DAMAGE CHARACTERISTIC OF BRIDGES DUE TO 2010 CHILE EARTHQUAKE

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Abstract

This paper presents damage of bridges during the 2010 Chile earthquake based on site investigations. Damage feature of bridges including the effect of in-plane rotation of bridges, insufficient seat length, absence of anchor between superstructure and substructures at bearings and lack of bearing capacity of foundations is presented. Design practice of bridges in Chile is also introduced.

Introduction

Chile earthquake occurred off coast of the Maule Region of Chile along the boundary between Nazca and South American tectonic plates at 3:34 (local time) on February 27, 2010. The moment magnitude Mw was 8.8. Aftershock region extended about 700 km and 200 km in NS and EW directions, respectively. The earthquake resulted in significant damage in wide area from Valparaiso (120 km northwest of Santiago) to Arauco (100 km south of Concepcion) as shown in Fig. 1. Thirty one investigated sites of bridges which will be described later are also shown in Fig. 1. The epicenter of the earthquake was 400 km south-southwest of Santiago, and 100 km north-northeast of Concepcion, the second largest city in Chile.

In Chile, significant earthquake repeatedly occurred in the past. Even after the late 20 century, a Mw 9.5 Valdivia earthquake occurred in 1960 at 800 km south of Santiago, which was the largest in size ever recorded, followed by the 1985 Mw 8.0 earthquake off San Antonio, and the 1995 Mw 7.8 earthquake at Antofagasta. The 2010 Chile earthquake developed extensive damage to buildings, transportation and lifeline facilities, and industrial facilities. Tsunami extended the damage along the coastal regions.

The authors investigated the damage of transportation facilities from March 28 to April 4, 2010. We were dispatched by Japan Society of Civil Engineers with support of Japan International Cooperation Agency and World Federation of Engineering Organizations. Damage investigation was conducted at thirty one sites along Route 5 (Pan American) highway as well as in Santiago, Constitucion, San Antonio, Concepcion and

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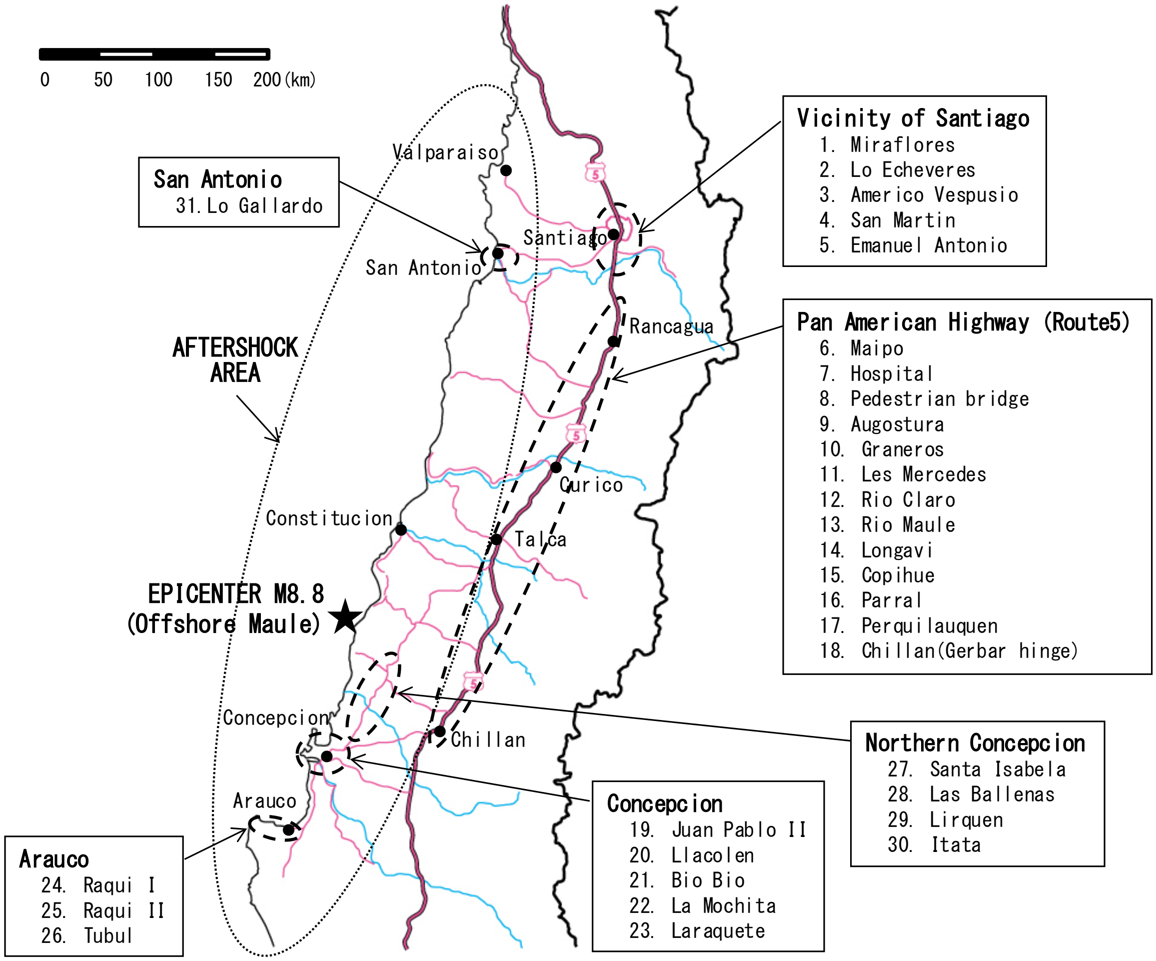


