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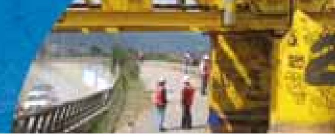

Preliminary analysis of the seismic response of bridges during the Chilean 27 February 2010 earthquake

Análisis preliminar del comportamiento sísmico de puentes durante el terremoto chileno del 27 de febrero del 2010

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A preliminary analysis of the seismic behaviour of bridges during the February 27th Chilean earthquake is presented in this paper. The traditional characteristics of reinforced concrete Chilean bridges have been changing in the last two decades, especially in concessions roads. Therefore, three different superstructure configurations are indentified. In newer bridges designs, the diaphragm has been eliminated and in some bridges, the reinforced concrete shear key, or stoppers, has been replaced by weaker steel shear keys. Because of the earthquake, 10 relevant public bridges collapsed and 91 concession bridges, which include 52 pedestrian bridges, suffered damage or collapse. The observed damage in selected bridges around Santiago and Concepción are described here. The seismic behaviour of bridges was affected not only by the structural configuration but also by the foundation soil. Findings from this analysis reveal that the strength and stiffness provided by the steel shear keys was inadequate, and that the absence of diaphragms was found to be detrimental to the integrity of the superstructure. It is also concluded that skewed bridges are more susceptible to unseating of the superstructure and that the foundation soil played an important role in the seismic behaviour of some bridges, especially for soft and saturated soils responding in an undrained condition.

Keywords: bridge, seismic behaviour, diaphragm, shear key, skewed bridges

En este artículo se presenta un análisis preliminar de la respuesta sísmica de puentes debido al terremoto de Chile del 27 de Febrero. Las características tradicionales de los puentes de hormigón armado Chilenos ha ido cambiando durante las últimas dos décadas, especialmente en las autopistas concesionadas. En consecuencia, tres diferentes tipologías de superestructuras son identificadas. En los diseños más recientes, el diafragma se ha eliminado en algunos puentes, y la llave de corte, o tope lateral, de hormigón armado ha sido reemplazada por una llave de corte más débil de acero. Debido al terremoto, 10 puentes públicos de importancia colapsaron y 91 puentes concesionados, donde se incluyen 52 pasarelas peatonales, sufrieron daño o colapso. Se describe en este documento el daño observado en algunos puentes seleccionados en Santiago y Concepción. El comportamiento sísmico de los puentes fue afectado no solo por la configuración estructural sino también por el suelo de fundación. De este estudio se pudo concluir que la resistencia y rigidez provista por las llaves de corte de acero fue inadecuada, y que la ausencia de diafragmas es perjudicial para la integridad de la superestructura. Adicionalmente se concluyó que los puentes esviados son más susceptibles a la caída del tablero, y que el suelo de fundación tuvo un rol importante en el comportamiento sísmico de algunos puentes, especialmente en suelos blandos o suelos saturados que se comportaron de manera no drenada.

Palabras clave: puente, comportamiento sísmico, diafragma, llave de corte, puentes esviados

Introduction

In February 27, a magnitude M_w = 8.8 subduction earthquake struck the central south region of Chile affecting about 80% of the population of the country. From the earthquake and tsunami, 524 people were killed according to the Ministry of Interior (2010) and about 440.000 dwelling units were damaged or destroyed only in the regions of O'Higgins, Maule and Bío Bío (CEPAL 2010). The road infrastructure of Chile was severely affected and the estimated repair cost is 850 million dollars (MOP 2010b). Any disruption of bridges and road infrastructure after an earthquake impacts society because it makes vulnerable the terrestrial communication. In the case of Chile, the connectivity given by route 5 is critical and any disruption in this route is not desirable. Therefore, bridges should not only resist strong earthquakes, but should also be operative as soon as possible after the event. For this reason, there is special attention in the engineering community about the seismic design of bridges.

The road infrastructure of Chile consists of 80.400 km of roads from which 17.500 km are paved. From these paved roads, 2.500 km are concession roads which are characterized by having higher standards (MOP 2010b). The total number of bridges in Chile by 2008 was about 12.000, were 10.150 corresponded to public bridges and 1.850 corresponded to concession bridges (MOP 2008). After the earthquake, 10 public bridges collapsed (Lo Gallardo, Los Morros, Itata, Llacolén, Juan Pablo II, Bío Bío, Raqui I, Raqui II, Tubul y Quilicura), and 91 concession bridges suffered damages or collapse. From these concession bridges, 10 were overpasses, 14 underpasses, 15 bridges and 52 pedestrian bridges. These damaged bridges represented 1.6%, 2.9%, 2.3% and 11.8% of the total number of concession overpasses, underpasses, bridges and pedestrian bridges, respectively (MOP 2010b).

Before analyzing the observed damage in selected bridges around Santiago and Concepción, a background of seismic design of bridges is presented. Additionally, the three common types of superstructures in reinforced concrete bridges, which were indentified in the reconnaissance, are described.

Background on seismic design of bridges

The seismic design of Chilean bridges is based on the *Manual de Carreteras* (2002), which was developed by the Ministry of Public Works (MOP). This code was based on the Standard Specifications of Highway Bridges (AASHTO 1996), but has modifications related to particular Chilean conditions (*eg.* seismicity, foundation soils, construction materials and river hydraulics). The *Manual de Carreteras* was updated in 2010 (MOP 2010a) based on the newer version of the Standard Specifications of Highway bridges (AASHTO 2002). However, the Transport Office (*Dirección de Vialidad*) also accepts the use of AASHTO LRFD Bridge Design Specification (2004) as a design code for bridges.

