Bridge Lessons Learned from the Chile Earthquake in 2010

by

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ABSTRACT

On February 27, 2010, a devastating earthquake, measuring 8.8 on the Richter scale, struck off the coast of the Maule region of Chile affecting a large area including Chile's two biggest population cities: Concepcion and Santiago, the Chilean capital. A Transportation Infrastructure Reconnaissance Team (TIRT) was soon organized by the Federal Highway Administration of the U.S. Department of Transportation, which performed a thorough post-earthquake investigation of highway infrastructure from April 4 to April 13, focusing on structural and geotechnical concerns on and around bridges, and on retaining walls,. The reconnaissance team was greatly assisted by the Ministry of Public Work of Chile and two local Universities in Chile: University of Catholic and University of Chile (both located in Santiago). This paper presents the summary of the preliminary findings of the earthquake performance of the transportation infrastructure which the team visited during the reconnaissance.

INTRODUCTION

Background

The country of Chile has experienced earthquakes throughout its history. Records from the USGS indicate that since the beginning of the 20th century there has been numerous earthquakes of magnitude 8.0 or greater. Specifically earthquakes of magnitude 8.2 occurred during the years 1906, 1943 and 1960; a magnitude 8.0 in 1985; a magnitude 8.2 in 1960 which was a foreshock that occurred the previous day to the great magnitude 9.5 Chilean earthquake. The M9.5 Chilean earthquake is the largest earthquake ever recorded in the history of the world.

The Maule offshore earthquake on February 27, 2010, was of magnitude 8.8 and lasted more than 2 minutes. It is estimated that this earthquake was approximately 520 times more powerful than the earthquake that devastated Haiti in January 2010. Several aftershocks, including 9 events with a magnitude exceeding 6.0, occurred in the days following. The M8.8 event is the fifth largest earthquake recorded in modern times. Many bridges and tunnels, constructed with seismic design codes similar to the current US and European codes, performed well while many were also damaged. This earthquake was characterized by its very long duration, strong ground motion, and which additionally created tsunamis across the region.

Because of the size of the event, the intensity of the ground shaking, the Chilean geological similarity in the Maule region to Northwestern United States (Washington and Oregon States' subduction zones), and the comparable nature of the infrastructure construction and seismic design codes used, the potential existed to learn much on the performance of transportation infrastructure with regards to earthquake engineering. In recognition of the importance of this earthquake to the US, the FHWA contacted the Ministry of Public Works of Chile, and organized a team representing federal, state and academia with knowledge of earthquake engineering and performance.. This FHWA Transportation Infrastructure Reconnaissance Team (TIRT) was dispatched to Chile on April 3rd to perform comprehensive earthquake reconnaissance with a joint member from the Earthquake Engineering Research Institute (EERI). The TIRT was supported by the local bridge engineers from the Ministry of Public Works (MOP) and academic researchers from the Pontificia Universidad Católica de Chile and the University of Chile.

The TIRT visited more than 32 transportation infrastructure sites, including highway bridges and port facilities located from the city of Santiago down to the city of Tubul (the southern city around the second largest city of Chile, Concepcion). The specific locations, identified as a white dot with a black center and a white number, are shown in the Figure 1. Table 1 gives more details on these sites including names, GPS locations and city/ county names.