Fig. 1 Sites and bridges where damage investigation was conducted

Arauco as shown in Fig. 1. Because there were sites where upper and lower bound bridges are separated, the total number of bridge investigated was forty six by counting the upper and lower bound bridges independently. Note that a bridge may include several to several tens spans. This paper introduces the damage of bridges. Design practice of bridges in Chile is also presented.

Chilean Practice of Seismic Design of Bridges

Conservative seismic design force has been used in Chile based on Manual of Highways in Chile. Design force is specified in three ways depending on type of bridges. For example, for either simple span or two span bridges with a span shorter than 70 m and the height from the design ground surface to the superstructure shorter than or equal to 12m, the design lateral seismic coefficient k_h is given as

$$k_h = c_I c_{GC} \frac{A_0}{2g} \geq 0.1 \qquad (1)$$

in which c_I : importance factor (1.0 and 0.8), c_{GC} : ground condition factor (0.9, 1.0, 1.2 and 1.3 for Group I, II, III and IV, respectively), and A_0 : standard peak ground acceleration depending on zones (0.2g, 0.3g and 0.4g in Zone I, II and III, respectively). Assuming that c_I =1.0 and c_{GC} =1.3, the design lateral seismic coefficient k_h becomes 0.13-0.26 depending on the zone.

On the other hand, for those bridges with span shorter than 70m and the height from the design ground surface to the superstructure higher than or equal to 12m and shorter than 25 m, the design lateral seismic coefficient k_h is evaluated depending on the fundamental natural period T as

$$k_h = \begin{cases} 1.5 \times c_I c_{GC} A_0 / g & T \leq T_1 \\ \dfrac{c_I c_2 c_{GC} A_0}{g T^{2/3}} & T > T_1 \end{cases} \qquad (2)$$

in which c_I , c_{GC} , A_0 are the importance factor, ground condition factor and standard lateral seismic coefficient shown above, and c_2 is parameter depending on ground condition (0.513, 0.627, 1.182, and 1.598 for Group I, II, III and IV, respectively), and T_1 is a period (s) depending on ground condition (0.2s, 0.3s, 0.7s and 1.10s for Group I, II, III and IV, respectively). Fig. 2 shows the seismic coefficient k_h assuming A_0 =0.4g and c_I =1.0. The highest lateral seismic coefficient is 0.78 until 1.1 s. at the group IV soil condition in the Zone 3.

