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Fragility Assessment in RC Bridges Considering the Cumulative Damage over Time Due To Earthquakes

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***Abstract* -** This study presents a criterion to estimate the structural fragility of reinforced concrete bridges considering the cumulative damage over time caused by seismic sequences. Fragility curves are obtained for different instants of interest associated with different levels of performance. The occurrence of seismic loads are characterized by a stochastic Poisson process. Uncertainties related with the construction processes of the structural elements and the manufacturing of the materials are considered. Fragility curves at the instants of 0, 50, 75, 100 y 125 years are obtained from nonlinear dynamic analysis which the maximum drift of the bridge deck is considered as the parameter of seismic demand measurement. The criterion is illustrated in a continuous RC bridge designed to satisfy a performance level equal to 0.002, the bridge is structured by AASHTO – type beams and circular columns. The structure is located in Acapulco, Guerrero, Mexico. The probability of exceedance of the drift design threshold of 0.002 associated with intensities of 0.05 sa/g increases up to 58% between 0 years (without damage) and 125 years after the bridge construction when the cumulative damage given by seismic sequences over time is considered.

***Keywords*:** fragility curves, cumulative damage, bridges, non-linear dynamic analysis, seismic sequences.

1. Introduction

Acapulco city is one of the main tourist destinations in Mexico. Nationwide, it is the sixteenth largest metropolis in the country and the twenty-first most populous city. Due to the above, highway networks play a crucial role in the transfer of raw materials, people, emergency equipment, etc., making important to keep communication lanes in good conditions. Acapulco city is located on the subduction zone of two tectonic plates, the North American plate and the Cocos plate, setting it in a highly seismic region. With the above, this research has concentrated on considering seismic loadings as the dominant hazard at the study site.

Earthquakes cause economic losses when the infrastructure present several cases of structural damage, collapses, interruption of the economic activities, etc. In addition to this, the absence of maintenance plans for the structures in our country, causes the accumulation of the seismic structural damage over time. The design philosophy has the aim to conceive structures to satisfy a pre-established level of performance. This has implications since the strength and solicitations involved in the design are random variables and it is difficult to characterize an exact value of strength and maximum expected solicitation. Consequently, the probability that a structure fails during a certain period has a high value. Therefore, it is important to estimate the probability that the structure exceeds a certain level of damage as a time function.

Based on the above, many researches have been interested in proposing methodologies that allow obtaining exceedance probabilities of different performance levels. [1] proposed fragility curves in highway bridges under seismic events taking into account a spatial variation. [2] presented a methodology to estimate the structural fragility in highway bridges considering the effect of base isolation. [3] evaluated non-destructive in RC columns to assess the damage in bridges, although, present fragility curves. [4] considered soil-structure interaction to estimate structural fragility analysis of a highway bridges. Some authors have focused on analytical fragility curves for highway bridges in Turkey by considering the critical response of some bridge components [5]. [6] presented retrofit measures to evaluate seismic performance of steel bridges located in New York. [7] presented a methodology to assess the seismic vulnerability of highway bridges in Quebec. [8] presented analytical fragility curves for a multi-span continuous concrete bridge for different performance levels. On the other hand, [9] proposed a methodology to estimate the fragility of a multi column bridge bent retrofitted with different rehabilitation techniques and their impact on the vulnerability of a bridge bent. [10] presented a condensation of the most common fragility analysis techniques. [11] presented an analysis to estimate the structural vulnerability in RC bridges subjected to aftershocks. Other studies have focused on the seismic fragility of RC concrete bridges taking into account the rehabilitation by using steel fibbers [12]. [13] estimated the vulnerability of RC bridges by using artificial neural networks. [14] presented a methodology to estimate fragility curves on RC bridges subjected to earthquakes and chloride-induced corrosion. [15] evaluated the performance of a three – span bridge isolated with elastomeric rubber bearings in Bangladesh and generate fragility curves. [16] presented a methodology to evaluate seismic fragility on RC bridges subjected to seismic sequences taking into account the cumulative damage. There are investigations to evaluate fragility curves in reinforced concrete structures considering earthquakes in India and Romania [17–18].