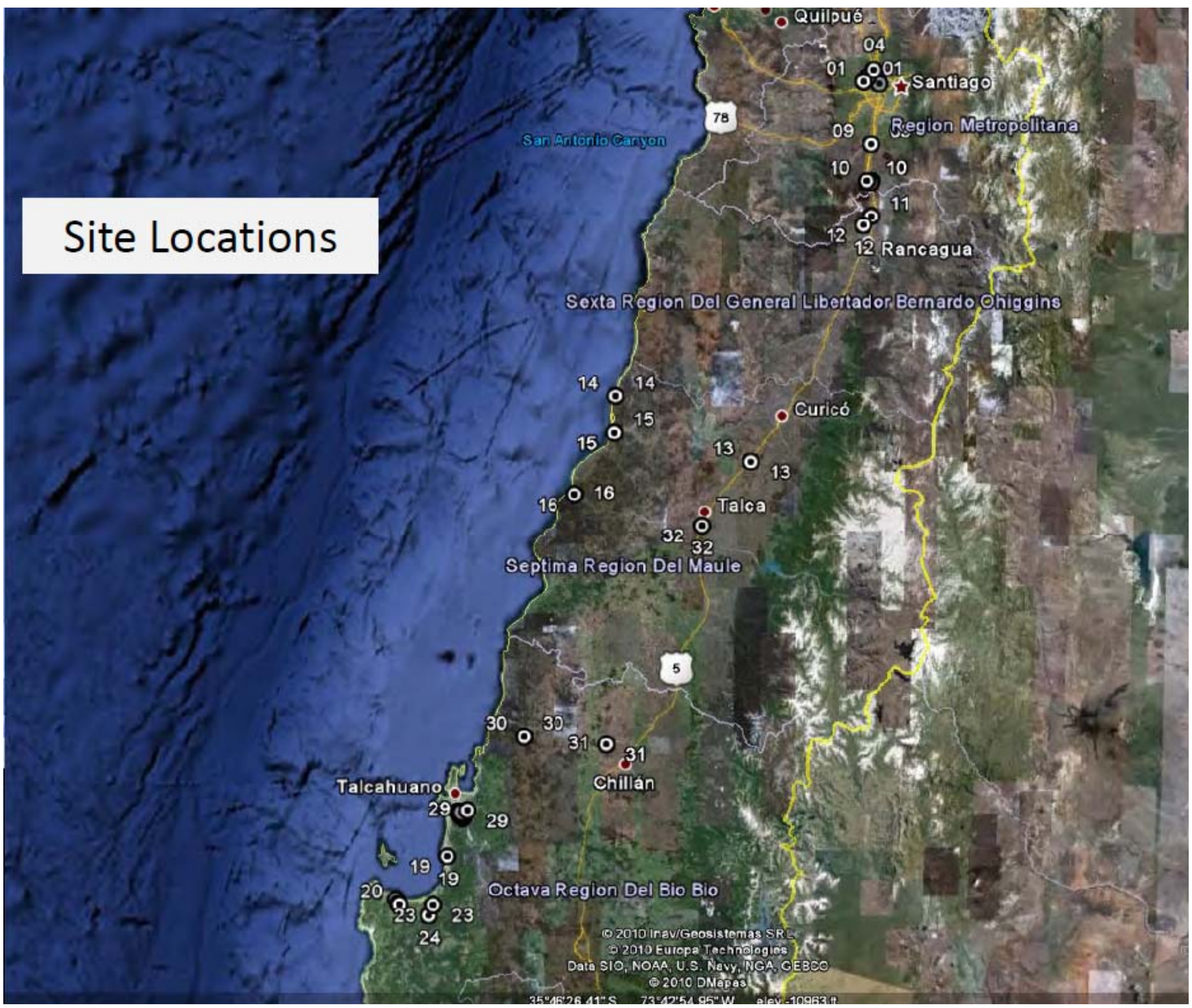


Figure 1 Site locations visited by the TIRT

Table 1 Specific site’s structural names and GPS locations

Site No.	Site Name	Location	Lat (S) (deg)	Long (W) (deg)
1a	Américo Vespucio/Miraflores eastbound	Santiago	-33.39	-70.77
1b	Américo Vespucio/Miraflores westbound			
2a	Américo Vespucio/Lo Echevers eastbound	Santiago	-33.38	-70.75
2b	Américo Vespucio/Lo Echevers westbound			
3	I-5/14 de la Fama	Santiago	-33.40	-70.68
4	I-5/KM 13.8 Pedestrian bridge	Santiago	-33.34	-70.71
5	M. Antonio Matta/Quilicura Railway Crossing	Santiago	-33.37	-70.70
6a	Américo Vespucio/Independencia westbound	Santiago	-33.37	-70.69
6b	Américo Vespucio/Independencia eastbound			
6c	Exit ramp at westbound traffic			
6d	Entrance ramp at westbound traffic			
7	Avenida Romero Accesso Sur Km 42	Paine	-33.86	-70.72
8	Avenida Chada Accesso Sur KM 43.4	Paine	-33.87	-70.73
9a	I-5/Maipu (Viejo – old bridge)	Buin	-33.69	-70.72
9b	I-5/Maipu (current Rt.5 Bridge)	Buin	-33.69	-70.72
9c	Maipu Rairoad Crossing	Buin	-33.69	-70.72
10a	I-5/Paso Superior Hospital westbound	Buin	-33.86	-70.75
10b	I-5/Paso Superior Hospital eastbound			
11	Estribo Francisco Mostazal (Avenida Independencia)	Mostazal	-34.03	-70.72
12	I-5/Paso Inferior Las Mercedes	Rancagua	-34.07	-70.76
13a	I-5/Rio Claro	San Rafael	-35.18	-71.39
13b	I-5/Rio Claro			
14	Pichibudis	Iloca	-34.88	-72.16
15	Mataquito	Iloca	-35.05	-72.16
16	Cardenal Raúl Silva Henríquez	Constitución	-35.34	-72.39
17	Llacolén	Concepción	-36.83	-73.07
18	Cruce Ferroviario Cerro Chepe Cruce Rio Biobio	Concepción	-36.82	-73.07
19a	Puerto Coronel Muelle Norte	Coronel	-37.03	-73.15
19b	Puerto Coronel Muelle Sur			