The response modification factor R depends on type of structural components and directions. For example, R of a single column is 3.0 in both longitudinal and transverse directions, while it is 3.0 and 4.0 for a moment resisting frame pier in the longitudinal and transverse directions, respectively. Based on the design seismic coefficient by Eq. (2), the design seismic coefficient for the evaluation of inelastic static seismic force demand is nearly 0.2.

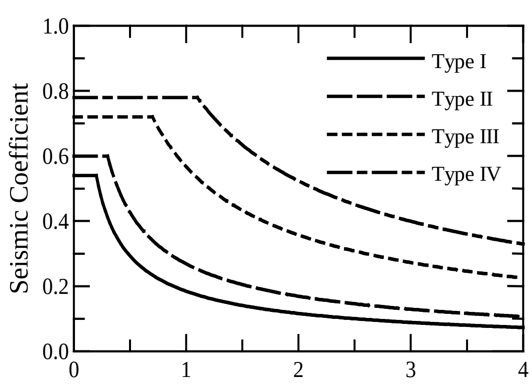


Fig. 2 Seismic coefficient by Eq. (1) assuming A_0 =0.4g and c_I =1.0

Seat length S_E is defined for preventing unseating of a superstructure from its substructure depending on the seismic response category as

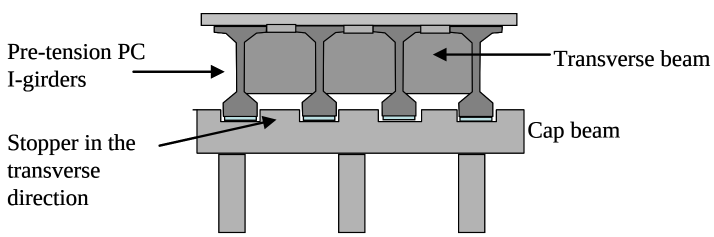
$$S_E = \begin{cases} (203 + 1.67l + 6.66h) \cdot c_{sk} \\ \qquad \cdots\cdots\cdots \text{category a and b} \\ (305 + 2.5l + 10h) \cdot c_{sk} \\ \qquad \cdots\cdots\cdots \text{category c and d} \end{cases} \tag{3}$$

where

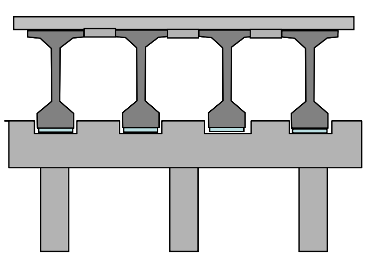
$$c_{sk} = 1.0 + 0.000125\alpha^2 \tag{4}$$

in which S_E : seat length (mm), l : span length (m), h : column height (m) and α : angle of skew in degree. The seismic response category is classified into four groups from a-rank to d-rank depending on the seismic risk, importance and the risk for scoring.

Assuming a straight bridge with l =30 m and h =10m, the seat length S_E becomes 0.48 m under category c and d based on Eq. (3). For reference, the minimum seat length S_E is 850 mm based on the Japanese practice under the same condition [JRA 2002]. Note that concession has been introduced for construction and operation of transportation facility in Chile since the mid 1990s. It seems that a different concept of seismic countermeasures has been introduced for bridges built in the concession regime. For example, in prestressed concrete (PC) I-girder bridges which are dominant in Chile, transverse beams have been generally set for connecting PC girders at mid span as well as at both ends based on the original Chilean practice. However they are eliminated in the bridges in the concession as shown in Fig. 3. Furthermore side blocks for limiting excessive offset in the transverse



(a) Original Chile Design


(b) Lateral beams are eliminated

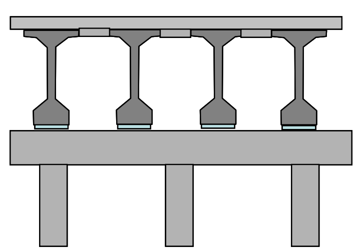

(c) Stoppers in the transverse direction are eliminated

Fig. 3 Type of PC I-girder bridges

direction, which are generally set in the Chilean practice, are also removed in the bridges in the concession. These modifications were most likely introduced for reduction of construction cost and period, which had resulted in extensive damage to bridges as shown below.