As observed in the cited works, there are not many methodologies that allow to estimate the cumulative damage over time due to earthquakes in RC bridges. The cumulative damage at each instant of time, *t*, is expressed in terms of the maximum drift at the bridge deck. Besides, it is deemed that bridges can develop a pre-establish performance level following the specified on Mexican Design Code [19]. The pre-establish performance level of the structure is 0.002; it is structured with AASHTO beams and circular columns and located in Acapulco, Guerrero, Mexico. With the results obtained in this paper, maintenance or inspection plans can be presented that are useful for extending the service life of structures.

2. Fragility curves over time

Fragility curves are constructed for RC bridge by using non-linear dynamic analysis. Maximum demands are assumed to follow a lognormal probability density function [20]. Fragility curves over time are obtained with the Eq. (1) below [16]:

$$P\left(D_{|y,t} \geq d\right) = 1 - \Phi\left(\frac{\ln d - \ln \bar{D}_{|y,t}}{\sigma_{\ln D_{|y,t}}}\right) \tag{1}$$

where $\bar{D}_{|y,t}$ is the median value of the structural demand given a seismic intensity, *y*, at the instant of time, *t*; and $\sigma_{\ln D_{|y,t}}$ is the standard deviation of the natural logarithm of demand for a given intensity, *y*, at the instant of time, *t*; *d* represents a pre-established demand threshold. $\bar{D}_{|y,t}$ and $\sigma_{\ln D_{|y,t}}$ can be estimated as shown in Eqs. (2) – (3), respectively:

$$\bar{D}_{|y,t} = exp\left(\frac{\sum_{i=1}^{n} ln\left(Di_{|y,t}\right)}{n}\right) \tag{2}$$

$$\sigma_{ln\, D_{|y,t}} = \left(\frac{\sum_{i=1}^{n}\left(ln\, Di_{|y,t} - ln\, \bar{D}_{|y,t}\right)^2}{n-1}\right)^{\frac{1}{2}} \tag{3}$$

where $Di_{|y,t}$ is the natural logarithm of the maximum demanded drifts associated with a intensity, *y*, at the instant of time, *t*; and *n* is the number of observations.

3. Cumulative damage process

This paper assumes that the structure does not receive any type of maintenance at any moment in time. The intensities are estimated from the seismic hazard curve associated with the fundamental period of the structure. The cumulative damage process is described in the following steps:

1. n simulated bridges are generated considering mechanic and geometric uncertainties.

2. Simulation of intensities associated with the n simulated models are generated.
3. Incremental dynamic analyses are accomplished where the scale factor of the aleatory earthquake is related to the maximum drift of the bridge deck.
4. i = 1
5. With the simulated intensities and waiting times, the $i-th$ simulated intensity is associated with the $n-th$ structural model with simulated properties related with the instant of interest, t. Then, a pair of simulated seismic events are subjected to the structure, which is associated with the $i-th$ simulated intensity and the $(i+1)-th$ simulated intensity. Seismic events are randomly selected. This events are scaled by a factor $\beta_m = i_{sim}/i_T$, which is the result of the ratio between the simulated intensity and the spectral acceleration associated with the fundamental period of the structure, T. Dynamic structural response is obtained by extracting the maximum drift, $D_{i_{|y,t}}$, at the bridge deck.
6. A seismic record is randomly selected, which is multiplied by a scale factor, ψe, that makes it able to producing the drift value, $D_{i_{|y,t}}$, obtained in step 4. In this point, the cumulative damage is given by two simulated seismic events, S_{ki}.
7. i = i + 1
8. A seismic record is associated with the $(i+1)-th$ simulated intensity.
9. The record, ri-1, is scaled by a factor $\beta_m = (i+1)_{sim}/(i+1)_T$, which is the result of the ratio between the simulated intensity with the spectral acceleration associated with the fundamental period of the structure, T.
10. A seismic signal composed by the accumulated seismic record, $S_{k(i-1)}$, and the seismic record, r_{i-1}, is obtained.
11. The maximum drift, $D_{(i-1)_{|y,t}}$, of the structure is obtained.
12. A random seismic record is multiplied by a scale factor, ψe, that produces the value of $D_{(i-1)_{|y,t}}$, is selected.
13. A seismic record, $S_{k(i-1)}$, is obtained that includes the cumulative damage up to the $(i+1)-th$ simulated seismic intensity.
14. The process is repeated from step 7 to 13.

