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Reliability Assessment of RC Bridges Under Seismic Loads

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ABSTRACT: An approach to estimate the probability of failure, reliability index β and mean annual failure rate by using closed mathematical expressions is presented. The approach is related to the probabilistic seismic demand hazard analysis proposed by Cornell et al., (2002). The uncertainties related to mechanical, geometric and seismic occurrences are considered. The reliability index β, probability of failure and mean annual failure rate are calculated for two different scenarios such as: a) consideration of aleatory uncertainties and 2) both aleatory and epistemic uncertainties are considered. The approach is illustrated on two renforced concrete bridges (RC) structured by AASHTO type beams and circular columns located in Acapulco, Guerrero, Mexico. Recommendations on the importance of considering uncertainties in the reliability analysis are provided.

1. INTRODUCTION

In Mexico, research on the seismic behavior of bridges has not been given the same priority as in case of buildings; proof of this is that there is no comprehensive standard for designing such systems for any type of enviromental load, which implies a certain risk for the population. Bridge structures are vital for communication and their failure or poor operation generates collateral damage such as road traffic problems, economic losses, difficulties in mobilizing emergency equipment and other problems. Due to several environmental factors that may affect RC bridges throughout their lifespan, the structural integrity of their components can deteriorate, leading to a reduction in their structural elements and changes in their reliability. As a result, it is essential to develop approaches to define the reliability levels of RC bridges. Many experts have put forward different techniques for evaluating structural reliability. For example, some researchers use Monte Carlo simulation techniques with FORM and SORM methods to compute the probability of failure (Zhang and Du 2010; Huang et al. 2018), while others develop algorithms based on the Kriging metamodel to assess structural reliability (Jian et al. 2017; Chen et al. 2019). The estimation of the probability of failure using Markov models in road bridges (Chang et al. 2018; Dizaj et al. 2021) and in railway bridges (Gu and Li 2019; Nazarizadeh et al. 2022) have been proposed. The estimation of the structural reliability of steel bridges by considering fatigue deterioration have been presented (Wang et al. 2021; Nie et al. 2022).

The degree of uncertainty faced by engineers during the design process requires the use of concepts and methodologies related to probability theory. Some studies highlight the correct utilization of probability distributions in a reliability evaluation of structures (Quesenberry and Kent 1982; Ditlevsen 1994). Other authors focus on including uncertainties involved in structural reliability related to specific structures such as underground structures (Wu et al. 2019; Zhao 2022), transmission lines (Kulhawy et al. 2006; Rezaei et al. 2016) and vessel structures (Soliman et al. 2015; Niu et al. 2021). Other studies have proposed closed-form mathematical expressions to determine structural reliability by considering aleatory and epistemic uncertainties (Vamvatsikos and Dolšek 2011; Tolentino et al.

2012; Tolentino and Ruiz 2014, 2015; Tolentino and Carrillo-Bueno 2018).

In this study, the structural reliability is expressed in terms of probability of failure, reliability index and mean annual failure rate using simplified closed-form mathematical expressions that consider both aleatory and epistemic uncertainties. This methodology is illustrated on two bridges located in Acapulco, Guerrero, Mexico.

2. RELIABILITY APPROACH

In recent years, there have been various attempts to use probabilistic methods to calculate reliability indexes due to their ability to take into account the inherent uncertainty and variability of real-world systems and provide a more accurate and comprehensive assessment of reliability. The survival probability of a system can be defined as $P_S = e^{v_F t}$ (Yang et al. 2004, 2006), where v_F is the annual failure rate and t is the time. Therefore, the probability of failure is defined as $P_F = 1 - e^{-v_F t}$ (Tobias and Trindade 2011). v_F can also be expressed as:

$$v_F = -\frac{\ln(1-P_F)}{t} \qquad (1)$$

On the other hand, the mean annual failure rate, $E(v_F)$, that takes into account both aleatory and epistemic uncertainties is obtained through the following closed-form expression (Cornell et al. 2002):

