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CAPACITY EVALUATION OF STEEL STOPPERS OF REINFORCED CONCRETE CHILEAN BRIDGES

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SUMMARY

The road infrastructure of Chile was severely affected by the 2010 earthquake with magnitude Mw=8.8. Of the nearly 12,000 highway bridges in Chile, approximately 300 were damaged including 20 with collapsed spans. The most common observed damage mode was the failure of shear keys, or stoppers, that are used to restraint the transverse displacement of the superstructure. Traditional Chilean bridge design, which consider reinforced concrete (RC) shear keys have been evolved during the last two decades, and steel stoppers have been introduced in these new designs. Steel stoppers were found to be a critical element during the earthquake and several bridges with such stoppers were damaged and collapsed. The objective of this study is to evaluate experimentally the capacity of steel stoppers of RC Chilean bridge. To achieve this objective, three full scale stoppers were tested under lateral load. The results of the experimental program are presented on this paper. Additionally, the lateral strength of three damaged bridges is estimated and compared with the requirements of the Chilean bridge design code. The results presented in this paper are useful for revising current bridge design code and for estimating the vulnerability of existing bridges.

INTRODUCTION

Damage of Chilean bridges after the 2010 Maule earthquake have been reported by several authors ([1], [2], [3] and [4]). Bridge damages can be classified in usual and unusual failure modes [3]. The usual failure modes are those frequently seen in earthquakes not just in Chile but elsewhere in the world. Usual failure modes observed included damage to connections between super and substructures, unseating of spans in skewed bridges due to in-plane rotation, and column damage and unseated spans due to permanent ground movement. Unusual failure modes included unseating of non skewed spans due to in-plane rotation of superstructure, plate girder rupture due to longitudinal inertial forces, scour and structural damage due to tsunami action and collapse of historic masonry bridge. Detailed descriptions of these damage are given elsewhere ([1], [2], [3], and [4]).

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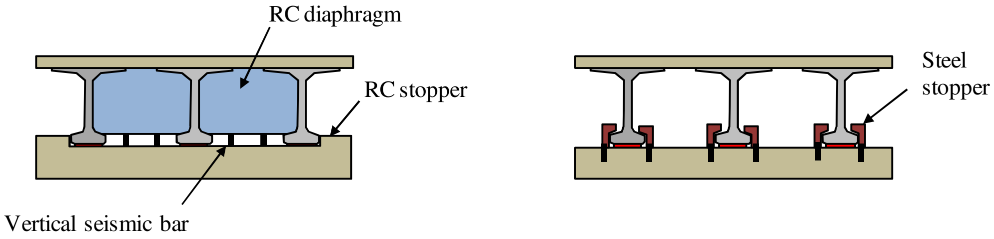
With the arrival of the concessions companies in the mid-90s, typical bridge design have been evolved [3]. Typical bridge design consist on a reinforced concrete (RC) slab with simply supported prestressed concrete (PC) girders, transverse RC diaphragm, RC stoppers, and vertical seismic bars for preventing uplift (Figure 1a). In the new designs, some bridges were constructed without diaphragms and in some cases, both the concrete stoppers and the vertical seismic bars were replaced by steel stoppers as shown in Figure 1b. The steel stoppers were designed to restraint the lateral displacement of the girder and to control the uplift of the superstructure.



(a) Typical bridge design (b) New bridge design

Figure 1: Cross section of Chilean bridges

During 2010 earthquake, new bridges designed with steel stoppers suffered considerable damage to its stoppers, lateral displacement of the superstructure, and three bridges collapsed. The collapsed bridges, which were located in Santiago, are the eastbound of Lo Echevers overpass, and the eastbound and westbound of Miraflores overpass. Figure 2a and 2b show the collapse of these bridges, where the damaged steel stoppers are encircled. The damage observed to the steel stoppers in the westbound of Lo Echevers overpass and the eastbound Independencia overpass is shown in Figure 2c and 2d, respectively. For the Independencia bridge (Figure 2d) the superstructure displaced laterally more than 0.5 m destroying the steel stoppers located at the south of the girders (left of the girders in Figure 2d). The girders of this bridge did not unseat because enough transverse seat width was provided at the cap beam.