Analogous to the AASTHO, the *Manual de Carreteras* classifies bridges in four seismic categories (a, b, c and d) based on three parameters: (1) the effective ground acceleration A_0, (2) the river bed seismic scour and (3) the bridge importance classification. This classification is shown in Table 1. The value of the effective ground acceleration is based on the seismic design code for buildings NCh433 (1996), which divides the country in approximately three parallel zones. Zone 1 corresponds to the eastern region located near the Andes Mountains, Zone 2 to the central valley and Zone 3 to the western coastal area. The effective accelerations of these zones, which are based on a probability of exceedance of 10% in 50 years, are 0.2g, 0.3g and 0.4g, respectively. The scour considerations are included to classify bridges in seismic categories because of the torrential characteristics of Chilean rivers. When a river stream exists, between 75% or 100% scour should be considered based on scour studies. However, MOP accepts scour to be between 50% and 75% only under justified circumstances. The bridge importance classification is defined by MOP considering two cases: I) essential bridges and structures and II) other bridges and structures.

Table 1: Seismic categories (*Manual de Carreteras* 2002)

Ao	scour, %	bridge importance	
		I	II
0,2g	0	a	a
	75	b	a
	100	b	b
0,3g	0	b	a
	75	c	b
	100	d	c
0,4g	0	c	b
	75	d	c
	100	d	d

For the seismic design of bridges, the *Manual de Carreteras* (2002) allows the use of four types of analyses: (1) seismic coefficient method, (2) seismic coefficient method modified by the structural response, (3) modal spectral analysis, and (4) time history analysis. For the modal spectral analysis the recommended spectrum given by the code can be used. Alternatively, a specific spectrum can be developed based on seismic hazard analysis. The minimum base shear given by the code depends on the selected analysis type, the soil characteristics, and the effective acceleration. However a minimum seismic coefficient of 0.1 is mandatory.

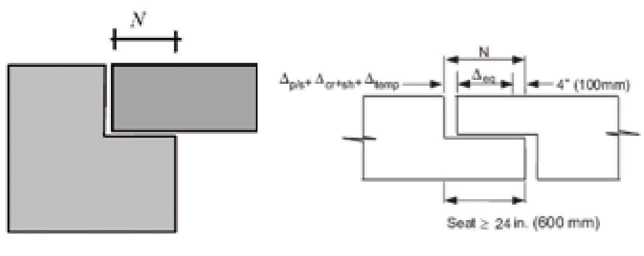
To obtain the design forces for the bridge elements, the forces obtained from the seismic analysis are divided by the modification factor R (*Manual de Carreteras* 2002). The modification factor values, which are given in the code, depend on the considered element and on the direction of analysis.

The seat width N, in abutments and piers, recommended by the *Manual de Carreteras* (2002) is based on the AASHTO recommendations. The minimum seat width N (in mm), shown in Figure 1a, is given by,

$$N \geq (203 + 1.67L + 6.66H)\cdot(1 + 0.000125\alpha^2) \quad \text{Categories a and b} \quad (1)$$

$$N \geq (305 + 2.5L + 10H)\cdot(1 + 0.000125\alpha^2) \quad \text{Categories c and d} \quad (2)$$

where L is the length in meters of the bridge deck up to the next expansion joints, or up to the end of the deck. In the case of an in-span hinge, L is the sum of the bridge deck length at each side of the hinge. For seats located in the abutments, H is the average height in meters of the columns supporting the deck, and $H = 0$ for single span bridges. For seats located in the piers, H is the average height in meters of both adjacent piers. Finally, the parameter α is the skew angle in degrees of the supporting points measured from a line perpendicular to the longitudinal direction of the beams.



(a) (b)

Figure 1: Definition of the seat width N a) *Manual de Carreteras* (2002) and b) Caltrans (2006)

The seat width recommended by the *Manual de Carreteras* (2002) is different than that recommended by Caltrans (2006). The seat width N (in mm), proposed by Caltrans where N should not be less than 600 mm, is

$$N \geq \Delta_{ps} + \Delta_{cr+sh} + \Delta_{temp} + \Delta_{eq} + 100 \quad (3)$$

where Δ_{ps} is the displacement attributed to pre-stress shortening, Δ_{cr+sh} is the displacement attributed to creep and shrinkage, Δ_{temp} is the displacement attributed to thermal expansion and contraction Δ_{eq} is the relative earthquake displacement demand,

$$\Delta_{eq} = [(\Delta_{D1})^2 + (\Delta_{D2})^2]^{0.5} \quad (4)$$

In equation (4), $\Delta_{D(i)}$ is the larger earthquake displacement demand for each of the adjacent frames of the bridge obtained from the analysis. To minimize unseating of the bridge decks, Caltrans (2006) recommends that the fundamental periods of vibration for adjacent bridge frames in the longitudinal and transverse direction satisfy the relation,

$$\frac{T_i}{T_j} \geq 0.7 \quad (5)$$

where T_i and T_j are the natural period of the more rigid and flexible frame, respectively. When equation (5) is not satisfied, the out-of-phase responses between adjacent frames increase. Therefore, the probability of longitudinal and transversal unseating and the probability of collision between adjacent frames at the expansion joints are increased. It is important to note that the period limit of equation (5) is not included in the *Manual de Carreteras* (2002).