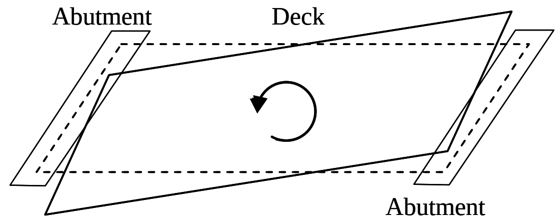


Fig. 4 Unseating resulted from rotation of skewed bridge

<u>Damage Due to Rotation of Bridges</u>

1) Rotation of Skewed Bridges

It is well known that skewed bridges tend to rotate in-plane directing from its obtuse corner to the acute corner under a strong ground motion. The rotation can result in unseating of a skewed bridge at the acute corners as shown in Fig. 4. To distinguish the direction of skewness from the bridge axis (longitudinal direction) and the direction perpendicular to the longitudinal direction (transverse direction), the direction parallel to the ends of a skewed bridge and the direction perpendicular to this direction are called herein skewed transverse direction and skewed longitudinal direction, respectively. Rotation of a skewed bridge is developed by various seismic actions such as pounding between a bridge and an abutment or between two adjacent bridges, and incoherent response of substructures.

2) Hospital Overcrossing

There were several skewed bridges which collapsed primarily or partially due to the rotation of bridges. One of the most typical examples of collapse due to rotation of skewed bridges was Hospital Overcrossing along the Highway No. 5 at crossing over railways, about 43 km north of Rancagua City. It was a two span simply supported PC I-girder bridge. The north-bound overcrossing collapsed while the south-bound overcrossing did not collapse as shown in Fig. 5 and Photo 1. Note that the north-bound overcrossing was constructed in the concession while the south-bound overcrossing was designed based on the original Chile design practice. There was another two span simply supported steel girder overcrossing beside the Hospital Overcrossing, however since it was a very old bridge, it is not described here. This old bridge was demolished after the earthquake. The north-bound overcrossing consisted of three PC I-girders with a skew angle of about 60 degree. Transverse beams were not set to I-girders. Piers and abutments of both the north- and south-bound bridges did not suffer serious damage although slight cracks were found. Photo 2 (a) shows seat at the north abutment of the north-bound overcrossing. The seat was 1.2 m wide in the longitudinal direction, so it was 0.85m wide in the skewed longitudinal direction. Neoplane pads were used to support PC girders without anchor. It is known from failure of the front wall at the south abutment shown in Photo 2(a) that a PC I-girder supported at the pedestal dislodged from this point. This shows that the north bridge of the north-bound overcrossing rotated in the clockwise direction. This is the direction anticipated to occur due to rotation of the north bridge. On the other hand,

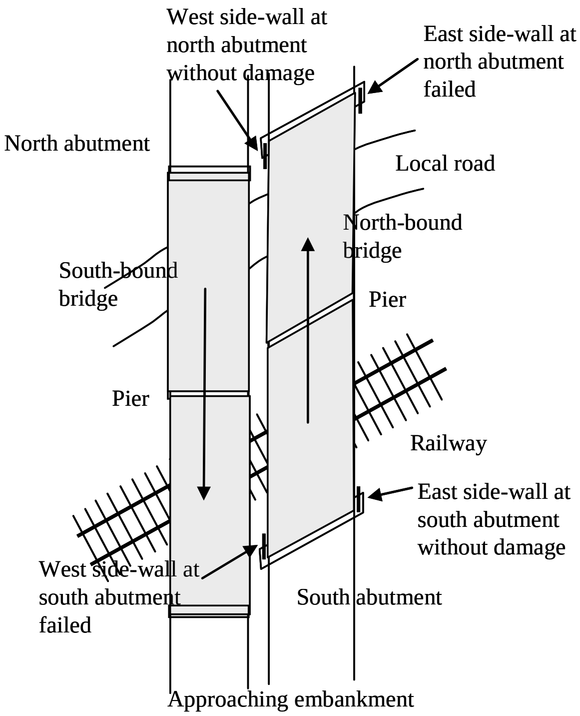


Fig. 5 Collapse of North-bound Hospital Bridge

Photo 1 Collapse of north-bound Hospital Overcrossing (Courtesy of Ministry of Public Works)

