SEISMIC VULNERABILITY OF CHILEAN HIGHWAY BRIDGES CONSIDERING GROUND MOTION DURATION

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Abstract

In recent years, megathrust subduction earthquakes, such as the events in Maule, Chile 2010, and Tohoku, Japan 2011, have caused severe damage to bridges, interrupting the connectivity and generating a high socioeconomic impact. Since these structures are the most vulnerable elements of a highway network, and megathrust subduction earthquakes are usually associated with high intensities and long durations, the objective of this study is to quantify the influence of ground motion duration on the response and corresponding damage of representative Chilean highway bridges.

For the above, nonlinear finite element models of typical Chilean bridges are performed in OpenSEES [1]. The models represent bridges in two conditions, namely, as built and repaired after the Maule 2010 earthquake. The representative bridge has three spans with prestressed concrete girders resting on elastomeric bearings, reinforced concrete multi-column bents and seat-type abutments.

In order to successfully capture the ground motion duration effect, the following three literature recommendations [2], [3] are followed; (1) the bridge models consider stiffness and strength degradation of structural components, (2) significant duration is considered as the most adequate ground motion duration measure, and (3) to isolate the effect of duration, two sets of spectrally equivalent ground motion records are used, namely, short and long duration sets. Then, incremental dynamic analyzes are performed and the structural response of the model subjected to both ground motion sets is compared. Seismic vulnerability, considering the influence of duration, is assessed through fragility curves. The importance of repair and retrofit measures, such as the inclusion of stiff concrete shear keys, the use of vertical seismic bars and anchoring the elastomeric bearings, is also evaluated. Additionally, a sensitivity analysis is performed, varying two key model parameters: shear modulus of elastomeric bearings and skew angle.

The results of this study indicate that ground motion duration influence the damage achieved by the representative Chilean highway bridge. It is also shown that this influence is structurally dependent. These results indicate that ground motion duration should be considered when assessing structures in areas where megathrust subduction earthquakes are expected.

Keywords: Seismic Performance, Duration, Bridge, Seismic Vulnerability.

1 Introduction

Chile is one of the most seismically active countries in the world, proof of this is that in the last decade there have been three earthquakes with a moment magnitude (M_w) greater than 8.0, namely, the Maule earthquake in 2010 ($M_w = 8.8$), the Iquique earthquake in 2014 ($M_w = 8.2$) and the Illapel earthquake in 2015 ($M_w = 8.3$). Because of these earthquakes the infrastructure in the country has been tested several times. In fact, after the Maule earthquake in 2010 where more than 30 bridges collapse, it was evident that the seismic performance of highway bridges in Chile was deficient. Hence, bridges were primarily repaired and retrofitted using reinforced concrete stoppers (shear keys) and increasing the seat-length at abutments and bents. After the 2010 earthquake these repair measures were collected in the new Chilean seismic design manual for bridges [4].

Seismic events of great magnitude, such as the Maule earthquake in 2010 or the Tohoku earthquake in Japan in 2011, which have caused severe structural damage in bridges and buildings, have been characterized by having significantly longer durations than those ground motions recorded in other parts of the world during high intensity earthquakes [5]. In general, interplate earthquakes in subduction zones such as those mentioned above, are associated with large magnitudes and long durations [5]. Consequently, this study aims to quantify the effect of long duration ground motion on the level of damage achieved by Chilean bridges.

There are several authors who have tried to identify the effect of ground motion duration on structural response, however, the conclusions of these studies differ widely [2]. Hancock et al. [2], in their state of the art review of the effect of duration on damage, concluded that ground motion duration influence depends on three main factors: (1) the damage metric used, (2) the duration definition adopted and (3) the structural model performed. Chandramohan et al [3] subsequently associated this lack of consensus on four problems identified in previous studies: (i) structural models that not consider adequate strength and stiffness degradation, (ii) shortage of long duration ground motion records, (iii) complexity of isolating the effect of duration from other characteristics of the seismic records and (iv) lack of agreement regarding a definition of duration that correlates with structural response.

Considering these difficulties, some studies have managed to identify a correlation between ground motion duration and structural response. Chandramohan et al. [3] and Belejo et al. [5] through nonlinear analysis, showed that, for high intensities, duration effects are evident. Some other authors, like Foschaar et al. [6], have also found that collapse capacity tends to decrease when increasing the duration of the ground motion. Despite the increasing number of studies in this subject area, there is still limited studies in bridges [7], [8], [9].

The objective of this study is to quantify the influence of ground motion duration in the particular case of Chilean highway bridges. With that aim, nonlinear models in OpenSEES [1] are performed, considering a Chilean typical highway bridge. Incremental dynamic analyses are performed to obtain fragility curves of the bridge subjected to spectrally equivalent short and long duration ground motion sets. Moreover, two conditions of the bridge are considered, namely, as built (original case) and repaired bridge, aiming to analyze if traditional repair/retrofit measures have some effect in the influence of duration on the structural response of the bridge. Besides, the influence of two relevant bridge parameters are studied: elastomeric shear modulus (G) and skew angle (β).

2 Description of the Typical Chilean Bridge

In this study, a representative Chilean highway bridge is selected as a case study. The bridge is selected based on the study of Cabrera [10], who concluded that, according to the number of linear meters built in the country, the typical Chilean bridge has a length of 40 to 200 meters and a superstructure and substructure made of reinforced concrete. Moreover, the typical bridge consists of prestressed concrete girders supported on elastomeric bearings and seat-type abutments.