Shear keys, or stoppers, are required in piers and abutments to provide lateral stability of the superstructure and they should be ductile enough to prevent unseating (*Manual de Carreteras* 2002). The shear keys should be taller than 30 cm and a gap between the shear key and the superstructure must be provided. The gap width should equal the seismic displacement plus 5 cm. However, it is not clear from the code how this displacement can be obtained. To design each shear key, half of the total transversal force of the superstructure should be considered. The design philosophy of shear keys suggested by Caltrans (2006) is somehow different from that of the Chilean code. The shear keys are designed to transfer lateral loads under service loads and small earthquakes. Therefore, additional lateral restraints such as strong pipes or sufficient seat widths have to be provided to avoid unseating during large earthquakes, especially in skewed bridges (Caltrans 2006). The idea of this philosophy is that the shear force transmitted to the piers and abutments is limited by the capacity of the shear keys. The design force V for the shear key recommended by Caltrans, which is different than that of the *Manual de Carreteras* (2002), is given by

$$V \leq 0.75 \sum V_{pile} \quad \text{and} \quad V \leq 0.3\ P_{dl}^{sup} \quad (6)$$

where $\sum V_{pile}$ is the sum of the lateral pile capacity and P_{dl}^{sup} is the axial dead load reaction at the abutment. In the case of abutments not founded on piles, for example on shallow footings, stoppers are designed with 0.3 P_{dl}^{sup}

Hold down devices are required by the *Manual de Carreteras* (2002) to anchor the superstructure to the piers and abutments. They are required to reduce the uplift of the deck which might reduce the unseating probability of the deck. The code requires hold down bars of at least 22 mm in diameter. The design forces of the hold down bars, given by the *Manual de Carreteras* (2002), is based on the vertical seismic coefficient K_v which is given by,

$$K_v = \frac{A_0}{2g} \tag{7}$$

Contrary to the *Manual de Carreteras* (2002), AASHTO (2004) requires hold down devices only when the vertical seismic forces on the girder's support are higher than 50% of the vertical reaction due to permanent loads. The design force of the hold down device is obtained with the larger load given by 10% of the reaction from permanent loads or 120% of the difference between seismic uplift forces and the reaction from permanent loads.

Finally, the *Manual de Carreteras* (2002) requires transversal diaphragms at the end of the deck to connect the I-beams or box girders. Diaphragms are also specified by AASTHO (2004) and are required to maintain the geometric section of the deck. However, the *Manual de Carreteras* (2002) establishes that diaphragms are not required in bridges located in seismic zones 2 and 3, if it can be demonstrated by numerical analysis that the bridge will behave adequately without a diaphragm.

Characteristics of Chilean bridge structures

Most of Chilean bridges are made of reinforced concrete and three types of superstructure are identified in the majority of these bridges, as it can be observed in Figure 2. For the three types of bridges, girders are connected to the piers or abutments using elastomeric bearing pads. The type 1 bridge corresponds to the traditional design which is characterized by having a concrete diaphragm connecting the girders, vertical hold downs and lateral concrete shear keys. The shear keys are installed at the edges of the diaphragm and some bridges also contained intermediate shear keys, which are not shown in Figure 2. The type 2 and type 3 bridges were introduced in Chile in the last two decades mainly by the concessions companies. The type 2 bridge is characterized by not having diaphragms, but it contains vertical hold downs and lateral concrete shear keys. Finally, the type 3 bridge does not have diaphragms neither vertical hold downs. For the latter type of bridge, the concrete shear keys are replaced by steel shear keys which are bolted to the pier transversal beam. A pair of shear keys is installed on each girder, as shown in Figure 2 for type 3. Since type 3 bridge does not have hold downs, the steel shear key is intended to prevent transversal displacement of the deck and also vertical support to the girder.

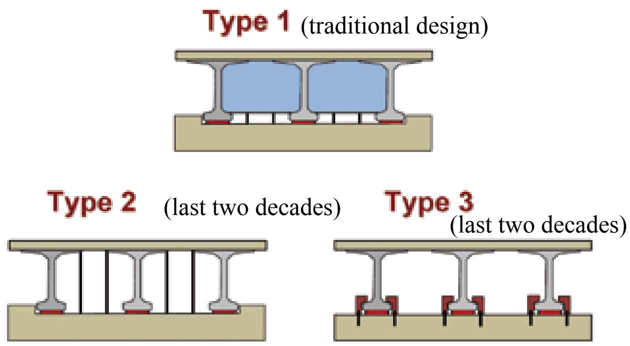


Figure 2: Types of superstructure configurations

Damage observed to selected bridges

This section summarizes the observed damage to selected bridges with the three types of configurations described above. At the end of the section, a detailed summary of the behaviour of the four bridges crossing the Bío Bío river is presented.

The bridges with better performance and that suffered the less amount of damage were mostly of type 1 configuration. The red ellipse in Figure 3a shows severe damage in the lateral shear key in the west pier of the Costanera Norte bridge over the Mapocho river. The presence of this damage suggests that energy was dissipated in the shear key during the earthquake. However, the bridge deck ended with residual displacement. Figure 3b shows damage in the lateral shear key of the older Lo Gallardo bridge, located in Llolleo and Figure 3c shows extensive damage in the lateral shear key of the Vespucio Norte bridge at Independencia. In the Vespucio Norte bridge, the vertical hold downs ended inclined and the effect of these bars on restraining the bridge collapse in this case may be questionable. From these observations, it can be concluded that the bridges with type 1 configuration behaved relatively good because they did not collapse and because damage was not induced to the piers and abutments.

