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Seismic Assessment of Repaired Bridges in Chile

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ABSTRACT

This study presents the current state-of-practice of repair measures for continuous multi-span prestressed concrete girder bridges in Chile. In addition to that, the seismic assessment of a repaired multi-span highway bridge is presented and discussed. The selected bridge suffered extensive damage during the 2010 Maule earthquake and was repaired by adding concrete diaphragms, shear keys, seismic bars and increasing seat-lengths. Non-linear static and non-linear time history analyses were performed in order to evaluate the effectiveness of the repair measures. Subduction ground motions recorded at several stations and relevant engineering parameters were used to assess damage states of different bridge components. The results of the study showed that the adopted repair measures are effective to control unseating of spans, however, minor to extensive damage is still expected.

1. INTRODUCTION

Chile is located on one of the most active subduction zones in the world. In fact, three major earthquakes of moment magnitude greater than M_w=8.0 have occurred within the last 8 years, namely, the 2010 Maule earthquake (M_w=8.8), the 2014 Iquique earthquake (M_w=8.2), and the 2015 Illapel earthquake (M_w=8.3). Because of this, the design, construction and overall seismic performance of the civil infrastructure in Chile are constantly being tested. In particular, highway bridges, which are essential structures for the transportation and communication among regions, were heavily damaged and some even collapsed during the 2010 Maule earthquake. Most of the damage and collapse of Chilean bridges was observed in continuous multi-span prestressed concrete girder bridges supported on seat-type abutments. The observed damage was attributed to in-plane rotation of the deck, which caused excessive displacement of the superstructure and led to unseating of spans (Kawashima, et al., 2011; Elnashai, et al., 2012; Buckle, et al., 2012; Schanack, et al., 2012). This damage,

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which is characterized by excessive displacements between the superstructure and the substructure as shown in Fig. 1, was caused by a lack of effective unseating-prevention devices, lack of transverse diaphragms and short superstructure seat-length on bents and abutments allowed by the seismic design criteria used at that time (MOP, 2002).

Fig. 1 Damage observed in highway bridges after the 2010 Maule earthquake. (Photos: M. Yashinsky, J.L. Seguel, F. Schanack)

After the earthquake, it was evident that the seismic performance of highway bridges in Chile was deficient. As a consequence, bridges were primarily repaired and retrofitted using reinforced concrete stoppers (shear keys) and increasing the seat-length at the supports (abutment and bents) as shown in Fig. 2.

Fig. 2 Repaired/Retrofitted Bridges. (Photos: J.L. Seguel)

Based on the observed damage, the Ministry of Public Works of Chile (MOP) on June of 2010 revised the seismic provisions and released a document with the seismic design criteria for the repair of damaged bridges and the design of new bridges. From that date until now, the design provisions have been under constant review and discussion from part of bridge practitioners, academics, and international experts. For that reason, this paper presents an overview and discussion of some of the major changes in the seismic design criteria for bridges in Chile after the 2010 earthquake. This study also presents the seismic assessment of a repaired highway bridge. The effectiveness of the repair measures is analyzed performing non-linear analyses and comparing the results of the repaired and the as-built condition. Non-linear time history analyses are performed utilizing subduction ground motions recorded at several stations and relevant engineering parameters are compared to element's damage states.

2. SEISMIC DESIGN PHILOSOPHY OF HIGHWAY BRIDGES IN CHILE

According to the Chilean Bridge Design Code (MOP, 2017), which is largely based on AASHTO (1996), bridges, and pedestrian bridges that do not have free span-lengths greater than 70 meters shall be designed to have a very low probability of collapse for a seismic event with a probability of exceedance of 10% in 50 years, i.e. return period of 475 years. The code also states that the seismic specifications are meant to achieve structures capable of performing;

(1) without damage, i.e. in the elastic range, for moderate seismic events,

(2) the damage on non-structural elements shall be limited for a seismic event of medium intensity

(3) the risk to life must be minimal and the structural components may result damaged as a consequence of a severe seismic event but collapse must be prevented.

Although the code states these three seismic performance objectives, the code does not provide seismic hazards or maps to determine the demands for seismic events of moderate and medium intensity, and the design is only carried out and checked for one performance level, which is collapse prevention. It is clear after the damage reported during devastating earthquakes (Buckle et al., 2012) that some of the performance objectives stated in the code were not fulfilled, suggesting that a performance-based seismic design methodology needs to be studied, evaluated and potentially incorporated in future revisions of the code. This would also point to the current trend of performing performance-based designs for highway bridges (NCHRP, 2013).

After the earthquake, it was evident, as shown in Fig. 1, that the seismic performance of bridges in Chile was deficient, mainly because of a lack of restraining devices that would prevent unseating of spans. Consequently, the Ministry of Public Works (MOP) on June of 2010 revised the seismic provisions and released an emergency document with the seismic design criteria for the repair and retrofit of damaged bridges and the design of new bridges (MOP, 2010). In 2017, the Ministry of Public Works decided to update the emergency document of 2010 by a document that presents further details and rationale for the seismic design of conventional bridges

(MOP, 2017). The main modifications introduced into the code were aimed to avoid the unseating of spans by:

(1) Increasing the seat-length requirements by using Eq. (1). Where, S is the required seat-length, L is the span length, L_θ is the total continuous bridge length, α_E is the rotation angle (usually taken as 2.5°), and θ is the skew angle.

$$S > 0.7 + 0.005L\ [m] \quad \text{for straight bridges}$$
$$S_\theta > 2L_\theta sin\left(\frac{\alpha_E}{2}\right)\cos\left(\frac{\alpha_E}{2} - \theta\right) \quad \text{for skewed bridges} \tag{1}$$

(2) Recommending unseating-prevention devices found in the Japanese bridge code as shown in Fig. 3.

(3) Instructing that all the bearings must be anchored to the substructure and corresponding girder since many elastomeric bearings walked out from their supports.

(4) Using vertical seismic bars to limit the vertical motion of the deck and to avoid having tensile forces in bearings.

(5) Stating that transverse diaphragms, and shear keys must be implemented at abutments and piers as shown in Fig. 4.

Regarding shear keys, in the 2017 update, a new design philosophy was set in place. This new philosophy indicates that shear keys should be designed to behave as a structural fuse, i.e. shear keys should get damaged or failed during a major seismic event but the bridge integrity should be preserved. Because of this, the new code states that the shear key width must be between 40 [cm] to 70 [cm]. Additionally, the code suggests using special cold joints in order to induce the shear key failure in the interface between the shear key and the cap beam.