Since several existing bridges of the concessions companies are detailed with steel stoppers, the question that wants to be answered in this study is what is the strength provided by these stoppers. Figure 3a shows the drawings of steel stoppers and Figure 3b shows a picture of a damaged stopper located in a bridge of Vespucio Norte concession highway. This stopper was bolted on top of a concrete base, but other stoppers were bolted directed to the cap beam. The stoppers are detailed with a 220 x 150 x 25 mm base plate which contain oval holes for bolting the stopper to the pier cap or abutment with two 25 mm threaded reinforcing bars. Below the washer of each of these anchor bars, an additional 90 x 90 x 20 mm steel plate is specified. The dimension of the vertical plate of the stopper is 338 x 220 x 15 mm and the dimension of the vertical stiffener is 338 x 135 x 20 mm. The specified distance between the anchor bolt and the end of the base plate is 90 mm. However, this specified distance can not be achieved because the oval holes specified in the base plate allow a distance that vary between 50 mm to 80 mm. In fact, the distance between the anchor bolt and the end of the base plate is about 75 mm in the stopper shown in Figure 3b. Finally, the specified gap distance between the stopper and the bottom flange of the PC girder is 35 mm.

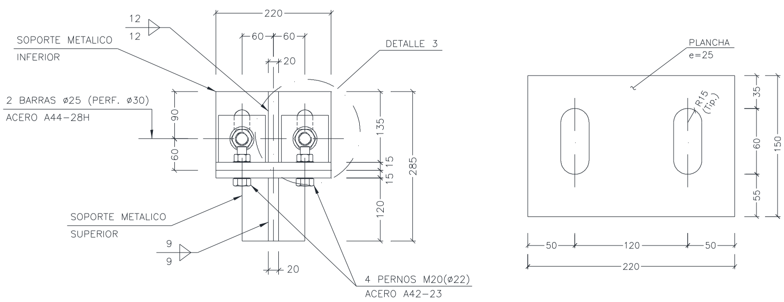
(a) Collapse of eastbound Lo Echevers overpass

(b) Collapse of eastbound Miraflores overpass

(c) Damage of westbound Lo Echevers overpass

(d) Damage of eastbound Independencia overpass

Figure 2: Observed damage to bridges with steel stoppers



(a) Steel stopper drawings

(b) Damaged to steel stopper

Figure 3: Steel stopper detailing

LATERAL STREGTH REQUIEREMENTS IN CHILEAN CODE

The Chilean bridge design code [5], which is based in the AASTHO code [6] specifies four seismic analysis procedures depending on the bridge characteristics: (1) seismic coefficient (SC) method, (2) modified SC method, (3) modal spectral analysis, and (4) time history analysis. The SC method uses a constant SC of $SA_o/2g \geq 0.10$ where A_o is the effective acceleration and S depends on the soil classifications. The modified SC method depends on the fundamental period of vibration and the SC is limited by $1.5SA_o/g$. For this method the SC decay with the fundamental period of vibration and the minimum value of SC is given by $0.25SA_o/g$.

Shear keys or stoppers are required at the piers and abutments to provide lateral stability of the superstructure. The strength reduction factor specified for shear keys or stoppers is R=1 and each shear key should be designed with half of the transversal seismic force [5]. Shear keys should be ductile enough to prevent unseating of the spans, they should be taller and 30 cm, and a minimum gap of 5 cm should be provided between the shear key and the superstructure [5]. Additionally, to prevent unseating of the bridge girders from the relative displacement induced by earthquakes sufficient seat width and/or unseating devices must be provided at the abutments and intermediate piers ([7], [8], and [9]).