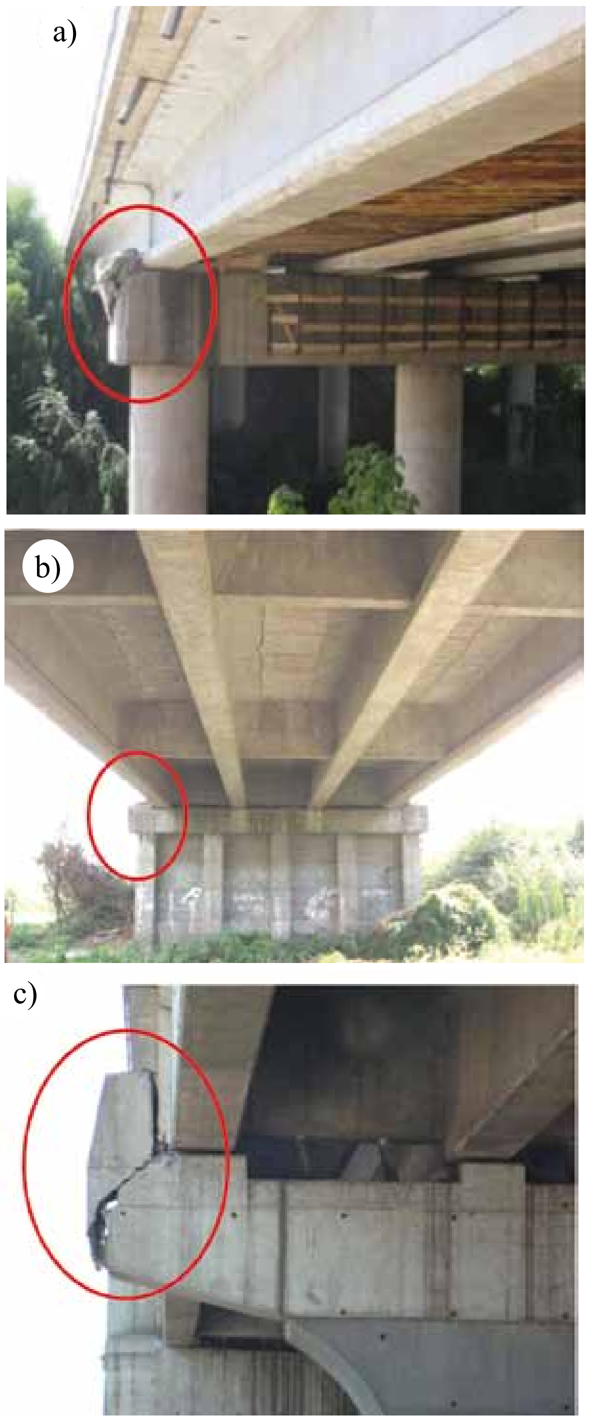


Figure 3: Damage in bridges of type 1 configuration: a) Costanera Norte, b) Llolleo and c) Vespucio Norte at Independencia

The type 2 bridges, which are characterized by not having diaphragm, suffered larger amount of damage than type 1 bridges. Figures 4a and 4b highlight in red ellipses the damage and displacement of Azufradero underpass located in the South access to Santiago. The girders of this bridge translated and rotated ending with a residual displacement Δ_h of about 0.5 m, as shown in Figure 4a. The vertical hold downs ended with a residual angle θ of about 45º. For this bridge, the girder impacted the shear key of the central pier causing damage to the web of the edge girder, as shown in Figure 4b.

The underpass Chada located at the South access to Santiago suffered considerable lateral displacements Δ_h of the deck and failure of the embankment connecting to the abutment, a settlement of the embankment Δ_v of 60 cm was estimated as highlighted by the ellipse in Figure 4c.