$$E(v_F) = k\left(\frac{\hat{C}}{a}\right)^{-\frac{r}{b}} e^{\left[\frac{r^2}{2b^2}\left(\sigma^2_{lnD|y}+\sigma^2_{lnC}+\sigma^2_{UD}+\sigma^2_{UC}\right)\right]} \qquad (2)$$

where k and r are shape parameters of the seismic hazard curve, $v(y)$, a and b are the elements of the median demand, $\widehat{D}$. σ^2_{lnC} and $\sigma^2_{lnD|y}$ are the variances of the natural logarithm of the capacity and demand, respectively. Assuming that v_F follows a Poisson stochastic process. Hence, v_F is equal to $E(v_F)$ and making an equality with Eq. (1) and Eq. (2), the probability of failure, P_F, that considers the uncertainties related to both aleatory and epistemic uncertainties is obtained as follows (Herrera et al. 2022):

$$P_F = k\left(\frac{\hat{C}}{a}\right)^{-\frac{r}{b}} e^{\left[\frac{r^2}{2b^2}\left(\sigma^2_{lnD|y}+\sigma^2_{lnC}+\sigma^2_{UD}+\sigma^2_{UC}\right)\right]}\theta \qquad (3)$$

where:

$$\theta = \frac{e^{-kte^{\sigma_{A,E}}\cdot\left(\frac{\hat{C}}{a}\right)^{-\frac{r}{b}} - \sigma_{A,E}\left(e^{kte^{\sigma_{A,E}}\left(\frac{\hat{C}}{a}\right)^{-\frac{r}{b}}} - 1\right)\left(\frac{\hat{C}}{a}\right)^{\frac{r}{b}}}}{k} \qquad (4)$$

where θ is a correction factor and $\sigma_{A,E}=\sigma^2_{lnD|y} + \sigma^2_{lnC} + \sigma^2_{UD} + \sigma^2_{UC}$. Based on the probability of failure, the reliability index β is as follows (Tolentino et al. 2018):

$$\beta = -\Phi^{-1}[P_F] \qquad (5)$$

where Φ is the standard normal distribution function.

3. EXAMPLE OF APPLICATION

Two continuous RC bridges consisting of four lanes illustrate the reliability approach. The first bridge, referred to as case 1, is constructed with three spans and has a height of 7 meters and a total length of 100 meters (see Figure 1). The second bridge, referred to as case 2, comprises four spans with a total length of 140 meters and a height of 8 meters (see Figure 2). The figures show the cross–section of the bridge, dimensions and reinforcement of columns and cap beams sections of both cases of study. Each case of study has a compressive strength, f'c, equal to 30 MPa in cap beams and columns; for AASHTO beams is considered a value equal to 45 MPa. The structures are designed to comply with a drift level equal to 0.004 and are located in Acapulco, Mexico. Fundamental periods equal to 0.40 and 0.25 s are obtained for cases of study 1 and 2, respectively.