The representative bridge selected is a three-span prestressed concrete girder bridge constructed in 1990. It is 92 [m] long, 12.3 [m] wide and has a skew angle of 30° approximately. It has seat-type abutments and two bents composed of 5 round columns and a rectangular cap beam.

From this structurally representative bridge, two cases are considered. The first one corresponds to the as built case, hereinafter referred to as Original Bridge, which was constructed before the 2010 Maule earthquake. The second case corresponds to the repaired / retrofitted bridge, which includes the repair measures taken after the Maule earthquake in 2010, hereinafter referred to as Repaired Bridge. The main differences between the two bridges are in the components that restrain the lateral displacement of the superstructure in bents and abutments (see Fig. 1). The original bridge (as-built case) is simply supported on elastomeric bearings, does not have diaphragm, has steel stoppers in both bents and abutments, and have additional concrete external shear keys in abutments. On the contrary, the repaired bridge has elastomeric bearings fixed to the substructure and superstructure, diaphragms, seismic bars and both interior and exterior concrete shear keys, as shown in Fig. 1.

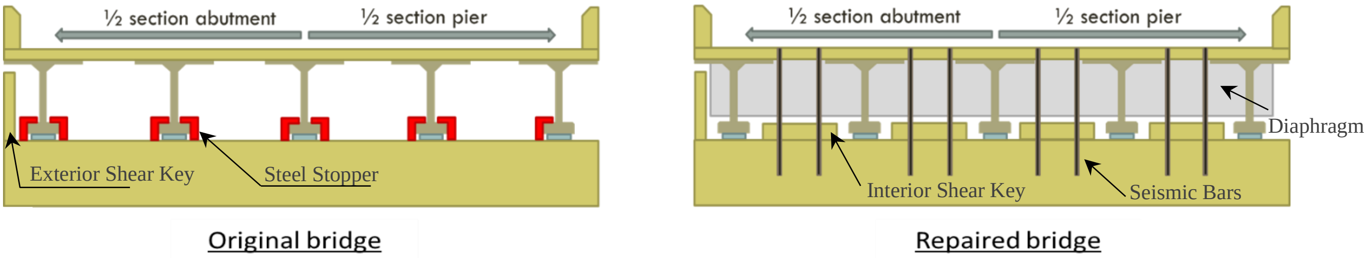

Fig. 1 - Cross section of the case studies: original and repaired

3 Numerical model

To carry out the study, nonlinear models for both bridge cases, original and repaired, were developed using OpenSEES [1]. Special emphasis was placed on adequately representing the nonlinear behavior, especially in terms of strength and stiffness degradation of columns, elastomeric bearings, shear keys and abutments. The modelling details are shown in Fig. 2.

The superstructure was modelled using elastic beam-type elements, since no inelastic incursions are expected due to the structural configuration of the bridge [11]. Translational and rotational masses are assumed according to the recommendations of Aviram et al. [12]. For the columns, fiber type sections assigned to nonlinear elements are used, adequately characterizing the structural contribution of the reinforcing steel and concrete, both confined and unconfined. In both bridge cases, the impact between the deck and abutment is included using a Hertz model modified by Muthukumar [13]. Additionally, the backfill soil is considered using a hyperbolic material, as proposed by Shamsabadi [14]. Internal shear keys are considered using the equations proposed by Megally et al. [15] but with a trilinear force displacement relation as recommended by Goel & Chopra [16], represented by two materials in serie: *hysteretic* and *elasticPPGap*.

In the case of the original bridge, elastomeric bearings are included following the friction model of Coulomb, as proposed by Steelman et al. [17], with an initial shear modulus of 13 kgf/cm^2, according to the Chilean bridge design manual [18]. Steel stoppers are modelled with two materials in serie: *hysteretic* and *elasticPPGap* materials, according to the recommendations of Rubilar [19].

In the repaired bridge, the previous elastomeric bearings were replaced and fixed to the substructure and superstructure. In order to represent adequately the new elastomeric bearing behavior, a calibration was performed to adjust a model (elastomeric Bouc-Wen material) to experimental data. On the other hand, seismic bars were included according to a model proposed by Martinez [20], which was based on an experimental campaign. Finally, the internal RC shear keys were modelled as recommended by Ramanathan et al. [21] considering two materials in serie: *elasticPPGap* and *MinMax*.