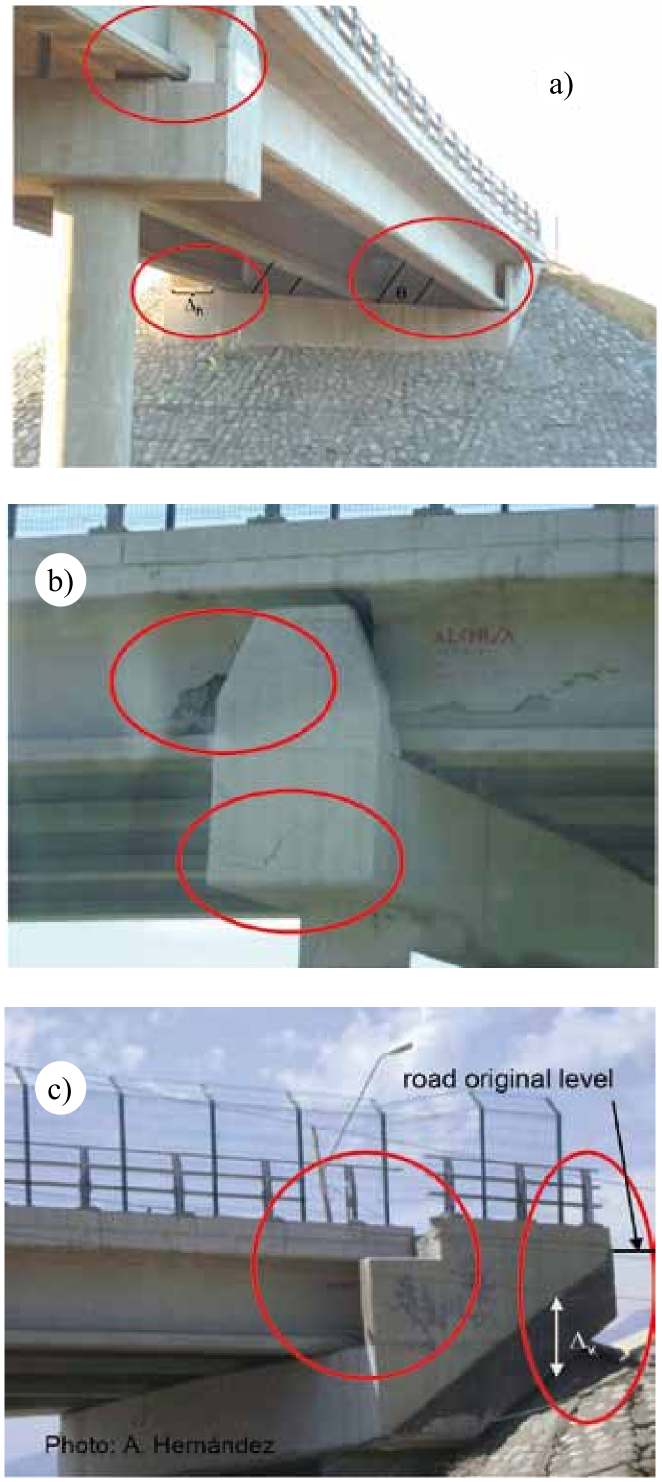


Figure 4: a) and b) Damage in bridge of type 2 configuration, underpass Azufradero, and c) embankment settlement in underpass Chada.

Another type of damage that was observed in type 2 bridge configurations occurred at the girders. Figures 5a and 5b show serious damage in girders of the underpass Chada. Due to the earthquake motion, the girders impacted the shear key, which caused damage in both the girders and the shear key. In fact, Figure 5b shows a complete bending failure of the girder in the weak axis. This type of damage in the girders is not desirable because the girders may not be reparable and a new deck is required to repair the bridge. It is concluded that the use of a diaphragm in this bridge would have distributed the horizontal load of the impact throughout every girder of the deck, minimizing the damage in the edge girder.

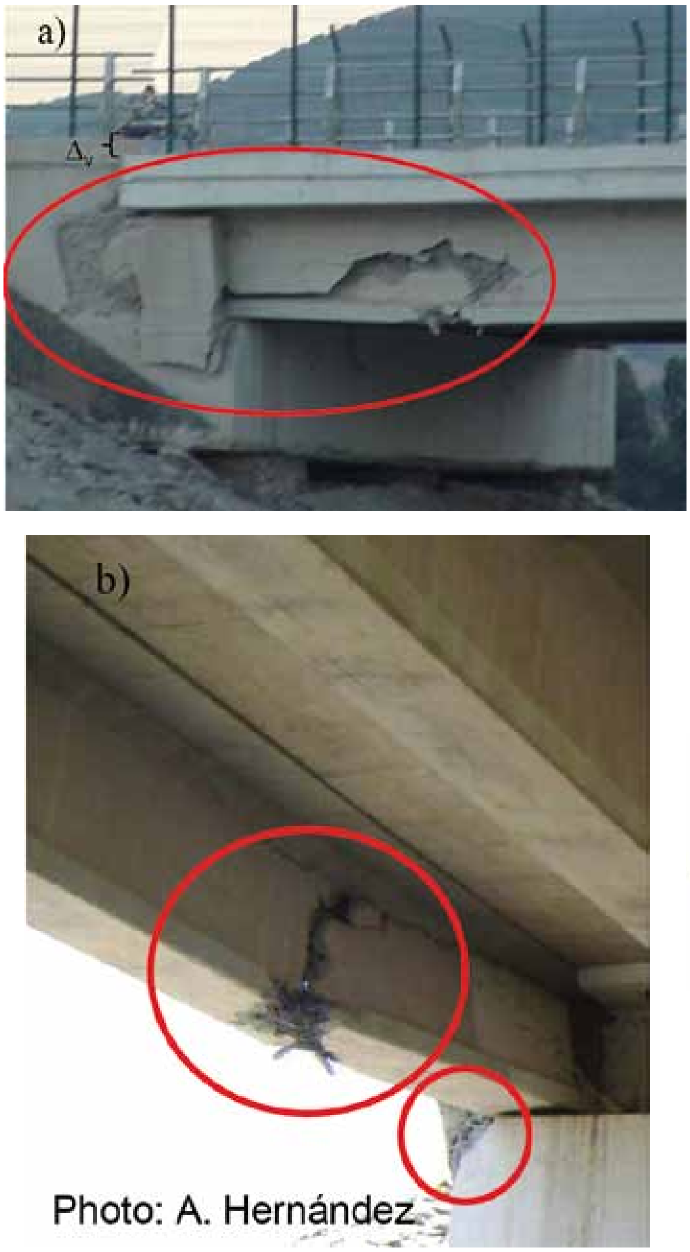


Figure 5: Damage in girders of type 2 bridge configuration, underpass Chada, South access to Santiago.

