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SEISMIC ASSESSMENT OF SKEWED HIGHWAY BRIDGES IN CHILE

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Abstract

The 2010 Maule earthquake (moment magnitude 8.8) had a significant socioeconomic impact on the Chilean highway network, in great part, by the extensive damage in bridges. Around 300 bridges suffered different levels of damage during the earthquake, which exposed a sequence of vulnerabilities in these structures that were attributed to deficiencies in the Chilean bridge design code. Some of these deficiencies were inadequate strength and stiffness provided by shear keys, absence of diaphragms and short seat support lengths at abutments and bents, among others. Despite the Chilean code was modified after the 2010 earthquake, there are still technical gaps in the assessment, design, and repair and retrofit of these structures. This situation is particularly important in skewed bridges since it is recognized that this type of bridge is more vulnerable because of their susceptibility to develop excessive in-plane deck rotations, consequently having greater probability of collapse if they have insufficient seat lengths at bents and abutments. Therefore, in order to improve their resilience and improve the design code, a comprehensive study on the seismic behavior of Chilean skewed bridges must be performed. With this aim, the research is focused on assessing the seismic performance of a set of Chilean skewed highway bridges that were damaged during the 2010 Maule earthquake.

To achieve the objective of this study, refined nonlinear tridimensional models for skewed Chilean bridges are performed in OpenSees. Different case studies of prestressed concrete girder bridges with multi-column bents and seat-type abutments are considered in their as-built and repaired conditions. The repaired bridges consider an increase on seat lengths, anchorage of elastomeric bearings in some cases, addition of transverse diaphragms between girders, vertical seismic bars and internal concrete shear keys as compared to the as-built bridges. In order to assess the seismic performance of both bridge configurations, fragility curves are obtained through incremental dynamic analysis (IDA). The nonlinear time history analyzes are performed using subduction ground motions recorded at several stations in order to account for record-to-record variability. Relevant engineering demand parameters (EDPs) are used to assess damage states of different bridge components. The results of this study show that the adopted repair measures have a significant effect on seismic performance. Furthermore, the results indicate that a shift on the inelastic demands occurs when a bridge is repaired using the Chilean common practice of adding vertical seismic bars and shear keys.

Keywords: highway bridges, seismic performance, fragility curves

1. Introduction

In February 27, 2010, a subduction earthquake of magnitude 8.8 on the moment magnitude scale occurred with an epicenter on the offshore of Maule region, Chile, causing important human and material losses. In particular, damage to structures was characterized by the collapse of a few, but not less important, bridges, overpasses and pedestrian bridges, which were designed according to the Chilean Bridge Design Manual (MC) [1]. The main causes of these failures were an inadequate transversal restriction and short seat lengths for girders, which were simply supported on elastomeric bearings. In the absence of the above, excessive displacements and rotations can occur, leading to major damage or collapse due to unseating of the superstructure [2] [3]. Skewed bridges are especially vulnerable to this failure mechanism, since they tend to rotate even more, due to the impact forces produced by pounding between the deck and the abutments [4]. An extensive study of the seismic behavior of "Chilean type" skewed bridges is not yet performed and are still technical gaps in the assessment of skewed bridges.

It is well known that bridges integrate an essential part of a country infrastructure, since they ensure the connectivity of the transport network, hence, an adequate performance level must be ensured to keep their serviceability, even after a seismic event, i.e. collapse is not admitted in any case. Following this philosophy, the Chilean bridge design manual [5] was improved after the 2010 earthquake, but there are still numerous existing bridges with deficiencies, namely, absence of diaphragm between girders, weak steel stoppers and concrete shear keys and short seat lengths. Fig. 1 shows the typical superstructure configurations observed in Chilean highway bridges. Fig. 1(a) and (b) show the bridge configurations allowed by the design code before the 2010 earthquake, while in the current bridge design code, i.e. updated after 2010, the type (a) and (b) are not allowed, and a type (c) configuration must be considered. In other words, new bridges and damage bridges repaired after the 2010 must include diaphragms, interior reinforced concrete shear keys, vertical seismic bars, and longer seat lengths [6].

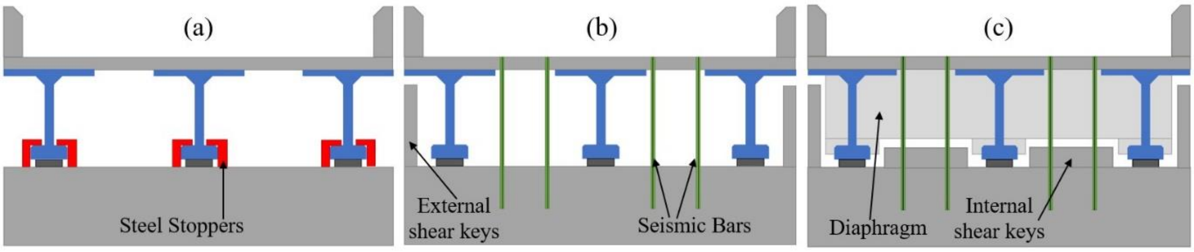

Fig. 1 – Typical Superstructure Configurations. (a) with steel stoppers, without diaphragm. (b) with external concrete shear keys and seismic bars, without diaphragm. (c) with concrete shear keys, seismic bars and diaphragm.