4. Intensities and waiting times

The aim of this research is to consider that the structure accumulates damage over time, so it is necessary to estimate the waiting time intervals between the seismic movements. Simulation of seismic intensities is accomplished based on the seismic hazard curve, SHC, of the site for T_0 = 0.40 s. Seismic hazard curve indicates the number of occurrences of an event that exceeds a certain intensity level per unit of time. Simulation of intensities is achieved based on the cumulative distribution function, CDF, of the SHC as shown in the Eq. (4):

$$F(y) = 1 - \frac{SHC_{FIT}}{v_0} \tag{4}$$

where $\mathrm{SHC}_{FIT} = \left(\frac{y}{y_0}\right)^{-r}\left(\frac{y_{max}-y}{y_{max}-y_0}\right)^{\varepsilon}$ is the expression that fitted the SHC; y_0 is the seismic intensity necessary to produces structural damage in the structure. In this particular case $y_0 = 1\ \mathrm{m/s^2}$ and is associated with an exceedance rate equal to $v_0 = 0.07737$; $y_{máx}$ represents the maximum value of seismic intensity in the SHC; r and ε are the adjustment constants; y is all the possible seismic intensities into the SHC. On the other hand, to simulate the waiting times between seismic events, it is assumed that these follow a Poisson-type process then, the events follow an exponential distribution. Making some arrangements in the CDF of the exponential distribution, the waiting times between seismic occurrences are estimate as $T_i = -\left|\frac{ln(u)}{v_0}\right|$, where u represents random numbers between 0 and 1 with uniform distribution.

5. Illustrative example

Fragility curves are obtained for a reinforced concrete bridge designed to perform a drift threshold of 0.002. Instants from 0 to 125 years after the bridge construction are considered. The structure has a total length of 175 m and height clearance of 8 m. for analysis and design, a compressive concrete strength, f'c, for cap beams and columns are equal to 29.42 MPa and a value of 39.23 MPa is consider for AASHTO type beams. The system period is equal to 0.40 s and it is located in Acapulco city, Mexico. Figure 1 shows the geometry and design of cap beams and columns. Furthermore, figure 2 shows the transverse section of the structure and figure 3 shows the longitudinal section.

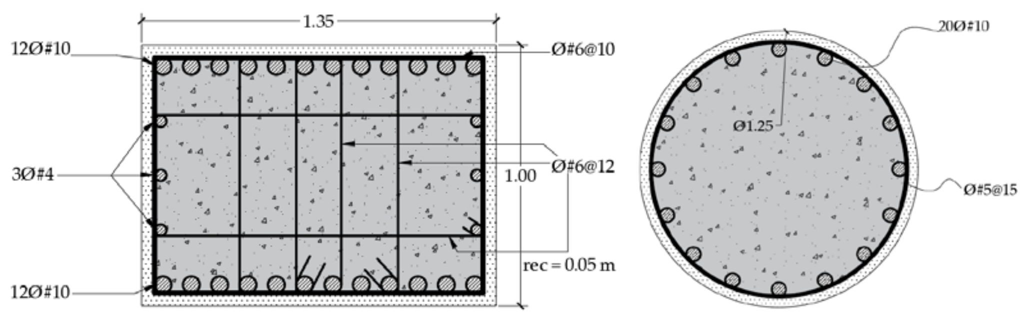


Fig. 1: Geometry and design of: (a) cap beams; (b) columns.

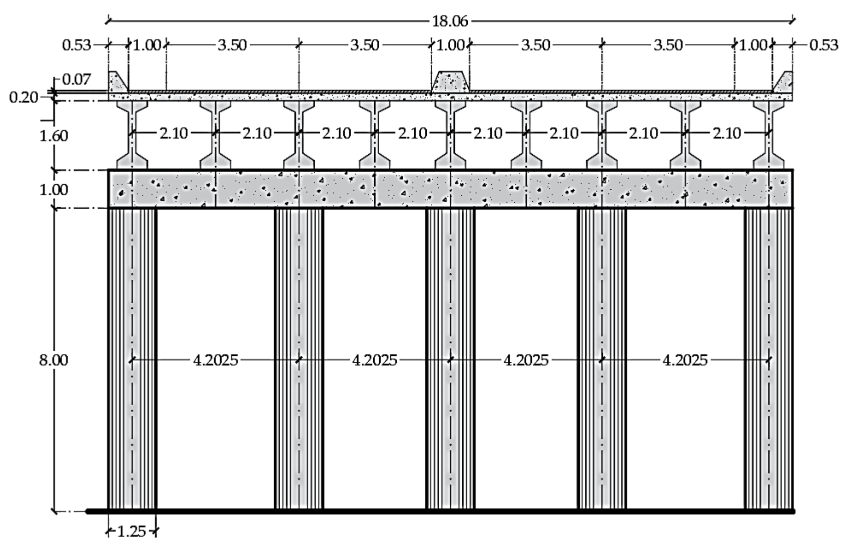


Fig. 2: Transverse section.