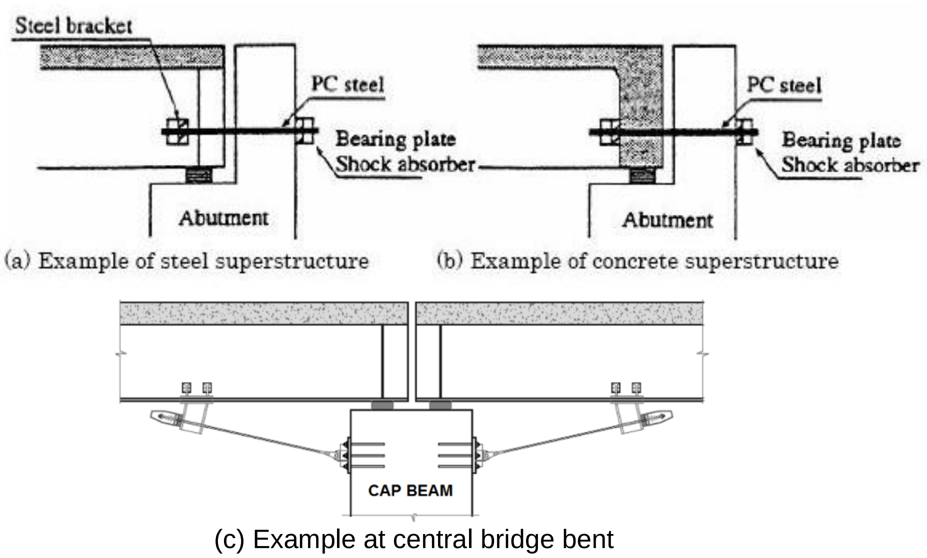


Fig. 3 Example of unseating-prevention devices (elevation view). (MOP, 2017)

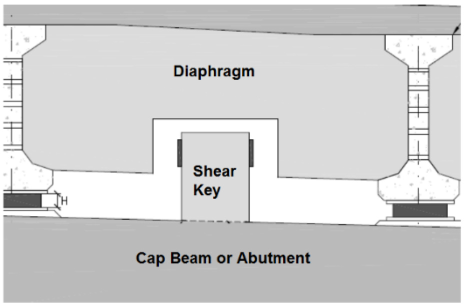


Fig. 4 Use of shear keys and diaphragms. (MOP, 2017)

3. REPAIRED BRIDGES IN CHILE

After the 2010 Maule earthquake, approximately 300 bridges were damaged and some of them even collapsed (Buckle, et al., 2012). Most of the bridges that were damaged during the earthquake corresponded to multi-span prestressed precast concrete (PC) girder bridges. The damage observed on those bridges was caused by large displacements of the superstructure relative to the substructure, leaving a large number of bridges with residual deck displacements, shear key damage, bearing damage, and damage in the web of precast girders as shown in Fig. 1.

In order to repair and retrofit the damaged bridges, the procedures carried out mainly consisted on:

1) restituting the deck to its original position by using vertical and horizontal hydraulic jacks (Fig. 5a). This procedure also allowed replacing the damaged elastomeric bearings for new ones (Fig. 5b)

2) repairing the damage in PC girders by removing all the loose concrete and damaged bars to then applying a bonding agent in conjunction with repairing mortar with fiber. In some cases, cracks were injected with special epoxy and girders were retrofitted using carbon fiber reinforced polymer (CFRP) as shown in Fig. 5c.

3) replacing expansion joints (Fig. 5d), and adding transverse diaphragms, seismic bars and shear keys.

4. SEISMIC ASSESSMENT OF A REPAIRED BRIDGE

As stated before, many bridges, most of them overpasses, suffered extensive damage during the 2010 earthquake. Among the bridges that collapse, we can found a few bridges in the city of Santiago, capital and largest city of Chile, with a population of over 7 million.

Despite Santiago being more than 400 [km] away from the 2010 earthquake epicenter, large ground accelerations were recorded, causing extensive damage in some bridges, to name a few, Miraflores overpass, Lo Echevers overpass, Americo Vespucio Bridge, Independencia overpass, Chada overpass, among others (Kawashima, et al., 2011), (Buckle, et al., 2012).

This study takes the Lo Echevers Bridge as a case study in order to assess the seismic performance and effectiveness of the repair and retrofit measures taken after the earthquake. It is important to note that the repair and retrofit measures adopted in this bridge were done using the emergency design criteria of 2010 (MOP, 2010).

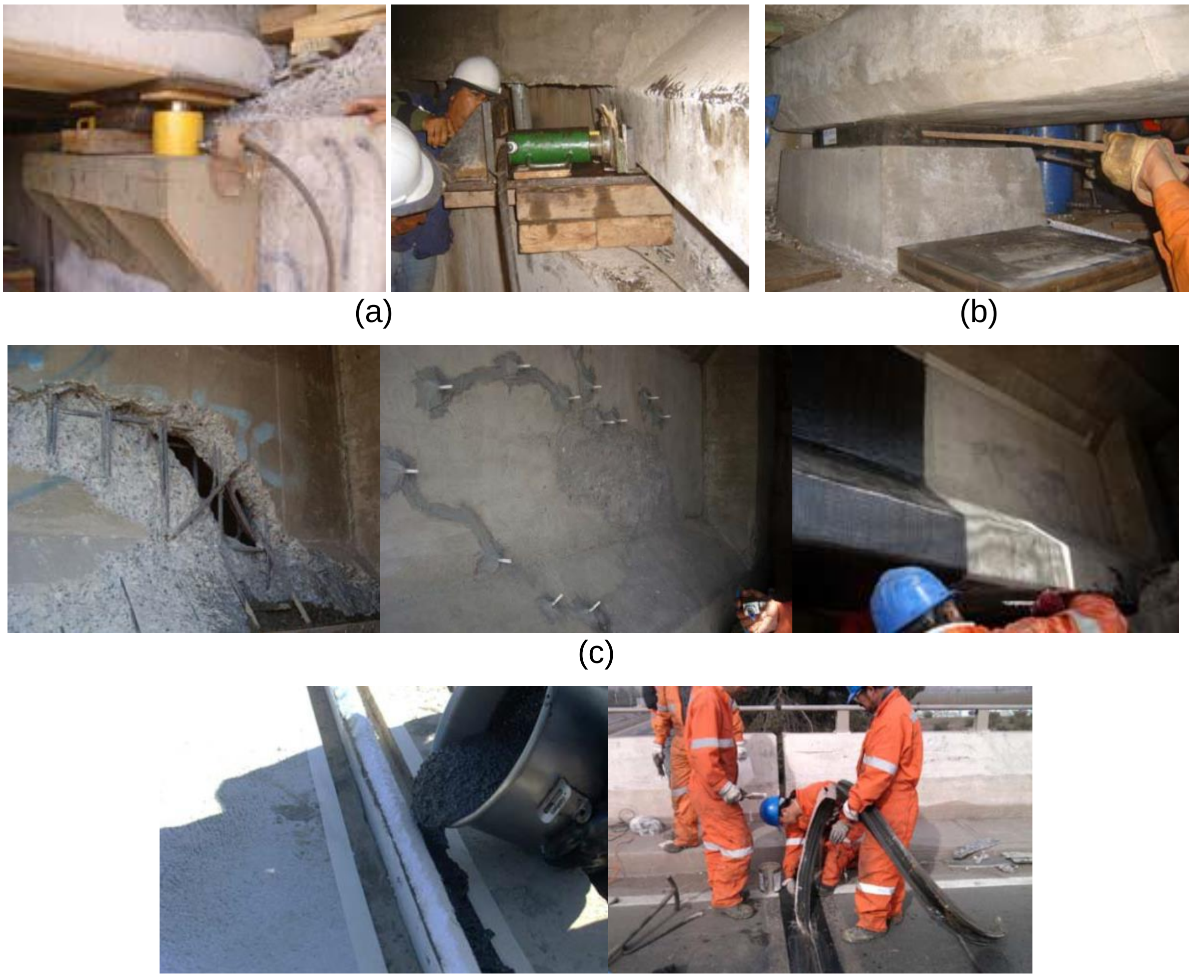


Fig. 5 Procedures to repair and retrofit damaged bridges after the 2010 Chile Earthquake. (Photos: D. Ortiz-TECNOAV)

4.1 Bridge Description

Lo Echevers Overpass is a three-span prestressed concrete girder bridge belonging to the Vespucio Norte highway that collapsed during the 2010 earthquake. The bridge was constructed in 1990 and is 92 [m] long, 12.3 [m] wide and has a skew of 33° as shown in Fig. 6. The bridge has seat-type abutments and two bents that are composed by 5 round columns and a cap beam.