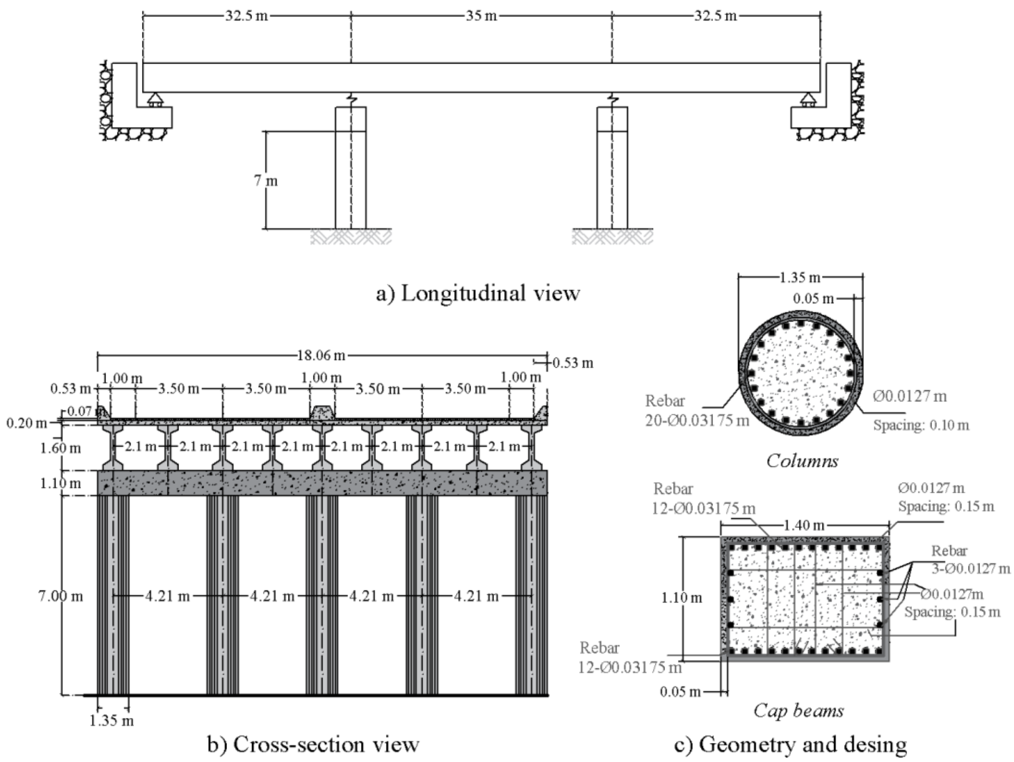


Figure 1. Case of study 1.

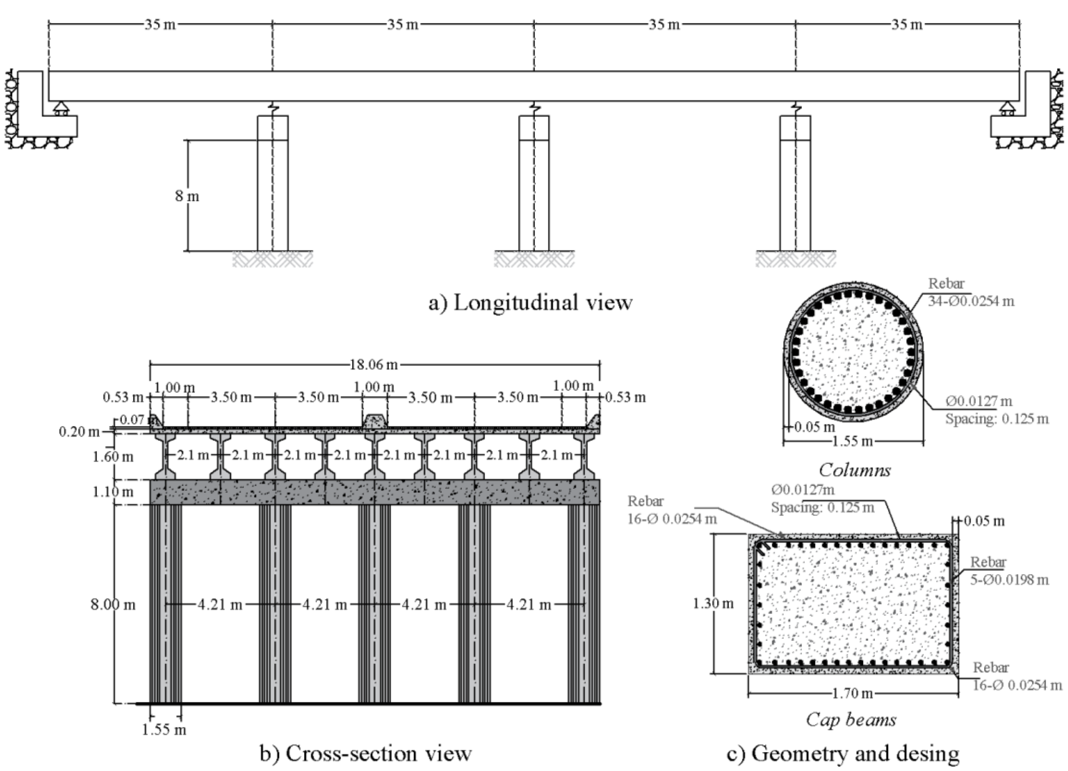


Figure 2. Case of study 2.