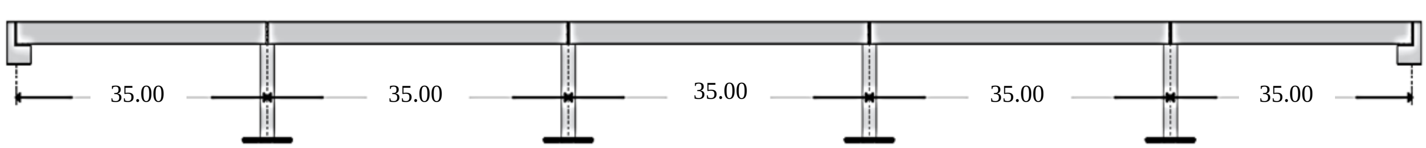


Fig. 3: Longitudinal section.

5.1. Uncertainties in mechanic and geometric properties

The uncertainties related to the manufacturing processes of the materials, as well as, uncertainties related to the construction processes, are considered in this research. Based on the above, table 1 shows the geometric and mechanic uncertainties.

Table 1: Geometric and mechanical uncertainties.

Element	**Bias factor λ**	**Coefficient of variation V**	**Reference**
Beams caps base	1.01	0.04	[21]
Beams caps height	1.000	0.025	[21]
Column width	1.005	0.04	[21]
Slab thickness	0.00381	8.661	[22]
Columns, beams caps and slab, f'c (MPa)	1.27	0.160	[21]
AASHTO beams, f'c (MPa)	1.16	0.127	[21]
Steel diameter #≤1/2; fy (MPa)	1.097	0.081	[23]
Steel diameter #>1/2; fy (MPa)	1.068	0.037	[23]
Steel diameter #≤1/2; fu (MPa)	1.180	0.039	[23]
Steel diameter #>1/2; fu (MPa)	1.155	0.022	[23]

5.2. Structural demand over time

The structural demand is obtained based on the cumulative damage process described in section 3 of this work. It is important to mention that fifty models with simulated mechanical and geometric properties are built. Uncertainties related with randomness of seismic loadings are considering by the construction of fifty histories of seismic intensities and occurrences. Instants of interest of 0, 50, 75, 100 and 125 after the construction of the bridge are considered. The time instant equal to 50 years corresponds to the life service of the structures according to de Mexican Design Code [19]; t = 75 years is associated with the service limit state according to the AASHTO code [24] and 100-125 years correspond to different time of interest.

The nonlinear response is obtained through the philosophy of concentrated plasticity in the extreme of the elements such as cap beams and columns. It is assumed that the lateral stiffness is provided by these structural elements. Furthermore, it is considered that the bridge deck only transmits dead loads. The failure mechanism is achieved when plastic hinges appear in all columns and cap beams. Ruaumoko 3D program [25] is used in order to obtain the nonlinear response. The moment-curvature diagram for RC is made by using confined concrete and stress-strain model for steel reinforced provided by [26] and [23], respectively. The moment-rotation relationship is estimate by using the Modified Takeda hysteresis rule.

Due to the above, figure 4 shows the median of the structural demand, $\overline{D}_{|y,t}$, at the end of each time instant considering the cumulative damage caused by earthquakes. It is observed that the median value of demand increment increase as time progresses. It is noticed that the initial ordinates of the demand curves increase as time progresses (Sa/g=0.05). These initial ordinates present values of 0.0003, 0.0006, 0.0012, 0.0017 and 0.0025 associated with the instants of 0, 50, 75, 100 and 125 years, respectively. This implies that for intensities of 0.05 sa/g the seismic demand increases by 188%, 368%, 529% and 782% for the instants of time of 0, 50, 75, 100 and 125 years, respectively. The above suggest that the structure experimented diverse damage levels over time and when it was evaluated for smaller intensities such as 0.05 sa/g, the structural response increases. For the case of the instant of 125 years at 0.10 sa/g, it is obtained a value of $\overline{D}_{|y,t}$ equal to 0.002 and as it can observed that the pre-establish design drift threshold of 0.002 has reached for the instant of 0 years (without damage) at 0.3 sa/g. This implies an increment of 200% at the intervals of 0 to 125 years for intensities of 0.10 sa/g. On the other hand, it can be observed that the collapse condition is reached when seismic intensities present values higher than 0.75 sa/g.