Site No.	Site Name	Location	Lat (S) (deg)	Long (W) (deg)
20	Raqui 1	Raqui	-37.25	-73.44
21	Raqui 2	Raqui	-37.25	-73.44
22	Tubul	Raqui	-37.23	-73.46
23	El Bar	Arauco	-37.26	-73.24
24a 24b	Ramadillas (old) Ramadillas (new)	Arauco	-37.31	-73.26
25	Juan Pablo II	Concepción	-36.82	-73.09
26	Old Biobio	Concepción	-36.84	-73.06
27	La Mochita	Concepción	-36.85	-73.06
28	Via Elevada 21 de Mayo/Cruce Ferroviario	Concepción	-36.82	-73.07
29	Rotonda General Bonilla	Concepción	-36.81	-73.03
30	Itata	Coelemu	-36.47	-72.69
31	San Nicolás	San Nicolás	-36.50	-72.21
32	Muros Talco (SW)	Talca	-35.48	-71.67

The Transportation Infrastructure Reconnaissance Team (TIRT)

Objective/ Mission

The Transportation Infrastructure Reconnaissance Team's mission was to conduct a thorough post-earthquake investigation concentrating on structural and geotechnical issues on and around bridges, and on retaining walls in the areas affected by the earthquake, including the cities of Concepcion and Santiago. Performance including damaged and non-damaged conditions, were to be carefully documented and analyzed. The lessons learned and information received from this particular reconnaissance were to be studied and the results used to assess, refine and improve current design codes and standards that benefit both countries and the general engineering community.

Team Members

The team members of the TIRT were comprised of 6 members led by the FHWA Office of Infrastructure R&D and included three representatives from the FHWA, one from the State DOT representing AASHTO, and two from academia.

The TIRT members included (Figure 2):

- Federal Highway Administration: Dr. W. Phillip Yen (Team Leader), Dr. Jeff Ger &Mr. Daniel Alzamora

- Washington State Department of Transportation: Mr. Tony Allen
- University of Nevada: Prof. Ian Buckle
- Missouri University of Science and Technology: Prof. Genda Chen

Mr. Juan Arias with support from the Earthquake Engineering Research Institute also joined the TIRT. Local support was received from Mr. Mauricio Guzman, and Ms. Sandra Achurra of the Ministry of Public Works of Chile; and Mr. Rodrigo Oviedo of the Pontificia Universidad Católica de Chile. Figure 2 shows this group as they are getting ready to head out for the investigation.

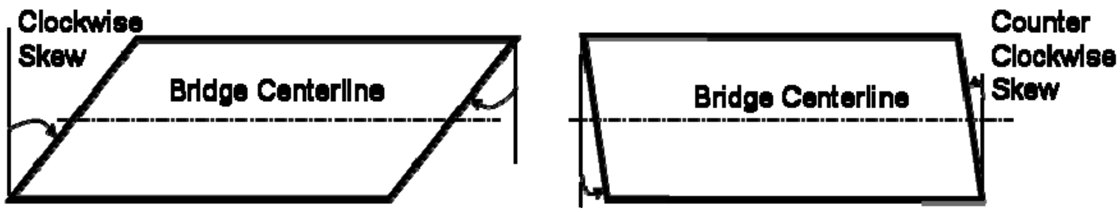
Figure 2 Team members of the TIRT and supporting members from Chile

LESSONS LEARNED FROM THE EARTHQUAKE

Super Structure Rotation

Skew Bridges

For clarity in the following discussions, the direction of the skew of a bridge is illustrated in Figure 3. It is defined as the direction of rotation from the transverse line (perpendicular to the bridge centerline) to the skew side or abutment back wall of the bridge. For example, figure 3(a) indicates a clockwise skew and figure 3(b) is for counter clockwise rotation.



(a) Clockwise skew (b) Counter clockwise skew

Figure 3 Notations on skew direction

Table 2 summarizes the characteristics and the earthquake-induced damage patterns of the bridges with 20° or more skews. It lists the bridge name, bridge orientation, skew angle, skew direction, material used in girder, presence of diaphragms, transverse displacement at intermediate bents, diagonal symmetry (diagonal line rotating), and direction of deck rotation. The combination of these parameters will help determine the mechanism of bridge deck rotations.