3.1. Expected properties of materials, loads and structural sections.

The uncertainties related to the manufacturing and construction processes of the materials are used to calculate the potential bias that is not considered when nominal properties are used. Uncertainties have a significant influence by either increasing of decreasing the structural response. Table 1 presents the statistical metrics for concrete strength (Nowak et al. 2011).

Table 1. Mechanical uncertainties in concrete.

$f'c$ (MPa)	Bias factor	Coefficient of variation
27.60	1.24	0.15
31.00	1.21	0.14
41.40	1.15	0.125

The yield stress of the reinforcement steel is used based on the parameters suggested by Rodríguez and Botero (1995) (see Table 2).

Table 2. Uncertainties for steel reinforcement.

Concept	f_y (MPa)	ε_{sh} (mm/mm)	f_u (MPa)	ε_{su} (mm/mm)
Mean	440.02	0.0066	713.92	0.11
SD	16.57	0.0022	16.28	0.012

The uncertainties associated with structural and nonstructural elements are shown in Table 3. Table 4 shows the statistical parameters for the structural components of the bridge.

Table 3. Statistical parameters for structural and non – structural elements Nowak et al. (2011).

Concept	Bias factor	Coefficient of variation
Factory items	1.03	0.08
Site elements	1.05	0.10
Asphalt	1.00	0.25
Non – structural elements	1.03 – 1.05	0.08 – 0.01

Table 4. Statistical parameters for structural elements (Ellinwood et al. 1980).

Concept	Mean (m)	SD (m)
Slab width	+7.62E-04	6.6E-03
Beam height	-5.334E-03	6.35E-03
Beam width	+2.54E-03	3.81E-03
Columns dimensions	+1.524E-03	6.35E-03
Cover	+8.128E-03	4.318E-03

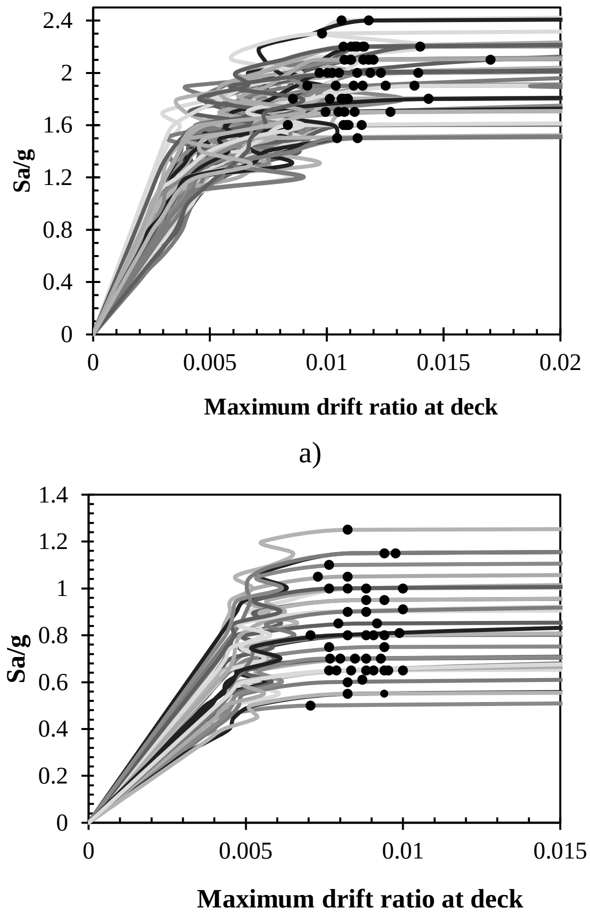
Yu et al., (2016) perform a sensitivity analysis to provide the influence and significance of the uncertainties on the behavior of the RC structures by considering seventeen uncertain parameters. The authors observed that the uncertainties involved in gravity loads, geometry in structural elements, mechanical properties of concrete and reinforcement steel present the greatest influence on the structural response. The above suggests that these factors should be given particular attention for design or evalution of RC structures.