Las Mercedes bridge, of type 2 configuration, is located in bypass Rancagua and it was seriously damaged. A drawing of this bridge is shown Figure 6a. Since the bridge has two spans with a central pier, this cross section is representative of several bridges located in Route 5. In this case, the abutments and pier are founded on shallow footings resting on compacted granular material, gravel and sand with relative densities higher than 80%. The observed damage in this bridge is shown in Figure 6b, where the girders underwent horizontal displacement and they unseated from the abutment. As a result, the lateral shear key was completely destroyed at the abutment, and the vertical hold downs were unable to control the vertical acceleration in order to minimize the lateral displacement. Fortunately, the girders of this bridge were supported by the embankment, allowing traffic under the bridge. The extensive damage observed in this bridge could be explained in part by the subsoil characteristics. Based on the information of the drawing, the foundation subsoil is a saturated and soft material composed mainly of sand and silt. Although the geotechnical exploration only reaches 5 m depth, perhaps this material may even extend deeper. Under this condition, it can be explained in part that the severe lateral and vertical displacements experienced by the bridge deck may be associated to seismic amplification of the saturated and soft subsoil.

The collapse of Los Pinos underpass, type 2 bridge configuration, is shown in Figure 7. This bridge is located in the South access to Santiago and the deck collapsed in both spans. The shear keys on the abutment failed and the girders fell down due to rotation of the deck and due to insufficient seat width.

Figure 6: Damage in Las Mercedes underpass, bypass Rancagua, route 5.

Figure 7: Collapse of Los Pinos underpass, South access to Santiago.

The type 3 bridge configuration, which is characterized by not having diaphragm and by having steel shear keys, was the most damaged type of bridge of the concession roads. The detail of the typical steel shear key, which is also used as a vertical hold down, is shown in Figure 8a. To prevent the lateral displacement of the girder, the steel shear key is connected to the abutment or pier using two bolts. It can be concluded that the lateral stiffness of the shear key is low because it can rotate through an axis parallel to the longitudinal axis of the bridge that pass through the bolts, as shown in Figure 8a. Figure 8b shows the lateral displacement of the bridge deck of Vespucio Norte at Independencia. The lateral displacements of the girders are marked with arrows and red ellipses in Figure 8b. The deck displaced laterally towards the south about 20 cm destroying the shear keys and leaving the deck in a serious risk of collapse, as shown in Figure 8c. Due to this risk, the bridge was shored to allow traffic underneath.

The bridges at Lo Echevers and Miraflores in Vespucio Norte, of type 3 configuration, collapsed both in a similar catastrophic manner. At Lo Echevers only the east bound of the overpass failed, whereas both bounds failed at the overpass Miraflores. The collapse of the east bound of Lo Echevers overpass is shown in Figure 9a. The collapse of this bridge was caused by the failure of the lateral shear keys and the rotation of the deck as shown in Figure 9c. The cause of the rotation of the deck is attributed to the angle of skew, which is about 33° in this bridge (Figure 9b). To reduce excessive rotation of the deck in a 50º skewed bridge, Watanabe and Kawashima (2004) studied three different cable restrainers as possible solutions based on analysis of rotation mechanisms. Finally, the extensive damage in Lo Echevers and Miraflores could also be attributed to the eolic soil deposits which are much softer than the gravel deposits usually found in others areas of Santiago.

