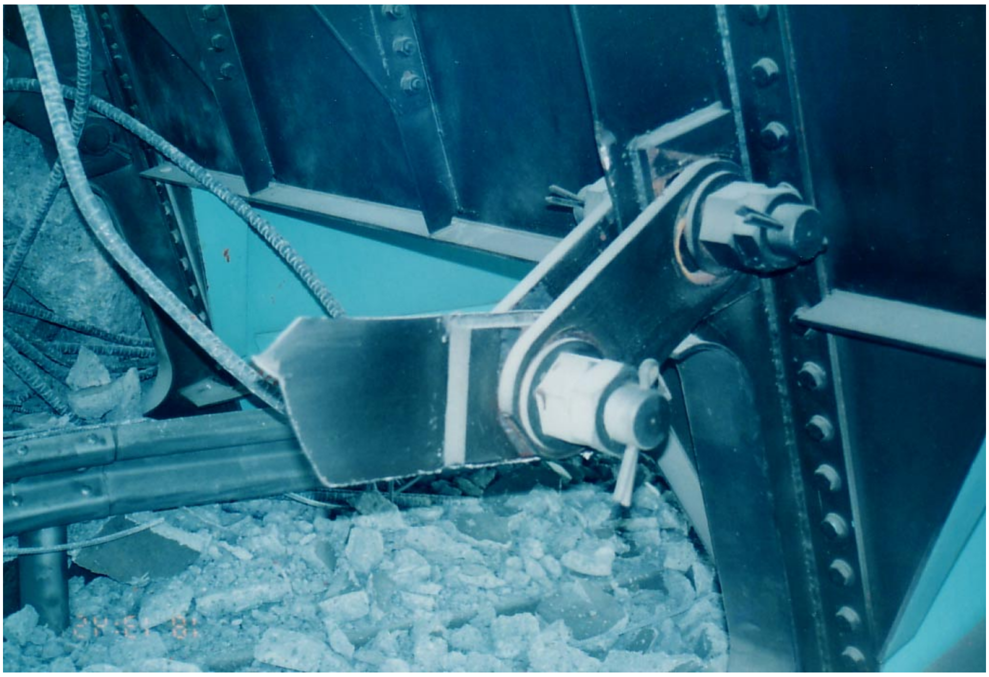


Photo 3.8 Failure of Unseating Prevention Device, Kobe Earthquake

Unseating prevention devices also affect the total response of a bridge system. A good example for this is the collapse of an approach span of the Nishinomiya Bridge system, Hanshin Expressway, as shown in Photo 3.9. The main bridge was a Nielsen Lohse bridge with a mass of 12,000 t, while the approach span was a steel plate girder bridge with a mass of 1,900 t. These two structures were tied together by plate-type restrainers. The damage was initiated by failure of fixed-bearings of the main bridge. This allowed large response displacement of the main bridge to take place, and the main bridge pulled the approach span, which resulted in failure of the fixed-bearings in the approach span. As a consequence the approach span dislodged from its support when the decks moved in the other direction. The unseating prevention devices were not strong enough to support the approach span once it dislodged from the support.

Photo 3.9 Collapse of an Approach Span, Nishinomiya Bridge, 1995 Kobe Earthquake

d) Damage of long span cable-supported bridges

The world-longest Akashi Straight Bridge (refer to Photo 3.10) was under construction when the earthquake occurred. Two abutments and two main towers were already completed, and cables hanged a part of the superstructure. The fault crossed the bridge between two tower foundations (P2 and P3). This resulted in the permanent movements and rotations of the anchorage and tower foundations as shown in Fig. 3.4 (Saeki et al 1997, Yasuda et al 2000). P3 tower foundation and A4 anchorage were dislocated 1.3m and 1.4m, respectively, relative to 1A anchorage and P2 tower foundation. This increased the center-span length from 1990m to 1990.84m, and the total bridge length from 3910m to 3911.09. Survey after the earthquake revealed that there was not any damage in the structures except the permanent movements. The superstructure was erected by slightly changing the length.