3.2. Non-linear analysis

The inelastic response of the systems is given by the appearance of plastic hinges on both ends of the columns and cap beams, which means that these structural members contribute to the lateral stiffness of the bridge structure. It is important to mention that the deck only transmits dead loads. For this purpose, Ruaumoko 3D program (Carr 2003) is used to calculate the non-linear response of the structural system. The Modified Takeda hysteresis rule (Otani 1974) is used. The moment-rotation relation for each structural element is estimated using a stress-strain constitutive model of confined reinforced concrete reported by Mander et al. (1988) and Rodríguez and Botero (1995) for Mexican steel reinforcement.

3.3. Ground motions

The ground motion data set used as input for the seismic simulations has been extracted from two seismographic stations close to the study cases "Acapulco Centro Cultural" (ACAC) and "Acapulco Diana" (ACAD), which complies with forty ground motions recorded during the past decades that are related to events with a magnitude between 4 and 7.5 (Tolentino and Carrillo-Bueno 2018). The dominant period of the soil site ranges from 0.3 to 0.64 s while the maximum ground acceleration of ground motions varies from 0.019 to 0.44 Sa/g. The epicentral distance is about 170 km.



b)

Figure 3. Structural capacity due to earthquakes: a) case of study 1 and b) case of study 2.

3.4. Structural capacity assessment

The structural capacity is evaluated by performing incremental dynamic analysis (IDA). The IDA curves (see Figure 3) present three distinct phases: 1) structural elastic behavior, 2) inelastic response zone and 3) the point near failure, which is identified with a black dot. Considering that incipient failures follow a lognormal distribution (Rosenblueth and Esteva 1972; Cornell et al. 2002). The median and standard deviation of the structural capacity are included in Table 5.

Table 5. The structural capacity of cases in the study.

Case	$\hat{C}$ (m)	σ_{lnC}
1	0.0114	0.18
2	0.0086	0.21

3.5. Structural demand evaluation

The median of structural demand, $\widehat{D}$, is calculated by non-linear dynamic analysis. The seismic records are scaled up until failure of the structure occurs. Figure 4 shows the median of the structural demand and the fitted function. For case study 1, the parameters that described the shape of the structural demand are $a = 0.00313$ and $b = 2.092$. On the other hand, values equal to 0.01651 and 1.5 are obtained for variables a and b, respectively in case of study 2. The following is observed: a) the median of structural demand increases proportionally to the seismic intensities; b) seismic intensities less than 0.45 Sa/g do not generate structural damage in both analysis cases.

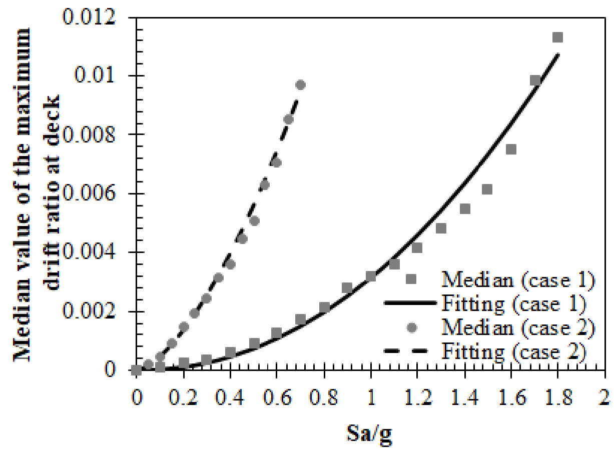


Figure 4. The median of the structural demand.