In this study, the seismic performance of a set of Chilean skewed bridges in their original design, before the 27F Maule earthquake, and after the repair measures are compared; thus, reflecting the changes made to the design code as described above. With this aim, refined models of these bridges are performed in OpenSees [7]. Time history analyses are carried out to assess the seismic response of bridges subjected to ground motion records scaled to the Chilean bridge design spectra [5] and afterwards fragility curves are obtained using incremental dynamic analyses [8].

2. Case Studies

The selected case studies are located in the central zone of Chile and represent the characteristics of typical Chilean skewed highway bridges: designed with seat-type abutments, multiple-column bents, precast prestressed concrete girders and elastomeric bearings. For this research, four Chilean bridges (overpasses and underpasses) are considered in their original and repaired state, i.e. before and after the 2010 Maule

Earthquake. These bridges are located in the Chilean seismic zone 2, with a design peak ground acceleration of 0.3 [g] [5], on soil types 2 or 3.

Table 1 – Case Studies

Bridge	Location	Seismic Zone	Soil Type	Skew [°]	N° Spans	N° Girders	N° Columns per bent
Las Mercedes	Route 5	2	*	11.1	2	3	2
Lo Echevers	Vespucio Norte	2	II	29.9	3	5	5
Los Pinos	Route 5	2	III	28.2	2	5	4
Miraflores	Vespucio Norte	2	II	21.6	3	5	5

* No data available. For the purpose of the study, the analysis was made for a type III soil in this case.

General Information for the Case Studies			
Bridge	**Original (O)**	**Repaired - Retrofitted (R)**	**Damage observed**
Las Mercedes (LM)	No Diaphragm, no internal shear keys, external concrete shear keys, seismic bars without diaphragm.	Diaphragm, internal and external concrete shear keys, seismic bars.	Rotation about the center bent, unseating. Shear keys completely damaged.
Lo Echevers (LE)	No Diaphragm, steel stoppers, external concrete shear keys in abutments, no seismic bars.	Diaphragm, internal and external concrete shear keys, seismic bars. Elastomers with anchorage.	Rotation of the deck. Superstructure unseating due to failure of steel stoppers.
Los Pinos (LP)	No Diaphragm, no internal shear keys, external concrete shear keys, seismic bars without diaphragm.	Diaphragm, internal and external concrete shear keys, seismic bars.	Shear keys failed, girders unseated due rotation of the deck and short seat lengths.
Miraflores (Mir)	No Diaphragm, steel stoppers, external concrete shear keys in abutments, no seismic bars.	Diaphragm, internal and external concrete shear keys, seismic bars. Elastomers with anchorage.	Rotation of the deck. Superstructure unseating due to failure of steel stoppers.

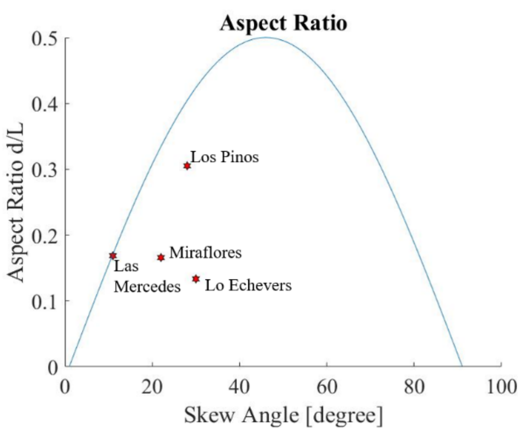

Fig. 2 – Aspect ratio for the case studies

In general, soil type 2 corresponds to soils with an in-situ shear wave velocity greater than 400 [m/s] in the first 10 [m], dense gravel, dense sand or hard cohesive soil. Soil type 3 corresponds to gravel, sand or cohesive soil with strength and stiffness properties less than type 2 [5].

Detailed properties and designations of the case studies are shown in Table 1.

The relationship between the aspect ratio (ratio between the deck width and the span length) and the skew angle of these bridges is shown in Fig. 2. The curve $2d/L = \sin(2\theta)$ represents the limits of skew angle for free rotation, which means that skewed bridges that fall inside the curve can freely rotate about any obtuse corner [4], [5], [9].

3. Ground Motion Records

In order to perform the analysis, ground motion records from the 2010 Maule (8.8M_w), 2015 Illapel (8.4M_w) and 2017 Valparaiso (6.9M_w) earthquakes were considered. For each case study, ground motions recorded in a station in the same seismic zone and on the same soil type of the respective bridge were considered. A baseline correction and a butterworth filter were applied to these records. Records were obtained by the Chilean

Seismological Center (CSN) [8] and are shown in Table 2. Soil classification was obtained from the station data [8] and from the study of Wilches et al. [9].

Table 2 – Accelerogram Stations Considered

Earthquake	Station	Coordinates	Zone	Soil type
Maule: February 27 2010	Curico	-34.991 -71.237	2	II
	Stgo La Florida	-33.514 -70.605	2	II
	Stgo Maipu	-33.509 -70.771	2	II
	Stgo Peñalolén	-33.501 -70.579	2	III
Illapel: September 16 2015	R02M	-33.473 -70.660	2	II
	R05M	-33.443 -70.534	2	II
	R20M	-33.665 -70.929	2	II
	R12M	"33.389 -70.622	2	III
	R18M	-33.508 -70.749	2	III
	R21M	-33.381 -70.796	2	III
Valparaíso: April 24 2017	R02M	-33.473, -70.660	2	II
	R05M	-33.443, -70.534	2	II
	R20M	-33.665 -70.929	2	II
	R12M	-33.389 -70.622	2	III
	R18M	-33.508 -70.749	2	III
	R21M	-33.381 -70.796	2	III