Photo 3.10 Akashi Straight Bridge

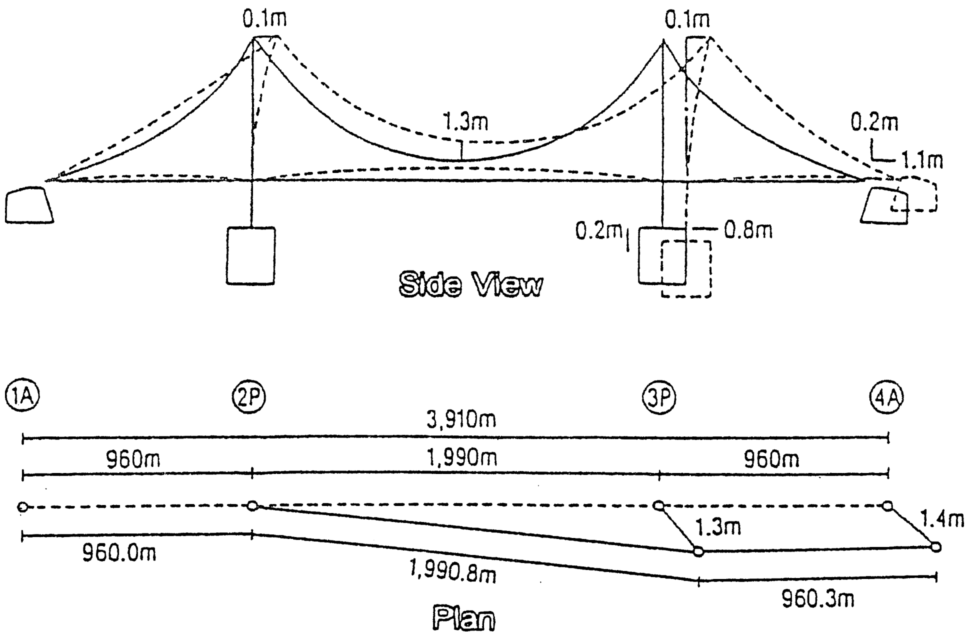


Fig. 3.4 Permanent Offset of Foundations, Akashi Straight Bridge, 1995 Kobe Earthquake (Saeki et al, 1997)

Higashi Kobe Bridge, a cable stayed bridge with 485m center span and two 200m side spans, suffered a controlled damage at dampers and bearings on one of the two end piers. Since reaction force of the deck resulting from the dead load was negative, a pair of steel bearings was provided to pull the deck down at the end pier. Vane-type rotating viscous dampers and window bearings were also provided at the end pier for the distribution of the seismic lateral force of the deck in longitudinal direction and the prevention of excessive transverse response of the deck resulting from high wind, respectively. The steel bearings suffered damage resulting in an about 0.4m uplift of the deck as shown in Photo 3.11. A connection of the rotating viscous damper suffered damage due to excessive longitudinal displacement as shown in Photo 3.12, and the window shoe also suffered damage due to excessive transverse force. Steel column and lateral beam of the pier suffered some flexural and shear buckling.

Photo 3.11 Uplift of Deck, Higashi-Kobe Bridge, 1995 Kobe Earthquake

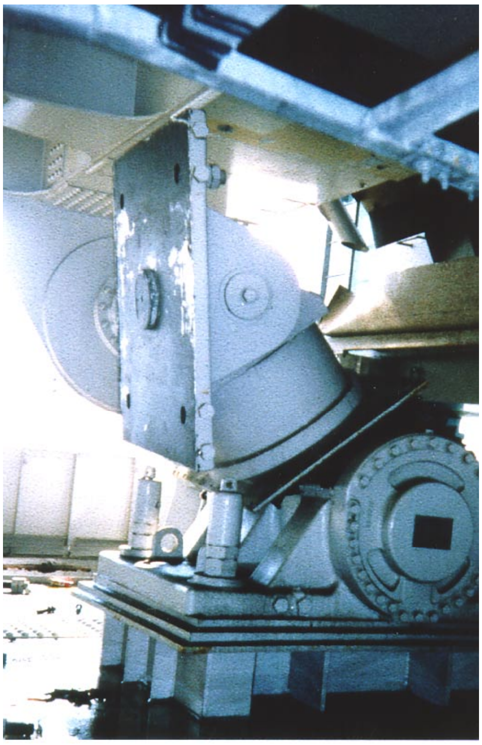


Photo 3.12 Failure of a Conector of Rotating Viscous Damper, Higashi-Kobe Bridge

Since the bridge was instrumented, accelerations were recorded at a foundation, a tower, ground surface and underground nearby the foundation. Acceleration at the top of a tower was over 1g in longitudinal and transverse directions. Since capacity of accelerometers was 1g, they were over-scaled. Peak ground motion was 0.43g and 0.45g in the longitudinal and transverse directions, respectively, at 33m below the ground surface, and 0.28g and 0.33g, respectively, at the ground surface. It is interesting to note that PGA was smaller at the ground surface than PGA at 33m below the ground surface. It was caused by strong nonlinearity of the ground and possibly soil liquefaction.