After the 2010 earthquake, the structure was repaired by replacing the deck for a new one, increasing the cross-section of the cap beam, replacing lateral steel stoppers, which had deficient seismic performance during the earthquake, for massive reinforced concrete shear keys, increasing the dimensions of elastomeric bearings, and including vertical seismic bars and transverse diaphragms at bents and abutments. The original and repaired bridge are illustrated in Fig. 7.

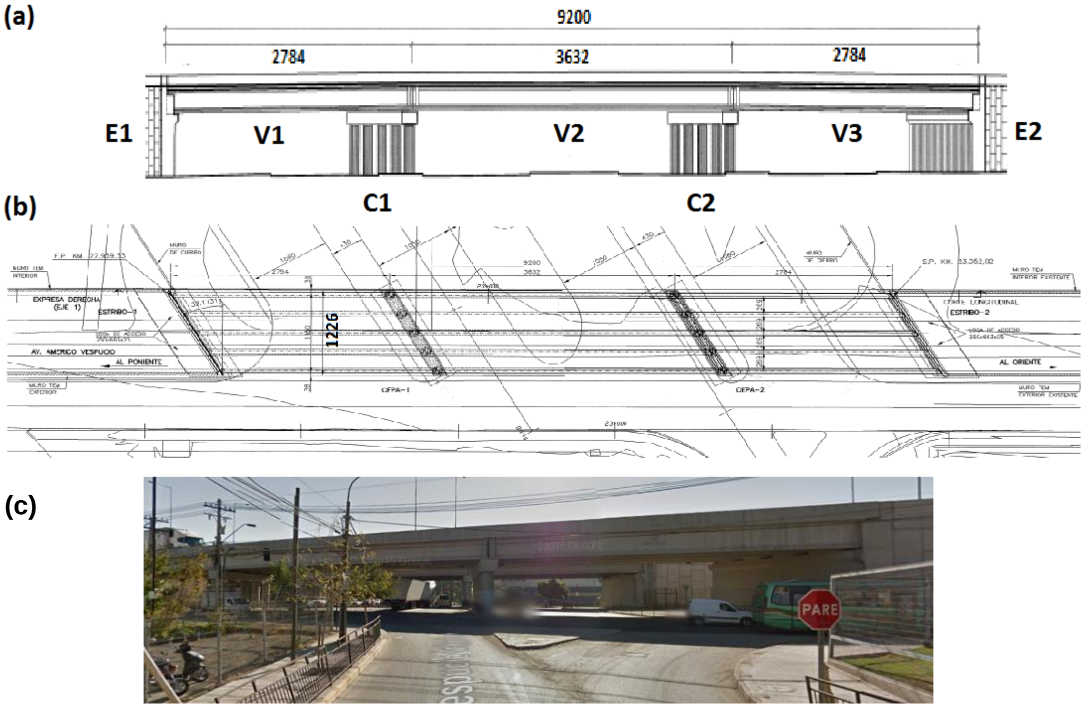
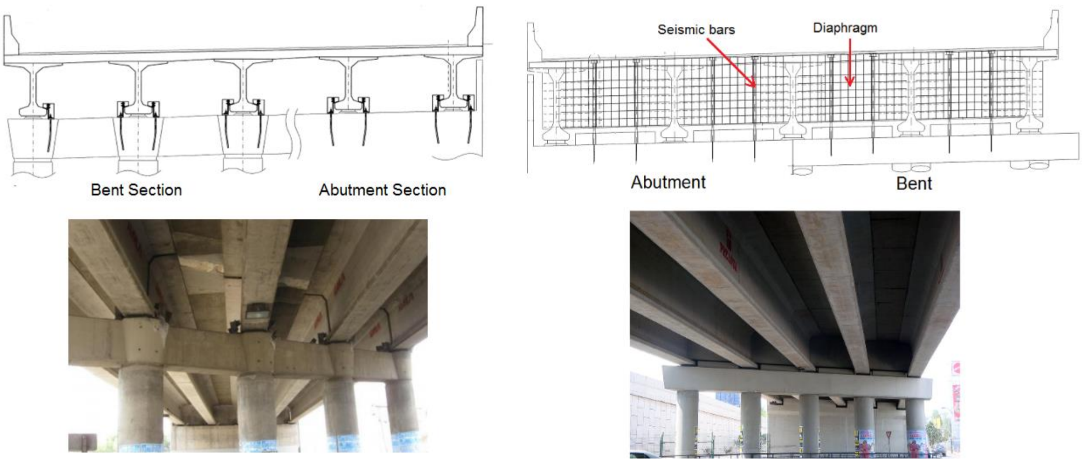


Fig. 6 Lo Echevers overpass (a) elevation view - units in cm, (b) plan view, (c) photo.



(a) (b)

Fig. 7 Lo Echevers overpass (a) original bridge, (b) repaired/retrofitted bridge

4.2 Numerical Model

In order to assess the seismic behaviour of the bridge, a 3D numerical model was generated in OpenSees (2013). The model considers different components that affect the seismic response of bridges. The superstructure of the bridge was modeled using a spine model composed by elastic frame elements since it was assumed that the superstructure remained in the elastic range (Caltrans, 2013). Translational and rotational masses were applied to the model based on tributary areas following the recommendations of Aviram et al. (2008).

For the bents, non-linear frame elements with distributed plasticity were adopted. The cross-section of these non-linear elements were discretized with steel fibers to account for the longitudinal steel reinforcing, and concrete fibers that take into account confined (core) and unconfined properties for the concrete. Appropriate constitutive laws were used, in such a way that the force-strain relationship of the section is derived from the integration of the uniaxial stress-strain relationship of the fibers. Likewise, the soil-structure interaction in the abutments was modeled by non-linear springs at the ends of the model with adequate non-linear force-deformation relationships, which are based on the AASHTO provisions (AASHTO, 2009). Fig. 8 shows the analytical model of Lo Echevers Bridge.

(a)
(b)
(c)

