Life-Cycle Cost of Bridges on Seismic Zones for Risk Management

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1. Abstract

In this paper the acceptable failure probability and the risk of important bridges, located on seismic zones, are calculated throughout the expected cost of failure consequences. Also, the bridge expected life-cycle cost is formulated in terms of the bridge seismic hazard and the potential consequences of failure. These consequences include aspects arising from the physical loss of the bridge to the human casualties and economical cost of the loss of service, which are estimated in monetary terms.

Current codes do not explicitly deal with this issue and in practice subjective estimations from experience are considered for some general cases.

Bridge reliability is an essential component of risk and in this paper is estimated in a simplified way and applied to the structural types given in the examples. Monte Carlo simulation techniques are used to explicitly account for the uncertainties. Initial and failure cost curves are determined for all possible seismic intensities and expected life-cycle costs conditional to these intensities are obtained. The unconditional expected life-cycle cost is calculated by convolution of the conditional costs by the occurrence probabilities of these intensities, which are obtained from the seismic hazard curve of the given site.

The procedure is illustrated throughout three reinforced concrete bridges located 1 on the soft soil of Mexico City and the other 2 on other sites with less seismic activity and different traffic volumes.

The results may be extended to get risk management policies for bridges and to improve the current Mexican codes and to enhance the practices on bridge design and maintenance on seismic zones.

Key words: seismic risk of bridges, expected life-cycle cost, bridge reliability, risk management

2. Resumen (Costo en el ciclo de vida de puentes en zonas sísmicas para administración del riesgo)

En este artículo se calculan la probabilidad aceptable de falla y el riesgo de puentes importantes, localizados en zonas sísmicas, a través del costo esperado de consecuencias de falla. Asimismo, el costo esperado en el ciclo de vida del puente se formula en términos del peligro sísmico y las potenciales consecuencias de falla del mismo. Estas consecuencias incluyen aspectos que abarcan desde la pérdida física del puente hasta las fatalidades, que se estiman en términos monetarios, y el costo económico de la pérdida de servicio.

Los reglamentos actuales no tratan explícitamente con estos conceptos y, en la práctica, se consideran estimaciones subjetivas, de la experiencia, para unos pocos casos.

La confiabilidad del puente es un componente esencial del riesgo y en este artículo se estima de manera simplificada y se aplica a los tipos estructurales tratado en los ejemplos. Se usan técnicas de simulación de Monte Carlo para tomar en cuenta explícitamente las incertidumbres. Se determinan curvas de costo inicial y de falla para todas las posibles intensidades sísmicas y se obtienen los costos esperados en el ciclo de falla para esas intensidades. El costo esperado incondicional en el ciclo de vida se calcula por convolución de los costos condicionales por las probabilidades de ocurrencia de las intensidades, que se obtienen de la curva de riesgo sísmico del sitio.

El procedimiento se ilustra para 3 puentes de concreto reforzado localizados, uno en la zona de suelo blando de la Cuidad de

México y los otros 2 en otros sitios con menor actividad sísmica y diferentes volúmenes de tráfico.

Los resultados pueden extenderse para obtener políticas de administración de riesgo de puentes y para mejorar los reglamentos mexicanos de puentes en lo relativo a las prácticas de diseño y mantenimiento de puentes en zonas sísmicas.

Palabras clave: riesgo sísmico de puentes, costo esperado en el ciclo de vida, confiabilidad de puentes, administración de riesgo.

3. Introduction

Several authors have presented their views about the purpose of designing a structure (Cornell, 1969; Rosenblueth and Esteva, 1972). For example, it has been said (Agarwal, 2003) that "the objective of a design process is to reach an acceptable probability that the designed structures will behave satisfactorily during their lifetimes. Therefore they are designed to withstand all possible loads and deformations during construction and normal use". Some modern codes emphasize the safety management (BS 8800, 1996). However, to date, none provides explicitly procedures or guidelines to manage the seismic risk throughout maintenance. Engineering judgement produces subjective estimations on the expected failure consequences but the risk management concept has not been quantitatively introduced on the codes so far.