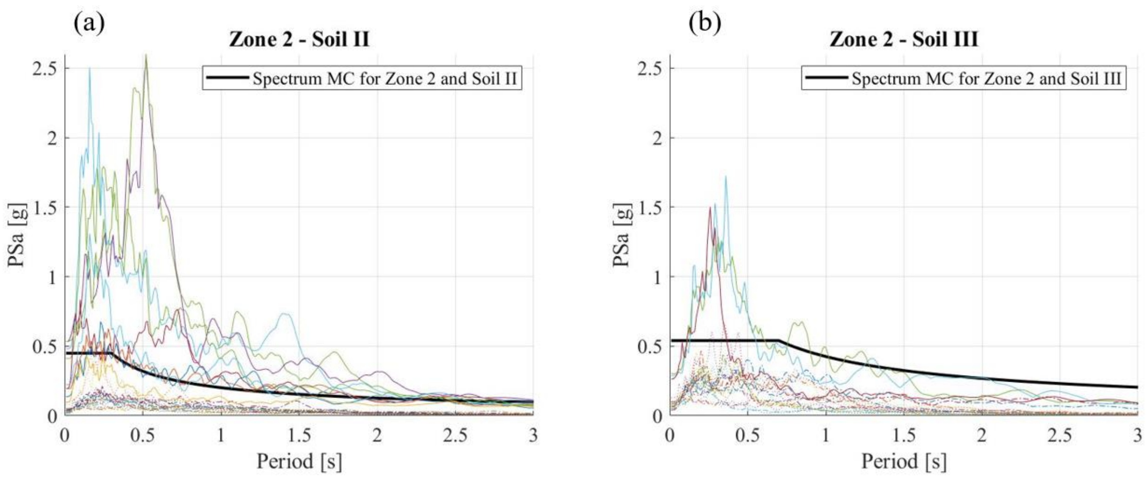

Fig. 3 – Response spectrum for the ground motions considered and bridge design spectra (MC) for seismic zone 2. (a) Soil type II. (b) Soil type III.

For each record and for each bridge, the geometric mean of the response spectrum at the fundamental period of the bridge, $PSa(t_n)$, of the three components of the record, two horizontal and one vertical, was calculated and scaled to a specific target value. For the time history analysis this target value corresponds to the acceleration at the fundamental period of the bridge for the design spectra [5]. For the incremental dynamic analysis this target value corresponds to each discretization of the range of intensities considered (from 0.2 to 2 [g]). Ground motion response spectrum without scaling and the design spectra, for each soil type, are shown in Fig. 3.

4. Modeling Overview

In order to perform the analysis for each case study in its original and repaired state, refined tridimensional nonlinear models were performed in OpenSees [7]. Special emphasis was put on modeling the nonlinear behavior of elements such as elastomeric bearings [12], shear keys [13], seismic bars, backfill passive pressure

[14] [15] and impact between the deck and abutments [16]. An overview of the modelling is shown in Fig. 4. Reference force – displacement relationships of materials are shown in Fig. 5.