EXPERIMENTAL PROGRAM

To obtain the strength of steel stoppers used in Chilean bridges three identical full scale stoppers (S1, S2 and S3) were tested under lateral load. The stoppers were tested in a 90° rotated position to simplify the load application, Figure 4. The vertical load was applied to the stopper using an hydraulic jack with a compression capacity of 200 kN. The jack load was transferred to the stopper using a half cylinder plate to maintain the vertical direction of the load. The load was applied to the stopper at a distance of 170 mm from the RC block, which corresponds to the approximate distance of the contact between the stopper and the bottom flange of the PC girder of bridges. At the top side, the jack was connected to a steel reaction beam which was anchored to the strong floor.

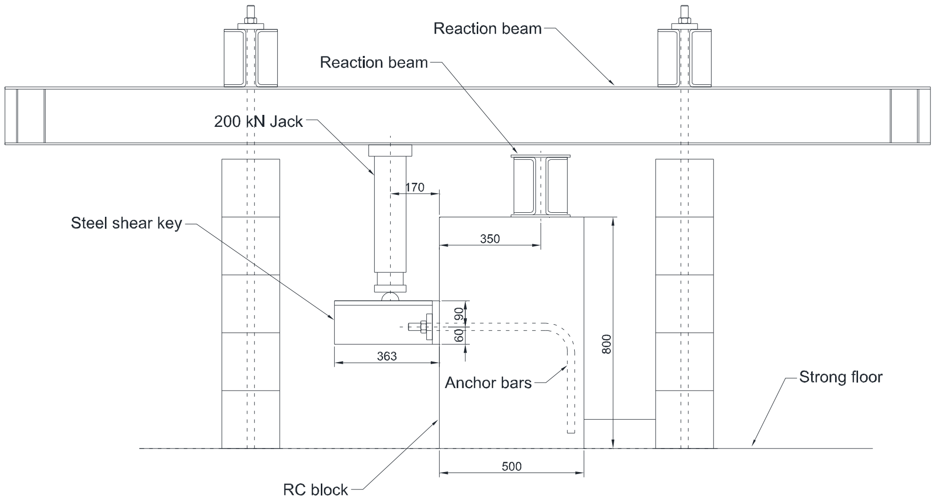


Figure 4: Test Setup

The stoppers were built with the described dimensions (Figure 3) using A36 steel. The upper pieces of the stopper, which are used to restraint the vertical displacement of the girders (Figure 3b), were not considered because the stoppers were not tested for uplift conditions. The stopper was bolted to a 800 x 500 x 600 mm RC block that simulate the cap beam using two 25 mm threaded reinforcing bars with nominal yield strength of 280 MPa. Since the location of the anchor bars is variable due to the oval holes of the base plate, conservatively a distance of 60 mm between the anchor bars and the end of the base plate was considered, Figure 4. Note that the allowable distance of the anchor bars range between 50 mm and 80 mm. The anchor bars were bend inside the RC block with a 90° hook to provide enough development length according to ACI code [10]. The RC block was anchored to the strong floor with a reaction beam and the horizontal displacement at the bottom of the block was restrained. The RC blocks were constructed using normal weight concrete with specified design strength of 20 MPa and maximum aggregate size of 13 mm. They were reinforced in both direction using 10 mm bars spaced at 100 mm. Figure 5s show pictures of the construction process of the RC blocks.

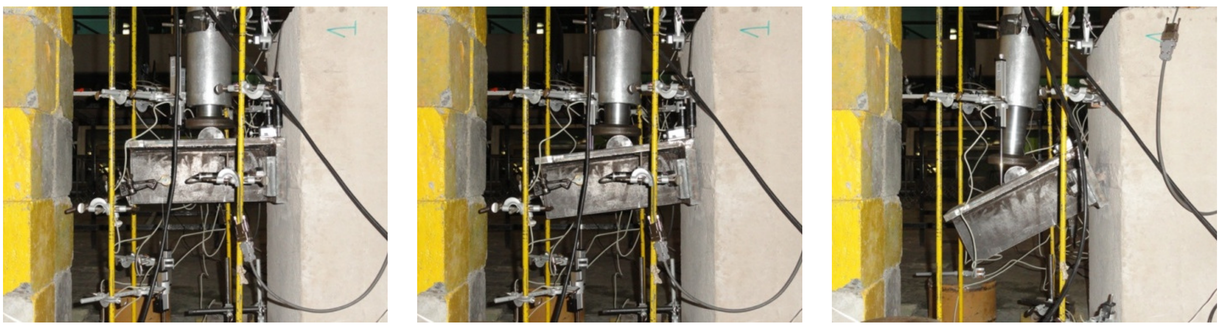
(a) Anchor bars (b) Concrete block reinforcement (c) Pouring of concrete