Structural reliability studies contribute to anticipate failures through an appropriate maintenance program (Nessim and Stephens, 1995). It has been recognized (Renn, 1998) that risk management is the process where Society reduces risks to tolerable levels and attempts to control them, monitor them and inform about them to affected people.

Cuevas and Robles (1994) sustain that the structural safety is defined during the design process, when the designer must verify that the resistance is over the demands that will act over it during its lifetime. Such descriptions have implicit the concept of structural reliability.

According to Meli (1994), the reliability of a structure is associated to a certain cost which should be minimized to balance safety with cost. Therefore, an optimization process should be performed where the objective function must include the initial cost of the work and the cost of the potential damages and other consequences in case that a failure occurs. Therefore, if C_t is the total cost of the structure, C_i is the initial cost, C_d is the cost of failure consequences and Pf the bridge failure probability:

$$C_t = C_i + C_d \cdot P_f \qquad (1)$$

The bridge reliability has been widely studied (Neves, *et al.*, 2003) and the bridge maintenance field has been enriched by important contributions (Estes and Frangopol, 1996; Das, 1999) including the formulation of procedures to estimate the deterioration, actual bridge condition and the remaining service life (Noortwijk, J. M. van and Frangopol, 2004; Thoft-Christensen, 1996). Also, cost functions for buildings design or retrofit has been proposed in the framework of cost/benefit and cost effective recommendations (Ang and De León, 1997; De León, 1996) and the optimal design and performance-based criteria has been advanced and improved (Frangopol, *et al.*, 2006a; Frangopol and Liu, 2006).

In this paper, some of these progresses are considered and the bridge acceptable failure probability is estimated in terms of the bridge failure consequences. Also, the bridge failure probability due to its seismic exposure is calculated to develop risk-based recommendations. These recommendations constitute a preliminary step towards the generation of risk management strategies to be applied to a regional bridge inventory in Mexico. Available structural reliability tools and seismic risk concepts (Esteva, 1969) are applied to obtain reliability-cost curves useful to provide basis to manage the bridge seismic risk either for design or retrofit purposes.

The bridge seismic risk is accounted for throughout the annual cumulative probability for maximum intensities (DDF, 1988) on the soft soil of Mexico City. This distribution is obtained from the seismic hazard curve for the site, previously reported (Esteva and Ruiz, 1989), and the assumption of Poissonian occurrence of significant earthquakes at the bridge location is taken into account. This distribution is used to randomly simulate the intensities and obtain the bridge failure probability.

The bridge limit state is defined in a simplified way, by considering the most probable failure mode, as identified from the experience of bridge design engineers and preliminary analyses of the selected bridges. Once the most critical structural component and failure mode were identified, the limit state was specified and the statistical properties of the participating variables were obtained. Then, the bridge conditional reliability and failure probability were calculated through Monte Carlo simulation for given values of potential seismic intensities and they were weighted by the occurrence probability of earthquake intensities for the bridge location to get the unconditional failure probability.

4. Acceptable failure probability

By recognizing the uncertainties inherent in the design process, especially the ones due to the seismic hazard, it has

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been proposed (Frangopol *et al.*, 2006b) to appraise the bridge performance by using the concept of expected life-cycle cost. From offshore technology practices, (Stahl, 1986), the expected life-cycle cost $E[C_t]$ is expressed in terms of the initial cost C_i and the expected failure/damage cost $E[C_d]$.

$$E[C_t] = C_i + E[C_d] \qquad (2)$$

where

$$E[C_d] = PVF(P_f)C_d \qquad (3)$$

And *PVF* is the present value factor. Given that this formulation includes all possible adverse events, either failure or damage that may occur within the bridge lifetime, the *PVF* considers all those potentially damaging events not just the worst scenario of total collapse. Also, the average damage cost C_d is composed by the costs of consequences:

$$C_d = C_r + C_f + C_e \qquad (4)$$

Where C^r is the repair/restitution cost, C_f is the cost related to fatalities and C_e is the economic loss due to the service interruption, user costs, while the bridge is repaired or rebuilt. PVF depends on the net annual discount rate r and the bridge lifetime T:

$$PVF = \frac{1 - \exp(-rT)}{r} \qquad (5)$$

If the initial cost C_i is expressed as a function of the failure probability, (Rosenblueth, 1986), the expected lifecycle cost becomes a function of the failure probability