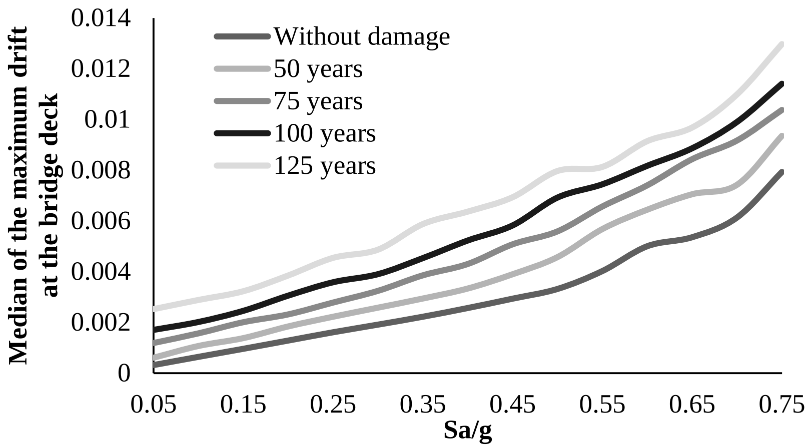
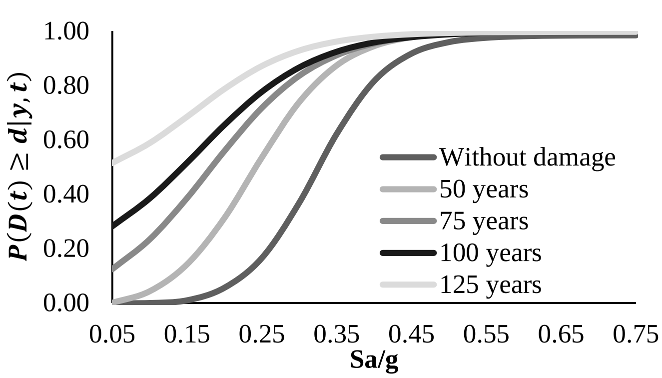


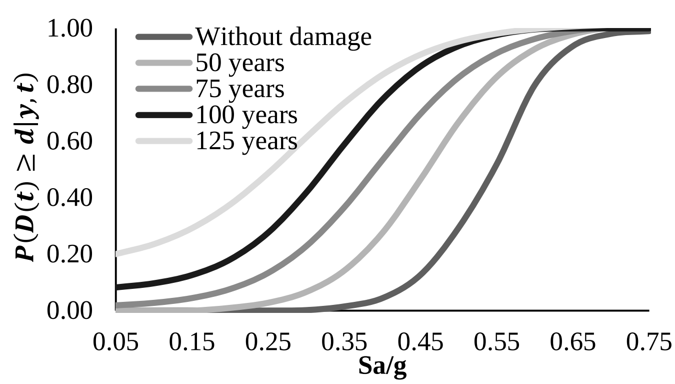
Fig. 4: Median value of the demand at the instants of 0, 50, 75, 100 and 125 years.