Figure 5: Construction process

The stoppers were subjected to monotonic static compression load. The load was applied under constant flow rate with a jack displacement of about 1 mm per minute. Loading continued until a residual strength of about 50% was achieved. Each specimen was instrumented with a total of 17 instruments: 2 load cells, 13 displacement transducers and 2 strain gauges. The load cells measured the vertical load of the jack. The displacement transducers were installed to measure the vertical and horizontal displacement of the stopper at different locations, the pullout displacement of the anchor bars, and the displacement of the RC block. Finally, the strain gauges were installed on the anchor bars to measure their tensile strain.

TEST RESULTS

The tests were conducted at a concrete age of 46 days. The measured concrete strength was 22.3 MPa and the measured yield strength and ultimate strength of the anchor bars were 323 MPa and 486 MPa, respectively. The three specimens demonstrated a similar behaviour where the steel stopper deformed as a rigid body and rotation of the stopper was observed at the connection with the RC block. When the specimens reached the maximum load, relative displacement between the anchor bar and the nut was observed because the nut was pulled out by the adjacent 90 x 90 x 20 mm steel plate. At the end of the tests, damage was observed to the threads of the anchor bolts and no damage was observed at the concrete of the RC block. The damage propagation of stopper S1 is shown in Figure 6.



(a) Initial state (b) At 90% of peak load (c) At the end of test

Figure 6: Damage propagation stopper S1

The load-displacement relationship for the three stoppers is shown in Figure 7. The vertical displacement at each specimen was measured with two displacement transducers, north and south, located at 250 mm from the RC block. Note that the vertical load was applied at 170 mm from the RC block. The displacement for each stopper, plotted in Figure 7, corresponds to the average displacement measured with the north and south transducers. The measured peak load was 81.5 kN, 85.4 kN and 85.9 kN, for stoppers S1, S2 and S3, respectively. The measured displacement at the peak load was 30.1 mm, 30.6 mm and 43.0 mm, for stoppers S1, S2 and S3, respectively. Therefore, the average stopper capacity is 84.3 kN at a peak average displacement of 34.6 mm. However, the capacity of the stoppers could vary significantly in actual bridges depending on the position of the anchor bars. After reaching the peak load, the strength of the stopper decreases in a somehow brittle manner. The tests were stopped at a displacement of about 80 to 100 mm. At this displacement level, the jack end plate touched the steel stopper at the side of the half cylinder plate.

The pullout horizontal displacement of the anchor bolt and the horizontal displacement of the 90x90x20 mm steel plate of the north side of specimen S3 is shown in Figure 8a. The details of the instrumentation is shown in Figure 8b, where an horizontal displacement transducer is used to measure the horizontal displacement of the steel plate. The pullout displacement of the anchor bar is measured with another displacement transducer (transducer not shown in Figure 8b) which is connected to the end of the anchor bolt using a string and a pulley. The horizontal displacement of the anchor bolt and the plate are similar until about 660 seconds, which correspond to approximately 50% of the peak load of specimen S3. After this time, a relative displacement between the steel plate and anchor bolt started to occur because due to the damage in the thread of the anchor bar and nuts.