(a) North abutment (b) South abutment

Photo 2 Evidence of rotation of the north-bound bridge

Photo 2(b) shows failure of the west side wall at the south abutment of the north-bound overcrossing. Note that the east side wall did not suffer any damage. This is in agreement with the anticipated direction of rotation. Consequently, this is also evidence that the south bridge of the north-bound overcrossing collapsed significantly affected by rotation of the bridge in the clockwise direction. Because transverse beams were not set to I-girders in the north bound bridges, it is likely that connection of I-girders by concrete slab deck could easily deteriorate once one of the three I-girders dislodged from its support, which in turn resulted in total collapse of the bridge.

On the other hand, Photo 3 shows the south-bound overcrossing which did not suffer damage. It was an almost straight overcrossing and this was in contrast to the north-bound overcrossing. Transverse beams were set between four PC I-girders. Side stoppers were also set at both ends on the top of lateral beam for preventing excessive superstructure drift in the transverse direction. It is considered that these countermeasures contributed to better performance in the south-bound overcrossing than the south bound bridge.

Photo 3 South-bound overcrossing which suffered almost no damage

3) Three Skewed Overcrossings in Santiago

Along the Metropolitan Santiago freeway "Americo Vespusio", there was a location at northwest of the city where two closely located overcrossings (Mira Flores Overcrossing and Lo Echeveres Overcrossing) collapsed and an overcrossing (San Martin Overcrossing) in between the two collapsed overcrossing did not suffer damage. Three overcrossing had similar structural properties. Outer and inner bounds were separated, and they were all three span simply supported skewed PC I-girder bridges with similar structural properties. They were supported by elastomeric bearings without anchor.

Both outer and inner bounds of Mira Flores Overcrossing collapsed as shown in Photo 4. It is obvious that deck rotation contributed to the collapse of this bridge. It had 22.5m+28m+22.5m long spans with a skew angle of about 70 degree. Two sets of moment resisting reinforced concrete pier consisting of a cap beam and five 0.9 m diameter columns supported a superstructure. The column suffered only minor flexural cracks, but bearing supports on two piers suffered extensive damage as shown in Photo 5. Photo 6 shows a side stopper for restricting drift in the transverse direction and uplift of I-girders. A pair of two side stoppers was set at both sides of an I-girder at the lower flange as shown in Fig. 6. Two anchor bolts with a diameter of about 13 mm were used to set a side stopper in position. However the anchors were too weak to limit lateral offset of the I-girders. Four abutments at both ends of two overcrossings did not suffered damage, however four side walls which were located beside the acute corners of the four collapsed bridges were destroyed. This is an evidence of the bridge rotation. The seat S_E was about 0.5 m at the two abutments.

The inner bound of Lo Echeveres Overcrossing collapsed while the outer bound bridge did not collapse but it was shored due to damage of supports. They were 26.5 m +34m+26.5m long bridges with skew angle of about 60 degree. The deck slab was continuous at the joints between adjacent decks. Photo 7 shows damage of supports as well as two side stoppers of the outer bound bridge. The side stoppers were weak to limit offset of the I-girders in the transverse direction.

On the other hand, San Martin Overcrossing did not collapse but suffered damage at their supports as shown in Photo 8. This provides a good understanding how the girders unseated due to failure of supports in Mira Flores and Lo Echeveres Overcrossings.