3.3 Kocaeli and Duzce, Turkey, Earthquakes

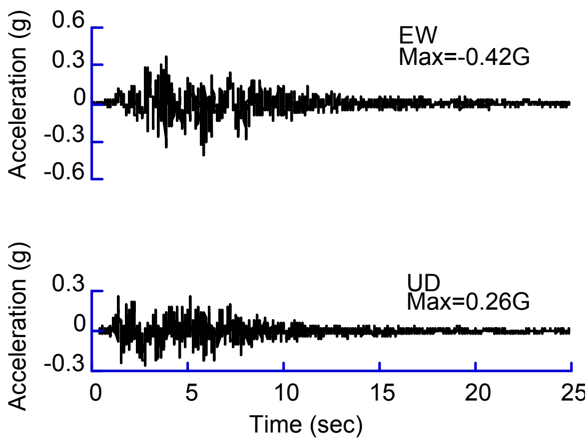
1) Kocaeli Earthquake

On August 19, 1999, a part of the right-lateral strike slip Anatolian fault ruptured in east-west direction for about 100km from Golcuk to Duzce. The moment magnitude M_w was 7.4. Since the Trans-European Motorway was parallel to the fault, extensive damage occurred where the fault crossed the Motorway. Damage was extensive around the city of Arifiye. Various evidence of the right-lateral strike slip rupture was observed. For example, a concrete fence offset 4.3m as shown in Photo 3.13. The fault resulted in a 3.6m offset of a 1.4m-diameter drainage pipe as shown in Photo 3.14.

Photo 3.13 4.3m Offset of a Concrete Fence (Arifiye)

Photo 3.14 3.6m Offset of a Drainage Pipe (Arifiye)

Fig. 3.5 shows the ground accelerations recorded at Sakarya, Adapazeri (Bogazici University 1999). The peak acceleration of fault parallel component was 4.07m/s^2. The fault normal acceleration was not recorded.



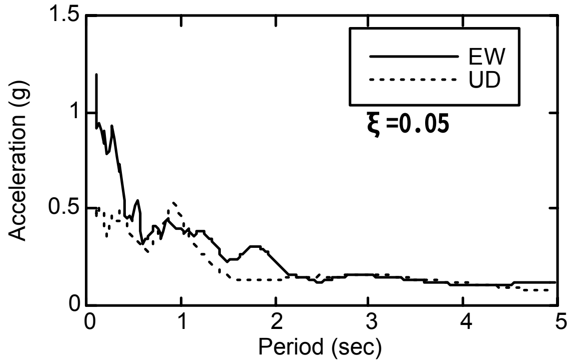


Fig. 3.5 Ground Motions Recorded at Sakarya, Adapazeri

The Arifiye Overpass, a 4 span simply supported prestressed concrete bridge, collapsed as shown in Photo 3.15. It was a skewed bridge with an angle of 65-degree. The fault crossed the bridge between A1 and P1 at an angle θ of 70-degree as shown in Fig. 3.6. The right-lateral strike-slip fault dislocated A1 in northeast direction relative to P1-P3 and A2. Representing D_0 the displacement amplitude along the fault, the relative extension and sway of A1 to P1-P3 in longitudinal and transverse directions, d_{LG} and d_{TR}, are $d_{TR} = D_0 \cdot \sin\theta$ and $d_{LG} = D_0 \cdot \cos\theta$. Assuming $D_0 \approx 4$m and $\theta \approx 70$-degree, the relative extension between A1 and P1 in longitudinal direction, d_{LG}, is 1.37m, which is much longer than the seat length of 0.6 m at A1 and 0.45 m at P1. The deck between A1 and P1 dislodged from the supports resulting from such a large relative displacement.

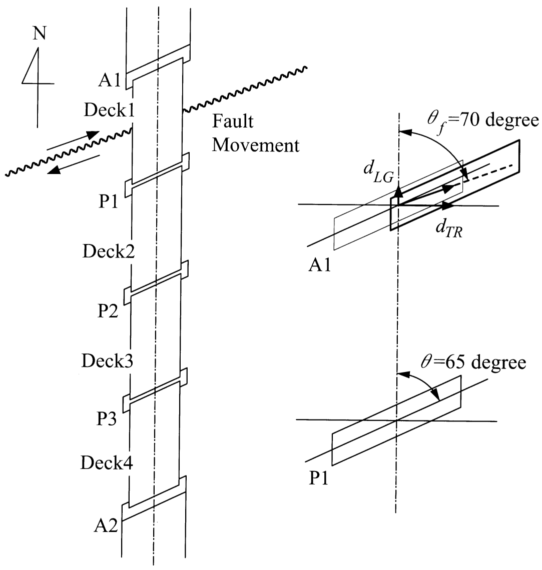


Fig. 3.6 Arfiye Bridge

Photo 3.15 Collapse of Arifiye Bridge

Near the Arifiye Overpass, there were several overpasses similar to the Arifiye Overpass. In general, their seismic performance was satisfactory. For example, the fault crossed a 2 span simply supported prestressed concrete overpass at its abutment as shown in Photo 3.16. However, the damage was limited.

Photo 3.16 Limited Damage of an Overpass where a Fault Crossed at its Abutment

2) Duzce Earthquake

A part of the Istanbul-Ankara Motorway at Bolu suffered extensive damage in the Duzce Earthquake with moment magnitude M_w=7.2, November 12, 1999. Fig. 3.7 shows the ground accelerations recorded at Duzce. Long period pulses are included in the records. Response acceleration of 0.05 damping ratio was 1.3 g at 0.8 second in the L-component.