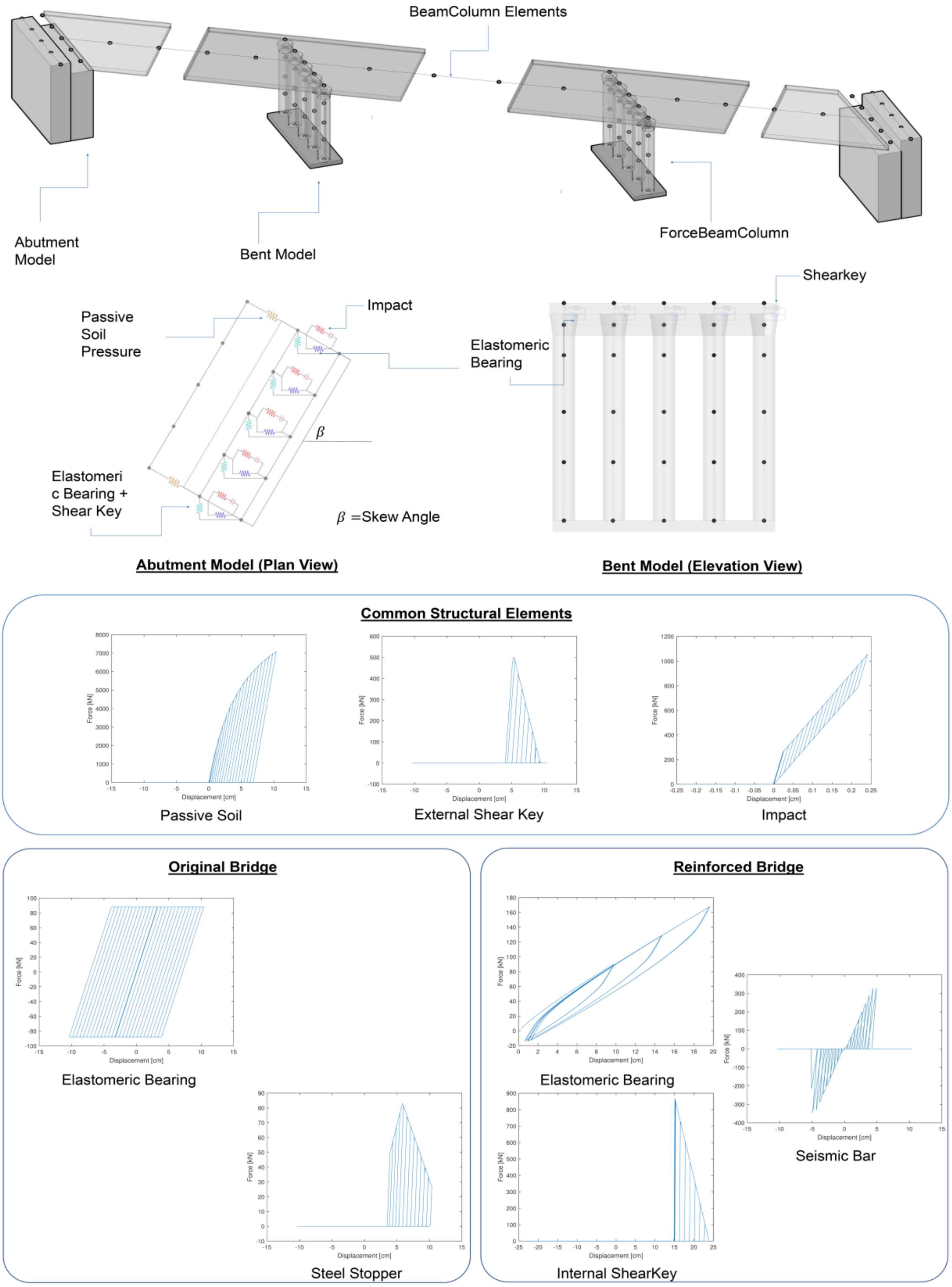

Fig. 2 - Numerical model in OpenSEES

4 Ground motion selection

In order to perform the incremental dynamic analysis to obtain fragility curves, it is necessary to establish some considerations, such as the duration definition, the scaling method to isolate the effect of duration and the ground motion sets.

4.1 Ground motion duration

There are more than 30 different metrics to quantify ground motion duration [22]. The problem is, generally, there is a poor correlation between them [2]. Thus, selecting a duration definition for the purpose of this study was not a trivial task. Previous studies have proposed some of the following metrics as potential candidates: bracketed duration, Arias Intensity, significant duration, cumulative absolute velocity, among others. Additionally, Chandramohan et al. [23] and Foschaar et al [6], with the aim of determining the most appropriate duration metric for evaluating the effects of ground motion duration on the structural response, analyzed and compared the previous mentioned metric against some desired properties of a robust metric. These desired properties are: (i) the duration metric is not correlated to the intensity measure, (ii) is unaffected by the scaling process, (iii) is efficient, (iv) does not bias the spectral shape in the selection process and (v) has an adequate correlation with the actual strong shaking duration.

Considering these requisites, these studies concluded that significant duration is the most appropriate measure for the purpose of analyzing the effect of duration on structural damage. Specifically, in this paper the significant duration 5-75% ($Ds_{5-75\%}$), which corresponds to the time interval that accumulates from 5% to 75% of the Arias intensity, was selected as the duration metric.

4.2 Ground motion scaling

To study the effect of ground motion duration, the structural response of the bridge is compared when subjected to short and long duration ground motion records. Records with a significant duration of less than 25 [s] are classified as short duration ground motions, consequently, ground motions with significant duration greater than 25 [s] are considered as long duration records.