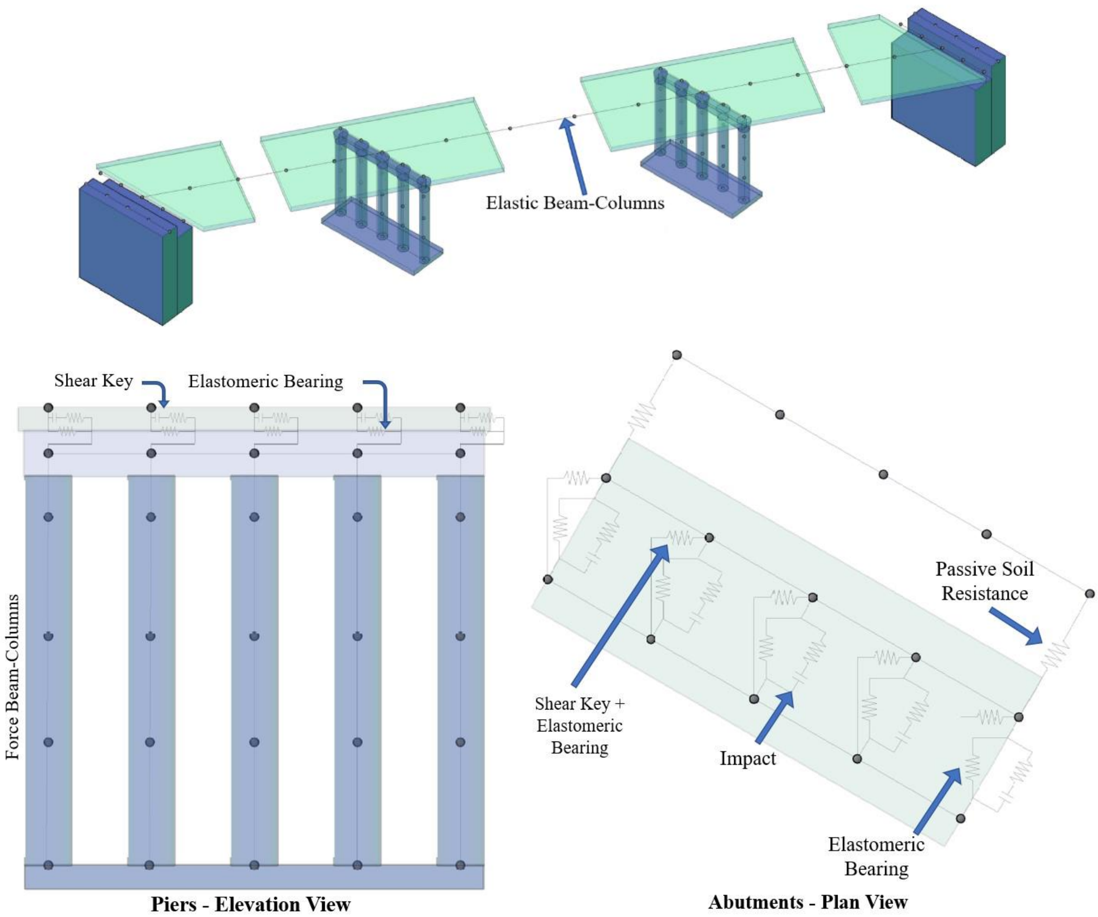

Fig. 4 – Modeling Overview

4.1 Superstructure

The superstructure was modeled using *elasticBeamColumn* elements from the Opensees library, with the composite section properties (deck and girders), since elastic behavior is expected in this bridge element [17]. Translational and rotational masses were assigned according to recommendations of Aviram et al. [18].

Elastomeric bearings without anchorage were modeled using a *flatSliderBearing* element, following a force-displacement relationship according to the Coulomb friction model proposed by Steelman et al. [12], with a shear modulus of 13 [kgf/cm^2] according to the Chilean bridge design manual [5]. On the other hand, for anchored elastomeric bearings, *elastomericBearingBoucWen* elements were considered. Modeling parameters for the elastomeric bearing were calibrated using experimental data.

Shear keys were modeled by *zeroLength* elements using nonlinear materials with an initial gap. For steel stoppers, the results of Rubilar [19] were used for modelling the material. They are based on an experimental campaign made by the same author. On the other side, internal and external concrete shear keys were modeled as proposed by Megally et al. [13].

The nonlinear behavior of seismic bars was calibrated using experimental data from a study conducted by Martinez [20]. The calibration process was performed in MatLAB using a global and local optimization algorithm. The material from the OpenSees library selected for calibration was the *Hysteretic* material. Vertical response was modeled using an *ElasticPP* Material with the steel properties of the vertical seismic bars.

4.2 Substructure and abutments

Fiber sections were used to model the bridge bents. The contribution of the reinforcing steel, and the confined and unconfined concrete for columns was considered in the fiber-section. *Forcebeamcolumn* elements with 7 integration points were used in the modeling. For superficial foundations linear springs with parameters according to ODOT [21] were considered. Piles were also modeled with linear springs representing the soil subgrade modulus [5].

Abutments were modeled using the spring configuration shown in Fig. 4. The shear key in transverse direction, the elastomeric bearing in longitudinal and transverse direction, and the impact between the deck and the abutment in a direction perpendicular to the abutment, are arranged in parallel. This is connected in series to the passive soil resistance material. A Hertz model modified by Muthukumar [16] was considered for modelling the impact between the deck and abutment (*ImpactMaterial)*. The backfill passive pressure force-displacement behavior was modeled with a *hyperbolicGapMaterial*, in the longitudinal direction of the bridge, based on CALTRANS [14] recommendations for seat type abutments and Shamsabadi [15].

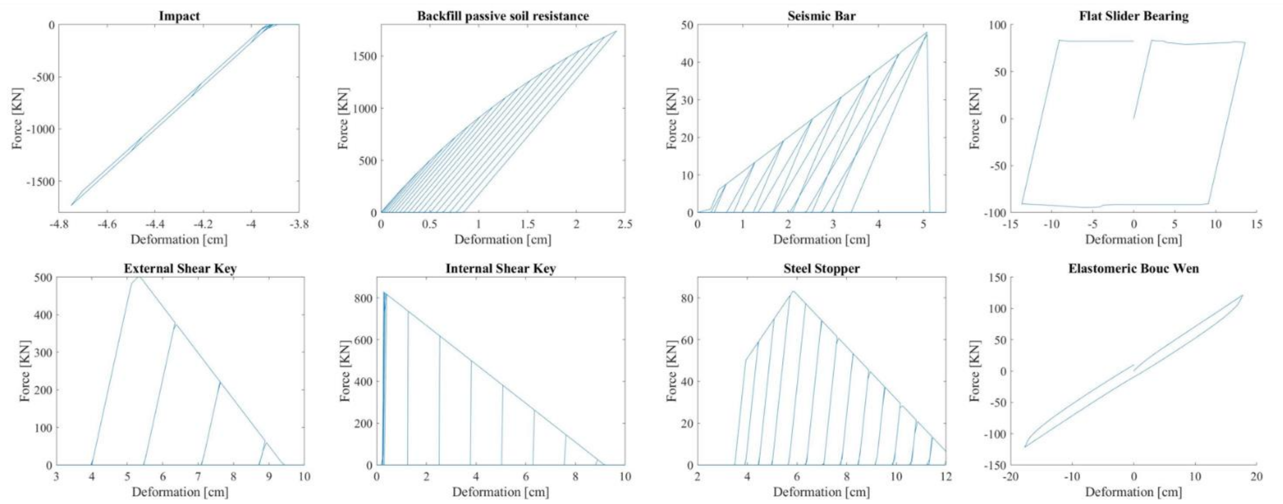

Fig. 5 – Force-Displacement relationship for different bridge components.