The Bolu Viaduct consisted of a 2313m long 59 span westbound deck and a 2273 m long 58 span eastbound deck as shown in Photo 3.17. Averaged span length was 40m. Superstructure consisted of 7 prestressed concrete U-beams. Although the U-beams were simply supported, the concrete deck was continuous for 10 spans. Hence, the Viaduct behaved as 10 span continuous bridges.

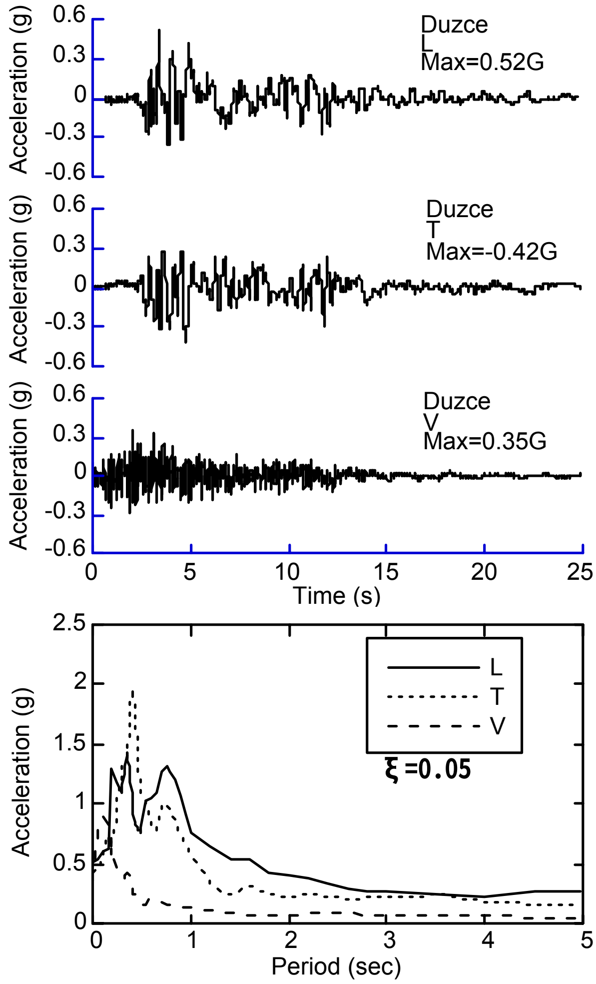


Fig. 3.7 Accelerations and Response Spectra at Duzce, 1999 Duzce Earthquake

Photo 3.17 Bolu Viaduct

They were supported by10-40 m high reinforced concrete columns. Most columns were taller than 40m. The columns were supported by pile foundations. When the earthquake occurred, the Viaduct was almost still under construction (nearly completed).

A unique supporting system was adopted in the Bolu Viaduct (Yilmaz 2000, Tiras and Sahiss 2001, Ghasemi et al 2000, Kawashima and Shoji 2000). A deck was supported by pot bearings that allowed multi-directional sliding. A mechanical energy-dissipating unit consisting of 8 C-shaped mild steel damping elements (EDU in Fig. 3.8) was provided at 10 columns, except one of the two deck ends. They achieved energy dissipation for multi-directional movements of the decks. Except at one of the two deck ends and the mid-column, a viscous damper stopper (VDS in Fig. 3.8) was provided, which connected to the energy-dissipating unit in series. The viscous damper stoppers allowed the deck to displace freely in longitudinal direction resulting from creep, shrinkage and thermal effects. Under seismic loading, the viscous damper stopper is locked so that the energy dissipating units dissipate energy.

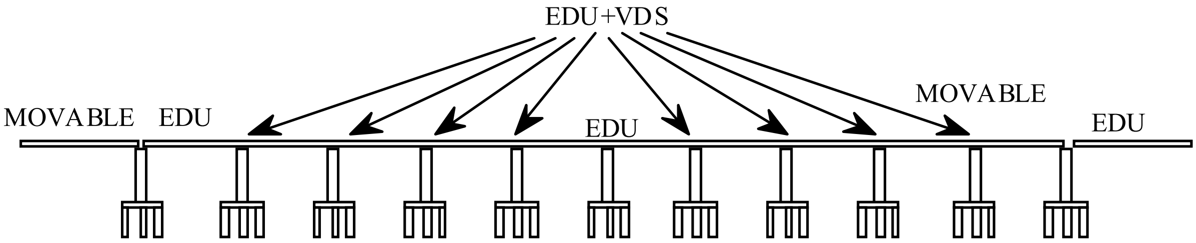


Fig. 3.8 Supporting Condition of Typical 10-Span Continuous Deck (EDU: Energy Dissipating Unit, VDS: Viscous Damper Stopper, Movable: Sliding Pot Bearings)

The site was located at a complex transition area between the Duzce fault and the North Anatoria fault (USGS 1999). From the preliminary design stage, it was known that the site is seismically active and a fault offset was anticipated (Yilmaz 2000, Tiras and Sahiss 2001). The Viaduct was designed assuming the maximum ground acceleration of 0.4g in accordance with the 1983 AASHTO Specifications with some modifications. Main difference from the AASHTO Guide Specifications was the response modification factor R; AASHTO specified a value of R=3 for a single column, while R=1 was used in design of Bolu Viaduct to ensure elastic response in the columns.