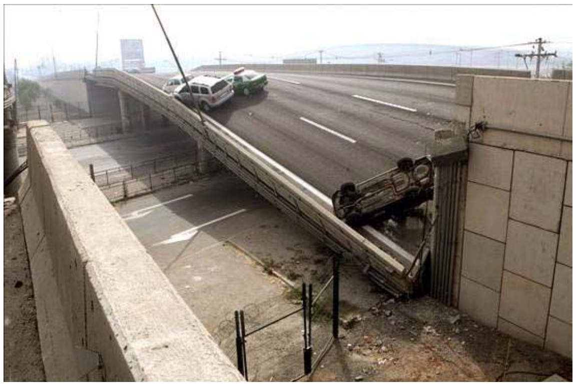


Photo 4 Collapse of Mira Flores Overcrossing along Americo Vespucio Freeway (After AP)

Photo 5 Failure of supports, Mira Flores Overcrossing

Photo 6 Device for constraint of I girders

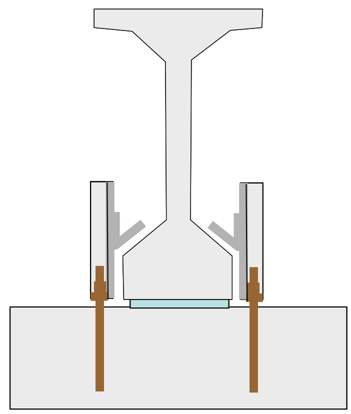


Fig. 6 Side stopper for restricting deck uplift and transverse drift

Photo 7 Damage of supports, Lo Echeveres Overcrossing

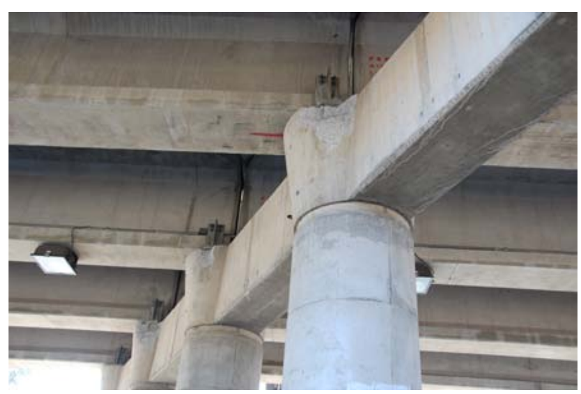


Photo 8 Damage of support of San Martin Overcrossing

Photo 9 Settlement of girders of Juan Pablo II Bridge (from right dyke)

Photo 10 Shear failure of a column at the right dyke, Juan Pablo II Bridge

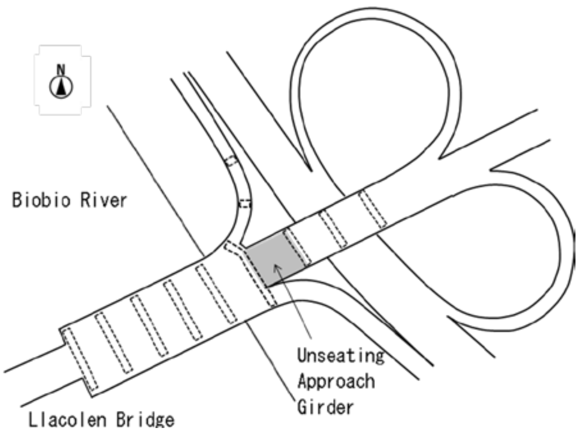


Fig. 7 Approach structure for Llacolen Bridge

Photo 11 Collapse of an approaching span

Damage of Multiple Span Bridges Crossing BioBio River in Concepcion

1) Juan Pablo II Bridge

Juan Pablo II Bridge is a 2310 m long, 21.9 m wide, 70 span simply supported bridge crossing Biobio river in Concepcion City as shown in Photo 9. It was built in 1974. This is a bridge with major importance for transportation in Conception. The decks are supported by moment resisting piers. As shown in Photo 9, extensive settlement of decks as high as 0.5 m was observed. Damage could be investigated only at an approaching span at the right river dyke. Only at this approaching span, three reinforced concrete columns collapsed in shear and a lateral cap beam suffered extensive damage due to shear. Photo 10 shows shear failure of one of the three columns at the right dyke. Because this bridge was built in the days when the importance of shear capacity was not recognized, the damage was resulted from insufficient shear capacity. The failure of the column resulted in extensive settlement of the bridge as shown in Photo 9. It is likely that the settlement was also developed due to insufficient bearing capacity of foundations.