5. Results and Discussion

5.1 Modal Analysis

A modal analysis was performed in OpenSees for each bridge, three periods are shown in Table 3. In general, the first and second mode are translational, and the third mode is rotational. For the studied bridges, the average of the fundamental period for original bridges is around 0.76 [s] and for repaired bridges is 0.73 [s]. In general, the first three periods are greater in the original bridge. This means the repaired bridges tend to be stiffer than the original, mostly by using anchored elastomeric bearings, vertical seismic bars and diaphragm between girders.

Table 3 - Periods of the case studies

Periods						
Bridge	Original			Repaired		
	T1	T2	T3	T1	T2	T3
Las Mercedes	0,74	0,72	0,71	0,72	0,71	0,70
Lo Echevers	0,86	0,81	0,79	0,83	0,73	0,66
Los Pinos	0,70	0,65	0,60	0,69	0,65	0,60
Miraflores	0,74	0,69	0,67	0,68	0,63	0,60
Average	**0,76**	**0,72**	**0,69**	**0,73**	**0,68**	**0,64**

5.2 Seismic assessment of bridges using the Chilean design spectra

Time history analyses were performed in OpenSees with ground motions scaled to the design spectra at each fundamental period of the respective bridge [5], as shown Fig. 6 for Lo Echevers bridge. Results are shown in Fig. 7, where each point represents the maximum relative displacement of the elastomeric bearing and maximum column displacement ductility for each ground motion and for the bridges in both conditions.

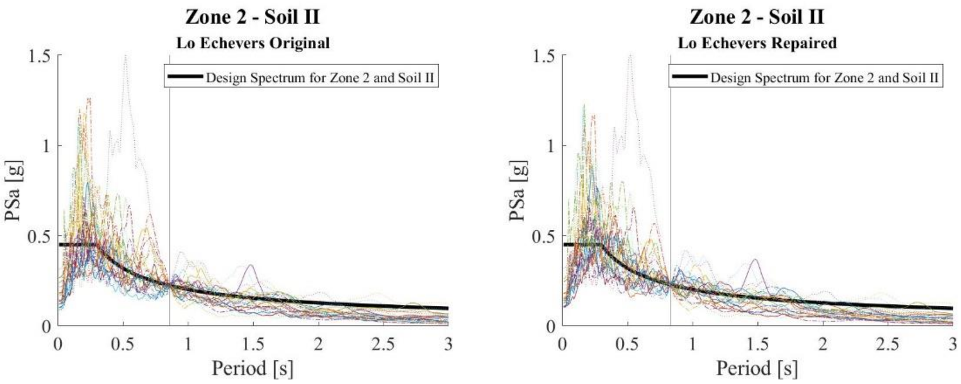

Fig. 6 – Scaled ground motions to the design spectra for Lo Echevers Bridge.

Limit states for both engineering demand parameters, namely, relative displacement of elastomeric bearings and displacement ductility of columns, obtained from Ramanathan [22] and from Billam & Alam [23], respectively, are shown in Table 4 and are also shown in Fig. 7. These are considered, in this section, to assess the response of the case studies subjected to ground motions scaled to the design spectrum [5], and, in the next section, to obtain the fragility curves for these bridges.

Table 4 – Limit States for Demand Parameters

Engineering Demand Parameter (EDP)	Limit states			
	Slight	Moderate	Severe	Collapse
Relative Displacement of Elastomeric Bearings [cm]	2.9	10.4	13.6	18.7
Displacement Ductility of Columns [μ]	1.0	1.2	1.8	4.8

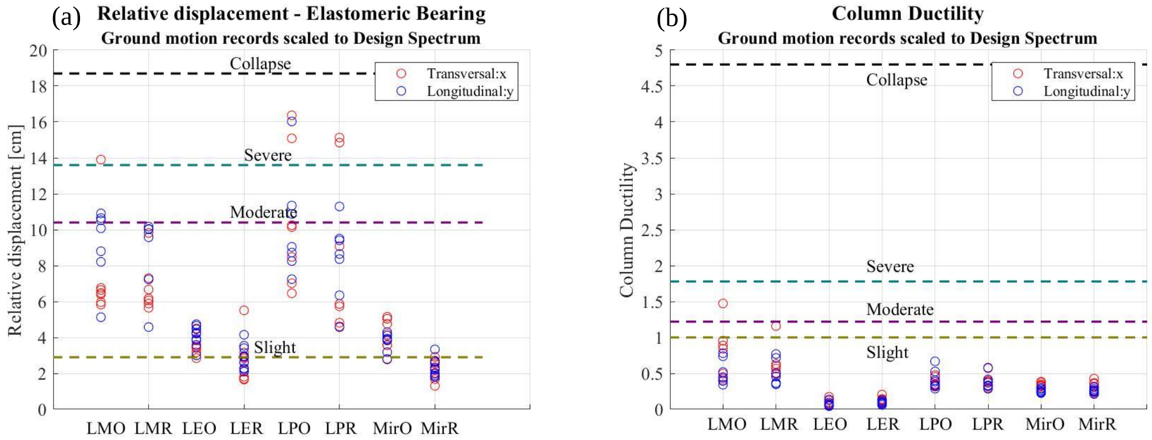

Fig. 7 – Time history analysis results. (a) Maximum relative displacement of the elastomeric bearings, (b) Maximum column ductility demand.