To isolate the effect of ground motion duration of other characteristics, such as intensity or frequency content, is not trivial when analyzing three dimensional models. The difficulty arises from the use of two or three orthogonal components of the earthquake and the inherent difficulty in obtaining equivalent spectra from different seismic records. For this reason, in the process of record selection, an attempt should be made to minimize the disparity in the response spectrum between long-duration and short-duration ground motions. In this study, two sets of short and long duration ground motion are used, following a methodology proposed by Belejo et al. [5]. The procedure consists of an optimization problem in which each pair of orthogonal components of the short duration record is rotated at an angle (θ) and scaled with a scale factor (SF), such that the mean square error (MSE) is minimized in a range of periods close to the natural period (T_n) of the structure ($0.3T_n$-$3T_n$). The process can be observed in Fig. 3a, where the optimization problem is solved for the variables θ and SF, obtaining a similar geometric mean spectrum between the short and long duration ground motions considered.

4.3 Ground motion sets

For this study, 42 ground motions, 21 of short duration and 21 of long duration, were used. The long duration set is based mainly in the events of Tohoku (Japan, 2011) and Maule (Chile, 2010). For each of the ground motions from the long duration set, a short duration ground motion is searched in the PEER NGA ground motion database [24]. The selection process consists in selecting the short ground motion that produces the minimum MSE according to the procedure explained in section 4.2. Fig. 4 shows the different significant durations of both sets of records.

After applying the previously explained process, it can be observed in Fig. 3b that there is no significant difference between the spectra of the sets, in a range of periods around the fundamental period of the bridge (in this case, T_n-=0,86 s). Because of this, any difference in the response of the structure subjected to both ground motion sets can be attributed primarily to the difference between the duration of each set.

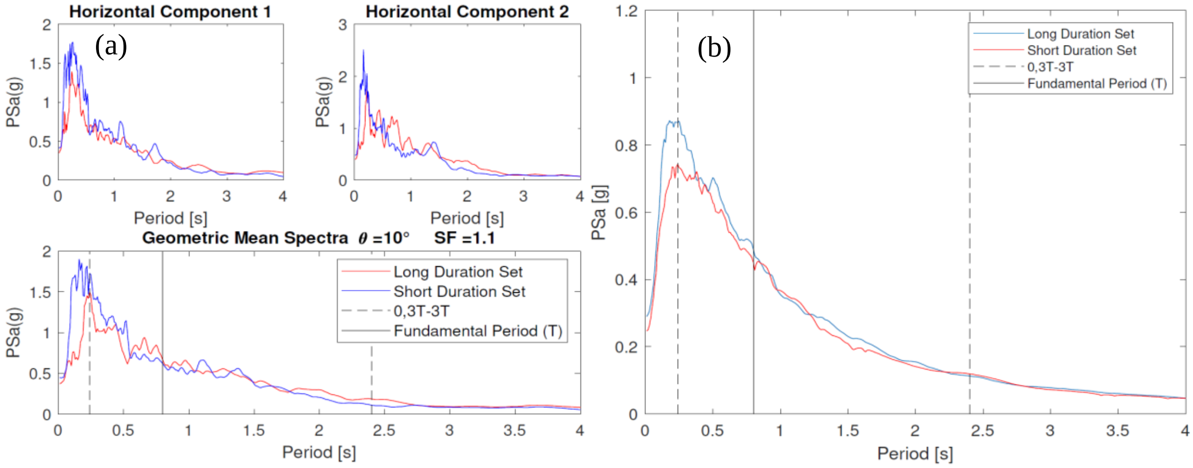

Fig. 3 Spectral matching procedure - (a) Example with a pair of ground motion records; (b) Complete set spectra comparison.

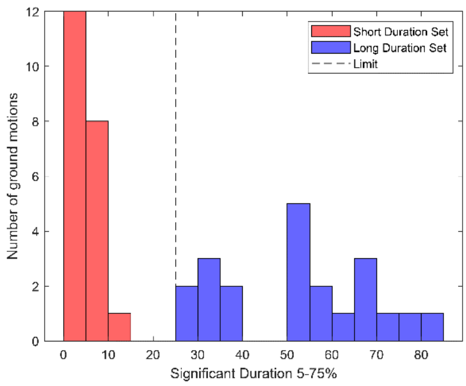

Fig. 4 – Histogram of significant durations for the selected ground motions.

5 Damage 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 1. Displacement ductility was used as engineering demand parameter for the multi-column bents, according to Billah & Alam, [25], while for the elastomeric bearings the displacements suggested by Ramanathan et al. [21] were considered since they are in agreement with the damage expected for the elastomeric bearings used in Chile. A nonlinear static analysis was performed to obtain the displacement that produces yielding (slight limit state) in the multi-column bent. The displacement that produces yielding in the multi-column bent is 1,89 [cm] in the longitudinal direction and 1,68 [cm] in the transverse direction.

Table 1 - Limit states

Structural component	Demand parameter	Limit states			
		Slight	Moderate	Severe	Collapse
Multi-column bent	Displacement ductility [μ]	1.0	1.22	1.78	4.8
Elastomeric Bearing	Relative displacement [cm]	2.9	10.4	13.6	18.7

6 Results and discussion

6.1 Nonlinear static analysis

In order to show the differences between the behavior of both bridge cases, a monotonic incremental static analysis – pushover – is shown in Fig. 5. It can be seen, in Fig. 5a that column behavior is similar in both cases since only a small change was made in the cap beam in the repaired bridge. On the other hand, when analyzing the displacement of a node on the superstructure respect to the forces on the bent base (Fig. 5b), it can be observed that both bridge cases have a completely different behavior. On the original bridge (blue curve), the force-displacement curve is initially dominated by the elastomeric bearing, before point (1). Then, when the gap is closed at point (2), steel stoppers begin to act until the moment when their maximum strength is reached at point (3). After the steel stoppers fail at point (4), there is no other structural element that could avoid lateral displacement. It is important to mention that point (3) represents the greatest force transfer to the substructure, but this is insufficient to bring the columns to yield, as shown by segmented lines in Fig. 5.