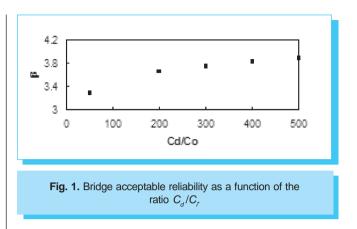$$E[C_t] = C_1 - C_2 \ln(P_f) + PVF(P_f)C_d \qquad (6)$$

The acceptable (optimal) failure probability may then be calculated by minimizing the expected life-cycle cost respect the failure probability

$$\frac{\partial E[C_t]}{\partial EP_f} = 0 \qquad (7)$$

$$P_f = \frac{0.434C_2}{PVF[C_d]} \qquad (8)$$

Given that the acceptable failure probability depends inversely of the cost of consequences, the safety requirement becomes stricter as those consequences are higher. Also, the requirement may be expressed in terms of the bridge reliability index.

$$\beta_a = \Phi^{-1}(1 - P_f) \qquad (9)$$

Fig. 1. Bridge acceptable reliability as a function of the ratio C_d/C_r

According to previous results, (De León *et al.*, 2006; De León *et al.*, 2007), the cost of consequences has been normalized to the initial cost and $C_2/C_i = 0.75$ for typical bridges. Also, for $T = 200$ years and $r = 0.08$, the bridge acceptable reliability has been plotted against the cost ratio C_d/C_i. See Fig. 1.

For the bridges considered here, it has been estimated that the costs of consequences are 800, 200 and 50 times (because of the user costs, traffic volume and bridge importance) the initial cost and, therefore, the acceptable bridge reliability a are approximately 3.99, 3.65 and 3.28, respectively for the examples shown ahead.

5. Bridge reliability

From the well known FORM, first order reliability method, the bridge reliability may be calculated, (Ang and Tang, 1984):

$$\beta = \frac{E(G)}{\sigma_G} \qquad (10)$$

Where G is the bridge limit state considering its exposure to seismic loads, $E(G)$ the expected value of such limit state and G its standard deviation. Although the bridge is a complex structural system, from previous analyses for typical bridges (De León *et al.*, 2007), the limit state has been conservatively approximated in terms of the failure of the most critical structural element. It was found that this element is one of the main piles and it is subject to a combination of axial load + bending. Therefore, G is calculated:

$$G = 1 - \left[\frac{P_A}{P_R} + \frac{M_A}{M_R} \right] \qquad (11)$$

Where P_A is the maximum acting axial load, P_R the axial resistant force, M_A the maximum acting moment and M_R the resistant

moment of the critical cross section. Given that P_A and M_A are a consequence of the random earthquakes that may occur during the bridge lifetime, these mechanical responses are random variables. Also, from the variability of materials properties, the resistances P_R and M_R are also random. The standard deviation σ_G is:

$$\sigma_G \approx \sqrt{\sum_{i=1}^{4}\left[\frac{\partial G}{\partial X_i}\right]^2 \sigma_{x_i}^2} \qquad (12)$$

In Eq. (3.3), X is the vector of acting and resisting axial loads and moments, such that, $X_1 = P_A, X_2 = P_R, X_3 = M_A$ and $X_4 = M_R$ and the derivatives are evaluated on the mean values. Therefore:
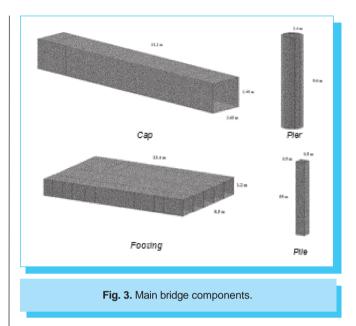
$$\sigma_G \approx \sqrt{\frac{\sigma_{P_A}^2}{\overline{P_R}^2} + \left[\frac{\overline{P_A}}{\overline{P_R}^2}\right]^2 \cdot \sigma_{P_R}^2 + \frac{\sigma_{M_A}^2}{\overline{M_R}^2} + \left[\frac{\overline{M_A}}{\overline{M_R}^2}\right]^2 \cdot \sigma_{M_R}^2} \qquad (13)$$

Where σ_{MR}, σ_{MA}, σ_{PR} and σ_{PA} are the standard deviations of the resistant and acting moments, and the resistant and acting axial loads, respectively.