It can be observed that, for the scaled records at the design spectra, most of the damage on elastomeric bearings is slight to moderate, except for Los Pinos (LP) underpass, which reaches severe damage for some records. This result can be caused by the existence of pile foundation in this specific bridge. In general, the original bridges reach slightly higher levels of damage than the repaired ones. For the column ductility demand parameter, almost all bridges suffer less than slight damage, excepting Las Mercedes (LM), in which in one record, it reaches the moderate damage state. This result may be a consequence of the smaller column cross section of Las Mercedes overpass compared to the other case studies.

5.3 Incremental Dynamic Analysis and Fragility Curves

In order to assess the seismic performance of the studied cases, fragility curves were obtained using IDAs [23]. The same EDPs, namely, elastomeric bearing displacement and column displacement ductility, and the pseudo spectrum of acceleration at the fundamental period $PSa(t_1)$ as intensity measure (IM) were used. IDAs were performed for each bridge and for both conditions, varying $PSa(t_1)$ with a step of 0.2 [g] from 0.2 [g] to 2 [g]. For a specific limit state, the discreet probability of exceeding an intensity "y" is computed as indicate in Eq (1). Where, n_i is the number of cases that exceeds the damage state and N is the total of simulations. Then, a log-normal cumulative distribution function is fitted using Eq (2) to represent the fragility curve, where λ_{IM} and ξ_{IM} are the parameters of the distribution.

$$P[LS|IM = y] = n_i/N \quad (1)$$

$$P[LS|IM] = \int_{-\infty}^{IM} \frac{1}{\sqrt{2\pi}\xi_{IM}} \exp\left(-\frac{(\ln(im) - \lambda_{IM})^2}{2\xi_{IM}^2}\right) d(im) \quad (2)$$

Ida Curves for transversal relative displacement of the elastomeric bearings are shown in Fig. 8 and Fig. 9. Ida Curves for column ductility demand are shown in Fig. 10 and Fig. 11. Fragility curves for elastomeric bearing relative displacement demand parameter, for the collapse limit state, are shown in Fig. 12 and Fig. 13. Fragility curves for column ductility demand parameter for all bridges are summarized in Fig. 14.

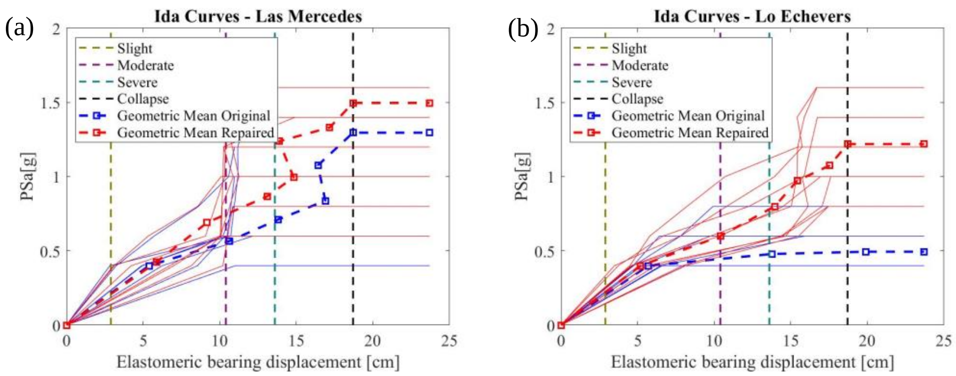

Fig. 8 - Elastomeric Bearing Displacement IDA Curves for: (a) Las Mercedes. (b) Lo Echevers.

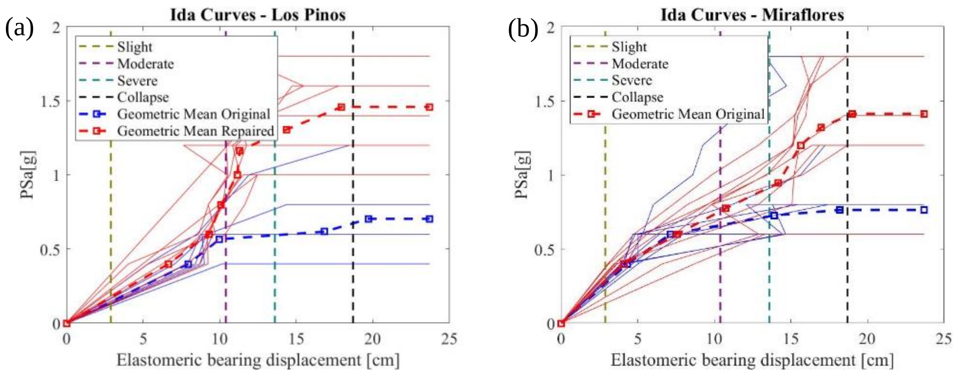

Fig. 9 – Elastomeric Bearing Displacement IDA Curves for: (a) Los Pinos. (b) Miraflores.

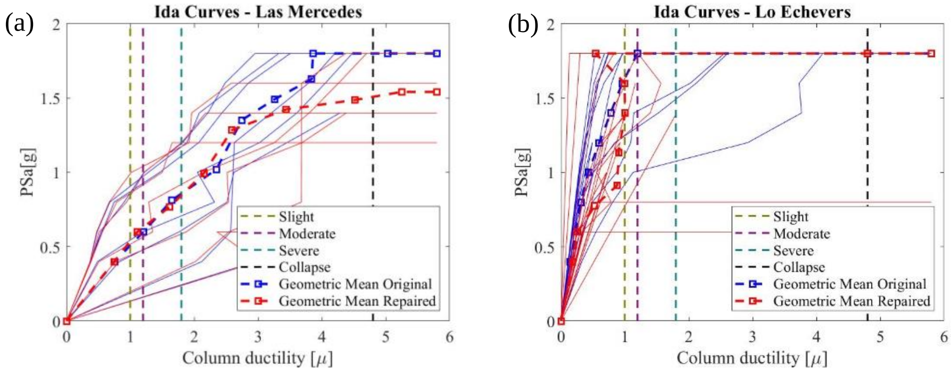

Fig. 10 – Column Ductility IDA Curves for: (a) Las Mercedes. (b) Lo Echevers.