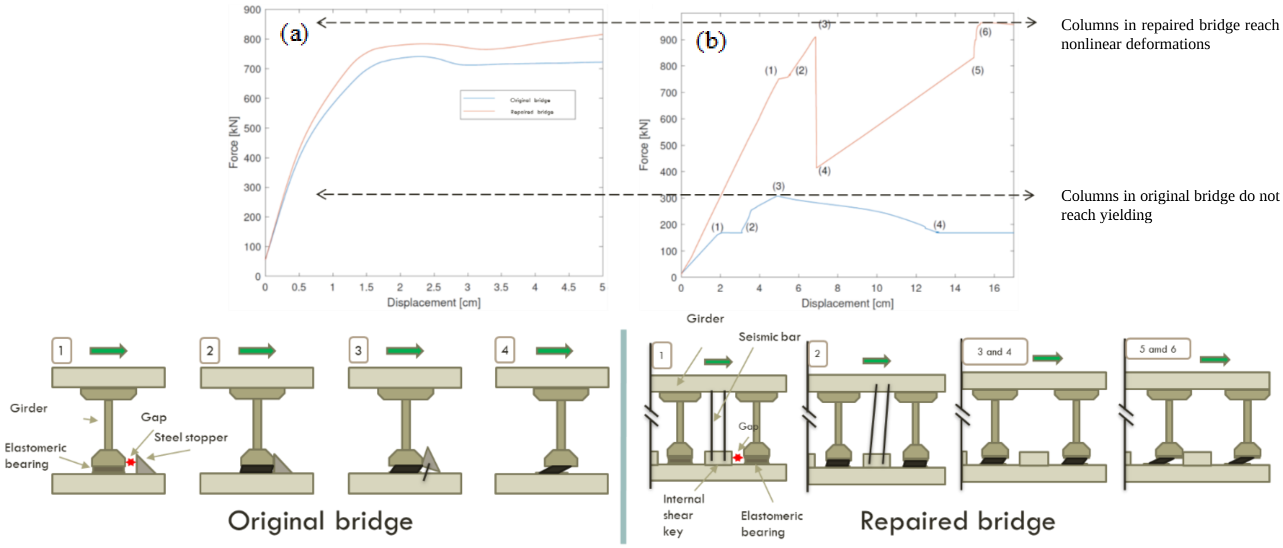

Fig. 5 - Nonlinear static analysis - Control node (a) in columns, and (b) in the superstructure

In the repaired bridge, the force displacement curve (orange curve) shown in Fig. 5b is first dominated by a combination of the elastomeric bearings and seismic bars, before point (1). After the maximum resistance of the seismic bars is reached, they fail in point (3). Between points (4) and (5) the response of the bridge is dominated by the elastomeric bearing behavior. At point (5) the gap between girders and internal shear keys is closed and these begin to act. After point (5) forces are transferred to the substructure and at point (6) there is no other structural element that could restrict the lateral displacement of the superstructure. The situation here is different, because these internal shear keys have enough strength to transfer bigger forces to the substructure and they can make the columns reach nonlinear deformations.

6.2 Incremental dynamic analysis

Incremental dynamic analyses were performed to obtain fragility curves by scaling the records so that the pseudo-acceleration spectral ordinate evaluated in the fundamental period varied from 0 to 2.5 [g], with a step of 0.1 [g]. For this, after achieving the spectral equivalence of the sets of short and long duration records, the pairs of ground motions are scaled to each of the intensity levels mentioned, obtaining the structural response of interest in each of these levels.

Fig. 6 shows that the columns of the original bridge do not reach any level of damage, independently of the intensity or duration of the seismic record. This is the result of the steel stoppers not having enough resistance to transfer a significant force to the substructure, which is also supported by the pushover analysis shown in the previous section.

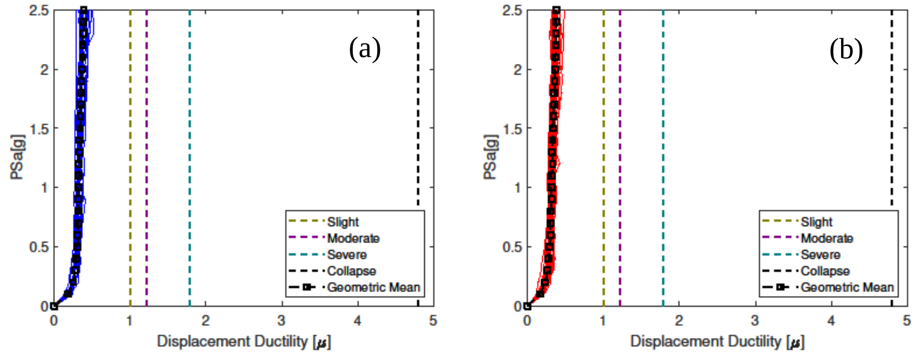

Fig. 6 - IDA curves for the original bridge considering displacement ductility of the multi-column bent – (a) Long duration set; (b) short duration set.