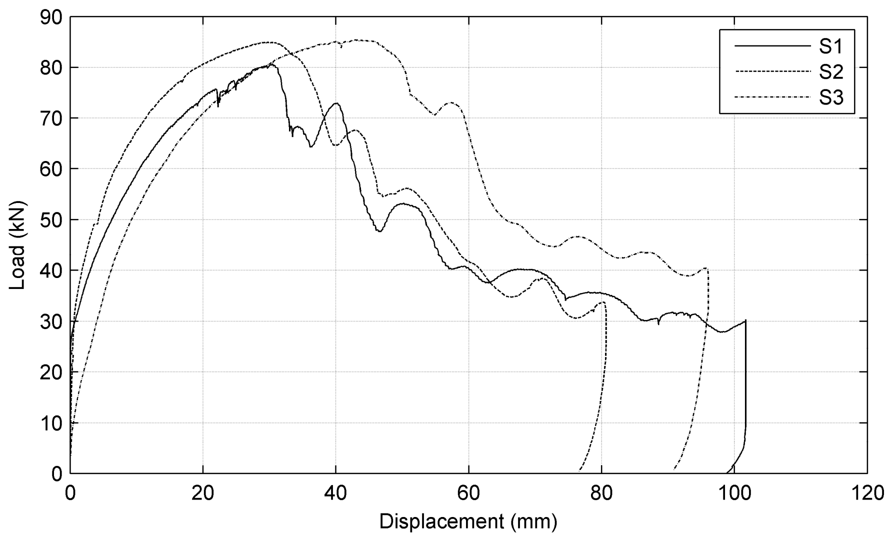
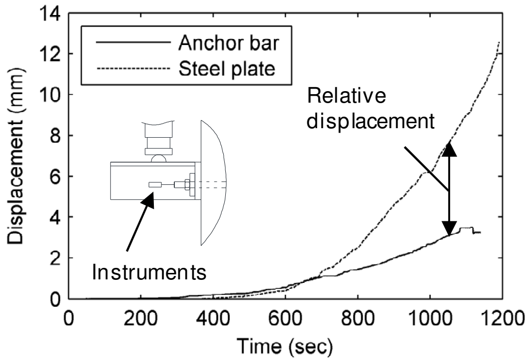


Figure 7: Load displacement-relationships

RESULTS DISCUSSION

The experimental capacity of steel stoppers is analyzed with three damaged bridges along Vespucio Norte concession highway in Santiago: Independencia, Lo Echevers and Miraflores bridges. The damage suffered by these bridges is described earlier where Lo Echevers and Miraflores bridges collapsed. The superstructure of the three bridges consisted on five simply-supported PC girders with a continuous deck with the cross section shown in Figure 1b. The eastbound Independencia overpass is a six-span structure with spans length of 21.67, 25.32, 22.36, 24.88, 27.95 and 17.54 m. The eastbound Lo Echever overpass is a three-span structures with spans lengths of 27.84, 36.32, and 27.84 m. Finally, the eastbound Miraflores overpass is a three span structure with spans length of 23.56, 26.68, and 23.56 m.



(a) Displacement versus time

(b) Instruments details

Figure 8: Behavior of anchor bar and 90 x 90 x 20 mm steel plate, stopper S3, north side

The weight of the longer span and the provided SC for the three considered bridges is summarized in Table 1. The weight is calculated using a RC deck of 22 cm thickness with a pavement of 5 cm thickness. The specific weight considered for concrete and pavement is 25 kN/m^3 and 24 kN/m^3, respectively. The weight of the girders and steel barriers is 13.5 kN/m and 7.0 kN/m, respectively. The provided SC is calculated as the quotient between the stoppers capacity and the weight of the superstructure of the longer span. Since each span contains five girders with a total of 20 steel stoppers, the capacity provided by 10 stoppers for the seismic action in one direction is 10 x 84.3 = 843 kN. Results from Table 1 shows that the SC of Independencia bridge is larger than that of Lo Echevers. This result contradicts the observed behavior during the earthquake where the Independencia bridge did not collapse and Lo Echevers did collapse.