Photo 12 Failure of concrete slab

Photo 13 Lateral drift of the pier supporting the span which unseated at the other end

Photo 14 Settlement around a pier

Photo 15 Uplift and outward drift at an inner end support

2) Llacolen Bridge

Similar to Juan Pablo II Bridge, this is also a critically important bridge crossing Biobio river at the center of Concepcion City. Main spans of the bridge did not collapse, but an approach span at the right collapsed as shown in Fig. 7 and Photo 11. It is obvious that seat length was very short, and this resulted in unseating of the approaching span. Photo 12 shows failure of concrete slab of the collapsed span. It is likely in the above mentioned PC I-girder bridges that similar failure due to its insufficient capacity of concrete slab resulted in collapse of a whole bridge once an I-girder unseated from its support.

Photo 13 shows inclined pier which supported the other end of the collapsed span. Around the bottom of the pier, 0.6 m deep 0.4 m wide settlement of soils was observed underneath the concrete pavement as shown in Photo 14. It is likely that sand liquefaction occurred around the span, and this developed excessive lateral displacement of the pier, which in turn resulted in dislodgement of the approaching span.

Photo 15 shows damage of an on-ramp curved three-span continuous box girder bridge close to the above mentioned collapsed approaching span. Curved bridges tend to have larger response in the outward direction than the inward direction due to the arch action [Kawashima and Penzien 1979]. Furthermore inner corners are likely to subject to uplift due to warping of the bridge. It is seen in Photo 15 that outward drift of the bridge as well as uplift at the inner corner are observed.

Photo 16 Collapse of Biobio Bridge

3) Biobio Viejo Bridge

Biobio Viejo Bridge was a 1646 m long 98 simply supported steel girder bridge built in the 1930s. It was an important bridge for crossing Biobio River in Concepcion City. It was repaired after partly damaged due to the 1960 Chile earthquake. It was known based on the investigation of Japan International Cooperation Agency (JICA) that this bridge was insufficient for normal transportation due to superannuation of structural members. So it was used only for pedestrian bridge. As shown in Photo 16, several spans each collapsed at three locations.

Photo 17 Maule Bridge

<u>Damage of Plate Girder Bridge</u>

Maule Bridge was a 913 m long plate girder bridge consisting of two eleven span continuous bridges as shown in Photo 17. Three steel girders were supported by neoplane pads at columns while they were supported by steel fixed bearings at abutments in both ends. Consequently large inertia force of superstructures concentrated at the fixed steel bearings on the abutments in the longitudinal direction. This resulted in rupture and buckling of web plates, flanges and stiffeners as shown in Photo 18. Because rupture of lower flange had already propagated to the web plate, it was very critical for collapse. As well as large inertia force of eleven spans, incoherent response between the abutment and neighboring columns developed large lateral force at the fixed bearings on the abutment. Buckling of diagonal braces and offset of steel girders in the transverse direction also occurred.

Photo 18 Rupture of web plates, flanges and stiffners of girder at fixed bearing

Photo 19 Collapse of Tubul Bridge

Photo 20 A new bridge constructed based on the original Chile seismic design

Photo 21 Transverse end beam and side blocks for limiting excessive transverse offset

Damage of Bridges Due to Soil Failure

There were bridges which suffered damage due to soil liquefaction in the coastal region. Tubul Bridge was an eight span simply supported plate girder bridge in Arauco. All eight spans collapsed as shown in Photo 19. Because piers extensively tilted and approaching road embankments slid and settled sideway, it is considered that extensive soil liquefaction occurred all around the bridge. Seat length was only 0.38 m at both abutments. Raqui II Bridge was a four span simply supported plate girder bridge located very close to Tubul Bridge. Two spans collapsed in a similar way with Tubul Bridge.