When analyzing the IDA curves related to the elastomeric bearing displacement, a difference is observed when the structure is subjected to long and short duration ground motions. It can be seen in Fig. 7 that for the repaired bridge, in order to reach slight and moderate limit states, there is no significant difference in the intensities that are needed for both duration sets. This result is because at low intensities there is no degradation and, thus, the long duration set cannot be predominant. On the contrary, to reach severe and collapse limit states, a slightly lower intensity is needed for the long duration set. This result can be explained because at high intensities occurs degradation and the long duration set, with its larger number of inelastic cycles, can degrade the seismic bars and concrete shear keys, allowing the elastomeric bearing to reach larger displacements. It is worth noting that the in-plane rotation of the deck also collaborates in increasing these displacements, i.e. when the superstructure rotates, the exterior longitudinal girders are closer to the external shear keys, which facilitates their degradation. Additionally, in both cases, short and long duration set, a significant increment of intensity level is appreciated around 15 [cm]. This increment is caused by the actual gap (15 cm) between the girder and the shear key. As concrete shear keys have great resistance, a higher level of intensity must be reached to increase the elastomeric bearing displacement.

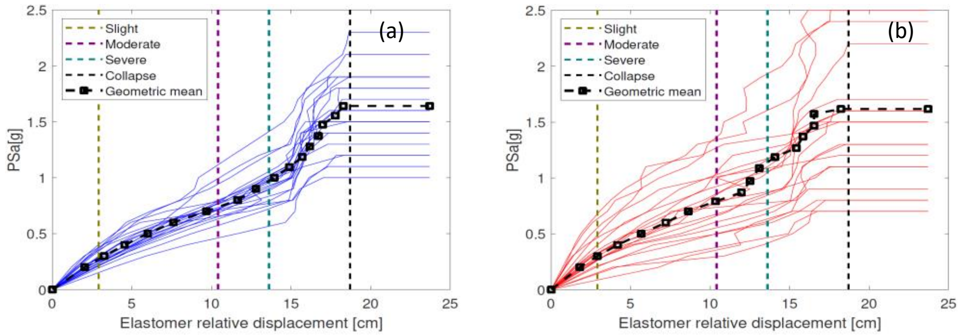

Fig. 7 - IDA curves for the repaired bridge considering elastomeric bearing displacement - (a) Long duration set; (b) Short duration set

6.3 Fragility curves

In order to evaluate the influence of duration on the structural response, fragility curves are obtained. These curves are a tool that describes the conditional likelihood of a structure of exceeding certain level of damage, given a determined level of ground motion intensity [25]. The fragility or conditional probability can be expressed as shown in Eq. (1).

$$Fragility = P[DM \geq DS|IM = y] \quad (1)$$

Where, DM is a damage measure; DS is the specified damage state of the structure or structural component; IM is the intensity measure level; and *y* is the condition realized of the ground motion intensity measure.

To derive analytical fragility functions using nonlinear time history responses of the bridges, this paper employs incremental dynamic analysis. The IDAs are developed using the scaling approach to relate the engineering demand parameter (EDP) to the ground motion intensity measure (IM). This method requires more computational effort than other methods, like cloud approach, because of the ground motion scaling to different IM levels. However, no previous assumption needs to be made in terms of probabilistic distribution function of seismic demand in order to develop the fragility curves. Each probability of a specified damage state is directly computed as indicate in Eq. (2):

$$P[DM \geq DS|IM = y] = {}^{n_i}/_{N} \tag{2}$$

Where: n_i is the number of damage cases for the damage state i, and N is the total number of simulation cases. After calculating the discrete exceedance probability, a continuous cumulative distribution function can be fitted. In this study a lognormal (see Eq. (3)) cumulative distribution function is used.

$$P[DM \geq DS|IM] = \int_{-\infty}^{IM} \frac{1}{\sqrt{2\pi}\xi_{IM}} exp\left[-\frac{(\ln(im) - \lambda_{IM})^2}{2\xi_{IM}^2}\right] d(im) \tag{3}$$

Where ξ_{IM} *and* λ_{IM} are the standard deviation and mean value of the IM of a determined damage state based on a lognormal distribution.