5.4. Fragility curves over time

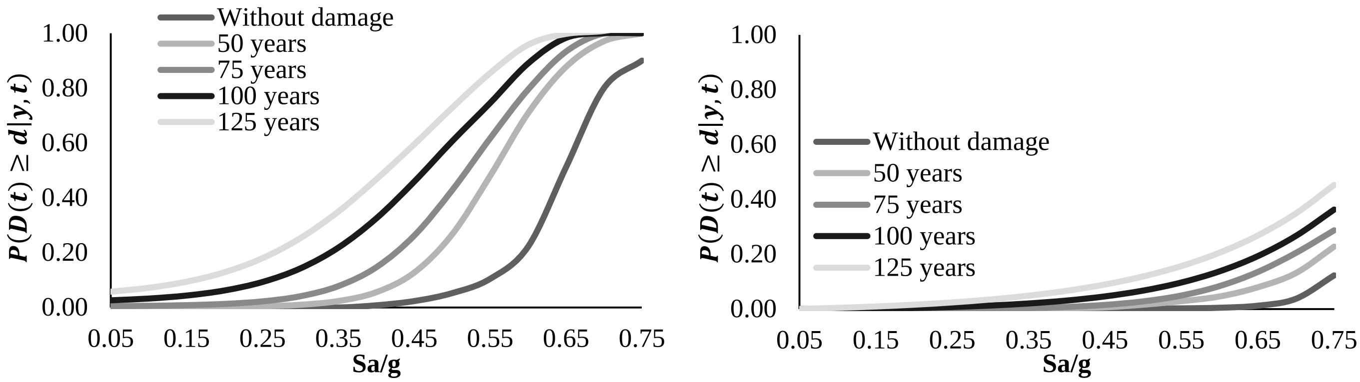
In figure 5a-d the fragility curves for the instants of 0, 50, 75, 100 and 125 years are generated based on different drift thresholds such as 0.002, 0.004, 0.006 and 0.012. The value equal to 0.002 is associated with to the drift threshold design; 0.004 is associated to the service limit state [19]; 0.006 is select as an intermediate threshold and 0.012 is related with the collapse limit state [19]. Figure 5a shows that the probability of exceeding 0.002 is close to zero for intensities smaller than 0.15 sa/g for 0 and 50 years. Besides, it is observed that for intensities greater than 0.55 sa/g, exceedance probabilities close to 1 are reached. It is important to observe that there are different initial ordinates associated with intensities of 0.05 sa/g in the set of curves, this is due to the consideration of cumulative damage over time. Figure 5b shows that the probability of exceeding 0.004 is reached for intensities greater than 0.70 sa/g and there are probabilities close to zero in the cases of 0 and 50 years for intensities not greater than 0.20 sa/g. on the other hand, figure 5c shows that the probability of exceeding the drift threshold of 0.006 is reached after 0.70 sa/g for the cases of 50, 75, 100 and 125 years. Probabilities of exceedance close to zero are reached before 0.25 sa/g for the instants of 0, 50 and 75 years. Finally, in figure 5d shows that the probability of exceeding the drift threshold of 0.012 (collapse limit) at any point in time is close to 1; which implies that the structure will not collapse under this limit state condition.



(a)



(b)



(c) (d)

Fig. 5: Fragility curves over time considering 0, 50, 75, 100 and 125 years: (a) 0.002, (b) 0.004, (c) 0.006, (d) 0.012.

6. Conclusions

An approach to obtain fragility curves that considers the effect of cumulative damage due to earthquakes was proposed. This methodology considers all the possible seismic intensities that may occur on the structure. The seismic intensities simulations were made from the seismic hazard curve which represents the seismicity of the site. The cumulative damage over time was obtained by considering any repair action at any point of time. With this approach, it is possible to design RC bridges for an established performance level. On the other hand, the cumulative damage over time was quantified in terms of the maximum drift of the bridge deck. Different instants of time were considered to evaluate the cumulative damage caused by seismic sequences which were 0, 50, 75, 100 and 125 years. This approach to obtain fragility curves is generalized and it can be used in other type of structures with different solicitations.

The methodology was illustrated in a RC bridge designed to performance a drift threshold of 0.002. Fragility curves were achieved for different drift levels, which were 0.002, 0.004, 0.006 and 0.012. On the other hand, it was observed that for time instants of 75, 100 and 125 years the drift threshold of 0.006 was exceeded, this means that for the time thresholds of 0 (without damage) and 50 years, there is a probability that the drift threshold of 0.006 is not exceeded. Respecting to the drift threshold of 0.012 at no instant in time was an exceedance probability close to 1 reached. Taking into account the previous results and the provisions of AASHTO design code about the life service of bridges (75 years), it is recommended to consider a design drift threshold smaller than 0.006. For the case of the design drift threshold of 0.002, it was observed that for intensities greater than 0.20 sa/g at the instant of 75 years, the probability of exceedance this drift threshold is equal to 0.56. The above implies that this drift threshold has a high probability of been exceeded at 75 years (life service) when a intensity greater than 0.20 sa/g, is presented. The fact that the structure exceeds the design drift threshold at 75 years does not mean that it will be in unfavourable conditions. This approach allows to determinate the conditions with which the structure has after it has been subjected to seismic sequences and it can be useful for decision-making. With this methodology, maintenance and inspection plans can be established in order to extend the service life of the system.

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