The Bolu Viaduct experienced the Kocaeli Earthquake before the Duzce Earthquake. Since it was located only 100km from the east end of the fault rupture, ground motion at the site was high enough to result in the energy-dissipating unit of 4.4-80 mm elastic displacements.

In the Duzce Earthquake, an approximately 2-2.5m right lateral offset crossed the Bolu Viaduct between Pier 45 and Pier 47 at an angle of approximately 20-30 degrees to the bridge axis.

As a result, some pile foundations tilted and rotated. Most decks moved in longitudinal and transversal directions. At several piers, the girders offset from the pedestal, and were hung only by the concrete decks as shown in Photo 3.18. As a result, the concrete decks suffered extensive shear failure. Some decks were about to fall from their supports.

Photo 3.19 shows damage of an energy-dissipating unit and a viscous damper stopper at Pier 48R. The C-shaped energy dissipating elements suffered damage, resulting from an excessive deck movement.

Photo 3.18 Girder Offset from Support, Bolu Viaduct

Photo 3.19 Damage of Energy Dissipating Unit and Viscous Damper Stopper, Bolu Viaduct

3.4 CHI CHI, TAIWAN, EARTHQUAKE

The Chi Chi Earthquake (M_w=7.6), September 21, 1999, occurred as a result of rupture of the Che-Long-Pu Fault. It was a thrust fault, and surface rupture occurred over 70km. Taiwan is a tongue of the Philippine Sea tectonic plate that is over-thrusting the Eurasia plate.

Fig. 3.9 shows the ground acceleration recorded at Shikhkang (TCU068) and response acceleration of 0.05 damping ratio, respectively (Lee et al 1999). Long pulses are included in the ground accelerations. In particular, EW component has a single pulse with peak acceleration of 4.9 m/s^2. Response acceleration of 0.05 damping ratio is about 1.5g at 0.5 second.

A number of bridges suffered extensive damage (Chang et al 2000, Kawashima and Shoji 2000). Most bridges suffered damage as a direct result of fault displacement.