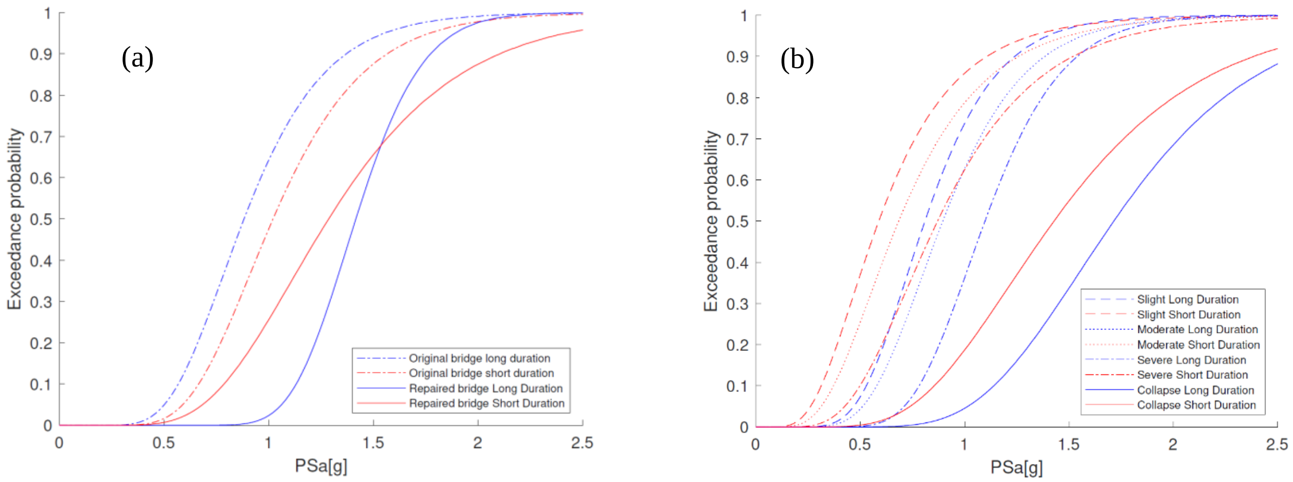

Fig. 8 (a) Elastomeric bearing collapse fragility curves, comparison between original and repaired bridge; (b) Displacement ductility in multi-column bent fragility curves for the repaired bridge.

Fig. 8(a) shows that for both bridge cases, when the intensities are under 0.5 [g], the difference between the fragility curves related to the elastomeric bearings for the long and short duration set does not vary considerably. This occurs because at low levels of intensity, there are no conditions for the existence of strength and stiffness degradation, so the long duration ground motion, with its greater number of inelastic cycles, does not produce any difference. On the other hand, for intensities over 2 [g], there is no difference either because in this case it is possible that the structural elements degrade with very few cycles. In the case of the original bridge, for intensities between 0.7 to 1.5 [g], the long duration set produces a probability of collapse 10% greater than that of the short duration set. This is explained by the degradation of the steel stoppers and concrete stoppers in abutments, a mechanism that is also influenced by the impact between the deck and the abutment, which produces superstructure rotation and closes the gaps between girders and stoppers. On the other hand, in the repaired bridge, there is an intensity range in which the short duration set is predominant till a certain level of intensity, after which, the long duration set produces a greater probability of collapse. This is presumably caused by two situations: (1) a higher level of intensity is needed so that the concrete shear keys can degrade and (2) the addition of the seismic bars and the change in the elastomeric bearings produces a greater re-centering of the superstructure, so it does not remain rotated, as in the case of the original bridge.

The results also show that the repaired bridge has, in general, a significantly lower probability of collapse for all the intensity levels considered. It is observed in the repaired bridge an increase of approximately 0.4 [g] in the average intensity that leads to elastomeric bearing collapse, considering the long duration set. With respect to the short duration ground motion set, the average intensity at collapse also increases, but to a lesser extent. This result indicates that the repaired/retrofitted measures, such as the implementation of concrete shear keys and anchoring the elastomeric bearing, are effective measures to reduce the likelihood of elastomeric bearing collapse, therefore, deck unseating. However, the addition of strong interior concrete shear keys produces that a greater percentage of the superstructure forces are transferred to the substructure, which means that columns can now enter the nonlinear range and get damaged, as shown in the fragility curves in Fig. 8(b).

For the original bridge, the probability that the columns exceed some limit state is zero, as shown in Fig. 6, therefore, the fragility curves are flat. In the case of the repaired bridge, the fragility curves for the multi-column bents are shown in Fig. 8(b). It is observed that the probabilities of exceeding any limit state are generally higher when subjecting the bridge to short duration records. It was observed that, in some cases, the long duration record, despite having a greater number of cycles, kept the multi-column bent in the linear range, while the short duration record, with fewer cycles, carried the multi-column bent to collapse. This is explained by some characteristics of the short-duration records, such as pulses, which would make it more damaging in this case. This result is prone to change if a more appropriate strength degradation model and a cumulative measure of damage is considered.

6.4 Sensitivity analysis

For the sensitivity analysis, two presumably influential parameters were varied: the elastomeric bearing shear modulus and the bridge skew angle. The shear modulus was varied from 10 to 16 [kgf/cm^2] with a step of 1.5 [kgf/cm^2], considering field experience. On the other hand, the skew angle varied from 0° to 60° with a 15° step, considering straight bridges and largely skewed bridges. For this analysis around 10.000 simulations were performed.

When considering an increase in the shear modulus (G) of the elastomeric bearing, fragility curves move towards higher levels of intensity, as shown in Fig. 9. This situation was expected, since to displace a stiffer elastomer, a higher level of intensity is necessary. In the original bridge, by increasing G, at low intensity levels, the short duration set is predominant since higher levels of intensity are necessary to obtain greater displacements in the elastomers and, consequently, to be able to degrade the steel stoppers with a greater number of cycles (long duration set). At higher levels of intensity, the long duration set is predominant. In the repaired bridge, a similar situation occurs, an increase in the intensity level is necessary, as G increases, to make the long duration set predominant. The effect of the change in G produces nothing more than moving the curves on the axis of intensities, however, this means a great change in the probability of collapse. For example, for an intensity level of 1 [g], it is observed that for the repaired bridge and elastomers with G = 10 [kgf / cm^2] there is a 20% of probability of collapse , while for the elastomer with G = 16 [kgf / cm^2] there is 0% probability at that intensity. This is relevant due to the uncertainty present in the field regarding this property.