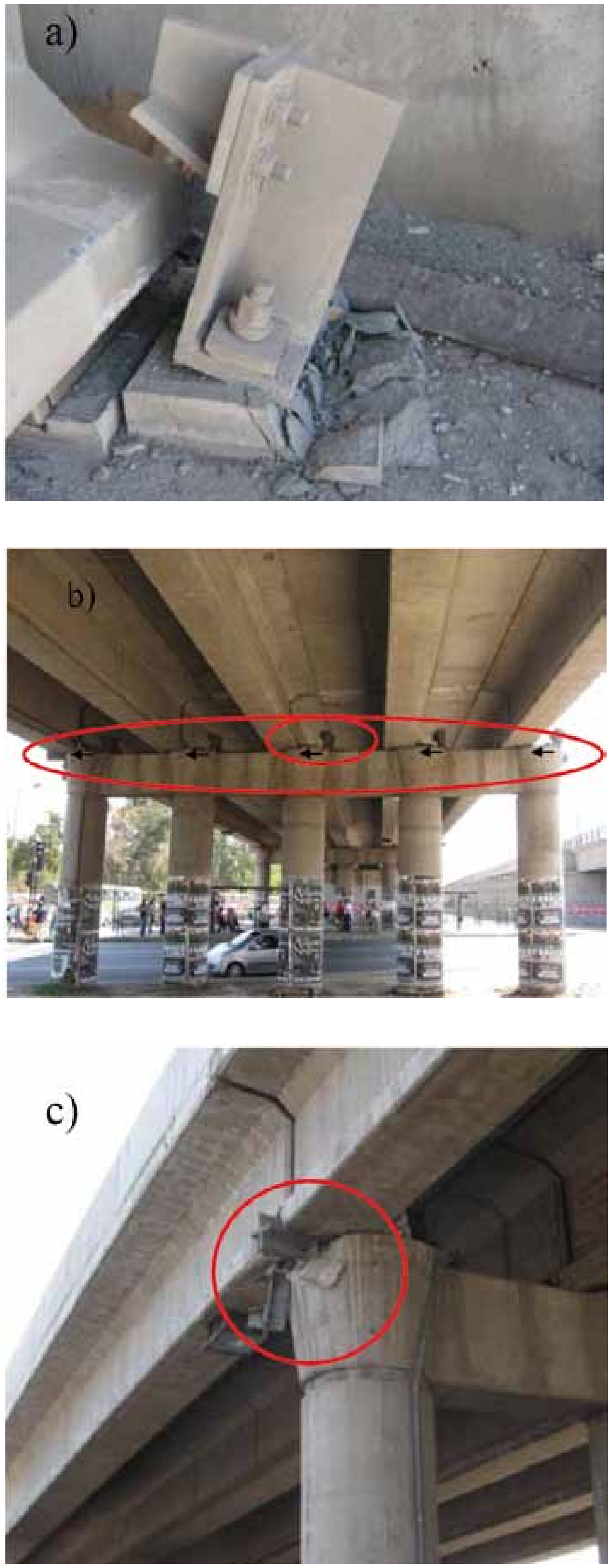


Figure 8: Damage in bridges of type 3 configuration: a) shear key failure detail b) and c) lateral displacement of girders in Vespucio Norte at Independencia.

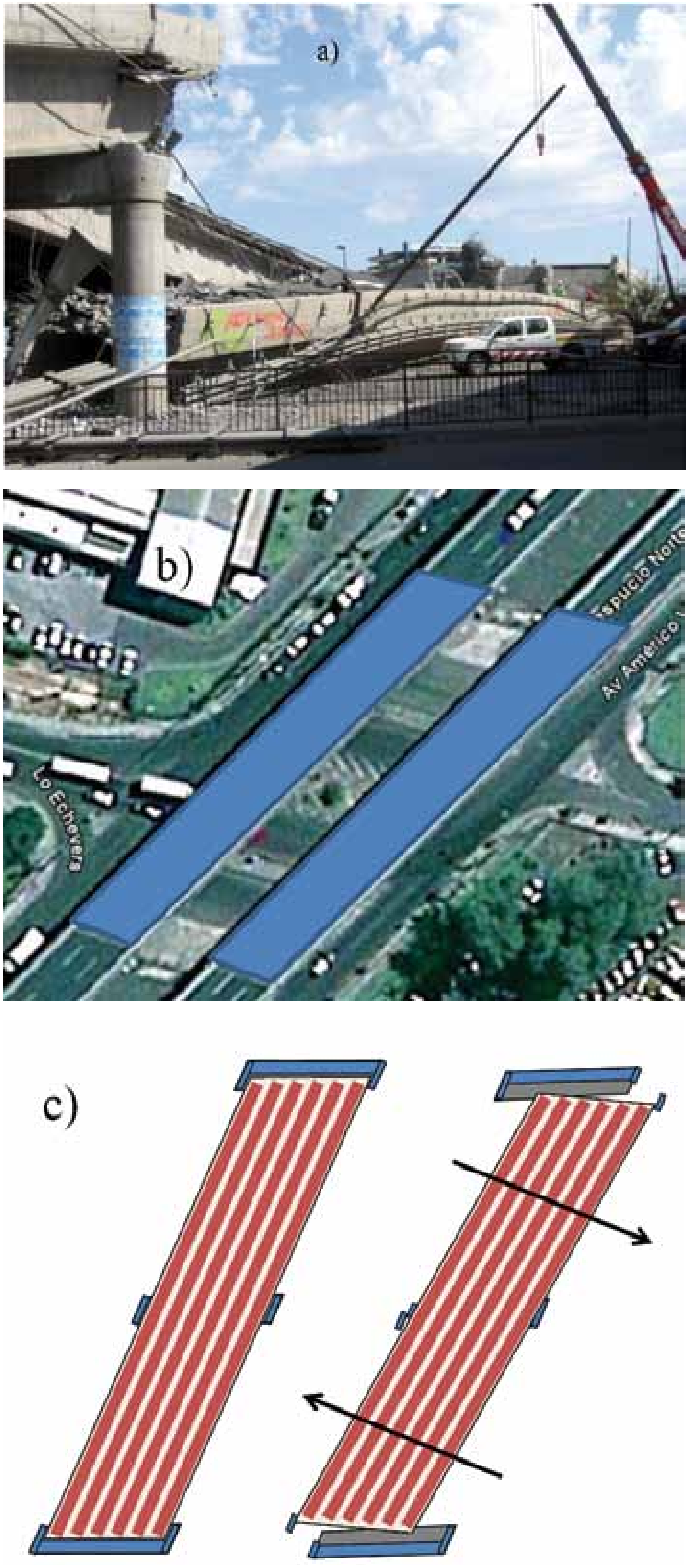


Figure 9: Lo Echevers junction a) bridge collapse, b) plan of skewed bridges and c) deck rotation

The four bridges crossing the 2 km wide Bío Bío River to connect Concepción with San Pedro de la Paz and subsequently with the province of Arauco, suffered different types of damage. These bridges are founded in a fine volcanic sandy soil, which is transported by the river from the Andes Mountains. The bridge with the lesser amount of damaged was the railway bridge, which was built in 1889. This bridge is structured with steel trusses and is founded on very deep laminated steel piles. The major problem of this bridge was the lack of rail alignment due to settlements and lateral displacements. This alignment was very sensible to aftershocks, which allowed the train service only after five months of repairs. Additionally, damage occurred in the abutment of the Concepción side caused by the displacements of a retaining structure of the road passing under the bridge. Figure 10a shows that the pedestrian path under the bridge settled about 0.5 m because of the lateral displacement and differential settlement (about 0.2 m) of the road retaining wall.