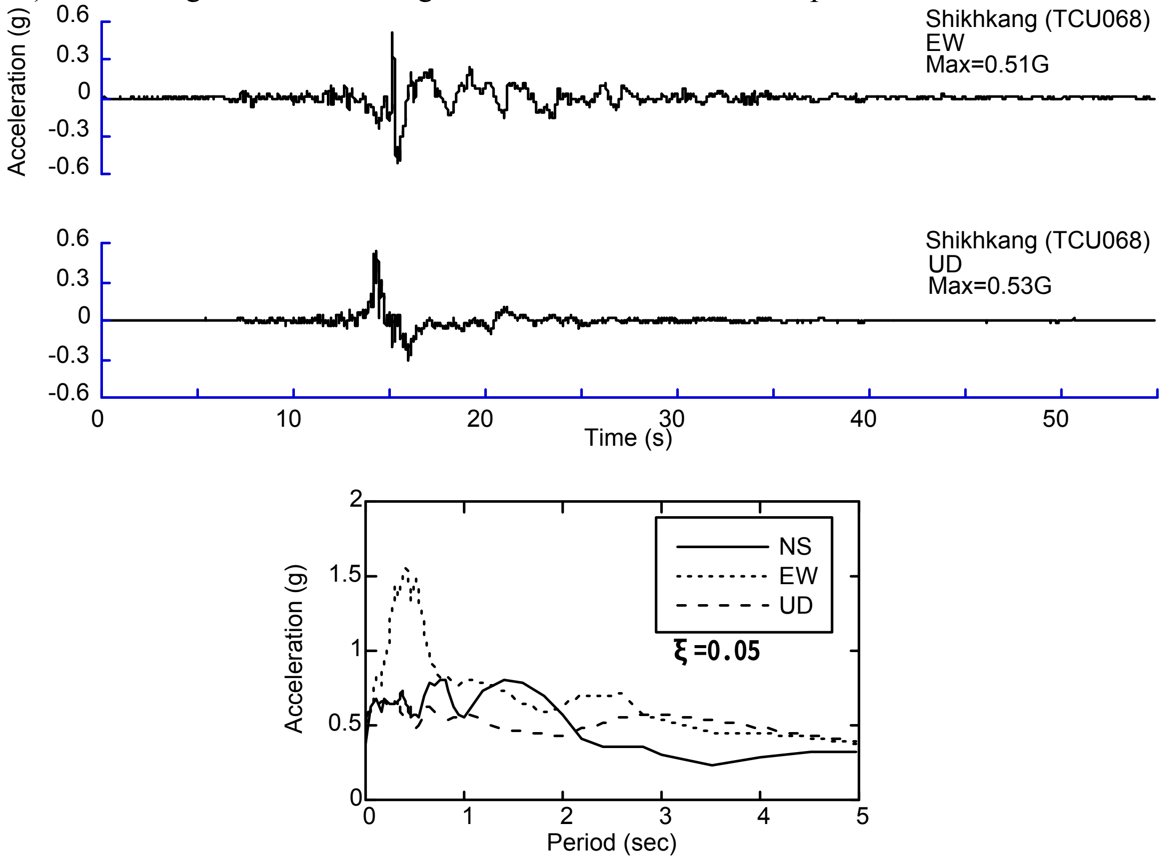


Fig.3.9 Accelerations and Response Accelerations with 0.05 Damping Ratio, Shikhkang (TCU068), Chi-Chi Earthquake (Lee et al 1999)

(a) Bei-Fong Bridge

Bei-Fong Bridge was a 13-span simply supported I-beam girder constructed in 1991. The fault crossed the bridge between A2 and P12 at an angle of 42-degree.The south most three spans collapsed as shown in Fig. 3.10 with other spans being free from damage. A2, P12 and P11 were up heaved about 3-4m, and P12 and A2 were laterally displaced about 3.5m and 4m, respectively, in downstream direction (west) as shown in Photo 3.20.

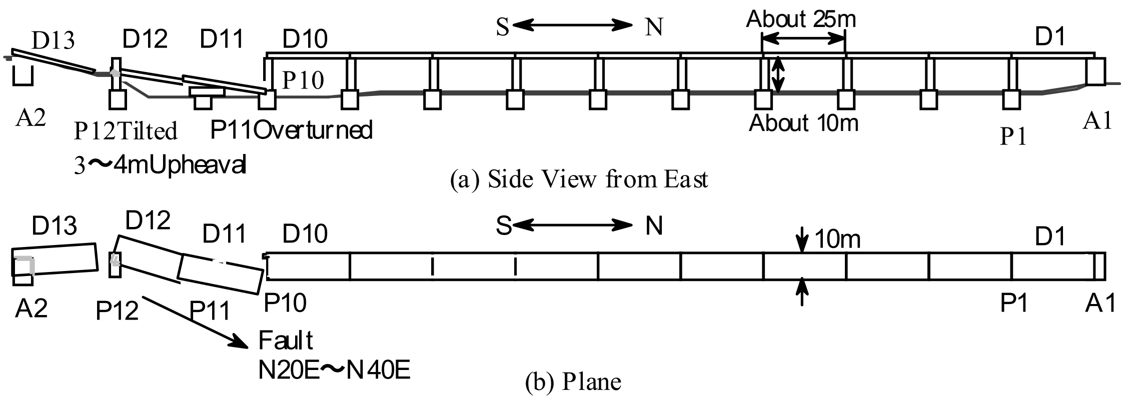


Fig.3.10 Collapse of Bei-Fong Bridge, Chi Chi Earthquake

Photo 3.19 Lateral Movement of A2 and P2, Bei-Fong Bridge, Chi Chi Earthquake

(b) Wu-Shi Bridge

Wu-Shi Bridge consisted of a northbound bridge (upstream, east) and a southbound bridge (downstream, west) as shown in Fig. 3.11. They were 17-span simply supported PC beam girders. The northbound and southbound bridges are identified here by putting “E” and “W”, respectively. The southbound bridge was newer than the northbound bridge. As shown in Photo 3.21, the northbound bridge was supported by 8.5m x 3m wall piers, while 5m x 2m rectangular reinforced concrete piers supported the southbound bridge.