The standard deviations are obtained from previously reported values and from the use of simplifying assumptions. The mean values are considered to be the bridge response to mean seismic intensities.

6. Application to selected bridges
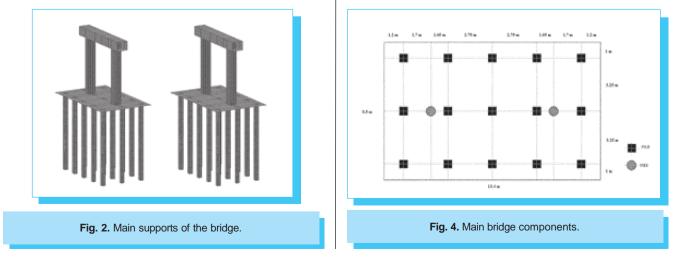## 6.1. Bridge on the soft soil of Mexico City

The structure is a bridge built on the Benito Juarez International airport area, in the transition seismic zone III, in order to improve the traffic conditions there. The bridge has a 400 m total span

Fig. 3. Main bridge components.

divided into 16 segments of 25 m each. The structural modeling was made through a finite element-based commercial software (RAM Advanse, 2006) and the sketches of the main structural members are shown in Figures 2 and 3.

Essentially, the main structural components of the bridge are: the transverse cap, two piers, the footing and the piles. Figure 4 shows the plant location and dimensions of the piers and piles. The mean reinforced concrete properties are $f'c = 250$ kg/cm^2 and $fy = 4200$ kg/cm^2.

The bridge structural type is typical for modern construction in Mexico and, given that it was built in a heavily populated area and it has a strong traffic demand, it was carefully designed and built.

Fig. 2. Main supports of the bridge.

Fig. 4. Main bridge components.

A family of bridge designs were obtained (RCDF, 2004; AASHTO, 2002) by varying the original design dimensions and steel areas. These designs allowed for a series of alternative designs to measure the variation of reliability with cost under specified seismic intensities. The bridge designs were analyzed under given maximum seismic coefficients c/g, using the typical spectral form for Mexico City, and according to the range of intensities as reported in Mexican seismic hazard and failure rates studies (Esteva and Ruiz, 1989). Table 1 shows a sample of the results obtained by varying the seismic coefficients from 0 to 0.60g at each 0.15g and for specific design alternatives. Table 1 contains the seismic coefficient, the rebars size, mean values of maximum axial load and moment and axial and moment resistances, reliability index and the initial costs obtained. Pier radius is 1.4 m and the number of rebars is 11.

Five alternative designs and the five maximum intensities shown in table 1, were considered and the corresponding reliability indices and initial costs were calculated. For the standard deviations, it was used CVA = 0.25 and CVR = 0.1 and the following simplifications were made (De León *et al.*, 2007):

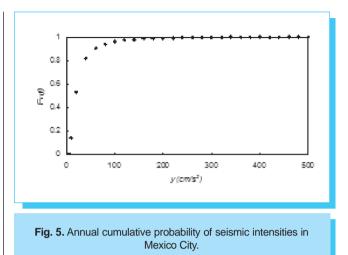$$CV_A = \frac{\overline{P}_A}{\sigma_{P_A}} = \frac{\overline{M}_A}{\sigma_{M_A}} \qquad (14)$$

$$CV_R = \frac{\overline{P}_R}{\sigma_{P_R}} = \frac{\overline{M}_R}{\sigma_{M_R}} \qquad (15)$$

All the initial costs are conditional to the ocurrence of the indicated intensity. In order to obtain the unconditional curve, the ordinates of the conditional curves need to be weigthed by the ocurrence probabilities according to the seismic hazard curve for Mexico City (Esteva and Ruiz, 1989). See Figure 5.