Table 2 Summary on bridges and bridge damage

Bridge Characteristics				Damage Pattern		
Name	Orientation	Skew/ Direction	Girder/End Diaphragm	Transverse Movement @ Intermediate Bents	Diagonal Symmetry	Rotation
Miraflores	NE-SW	20°/Counter clockwise	Concrete/No.	Small	Symmetric	Clockwise
Lo-Echevers	NE-SW	33°/Counter clockwise	Concrete/No.	Small	Symmetric	Clockwise
Romero	E-W	31°/Clockwise	Concrete/No.	Negligible	Symmetric	Counter clockwise
Hospital	NW-SE	40°/Counter clockwise	Concrete/No.	Significant	Symmetric	Clockwise
Quilikura	E-W	45°/Counter clockwise	Steel/Yes	Negligible	Symmetric	Clockwise

It can be clearly observed that all bridges listed in Table 2 consistently rotated about the centroid of the bridge superstructure in the opposite direction to their respective skew direction, regardless of bridge orientation, skew angle and direction, and presence of diaphragms. The fact that most of the bridges experienced small or negligible transverse displacements also indicates the dominant rotation effect in bridge superstructures. As a result, the acute corners of each bridge moved away from their abutments at both ends.

Based on the previous observations, the movement of a bridge superstructure can be illustrated in four steps in figure 3. Under the earthquake excitations (1a, 1b, and possibly 3 in figure 3), the bridge superstructures first moved towards one abutment (left as shown in figure 3) and impacted against the abutment back wall (2 in figure 3). The reaction from the back wall then turned the superstructure in a direction opposite to the skew direction (counter clockwise in figure 3). The rotational motion (3 in figure 3) was amplified due to the fact that the rotational vibration mode of the bridge superstructure is more sensitive to the ground motions as illustrated with the acceleration response spectra recorded at the Hospital Station in Curico, figure 4, since all the concrete girder bridges listed in Table 2 have superstructures supported on neoprene pads and restrained with vertical seismic bars. The superstructures are weakly restrained in plan with the fundamental vibration mode in translation. With continuing deck rotations, the acute corners at two ends of the bridge finally moved away from the abutments, knocking off the curtain walls and becoming unseated (4a and 4b in figure 4). Note that the possibility of having significant rotational ground motions at the bridge sites can further amplify the rotational motion (3 in figure 4).

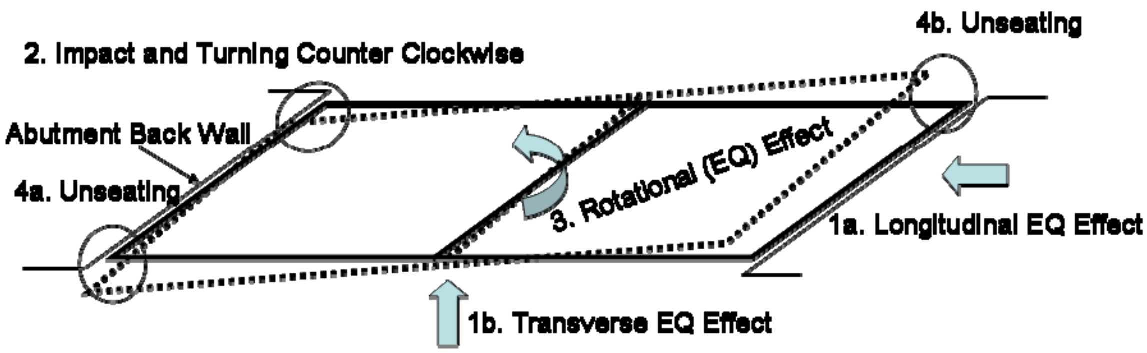


Figure 4 Deck rotation of a representative bridge (two spans shown in plan)

Straight Bridges

For the bridges with little or no skew (Chada and Las Mercedez), they rotated counter clockwise. Unlike the skewed bridges, the deck rotation of straight bridges cannot be explained by the skew effect. The possible factors contributing to significant rotations in these bridges are listed below:

1. The rotational mode of vibration of those bridges is very sensitive to ground motions. Any accidental eccentricity between the center of mass and the center of rigidity of the superstructure of a bridge could lead to substantial rotations.
2. The rotational component of ground motions could be significant.
3. The fault directivity effect could be significant since both bridges are approximately oriented along the E-W direction.