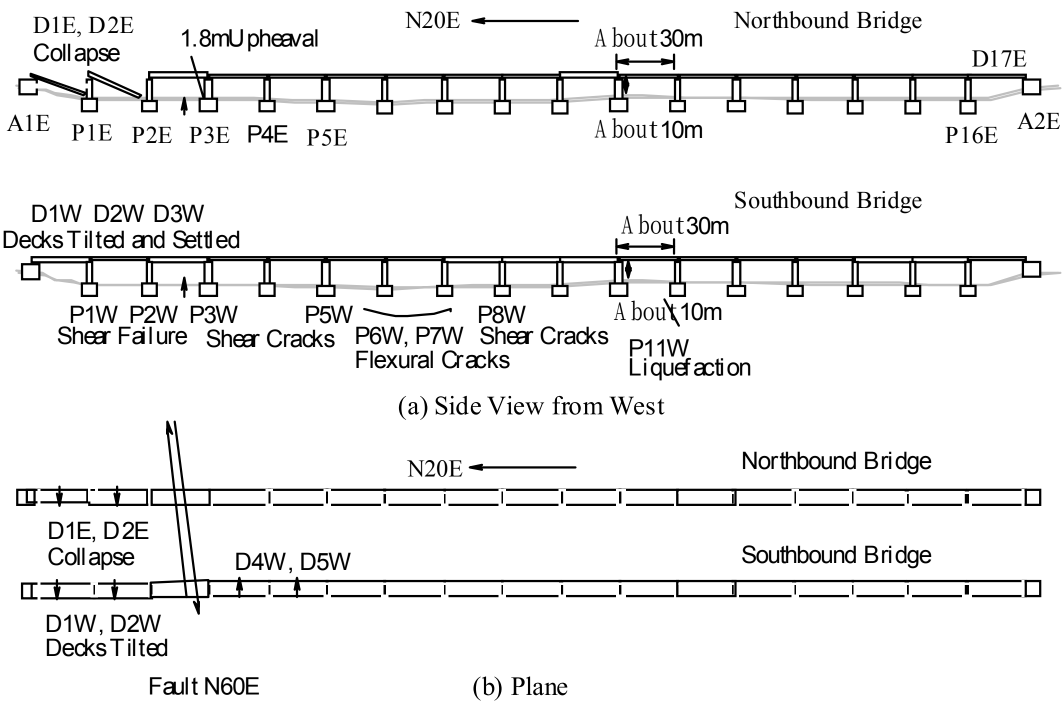


Fig. 3.11 Collapse of Wu-Shi Bridge

Photo 3.21 Collapse of Wu-Shi Bridge, Chi Chi Earthquake

A fault crossed the bridge between P2 (P2E & P2W) and P3 (P3E & P3W) at an angle of about 40-degree N60E). As a result, most piers in the southbound bridge suffered extensive damage. In particular, P1W and P2W failed in shear from east to west as shown in Photo 3.22. The caisson foundation at P3W suffered shear failure from east to west, as shown in Photo 3.23**,** as a direct result of fault movement.

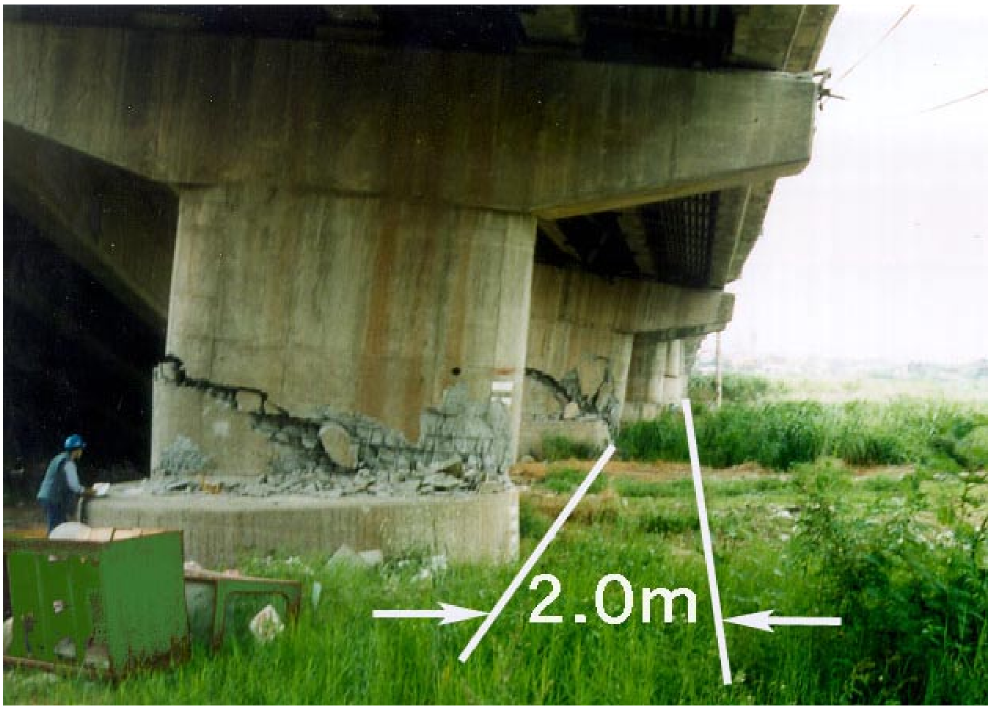


Photo 3.22 Lateral Movement of P1W and P2W, Wu-Shi Bridge

Photo 3.23 Shear Failure of Caisson Foundation Reslting from Fault Movement, Wu-Shi Bridge

On the other hand, damage of P2E and P3E was limited. P2E suffered several shear cracks from north to south at mid-height. P3E suffered a 100-150mm opening of concrete at 1.5m high from the foundation. Since strength of wall piers (P1E and P2E) was large enough, they did not collapse. However, this developed the large relative displacements between the piers and the decks, and this resulted in the collapse of Deck 1E (D1E) and Deck 2E (D2E).

c) Chi-Da Bridge
A two-span continuous prestressed concrete cable stayed bridge suffered damage at the connection between the pylon and the deck as shown in Photos 3.24 and 3.25. This was the damage resulted from ground motion effects. This bridge was still under construction when the earthquake occurred. The pylon and the deck were rigidly connected. The pylon suffered spalling off of covering concrete. A fact that the damage of pylon was more extensive on the surface in transverse direction suggests that the damage resulted from the pylon oscillation in transverse direction. Damping ratio is generally smaller in transverse direction than longitudinal direction in such a cable-stayed bridge (refer to 5.6 4). A 100-300 mm wide flexural crack penetrated the upper and lower slabs of the deck close to the connection with the pylon. The 7th cable from the bottom suffered damage at its cast-iron coupler, and it pulled out.