Fig. 10a shows that the skew angle does not play an important role in the original bridge if short duration records are considered. The fragility curves of short duration are quite similar for any of the skew angles considered. On the other hand, when considering long duration records, there are greater differences between the curves. Skewed bridges are clearly more vulnerable to long duration records than straight bridges. This phenomenon is attributed to the fact that, when there is an angle between the superstructure and the abutment, the impact between both elements induces plan rotation of the superstructure, which closes the gaps with the stoppers in the abutments and a greater degradation occurs with long duration records by having a greater number of cycles. The same situation explained previously occurs on the repaired bridge. At a certain level of intensity, for all skewed bridges considered, the long duration set acquires greater importance in the probability of collapse, due to the greater degradation of certain structural elements. It can also be observed that bridges with greater skew angles produce greater probability of collapse, which is explained by the large angle of impact forces between superstructure and abutment. Finally, it is shown that the straight repaired bridge is more vulnerable, considering the set of long duration records, than bridges with small angles (15 and 33°). This may be due to the excitation of other modes of vibration in the straight bridge or that there is some difference in the axial forces in the elastomers, which could produce, despite the 0° skew angle, a rotation of the superstructure.

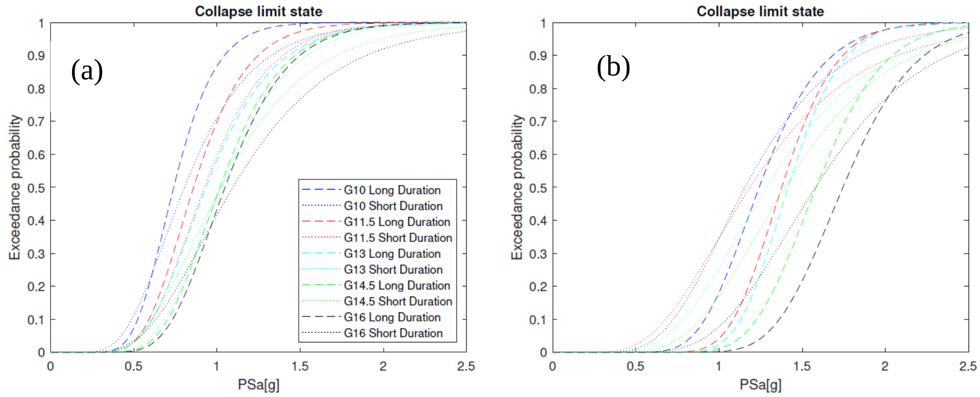

Fig. 9 - Collapse fragility curves varying shear modulus G (a) Original Bridge; (b) Repaired bridge

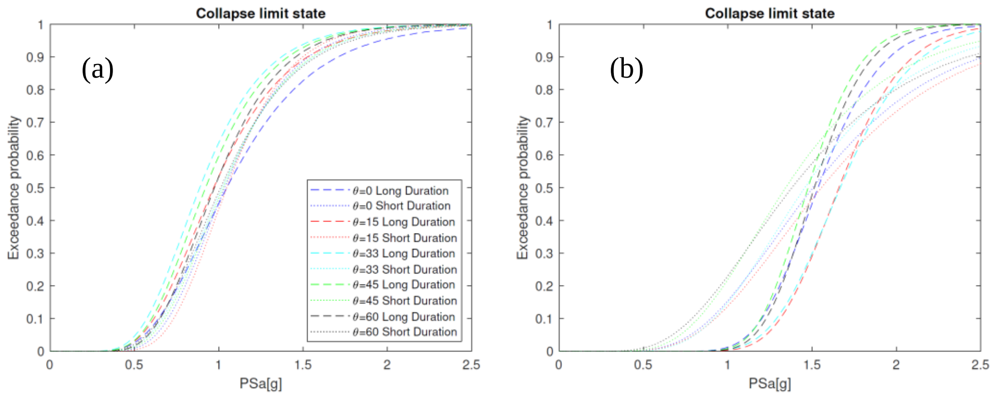

Fig. 10 Collapse fragility curves varying skew angle - (a) Original bridge; (b) Repaired bridge.

7 Conclusions

Based on the results obtained, the following conclusions can be drawn:

- A longer duration, in general, increases the likelihood of collapse, especially at relatively high intensity levels. The effect of the duration is mainly due to strength and stiffness degradation. In addition, the response of this type of bridge is highly influenced by the mechanism of rotation of the superstructure.
- The differences between the bridge in its original and repaired state are significant. Moreover, the effect of the ground motion duration depends on the analyzed case and the potential damage in the columns.
- The measures taken to repair and retrofit the original bridge showed an improved seismic performance. Furthermore, the probability of elastomeric bearing collapse for both long and short duration events was significantly reduced, at the cost of exposing the columns to damage.
- The bridges with greater skew angles are clearly more vulnerable to long duration ground motions. On the other hand, the value of G greatly influences the probability of elastomeric bearing collapse and allows the curves to shift to different intensity levels.

Future studies should incorporate the vertical component of the seismic record and should propose some measure to incorporate duration in bridge design codes, especially in areas where megathrust subduction earthquakes are expected.

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