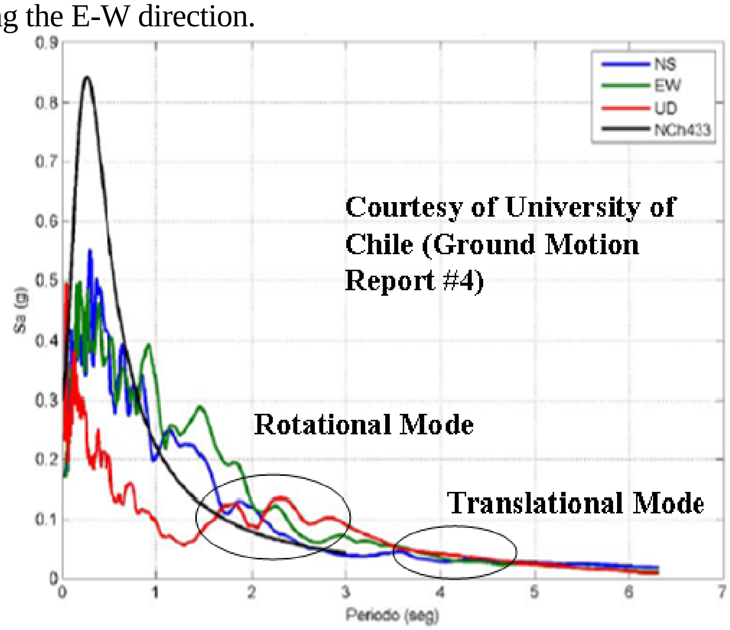


Figure 5 Rotational and translational mode responses to ground motions (University of Chile).

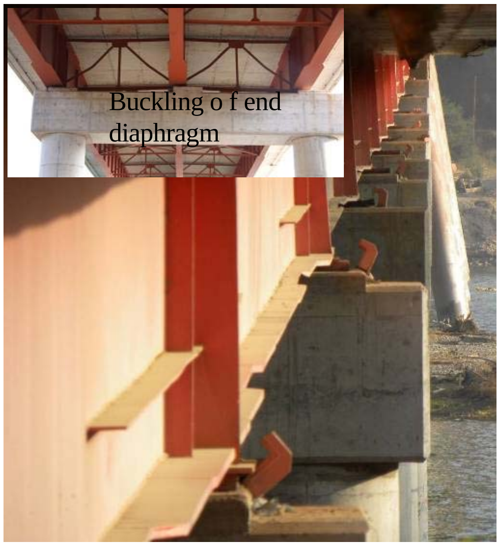
Considering both the skewed and straight bridges discussed, observations can be made on the overall plausible reasons for bridge deck rotation. Skew of bridges is a significant but not necessarily decisive factor contributing to the bridge rotation. The high sensitivity of rotational vibration mode of the bridge designs to ground motions, particularly rotational excitations, could be dominant. The fact that all bridges experiencing significant rotations are not far away from Santiago where soil conditions are relatively stiff according to the Chilean microzonation map also supports the possibility of having rotational ground motions at the bridge sites. Further analysis is required to understand the significance of rotational ground motions.

Girder Damage

Fracture of Steel Girder

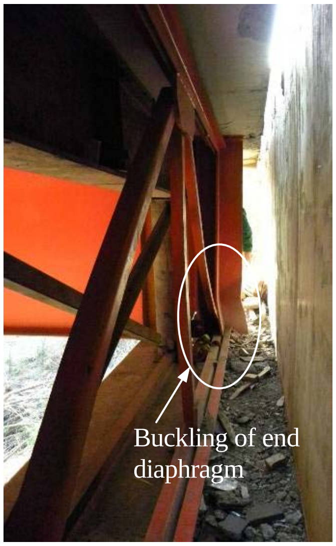
The superstructure of the Cardenal Raul Silva Henriquez Bridge is basically divided into two parts by the center expansion joints at bent 11. The NE portion of the bridge is supported by a concrete substructure and the SW portion is mainly supported on a steel substructure. Most bents of the bridge are supported on drilled shafts. During the earthquake, the two parts most likely vibrated separately.

The most plausible reason for the girder damage at each abutment, is excessive longitudinal force applied on the end support. During the earthquake, the majority of the inertia force on half of the bridge superstructure was resisted by the end support at each abutment. The excessive force resulted in either fillet weld fractures at the SW abutment or steel girder fractures in web and bottom flange at the NE abutment.



(a) Girder offset and crossframe buckling

(b) Girder damage at north abutment



(c) Crossframe buckling

(d) Temporary repair at NE abutment

Figure 6 Damage to the superstructure of the Cardenal Raúl Silva Henríquez Bridge.

Failure of Concrete Girders

Exterior PC girders in Chada and Romero bridges without diaphragms experienced out-of-plane block shear failures due to transverse impact loads from shear keys as illustrated in figure 7(a). When partial diaphragms are used between girders in bridges such as San Nicholas bridge, the bottom portion of an exterior girder can still experience significant shear crack as shown in figure 6-3(b). With the use of even partial concrete teeth between girders as seen in figure 7(c), both exterior and interior PC girders suffered no visible damage. The most severe damage occurred in the west portion of LLacolen bridge was a shear crack on one interior girder. This is because the concrete teeth provide sufficient lateral restraints on most of the PC girders, making them work together and share the transverse seismic force.

(a) Chada bridge with no diaphragms

(b) San Nicholas bridge with partial diaphragms

(c) West abutment of LLacolen bridge with concrete teeth

Figure 7 Exterior PC girder damage

Connections between Superstructure and Substructures

Shear key and Steel Stopper Failures

Concrete shear keys or shear keys in integral construction with curtain walls performed well as sacrificial devices to protect the substructure of bridges. Their failures were observed regardless of the presence of bridge deck rotations during the earthquake (e.g. Independencia, Chada, Romero, and Hospital bridges).