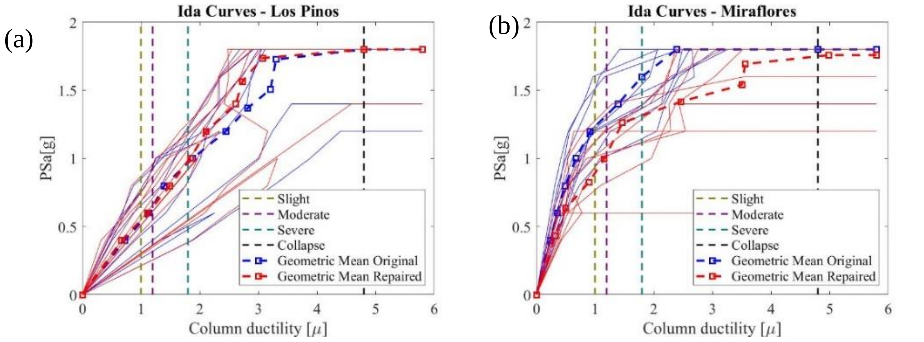

Fig. 11 – Column Ductility IDA Curves for: (a) Los Pinos. (b) Miraflores.

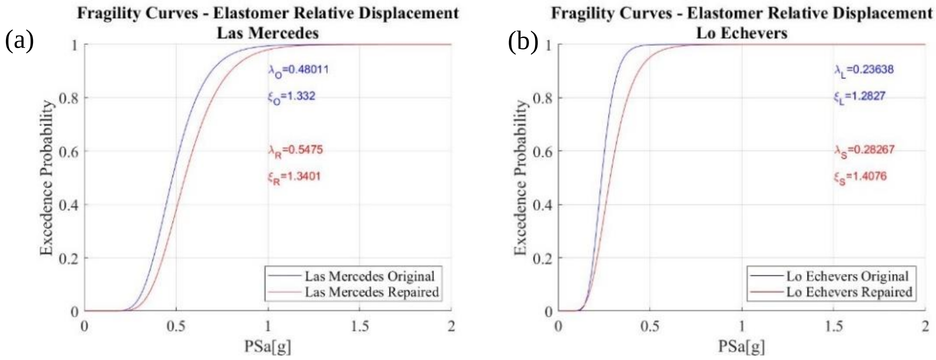

Fig. 12 – Fragility Curves for elastomeric bearing relative displacement: (a) Las Mercedes. (b) Lo Echevers.

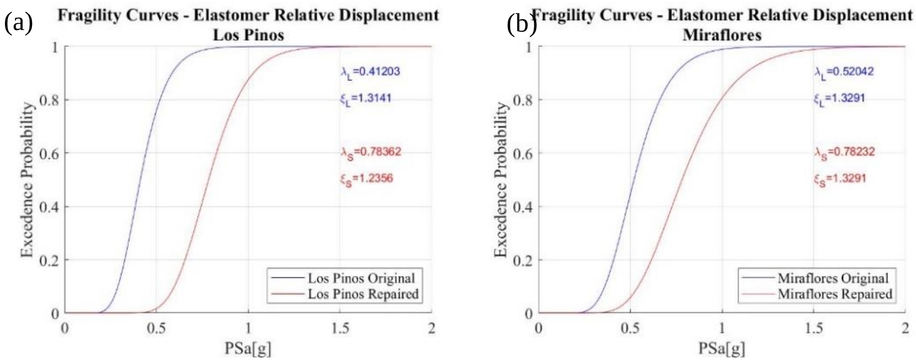

Fig. 13 – Fragility Curves for elastomeric bearing relative displacement: (a) Los Pinos. (b) Miraflores.

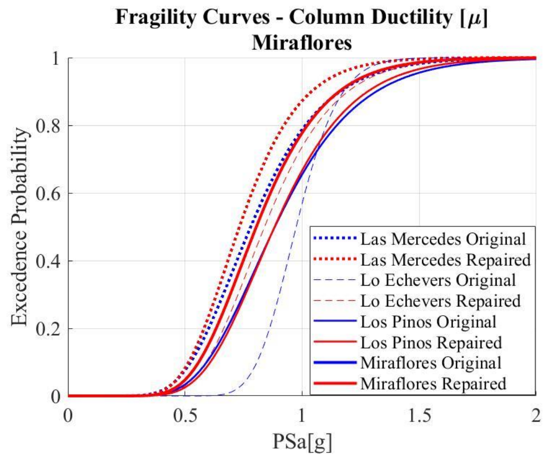

Fig. 14 – Collapse Fragility Curves for Column Ductility Demand Parameter. All bridges.