Figure 10: a) Abutment settlement and lateral displacement in Concepción railway bridge and b) collapse in the access to the Llacolén bridge in the Concepción side

The Llacolén bridge, built in 1999, is the newest bridge over the Bío Bío River. Beyond the Concepción side abutment, each pier is founded on a transversal row of six piles of 1.2 m in diameter. The bridge did not have diaphragms connecting the girders (type 2 configuration). The damage in Llacolén bridge is shown in Figure 10b,

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where the girders unseated from the piers at the Concepción side. In spite of this collapse, traffic could still be established by connecting the bridge deck with a lateral access from a perpendicular road. The other symmetric access was not functional for cars due to gaps between decks of up to 30 cm. Since this was the only available bridge to cross the Bío Bío river, a prefabricated temporary steel bridge was installed a few weeks later at the Concepción side to make possible a straight cross onto the bridge. The temporary steel bridge lasted about five month until the final reparation.

The Juan Pablo II Bridge suffered serious damage due to foundation differential settlements and structural failure of some piers and abutment. This bridge had been used since 1974 and was designed by the English company E.W.H. Gifford & Partners. This bridge, of type 1 configuration, is structured with piers having two columns founded on piles no more than 15 m deep. At the time of the earthquake, the bridge was under repair to fix a hole that passed through the slab of the deck. Figure 11a shows the bridge from the Concepción side, where differential settlements and resulting unevenness of the bridge deck can be observed. These

Figure 11: Juan Pablo II Bridge showing a) unevenness due to pier differential settlements and b) shear failure in reinforced concrete piers

differential settlements induced rotation of the piers causing transversal and longitudinal unevenness in the bridge superstructure. It may be concluded that the diaphragm of this bridge helped to keep the integrity and avoid unseating, which was not the case of the Llacolén bridge. The differential settlement, which occurred in four sections of the bridge, can be explained by the occurrence of soil liquefaction (GEER 2010). Additionally, in the Concepción side, shear failures were observed in reinforced concrete columns of the cap beams. These pier shear failures induced structural settlements of the bridge deck of about 50 cm to 70 cm, as shown in Figure 11b. In spite of the severe damage of this bridge, it was open for pedestrians, bicycles, motorcycles and even a few cars. However the traffic had to deal with the uneven surface when crossing the bridge.

Lastly, the "old" Bío Bío Bridge totally collapsed during the 2010 earthquake. It was built during the 1930's, but was open to traffic in the early 1940's. It was the first bridge for car vehicles over the Bío Bío River. The bridge length was 1650 m over the river and it had an extension of 220 m over a floodable area. The bridge decks were simply supported on hollow reinforced concrete piers by means of steel beams and the piers were founded on wood piles. According to Steinbrugge and Flores (1963) two spans of the bridge collapsed during 21st May 1960 and three spans collapsed during the 22nd May 1960 earthquake, on the Concepción side. Parts of the bridge colapse after the 2010 earthquake are shown in Figure 12, where many decks, not well restrained to lateral and longitudinal movements, fell down. As a consequence of the strong shaking, an initial deck fell down damaging the pier and causing the adjacent deck to fail and so on. It can be concluded that this bridge was not designed for strong earthquakes because of the inadequate lateral and longitudinal restraint of the bridge decks. Additionally, possible effects of liquefaction, amplification or foundation failure could also have occurred at this time, as it was suggested for the 1960 earthquake by Steinbrugge and Flores (1963).

Final remarks

The observed damage of selected bridges, after the 27 February 2010 earthquake, was presented in this paper. For the analysis of the damage, special attention was taken to three different bridge configurations that were identified

Figure 12: Sections collapsed of the Bío Bío Old Bridge

in the reconnaissance work. Bridge design has been changing in the last two decades, especially in concessions roads. The traditional superstructure design using diaphragms and reinforced concrete shear keys have been changed to more economic superstructures without diaphragms, neither vertical hold downs and with steel shear keys, which is defined as the type 3 bridge configuration in this paper. The type 3 configuration performed inadequately causing severe bridge damage and also collapses. The strength and stiffness provided by the steel shear keys was inadequate, and the absence of diaphragms was found to be detrimental to the integrity of the superstructure. Additionally, the absence of the diaphragm caused undesired damage in prestressed concrete girders because of the impact between the girder and the lateral shear keys or stoppers. It is also concluded that skewed bridges are more susceptible to collapse because the deck rotation in bridges with inadequate lateral restraint can result in unseating.

The foundation soil played an important role on the seismic behaviour of bridges, especially for soft and saturated soils responding in an undrained condition. This type of soils may have amplified the ground motion and may have reduced the undrained shear strength leading to lower bearing capacity of abutments and piers foundations.

Finally, it is concluded that bridges built before the 1990's, which followed the traditional use of diaphragms and concrete shear keys performed better than newer designs. For this type of bridges, the effectiveness of the vertical hold downs to restrict the bridge uplift is questionable and more research is require to understand the real benefit of these hold down elements.

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