Steel stoppers used in several bridges (e.g. Independencia, Miraflores, and Lo Echevers bridges) failed prematurely. The Independencia bridge with steel stoppers was out of service while the parallel bridge with concrete shear keys and diaphragms survived the earthquake with repairable damage. The two-bolt connection from each steel stopper to capbeams is too weak to resist any significant bending moment. However, once welded to steel girders, steel stoppers appeared functioned well to prevent lateral movement of girders as observed in Quilikura bridge. (See Figure 8)

(a) Collapsed bridge

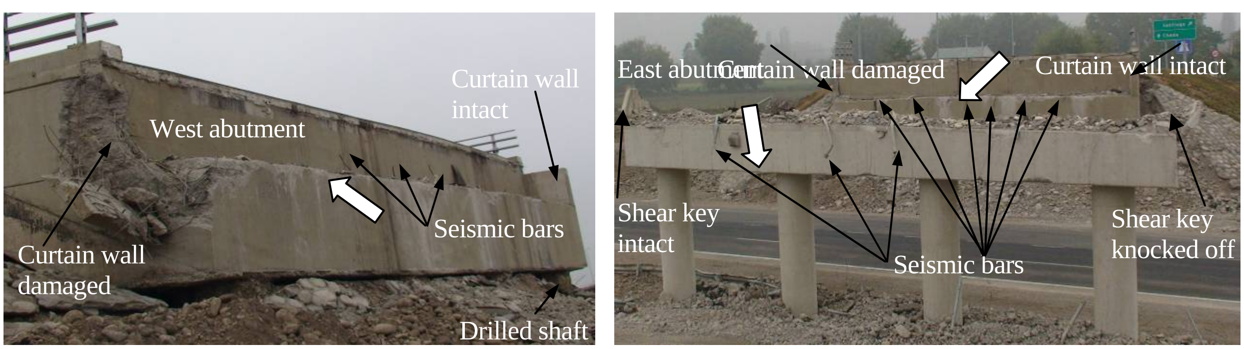
(b) Three unseated spans

(c) Displaced elastomeric bearing (d) Failure of a steel stopper

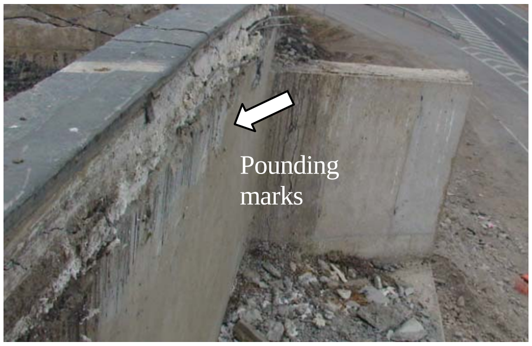
Figure 8. Collapse of the Lo Echevers Bridge (from MO, Chile).

Vertical Seismic Restrainers

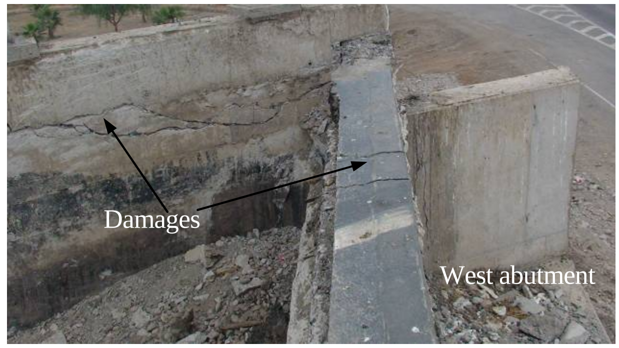
Vertical seismic bars were used in a number of bridges (e.g. Chada, Las Mercedes, LLacolen, Romero, Hospital, and Pichibudis bridges). In general, they are flexible, experiencing significant deformation during the earthquake. Almost all of them were well anchored into capbeams and decks since only those bridges with felled-off spans left signs for bar pullout. It was uncertain whether they have provided vertical restraints to bridge girders even though no sign of up-and-down jumping of girders was observed at bridge sites (see figure 9). This point can only be clarified after significant bridge analyses using recorded ground motions.



(a) Seismic bars at west abutment (b) Seismic bars at east abutment



(c) Pounding at west abutment



(d) Wingwall damage at west abutment

Figure 9 Abutment damage in the Avienda Romero Bridge.

Bridge Bearings

In general, bridge bearings functioned well during the earthquake. Several of them in bridges such as Lo Echevers and Cardenal Raul Silva Henriquez bridge shown in figure 10 were displaced significantly.