Table 1: Characteristics of bridges and provided seismic coefficient for the superstructure

Bridge	Superstructure length(m)	Longer span (m)	Deck width (m)	Number of spans	Superstructure weight (kN)	Provided SC
Independencia	139	27.95	12.06	6	4484	0.188
Lo Echevers	92	36.32	12.26	3	5874	0.144
Miraflores	73	26.68	12.26	3	4315	0.195

The provided SC is compared with the requirements of the SC procedures of the Chilean code [5]. The considered bridges are located in Santiago and are founded in soil type II, which implies A_o=0.3g and S=1.0. Therefore, the SC for the SC method is 0.150, and the maximum and minimum SC for the modified SC method is 0.075 and 0.450, respectively. It is concluded that the provided SC for the three bridges is at least 88% larger than the minimum value of 0.075 specified for the modified SC, but is smaller than the maximum value of 0.45, which apply to bridges with period lower than 0.3 sec. It is important to note that this analysis does not consider an amplification factor for the seismic action and does not consider a reduction factor for the capacity. Additionally, this analysis consider that all stoppers are acting at the same time, which is questionable due to the their relatively high stiffness.

CONCLUSSIONS

From the experimental results of steel stoppers subjected to lateral loads, and the analysis of three damaged bridges, the following conclusions can be obtained:

1) Steel stoppers used in Chilean bridges were detailed with oval holes which implies that the capacity depends on the on-site location of the anchor bars.
2) The experimental capacity of steel stoppers with anchor bars located at 60 mm from the edge of the base plate is 84.3 kN. The capacity is governed by the failure of the threads of the anchor bars and nuts.
3) The provided SC of the three bridges analyzed range from 0.144 to 0.195. The SC is at least 88% larger than the minimum value of 0.075, but it is smaller than the maximum of 0.45 specified by the Chilean code for structures with period shorter than 0.30 sec.
4) The provided strength and ductility of steel stoppers and/or the provided transversal seat width was not adequate because two bridges suffered unseating of spans.
5) The capacity requirements for stoppers should be revised in seismic codes to prevent unseating of spans.

AKNWOLEDGMENTS

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REFERENCES

1. Kawashima K, Unjoh S, Hoshikuma J, Kosa K. "Damage of bridges due to the 2010 Maule, Chile, Earthquake." Journal of Earthquake Engineering 2011, 15(7): 1036-1068.
2. Schanack F, Valdebenito G, Alvial J. "Seismic damage to bridges during the 27 February 2010 Magnitude 8.8 Chile Earthquake." Earthquake Spectra 2012, 28(1): 301-315.
3. Buckle I, Hube M, Chen G, Yen W, Arias J. "Structural performance of bridges in the offshore Maule earthquake of February 27, 2010." Accepted for publication Earthquake Spectra 2012, Chile Earthquake Special Issue.
4. U.S. Department of Transportation, "Post earthquake reconnaissance report on transportation infrastructure impact of the February 27, 2010, offshore Maule Earthquake in Chile." Federal Highway Administration Publication No. FHWA-HRT-11-030, March 2011, Washington DC.
5. MOP. "Manual de Carreteras, Volumen N°3, Instrucciones y Criterios de Diseño, Capítulo 3.1000." Dirección de Vialidad, Ministerio de Obras Públicas, Chile. 2010.
6. AASHTO. "Standard Specifications for Highway Bridges, 17th edition." American Association of State and Highway Transportation Officials, Washington, DC, 2002.
7. CALTRANS. "Seismic design criteria, Version 1.4." California Department of Transportation 2006, Sacramento, CA.
8. Hube MA, Mosalam KM. "Experimental and computational evaluation of in-span hinges in reinforced concrete box-girder bridges." Journal of Structural Engineering 2011, 137(11): 1245-1253.
9. Hube MA, Mosalam KM. "Parametric study and design recommendations for in-span hinges in reinforced concrete box-girder bridges." Journal of Bridge Engineering 2012, 17(2):334-342.
10. ACI. "Building code requirements for structural concrete and commentary, ACI 318-08.", American Concrete Institute 2008, Farmington Hills, MI.