3.6. Spectral acceleration hazard

The spectral acceleration hazard curve (SAHC) is known and it is associated with the fundamental period of the structures and site. SAHC is used to describe the seismic hazard that the structure could be exposed and it is described through the constants k and r proposed by Cornell et al. (2002). Therefore, the constants k and r are equal to 0.003513 and 1.6224, respectively for the bridge in case 1. For the case of study 2, $k = 0.002757$ and $r = 1.3145$.

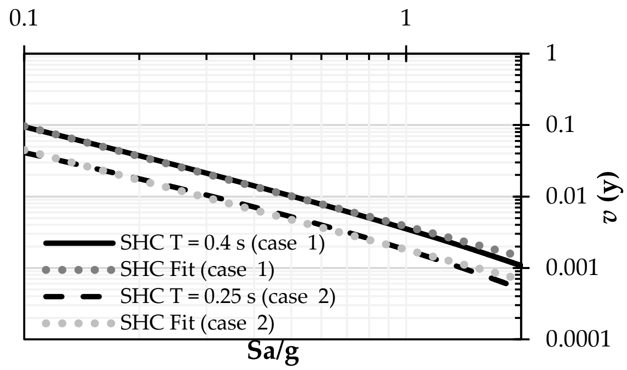


Figure 5. Spectral acceleration hazard curves.

3.7. Reliability estimation

The reliability index β, the probability of failure and the mean annual failure rate are computed by considering and not considering epistemic uncertainties; the parameters used are outlined in the previous section 3.5. The structural capacity, $\hat{C}$, is obtained in section 3.4. Epistemic uncertainties are considered with a value equal to 0.15 (FEMA-350 2000). Table 6 shows the reliability indicators calculated with and without considering epistemic uncertainties. It is observed that the mean annual structural failure rate with epistemic uncertainties resulted in 16.10% and 2% greater than the scenario without epistemic uncertainties for studies 1 and 2, respectively. On the other hand, the epistemic uncertainties lead reduce the return period ($1/E(v_F)$). In case 1 the return period is equal to 653 years and when these uncertainties are not considered, the return period is equal to 759 years. For case study 2, the return periods are equal to 376 and 389 years for with and without epistemic uncertienites, respectively. The significant difference in return periods is attributed to the higher failure probability in case 2. This probability is 57% and 50% higher when considering and not considering epistemic uncertainties in comparison to the results of case 1. For the cases studied, the reliability index β do not present acceptable values in accordance to AASHTO code ($\beta \geq 3.5$).

Table 6. Reliability indicators.

Analysis consideration	Case of study	$E(v_F)$	P_F	β
With epistemic uncertainties	1	1.531E-03	1.529E-03	2.96
	2	2.657E-03	2.653E-03	2.79
Without epistemic uncertainties	1	1.318E-03	1.317E-03	3.01
	2	2.595E-03	2.591E-03	2.80

4. CONCLUSIONS

A methodology estimate the reliability index, probability of failure and mean annual failure rate by using closed-form expressions is presented. This approach considers the uncertainties related to aleatory and epistemic.

The methodology was applied on two continuous bridges designed to perform a drift threshold equal to 0.004. The analysis considers uncertainties in mechanical, geometric properties and seismic loadings. A reliability index ≥3.5 is considered as an acceptable safety level according to AASHTO standards. The two bridges do not satisfy this specification. In case of study 1, the reliability index is 15.43% lower than the one recommended when epistemic uncertainties are considered and 14% lower when they are not considered. A similar situation occurs in case of study 2, where the reliability index is 20.29% and 20% less than the recommended level when epistemic uncertainties are not considered, respectively. This highlights the importance of considering epistemic uncertainties in the design and evaluation of bridges to ensure the safety and performance of the structure. Therefore, a design drift threshold of 0.004 is not recommended for this type of structure. It is suggested to explore lower values between 0.001 and 0.003.

The reliability indicators are important parameters to evaluate the safety and performance of a structure, making them useful for decision-making. The proposed methodology can be used to design any structural system with high social impact; such structures must have acceptable reliability levels after a certain environmental load. This methodology can be extended by considering the cumulative damage over time to formulate a prioritized schedule for guiding decision makers on the restoration of bridges based on their structural condition.

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