Girder Seat Length

The superstructure of a number of concrete or steel girder bridges (LLacolen, Miraflores, Lo Echevers, Romero, Hospital, Tubul, Bio-Bio, and pedestrian bridges) dropped off its support during the earthquake. In general, the support seat length is insufficient according to the latest AASHTO Seismic Design Specifications.

Column Shear Failures

Except for the ground settlement and lateral spreading effects (e.g. LLacolen and Juan Pablo II bridges), bridge substructures (capbeams and columns) received virtually no damage during the earthquake. This represented a successful design of bridge substructure. Several columns in the LLacolen and Juan Pablo II bridges failed in shear (See figure 10).

(a) Underneath first span over water (b) Shear failure in upstream column
Figure 10 Damage to intermediate bent under approach spans to Juan Pablo II Bridge.

Foundation Movement and Damage

Overview

In general, structural aspects of the foundations performed relatively well in this earthquake. With the exception of cases where liquefaction induced vertical and/or lateral soil movement was severe, foundations did not appear to suffer significant permanent deformations or significant damage, based on surficial observations, Most of the newer bridges included in the 32 sites visited were supported by shaft foundations (typically 1.5 m diameter), typically less than 30 m deep. These foundations appear to be relatively light compared to bridge foundations currently used in the USA in areas of high seismic hazard, yet they performed well in most cases.

To identify lessons learned regarding structural performance of foundations, two broad categories of geotechnical performance issues must be considered. These include foundation performance when liquefiable soils were likely not present, and foundation performance when liquefiable soils were present. For those sites where liquefaction likely occurred, geotechnical performance issues are further divided between the effects of liquefaction induced settlement, and liquefaction induced ground failure and lateral movement.

Liquefaction likely occurred in 15 of the 32 sites visited, however, it must be recognized that none of these sites were specifically designed to mitigate the effects of liquefaction (i.e., through use of ground improvement or foundation strengthening). This affords the opportunity to observe the effect liquefaction can have on foundation and abutment performance for structures that are otherwise designed using the AASHTO or similar specifications, depending on the age of the structure.

Effects of Lateral Spreading and Liquefaction Induced Ground Failure

What was most surprising was the good performance of bridge abutments retaining 4 to 8 m high approach fills over gently sloping ground even when severe vertical and horizontal approach fill deformation (e.g., 0.5 to1 m or more), likely due to liquefaction of soil below the approach fill, occurred (see figure 11). While there were a few cases where 50 to 150 mm of lateral movement of the abutment appears to have occurred, in most cases no discernable movement occurred. This may be the result of three dimensional effects reducing the lateral forces acting on the abutment foundations relative to what would be predicted assuming two-dimensional (i.e., plane strain) conditions. Furthermore, the liquefaction induced slope failure tended to follow the path of least resistance – i.e., in the direction perpendicular to the roadway and bridge centerline. For those cases where either the abutment or an interior pier is located on a general slope such as at a river bank, the beneficial three dimensional slope geometry is not present, and foundation and substructure movement and damage due to liquefaction induced ground failure was more likely to occur (e.g., Sites 17, 18, 19, and 25). These observations may have important implications for the strategy typically used for liquefaction design of bridges that could be used to advantage in both Chile and the USA.