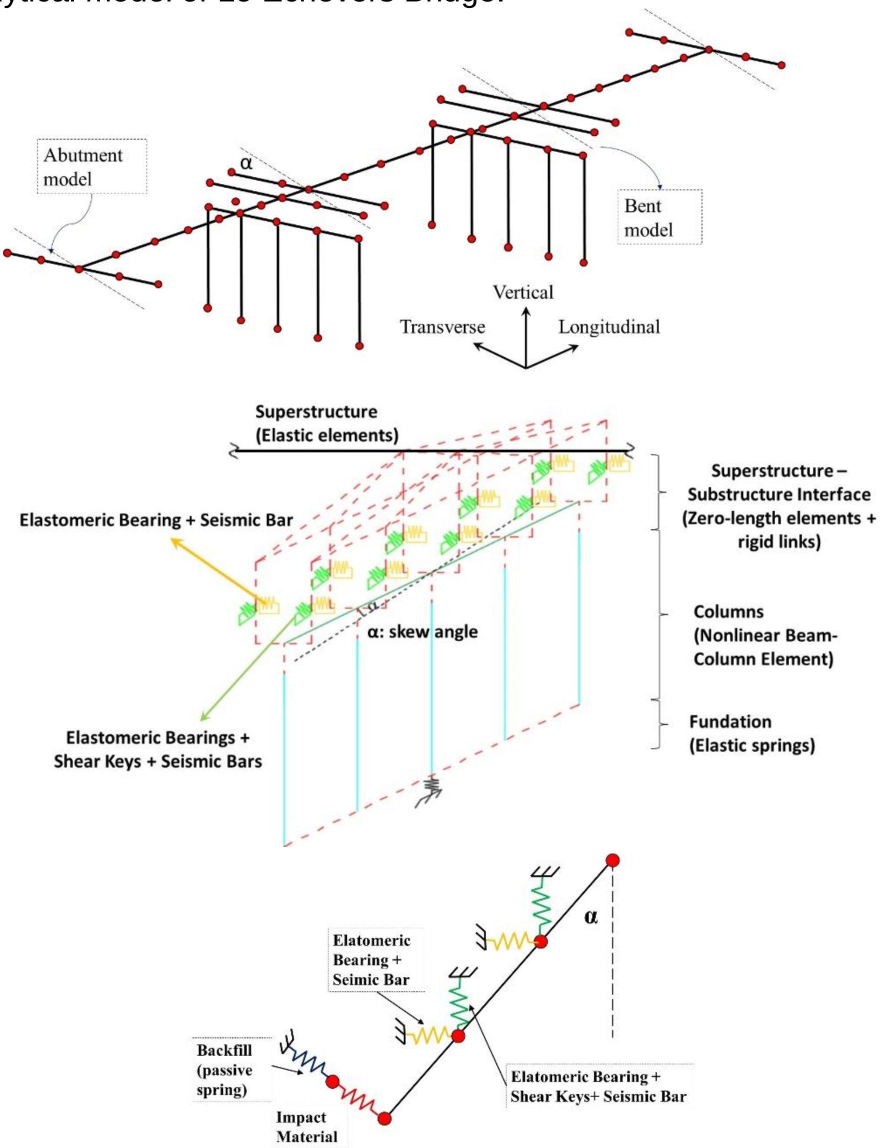


Fig. 8 3D numerical model of Lo Echevers Bridge. (a) General, (b) Bents, (c) Abutments

The force-deformation relationships considered for the different elements are shown in Fig. 9. The analytical model for the elastomers follows a Coulomb friction model as suggested by Steelman et al. (2012), which is based on the friction between the elastomeric bearing (neoprene) and the concrete. Additionally, a shear modulus (G) of 13 [kgf/cm^2] was used to determine the elastomeric bearing stiffness as recommended by the Chilean code (MOP, 2017). For the seismic stoppers, the original bridge had steel stoppers, which were modeled based on an experimental campaign carried out by Hube et al. (2012). In the repaired/retrofitted condition, the bridge has reinforced concrete stoppers that were modelled based on the studies of Megally et al (2001) and Goel & Chopra (2008). For both bridge cases, original and repaired, the shear keys were modelled by using a hysteretic material. The seismic bars, which are a common element in Chile but not in other earthquake prone regions, were modelled based on an experimental campaign carried out by Martinez et al. (2017).

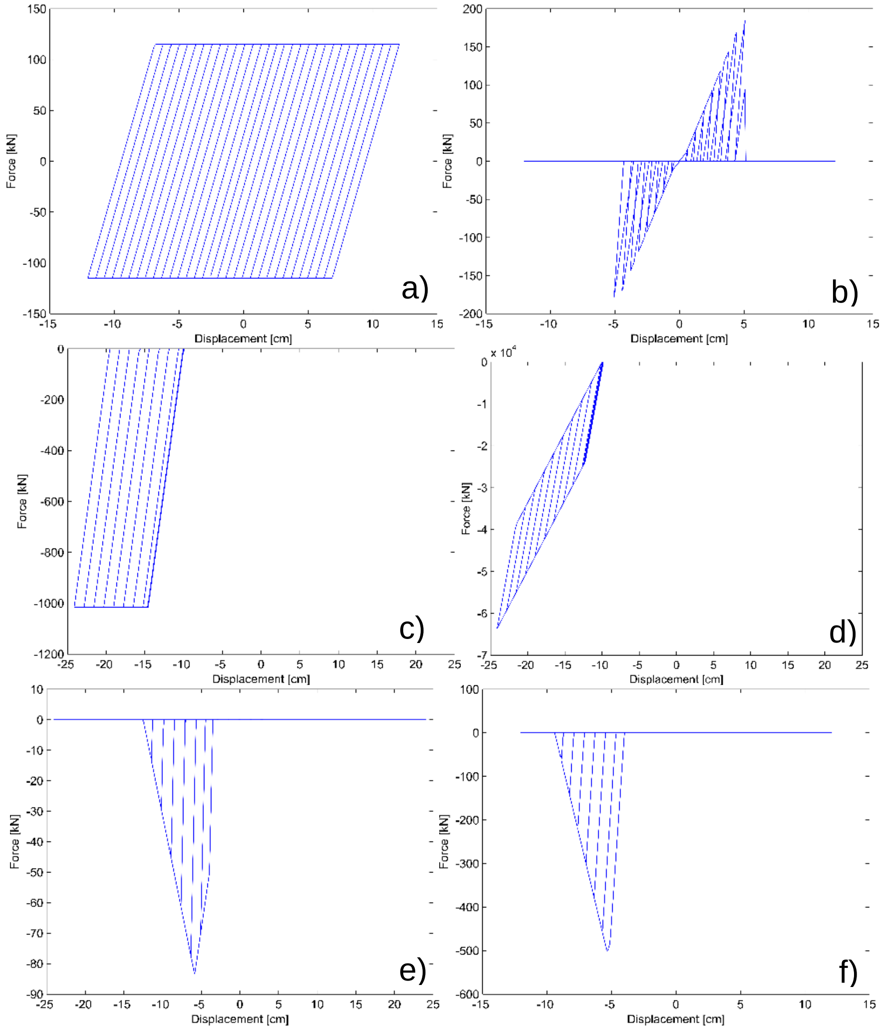


Fig. 9 Force-displacement curves for different elements. a) Elastomeric bearings, b) Seismic bars, c) Backfill, d) Impact material, e) Steel stopper, f) RC shear key

In order to obtain the dynamic properties of the bridge, modal analyses for the original and repaired condition were performed. For the original bridge, fundamental periods of 0.87, 0.86 and 0.39 [sec] were found for the first three modes of vibration, while 0.32, 0.31 and 0.23 [sec] were obtained for the repaired bridge. The large difference in the fundamental periods between the original and repaired bridge was caused by the inclusion of 16 vertical seismic bars for bent in the repaired bridge. Each seismic bar has a diameter of 32 [mm] with a yield stress of 420 [MPa]. The first three mode shapes are shown in Fig. 10.