Photo 3.24 Damage of Pylon, Chi-Da Bridge

Photo 3.25 Damage of Deck, Chi-Da Bridge

REFERENCES

AASHTO, (1983), "Guide specifications for seismic design of highway bridges.
Bogazici University Kandilli Observatory and Earthquake Engineering, Home Page, 1999
California Department of Conservation, (1994), "Processed CSMIP strong motion records from the Northridge, California Earthquake of January 17, 1994," Office of Strong Motion Studies.
Chang, K. C., Chang, D. W., Tsai, M. H. and Sung, Y. C., (2000), "Seismic performance of highway bridges," Earthquake Engineering and Engineering Seismology, 2(1), pp. 55-77.
Ghasemi, H., Yen, P. and Cooper, J. D., (2000), "The 1999 Turkish Earthquake: Post earthquake investigation of structures on the Trans European Motorway," Proc. 2nd International Workshop on Mitigation of Seismic Effects on Transportation Structures, pp. 337-348, National Center for Research on Earthquake Engineering, Taipei, Taiwan.
Japan Road Association, (1971), "Guide Specifications for Seismic Design of Highway Bridges," Maruzen, Tokyo, Japan.
Japan Road Association, (1987), "Guide specifications for earthquake hazard mitigation for road transportation facilities-pre earthquake countermeasures,"
Japan Road Association, (1980, 1990, 1996, 2002), "Design Specifications of Highway Bridges- Part I Common Part, Part II Steel Bridges, Part III Concrete Bridges, Part IV Substructures, and Part V Seismic Design," Maruzen, Tokyo, Japan.
Japan Road Association, (1995), "Reference for applying the guide specifications for reconstruction and repair of highway bridges which suffered damage due to the Hyogo-ken Nanbu Earthquake, to new highway bridges and seismic strengthening"
Japan Society of Civil Engineers, (1931, 1940, 1949, 1956, 1967, 1974, 1977, 1980), "Standard specifications of concrete structures"

Hoshikuma, J., Kawashima, K., Nagaya, K., (1994), "Stress-strain model for confined reinforced concrete in bridge piers," Journal of Structural Engineering, ASCE, 123-5, 624-633.

Kawashima, K. and Hasegawa, K (1994), "New seismic design specifications of highway bridges in Japan," Earthquake Spectra, 10-2, 333-356.

Kawashima, K. and Unjoh, S., (1997), "The damage of highway bridges in the 1995 Hyogo-ken Nanbu Earthquake and its impact on Japanese seismic design," Journal of Earthquake Engineering, 1-3, 505-541, Imperial College Press.

Kawashima, K., (1997), "The 1996 Japanese seismic design specifications of highway bridges and the performanced based design," Seismic Design Methodologies for the Next Generation of Codes, Fajfar & Krawinkler (Eds), Balkema, Rotterdam.

Kawashima, K. and Shoji, G., (2000), "Damage of transportation facilities in the 1999 Kocaeli and Duzce, Turkey Earthquakes and the 1999 Chi Chi, Taiwan Earthquake," 32 Joint Meeting, Panel on Wind and Seismic Effects, UJNR, NIST, Gaithersburg, MD, USA.

Lee, W. et al, (1999), "CWB free-field strong motion data from the 921 Chi-Chi Earthquake, Vol. 1, digital acceleration files on CD-ROM."

Ministry of Interior, (1927) "Guide Specifications for Road Structures.

Saeki, S., Kurihara, T., Toriumi, R. and Nishjtani, M., (1997), "Effect of the Hyogo-ken nanbu earthquake on the Akashi kaikyo bridge," Proc. 2nd Italy-Japan Workshop on Seismic Design and Retrofit of Bridges, Rome, Italy

Tiras, A. and Sahiss, T., (2001), "European transit motorway damaged bridge restoration after Kocaeli and Bolu Earthquakes," 80th Annual Meeting, Transportation Research Board, Washington, D.C., USA.

USGS, (1999), "Implications for earthquake risk reduction in the United States from the Kocaeli, Turkey, Earthquakes of August 17, 1999," Geological Survey Circular.

Yasuda, T., Moritani, T., Fukunaga, S. and Kawabata, A., (2000), Seismic behabior and simulation analysis of Honshu-Shikoku bridges," Journal of Structural Engineering, JSCE, 46A, 685-694.

Yilmaz, C., (200), "Design philosophy and criteria for viaduct no. 1," KGM and FHWA Workshop, KGM, Ankara, Turkey.