Performance of A Recentlt Built Bridge

There were a number of bridges which did not suffer extensive damage in the region under strong excitation. For example, Lirquen Bridge was a six span simply supported PC I-girder bridge, as shown in Photo 20, along No. 150 road in the suburbs of Concepcion City. It was recently built based on original Chile seismic design. Although the bridge was supported by five columns with different height, it suffered essentially no damage. As shown in Photo 21, transverse beams connecting three PC I-girders and side blocks for limiting excessive transverse offset are provided. Although PC I-girders were only supported by neoplane pads without anchor, no transverse offset developed.

Feature of Damage

1) Damage due to Absence of Anchor between Super- and Sub-structures at Bearings

Essentially in all bridges in Chile, superstructures rest on neoplane pads or elastomeric bearings free of anchoring to substructures. Consequently, superstructures can freely drift from their supports under extreme ground motions, and there were several bridges where this directly or indirectly resulted in unseating of superstructures from their substructures. Obviously full connection of superstructures to substructures can increase the seismic lateral force demand from a superstructure to substructures, so it may be wise to mitigate the demand by accepting a certain amount of slip to occur at bearings if excessive seismic lateral force applies from a superstructure to substructures. However because friction force is difficult to control, the free support at bearings without anchor can result in extensive lateral drift of superstructures relative to substructures. It is therefore considered that a certain connection between a superstructure and substructures is required at bearings in the region where extensive ground motion is anticipated. Furthermore it is generally appropriate to set unseating prevention devices so that excessive drift of superstructures can be prevented. Although side stoppers and vertical restrainers were set in some bridges, they were insufficient for this purpose.

2) Damage of Skewed Bridges due to In-plane Rotation

Several skewed bridges collapsed due to in-plane rotation. Absence of anchor at bearings and effective unseating prevention devices for limiting the rotation extended the damage. Because reaction force at bearings due to dead weight varies from an acute corner to an obtuse corner, lateral force demand as well as the seismic response of a skewed bridge is very complex. Once a side PC I-girder starts to unseat from the support at an acute corner, it is likely to develop total collapse of the bridge system because of lack of integrity of the bridge system resulted from absence of transverse beams connecting I-girders. More extensive research directed to effective measures is required. It is essential not to construct skewed bridges with large skew angles. Anchor of bearings to superstructure and substructure may be effective as shown above.

3) Damage Due to Insufficient Seat Length

Collapse of some bridges could be prevented if the seat of support was much longer. Because cost increase for extending seat is generally limited, it is recommended to adopt more redundant seat so that unseating of a superstructure from its supports could be avoided. Connection between adjacent superstructures or connection between a superstructure and its supporting substructure are also effective.

4) Damage due to Insufficient Shear Capacity of Substructures

Major structural components including columns and cap beams failed in shear and flexure in the bridges which were unreinforced or constructed in the early days as shown in Photo 10. In the damage shown in Photo 10, the shear capacity of concrete was probably overestimated in those days. Except those bridges built in the early days, bridges built in recent years did not suffer extensive damage at their columns.

5) Damage due to Lack of Bearing Capacity of Foundations

In the bridges in Arapco and Concepcion, several bridges collapsed due to excessive relative displacement between or lack of bearing capacity of foundations as shown in Photos 13 and 19. It is important to correctly consider the effect of soil liquefaction.

Conclusions

Chile earthquake provided valuable lessons on how bridge structures behaved under a Mw 8.8 earthquake. The damage of bridges may be classified into three categories; 1) damage of bridges constructed in the early days, 2) damage of bridges designed based on original Chile design, and 3) damage of bridges built in concession after the mid 1990s with insufficient insight to the seismic effects. In PC I-girder bridges which are dominant in recent bridges, lack of integrity of a bridge due to absence of transverse beams connecting I-girders and effective stopper mechanism in the transverse offset resulted in extensive damage in the category 3) bridges. This deficiency was particularly intensified in skewed bridges, which suffered extensive damage due to in-plane rotation of a whole bridge system. In contrast to the category 3) bridges, the category 2) bridges generally performed well without extensive damage except the bridges in coastal region where soil instability resulted in extensive damage. Based on clear difference of damage degree between category 2) and 3) bridges, it is important to learn that elimination of important seismic countermeasures resulted in extensive damage in the bridges constructed in concession.

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