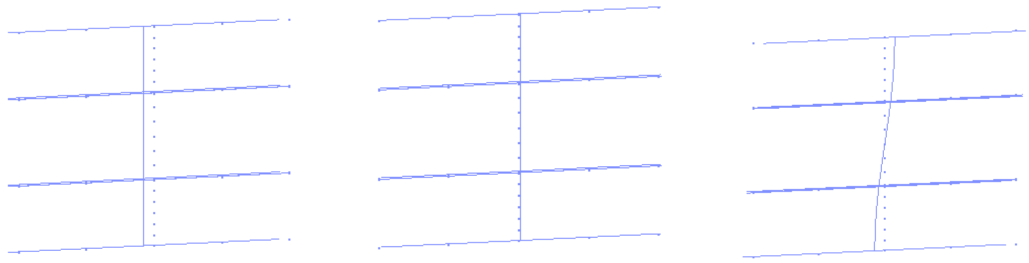


Fig. 10 Mode Shapes. a) Transverse, b) Longitudinal, c) Rotation

4.3 Ground Motion Selection and Scaling

With the aim of assessing the seismic performance of the original and repaired bridge, a selection of strong motion records was conducted in order to determine the inelastic demands imposed in the structure. 14 pairs of orthogonal horizontal subduction zone records were used in this study. The ground motions were chosen from the 2010 Chile earthquake (U. Chile, 2010). The ground motions used in the study are summarized in Table 1, which also includes the peak ground acceleration (PGA).

Table 1 Ground Motions

Station	Component	PGA (g)	Station	Component	PGA (g)
Santiago - Peñalolen	E-W	0.29	Concepcion	Longitudinal	0.39
	N-S	0.28		Transverse	0.27
Santiago - Centro	Longitudinal	0.23	Curico	Longitudinal	0.41
	Transverse	0.30		Transverse	0.47
Santiago - La Florida	E-W	0.13	Llolleo	Longitudinal	0.33
	N-S	0.19		Transverse	0.56
Santiago - Maipu	E-W	0.52	Valdivia	E-W	0.12
	N-S	0.52		N-S	0.08
Santiago - Puente Alto	E-W	0.26	Talca	Longitudinal	0.45
	N-S	0.27		Transverse	0.44
Viña - Centro	E-W	0.32	El Almendral	Longitudinal	0.22
	N-S	0.21		Transverse	0.27
Viña - El Salto	E-W	0.32	Constitucion	Longitudinal	0.54
	N-S	0.34		Transverse	0.63

Fig. 11 shows the 5% damped acceleration response spectra of each ground motion used in this study along with four response spectra representing the one specified in the Chilean Bridge Code, and three extra response spectra representing the Uniform Hazard Spectra (UHS) for the city of Santiago for events with return period of 475, 1000 and 2500 years (BID, 2016).

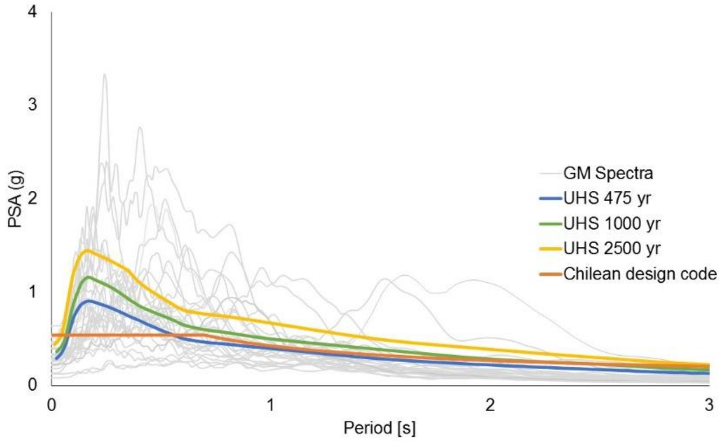
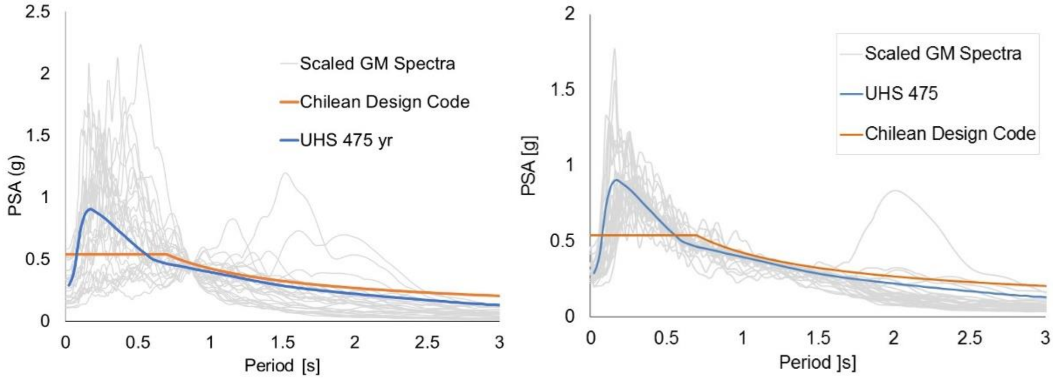


Fig. 11 Response spectra of each ground motion

In addition to the unscaled ground motions shown in Fig. 11, two scaling procedures were used in order to relate the level of damage of different components to a specific seismic hazard (475, 1000 and 2500 yr), i.e. the records were scaled to each return period. The first scaling procedure consisted on amplitude scaling the input records to the fundamental period of the structure (T_n), while the second approach was based on the spectral matching technique developed by Hancock et al. (2006). For the spectral matching a period range of $0.2T_n$ to $1.5T_n$ was set. The results of scaling the ground motions to the 475 year uniform hazard spectra using different scaling procedures are shown in Fig. 12.



(a) (b)

Fig. 12 Scaled response spectra to the 475 year UHS using (a) amplitude scaling, (b) spectral matching.

4.4 Damage Limit States

In order to determine the level of damage of different bridge components, limit states for the multi-column bents and elastomeric bearings were used as indicated in Table 2 . Displacement ductility was used as the controlling demand parameter for the multi-column bents (Billah & Alam, 2015), while for the elastomeric bearings the displacements suggested by Ramanathan et al. (2010) were used.

Table 2 Damage limit states

Component	Demand Parameter	Damage Limit State			
		Slight	Moderate	Extensive	Collapse
Multi-column bent	Ductility (μ)	1.0	1.22	1.78	4.8
	Displacement (cm)	1.6	2.0	2.9	7.9
Elastomeric bearing	Displacement (cm)	2.9	10.4	13.6	18.7

4.5 Results and Discussion

Non-linear static (pushover) and non-linear time history analysis were performed to assess the seismic performance of Lo Echevers Bridge.

The non-linear static procedure was used to determine the capacity curve of the multi-column bents, consequently, the yield displacement of the bent, which is used for defining the displacement ductility, was also obtained from this analysis. The capacity curves for the original and repaired bridge are shown in Fig. 13. Fig. 13(a) shows the capacity curve with the controlling point at the deck level. Consequently, the contribution of all elements located in the superstructure-substructure interface are considered (seismic stoppers, shear keys, seismic bars and elastomeric bearings). It is interesting to note that in the original bridge the gap between the steel stopper and the longitudinal PC girder is 3.5 [cm], while in the repaired bridge the gap with the RC shear key is 15 [cm] as inferred from Fig. 13(a). Thus, the response of the original bridge before 3.5 [cm] and after 13 [cm], which is the point at which the steel stopper fails, is controlled by the elastomeric bearing response. Fig. 13(a) also shows that the repaired bridge is stiffer and has more than twice the strength than the original bridge. This result was caused by the inclusion of the seismic bars and reinforced concrete shear keys in the repaired condition. Another important result is that the multi-column bent in the original bridge is not able to develop its full strength since the steel stoppers are not strong enough to get the columns to yielding as shown in Fig. 13(b). On the contrary, the columns in the repaired bridge can reach yielding.