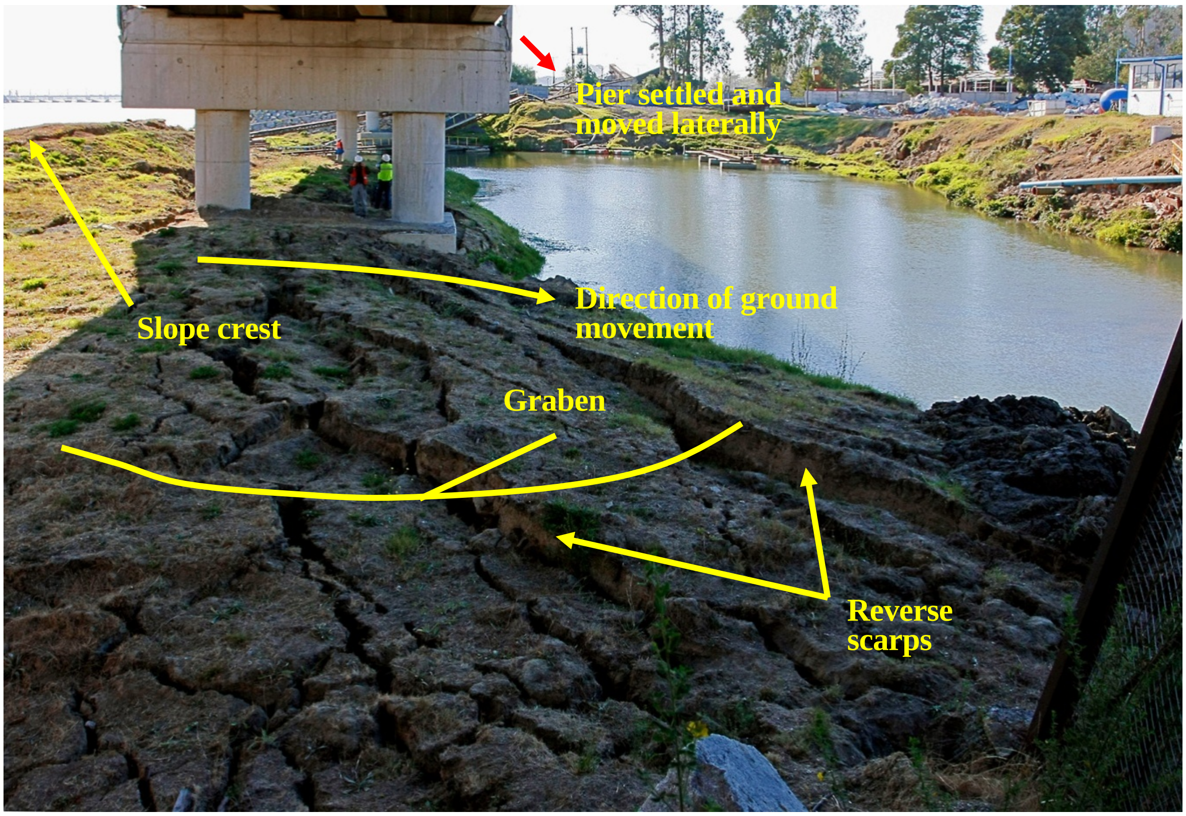


Figure 11. Severe ground failure at La Mochita Bridge.

Effects of Liquefaction Induced Settlement and Downdrag

Liquefaction induced ground settlement was observed at many of the bridge sites where liquefaction occurred. However, settlement of the bridge foundations only occurred for a few of those sites (i.e., Site No's 25 – Juan Pablo II Bridge, 27 – La Mochita Bridge (see Figure 12), and 31 - San Nicolas Bridge). In general, regardless of the amount of liquefaction induced ground settlement that occurred, the foundations did not settle significantly if these were supported underneath bya reasonably good bearing layer of soil. However, if the foundation was relatively shallow and not supported by a good quality bearing layer, significant settlement of the foundation did occur.

Figure 12 Ground failure of north approach fill for the La Mochita Bridge, looking toward the north away from the bridge.

Retaining Walls and Roadway Fills

Three types of walls were inspected by the TIRT: panel faced mechanically stabilized earth (MSE) walls using bar mat or steel strip soil reinforcement, modular block HDPE geogrid reinforced walls, and concrete gravity walls. Overall, retaining walls performed well during the earthquake. Tieback and soil nail walls appeared to suffer little or no damage based on observations made by others. True MSE abutments, where the bridge footing foundation was directly supported on top of the MSE wall (Site 11), also performed well, with no apparent deformation or damage to the walls. Note that most of these walls were designed using the AASHTO Standard Specifications. The observed wall performance appears to indicate that the AASHTO specifications, as applied in Chile, provide a safe design for seismic loading conditions.

There was no evidence of lateral sliding of the walls, the limit state that often controls wall design for seismic conditions in North American design practice (with one exception – see Chapter 5 for details). Where lateral movement of the wall face was observed, the movement was primarily rotational, with the maximum movement near the wall top, as it appears that the passive resistance at the wall toe in combination with friction along the wall base prevented significant translational movement.

Minor damage observed in several walls was due mainly to poor detailing. Inadequate coping details allowed a few of the top blocks to topple and fall off the wall (modular block walls); poor wall corner details or vertical full height joint details (such as occurred between the curtain wall and MSE wall retaining the approach fill sides) allowed panels to separate and wall backfill to spill out through the gaps in the facing. Stresses during seismic loading, especially for relatively tall walls (e.g., 9 m or more in height) appeared to be more pronounced at abrupt changes in wall geometry (e.g., corners and small radius changes in alignment), indicating the need for more robust wall facing designs in that type of situation. Soil reinforcement that is too short, especially near the wall top, and even more so if uniform low shear strength medium sand is used as backfill, can contribute to excessive wall, or at least panel movement.

CONCLUDING REMARKS

This paper provides a short summary of the damages the TIRT observed during the reconnaissance. The detailed descriptions and suggestions of the future research needs or design code changes will be published soon before or after this paper. There are so many things to learn from this particular earthquake, and the bridge seismic performance observed will need to be further analyzed from the recorded ground motion data to validate these findings. The help and support from Ministry of Public Works (MOP) of Chile, Catholic University of Chile and Univ. of Chile are greatly appreciated.

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