It can be observed that in all cases, the probability to reach the collapse limit state for the elastomeric bearings is greater in the original bridges than in the repaired bridges. This could be explained because all repaired bridges consider internal concrete shear keys, seismic bars and diaphragms that restrain the deck from excessive transverse displacements. Furthermore, in all cases, at 1.5 g there is an exceedance probability of 100% to reach the collapse limit state of elastomers. On the contrary, when column ductility is used as EDP, the IDA and fragility curves show that repaired bridges reach first the collapse limit state as compared to original bridges. This result is caused by the inclusion of very strong shear keys in the repaired bridges, allowing to transfer greater forces from the superstructure to the substructure, which in turn increases the demand in the columns.

Fragility Curves were also computed for the other limit states, obtaining similar results to the ones previously discussed. Fig. 15 shows the results for Lo Echevers overpass.

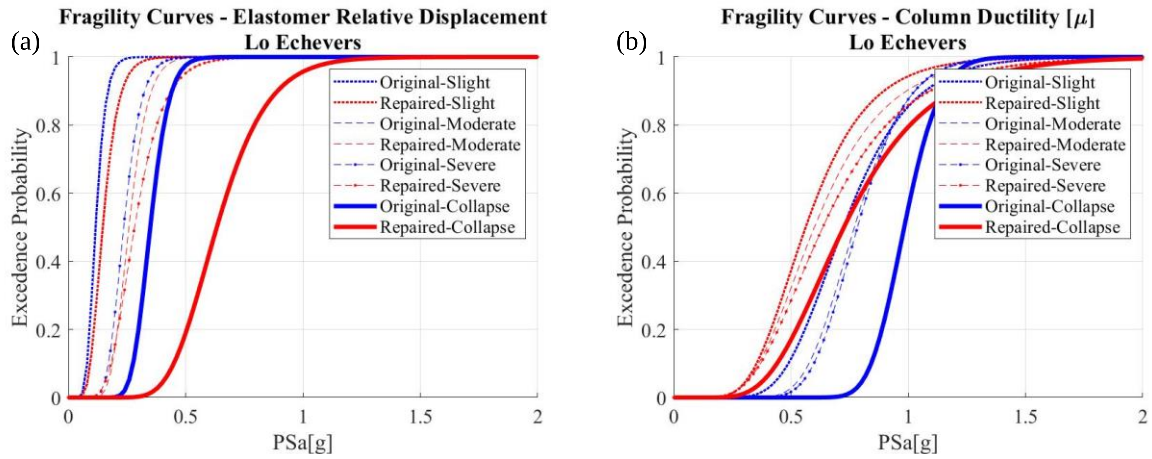

Fig. 15 - Fragility curves for all limit states for Lo Echevers Bridge. (a) Elastomer relative displacement. (b) Column Ductility.

For slight, moderate and severe limit states, at lower levels of intensity, fragility curves for the relative displacement of the elastomeric bearing seem to be very similar for the original and the repaired bridge. For higher levels of intensity, this difference begins to grow, being the original bridge more vulnerable. Nevertheless, for the collapse limit state the difference between the original and repaired bridge is observed even at low intensities, intensifying this difference notoriously at high intensities. This result can be explained by the inclusion of strong internal shear keys, since, at low intensities, and for the lower damage states, the gap between girders and shear key is not able to close, so the existence of the shear key does not play an important role as it does when high intensities are used. For the collapse limit state, the role of the shear key is crucial, and this is clearly observed in fragility curves for this state, where the exceedance probability of collapse is reduced from 90% to 20% for 0.5g.

Fragility curves for column ductility, show that for all limit states the repaired bridge reaches the limit state before the original bridge. This result is opposite to that found for the elastomeric bearings, which indicates that a shift on the inelastic demands occurs when a bridge is repaired using the Chilean common practice of adding seismic bars and strong shear keys. This was clearly observed in the fact that 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 very strong shear keys.

6. Summary and Conclusions

In this paper, the seismic performance of a set of Chilean skewed bridges, in their original and repaired conditions, was assessed through analytical models, modal analysis, time-history, and IDAs performed in OpenSees [7]. From the results and discussion presented in the paper, the following conclusions can be drawn:

- Time history analysis, using scaled ground motions to the design spectra [5], showed that the elastomeric bearings suffer slight to moderate damage. The response was slightly higher for the original bridges, which was expected since those bridges were damaged during the 2010 earthquake. For column ductility demand, almost all cases suffer less than slight damage for the scaled ground motions considered. It is worth noting that the demands during the 2010 earthquake may be considerably larger than the ones using the design spectra.
- In all cases, the exceedance probability of reaching any limit state in the elastomeric bearings is greater in the original bridge than in the repaired bridge for the same value of intensity. This difference becomes higher for high intensity levels and for the collapse limit state. For lower intensities, the gap between girders and shear keys is not able to close, so the difference between the original and repaired bridges is small since shear keys are not engaged. But, when the gap closes, shear keys can effectively restrain large transversal displacements of the superstructure, which usually occurs at the highest levels of intensity.
- When the column displacement ductility demand parameter is considered, the probability to reach any limit state is greater in the repaired bridge. This difference on exceedance probability becomes higher for the severe and collapse damage states. This result is a direct consequence of the inclusion of strong shear keys and diaphragms, which in turn causes an increase in the forces transferred to the substructure, making it more vulnerable.
- The modifications in the bridge design manual and repair measures, namely, inclusion of interior shear keys, seismic bars and diaphragms, cause that the inelastic demands that were originally concentrated in the elastomeric bearings could be effectively transferred to the substructure. As a result, deck unseating can be prevented, but this also means that a ductile design of the substructure is mandatory.

7. Acknowledgements

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