The results of a nonlinear analysis in the transverse direction for a selected pair of ground motions are shown in Fig. 14. The results show that the displacements of the superstructure in the original condition are larger than in the repaired condition (Fig. 14a). This result is a consequence of the lack of effective restraining devices in the original bridge. On the contrary, the displacements of the substructure are larger in the repaired condition since the inclusion of seismic bars and shear keys caused that greater forces were transferred to the substructure, indicating that the design of a ductile substructure is required in this case.

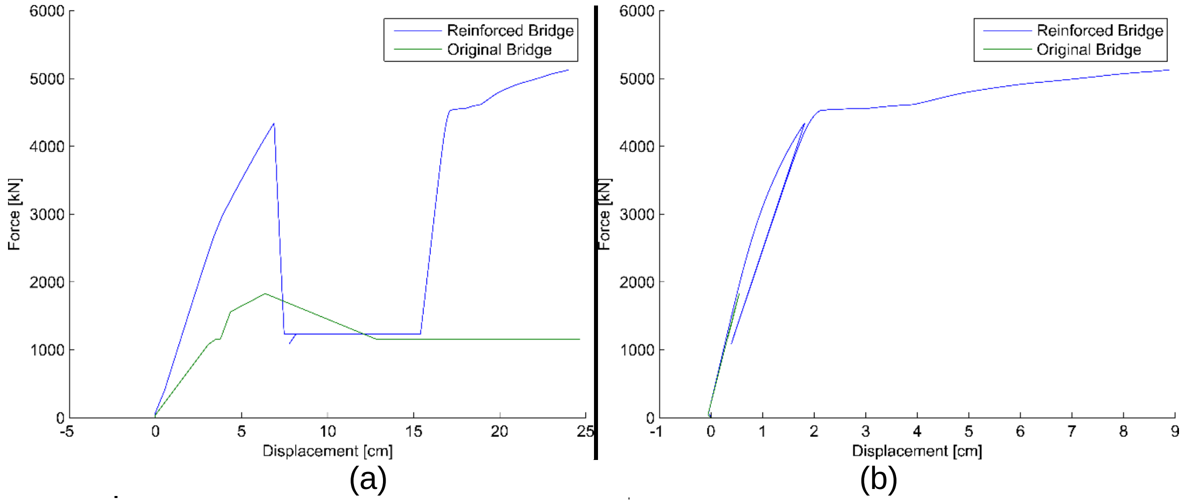


Fig. 13 Capacity curves for multi-column bents (a) considering elastomeric bearings, seismic bars and shear keys, (b) considering only the columns.

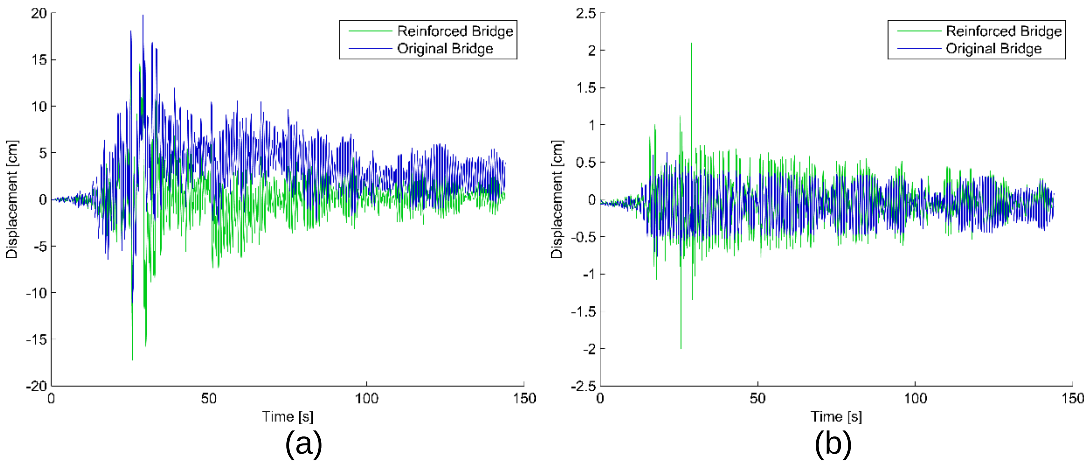


Fig. 14 Time-history response. (a) Superstructure, (b) Substructure

The damage states for the multi-column bent and elastomeric bearings using the unscaled ground motions are shown in Fig. 15. The resulting displacements for the elastomeric bearings (Fig. 15a) were recorded at the abutments and bents. These results show that the repair measures were effective in reducing the displacements demands in the elastomeric bearings. Moreover, a direct relation can be observed between the peak ground acceleration and the displacement demand in the bearings.

It is important to mention that in the original condition 100% of the displacement demands exceeded the slight, 50% the moderate, 20% the extensive and 4% the collapse damage states, while in the repaired condition 95 % exceeded the slight damage state and none of the records cause the bearings to collapse.

Regarding the damage states in the columns, it can be observed that in the original condition none of the demands exceeded the slight damage state, which indicates that the columns did not reach yielding. This result was also concluded from the capacity curves. For the repaired condition, on the contrary, 50 % of the ground motions caused column yielding and two records caused extensive damage.

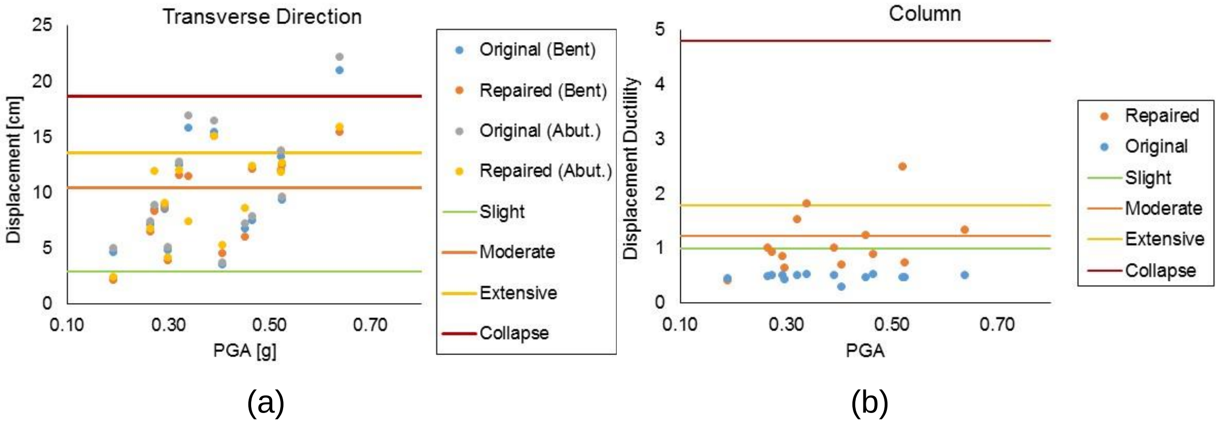


Fig. 15 Damage states using unscaled records. (a) Elastomeric bearings, (b) Columns

The damage states for the multi-column bent and elastomeric bearings using the scaled ground motions are shown in Fig. 16 and Fig. 17 for the amplitude scaling and spectral matching techniques, respectively.

In terms of damage states, it can be observed from Fig. 16 that all the ground motions caused slight damage in the bearings and as expected the damage in the bridge get more severe as the seismic hazard increase. This result becomes clear when the displacement demands obtained for the 475 and 2500 yr hazard levels are compared. While for the 475 yr hazard none of the ground motion exceeded the collapse damage state, 10 ground motions exceeded it for the 2500 yr hazard. It is important to note that even though the repair measures limit the displacements and improve the performance of the superstructure (elastomeric bearings), collapse damage state can still be reached for a severe ground motion (2500 yr event).

Regarding the damage states in the columns, similar conclusions to previous sections can be mentioned, i.e. in the original bridge none of the ground motions can cause column yielding since the forces transferred from the superstructure to the substructure are not enough to cause such state. On the contrary, in the repaired bridge many of the considered ground motions can demand the columns above its yielding and in cases of severe ground motions (2500 yr event), the columns may even be closed to reaching the collapse damage state. Again, this result raises a red flag about the importance of designing a ductile substructure where seismic bars and strong shear keys are used.

Comparing both scaling approaches, the results show that when the amplitude scaling technique is used a larger dispersion in the displacement demands is obtained. The spectral matching presents less dispersion, however, it is more sensitive to the selection of ground motions. This problem can be observed in the fact that larger displacement demands were obtained for the elastomeric bearings in the repaired bridge. This results was presumably caused by the lower acceleration demands obtained in the 1 to 2 [sec] period range when using the spectral matching technique (see Fig. 12b).

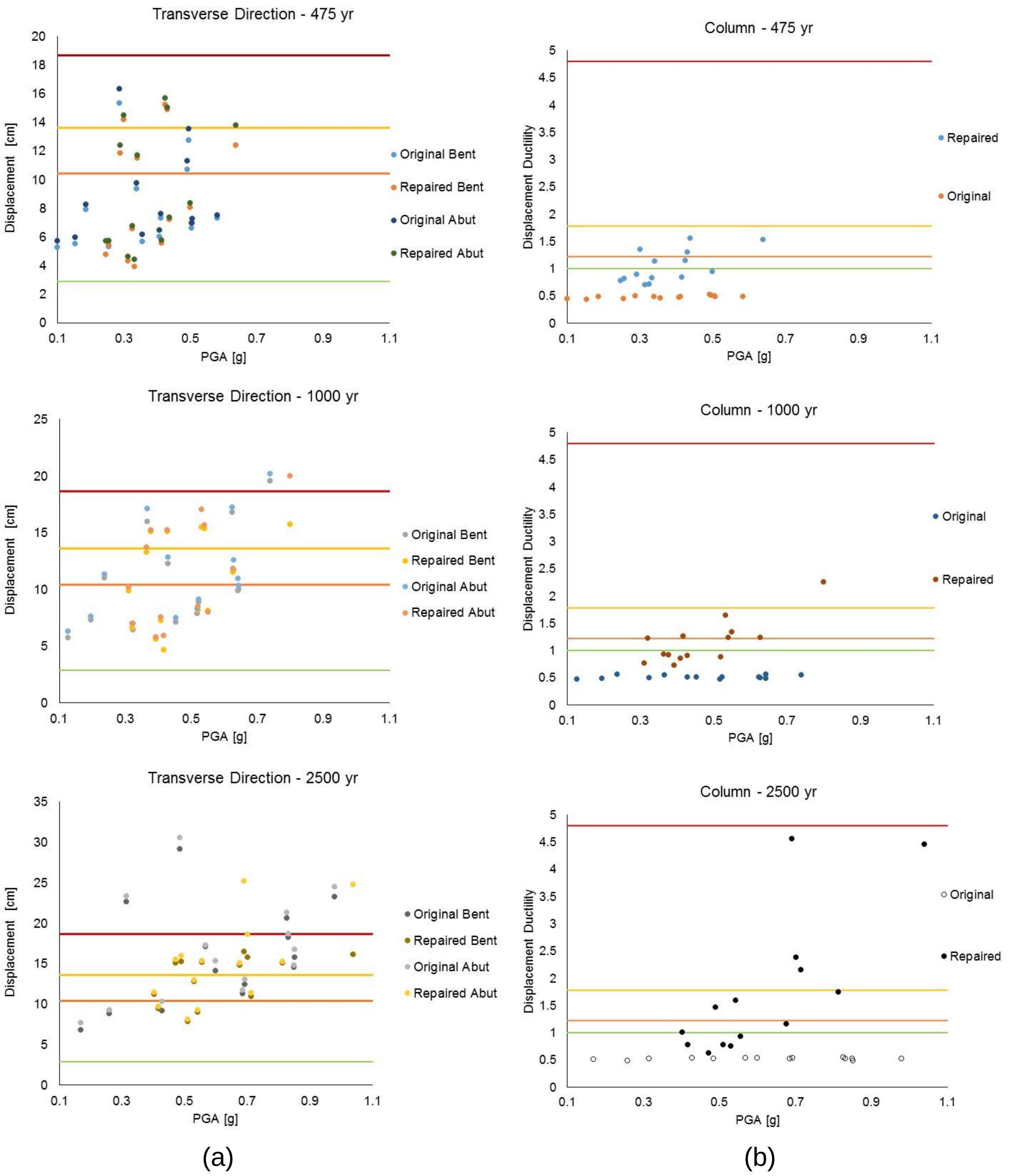


Fig. 16 Damage states using amplitude scaling. (a) Elastomeric bearings, (b) Columns

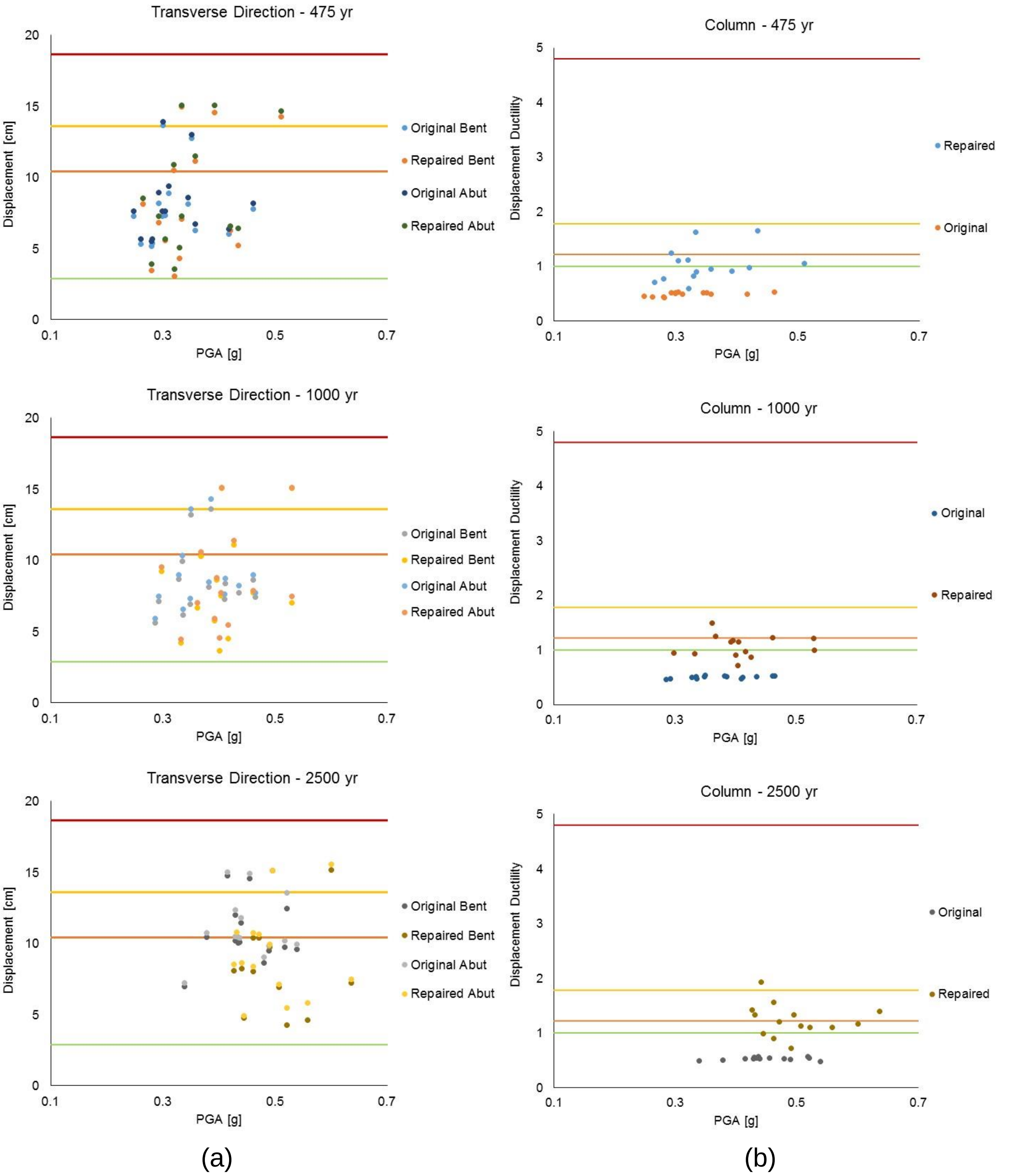


Fig. 17 Damage states using spectral matching. (a) Elastomeric bearings, (b) Columns

5. SUMMARY AND CONCLUSIONS

This work presented the damage observed in bridges during the 2010 Chile earthquake and measures taken for repairing them. The paper also presented the seismic design philosophy of bridges in Chile and the major modifications made to the Chilean bridge code after the earthquake. Then, the paper focuses on the seismic assessment of an overpass that collapsed during the 2010 earthquake. The damaged bridge was repaired by replacing the bridge deck and adding elements to limit excessive displacements in the superstructure, such us vertical seismic bars, transverse diaphragms and reinforced concrete shear keys. Numerical models to predict the behavior of the original and repaired system were developed. The numerical models were used to carry out non-linear static and time history analyses. The results from the non-linear static analysis showed that the repaired bridge is stiffer and has more strength that the original bridge. Furthermore, it was demonstrated that the multi-column bent was not able to reach yielding in the original bridge since the forces transmitted to the substructure were not enough to cause that state.

The results from the non-linear time history analysis showed that even though the repair measures used in Chilean bridges can prevent unseating of spans since they limit the displacement of the superstructure, extensive to collapse damage states for elastomeric bearings can still be expected under severe ground motions (2500 yr event). Regarding the seismic performance of columns, slight to extensive damage is expected in the substructure as a consequence of the greater forces being transmitted from the superstructure.

Finally, the results of this study indicates that a shift on the inelastic demands occurs when a bridge is repaired using the Chilean common practice of adding seismic bars and sturdy shear keys. This was clearly observed in the fact that for all the considered ground motions the inelastic demands in the original bridge were concentrated in the elastomeric bearings, while in the repaired bridge were transferred to the substructure, indicating that the design of a ductile substructure is mandatory when adding seismic bars and strong shear keys.

6. ACKNOWLEDMENTS

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REFERENCES

AASHTO, 1996. *LRFD Bridge Design Specifications, 2nd ed.,* Washington, D.C: American Association of State Highway and Transportation Officials.

AASHTO, 2009. *Guide Specifications for LRFD Seismic Bridge Design,* s.l.: American Association of State Highway and Transportation Officials.

Aviram, A., Mackie, K. & Stojadinović, B., 2008. *Guidelines for nonlinear analysis of bridge structures in California.,* s.l.: s.n.

BID, 2016. *Perfil de Riesgo de Desastre, Informe Nacional para Chile (in Spanish),* s.l.: Banco Interamericano de Desarrollo.

Billah, A. & Alam, M., 2015. Seismic fragility assessment of concrete bridge pier reinforced with superelastic shape memory alloy. *Earthquake Spectra,* 31(3), pp. 1515-1541.

Buckle, I. et al., 2012. Structural performance of bridges in the offshore Maule earthquake of 27 February 2010. *Earthquake Spectra,* 28(S1), pp. S533-S552.

Caltrans, 2013. *Seismic Design Criteria,* Sacramento, California: California Department of Transportation.

Elnashai, A. S. et al., 2012. The Maule (Chile) earthquake of February 27, 2010: Development of hazard, site specific ground motions and back-analysis of structures.. *Soil Dynamics and Earthquake Engineering,* pp. 229-245.

Goel, R. & Chopra, A., 2008. Role of shear keys in seismic behavior of bridges crossing fault-rupture zones. *Journal of Bridge Engineering,* 13(4), pp. 398-408.

Hancock, J. et al., 2006. An improved method of matching response spectra of recorded earthquake ground motion using wavelets. *Journal of Earthquake Engineering,* 10(1), pp. 67-89.

Hube, M. & Rubilar, F., 2012. *Capacity evaluation of steel stopers of reinforced concrete Chilean bridges.* Lima, Peru, The international symposium for CISMID 25th Anniversary.

Kawashima, K., Unjoh, S., Hoshikuma, J. I. & Kosa, K., 2011. Damage of bridges due to the 2010 Maule, Chile, earthquake. *Journal of Earthquake Engineering,* 15(7), pp. 1036-1068.

Martínez, A., Hube, M. & Rollins, K., 2017. Analytical fragility curves for non-skewed highway bridges in Chile. *Engineering Structures,* Volume 141, pp. 530-542.

Megally, S., Silva, P. & Seible, F., 2001. *Seismic response of sacrificial shear keys in bridge abuments,* CA: Departament of Structural Engineering.

MOP, 2002. *Manual de Carreteras,* Santiago, Chile: Ministerio de Obras Públicas, Dirección de Vialidad.

MOP, 2010. *Nuevos Criterios Sísmicos Para el Diseño de Puentes en Chile,* Santiago, Chile: Ministerio de Obras Públicas, Dirección de Vialidad.

MOP, 2017. *Manual de Carreteras,* Santiago, Chile: Ministerio de Obras Públicas, Dirección de Vialidad.

NCHRP, 2013. *Performance-Based Seismic Bridge Design, Synthesis 440,* Washington, D.C.: Transportation Research Board, National Academy of Sciences.

OpenSees, 2013. *Open System for Earthquake Engineering Simulation, Pacific Earthquake Engineering Research Center, University of California, Berkeley V. 2.5.0.* [Online]
Available at: http://opensees.berkeley.edu

Ramanathan, K., DesRoches, R. & Padgett, J., 2010. Analytical fragility curves for multispan continuous steel girder bridges in moderate seismic zones. *Transportation Research Record: Journal of the Transportation Research Board,* Volume 2202, pp. 173-182.

Schanack, F., Valdebenito, G. & Alvial, J., 2012. Seismic damage to bridges during the 27 February 2010 magnitude 8.8 Chile earthquake. *Earthquake Spectra,* 28(1), pp. 301-315.

Steelman, J. et al., 2012. Shear and friction response of nonseismic laminated elastomeric bridge bearings subject to seismic demands. *Journal of Bridge Engineering,* 18(7), pp. 612-623.

U. Chile, 2010. *University of Chile, Earthquakes of Chile.* [Online]
Available at: http://terremotos.ing.uchile.cl/registros/164