The conditional curves, for the prescribed intensities c/g indicated in the box, and the unconditional curve are shown in Figure 6.

By considering that the damage/failure cost is 800 times the initial cost, the expected failure cost and expected life-cycle cost are calculated.
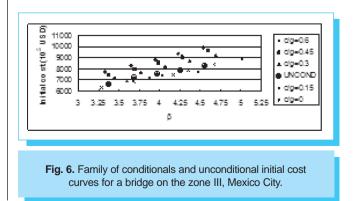

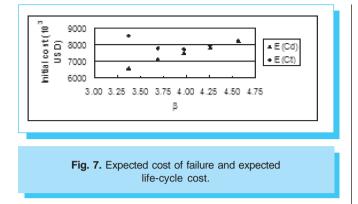**Fig. 5.** Annual cumulative probability of seismic intensities in Mexico City.

Plan areas of the bridge viaduct and under the bridge were considered to estimate, according to average traffic, the expected number of deaths given the bridge collapse during a strong earthquake. A fatality cost for an individual is taken as 0.2 million USD considering an individual with 25 years of remaining productive life, if he would not have died from the bridge collapse, and a GNP of 8000 USD. Then, if 800 people were died, the fatality cost would be 160 million USD. These high costs result because of the wide bridge extension (about 4.5 km).

Also, as the bridge cost is 7 million USD and the economic loss due to service interruption, user losses, was estimated to be 4400 million USD (coming from 880 million man-hours lost during the reconstruction period of a year, and 5 USD the average cost of hour lost per individual). These costs are also high because they cover the whole bridge system on the area nearby the airport.

From the conditional failure probabilities, the expected cost of failure is obtained and then the unconditional expected

Table 1. Sample of the calculations for cost-reliability curve.

c/g	Vars #	$P_A(T)$	$P_R(T)$	M_A $(T*m)$	M_R $(T*m)$	β	C_I ($USD)
0.00	22	228.97	817.77	529.78	541.29	3.28	6338.4
0.15	26	227.4	947.43	575.51	618.62	3.68	6918.6
0.30	28	225.82	1026.46	621.24	656.23	3.44	7193.6
0.45	30	225.82	1075.38	666.97	689.22	3.37	7468.7
0.60	32	222.67	1171.87	712.7	722.25	3.33	7743.7

Fig. 6. Family of conditionals and unconditional initial cost curves for a bridge on the zone III, Mexico City.

Fig. 7. Expected cost of failure and expected life-cycle cost.

cost of failure is calculated through the convolution with earthquake intensities occurrence probabilities. Finally, the unconditional expected life-cycle cost results from the addition of both unconditional curves: the initial cost and the expected cost of failure. The results are shown in Fig. 7.

6.2 Bridge Cuto

This is a reinforced concrete bridge with a structural system composed by a flat reinforced concrete slab supported by squared reinforced concrete piles. It has two 12.5-m spans and its photograph is shown in Fig. 8.

The bridge is located on a medium seismicity zone and its traffic demand is a lot less than the one for Mexico City and, as a result, the cost of consequences has been estimated to be 50 times the initial cost. The acceptable annual reliability is therefore 3.28.

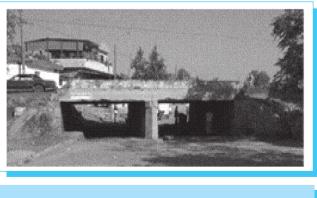The annual cumulative distribution of seismic intensities, for the Bridge Cuto, is shown in Fig. 9.

Fig. 8. Bridge Cuto.

By following the same procedure as for the bridge in Mexico City, shown in section 4.1, the curves for the initial, expected cost of failure and expected life-cycle cost are obtained for this bridge. The results may be seen in Figs. 10 and 11.
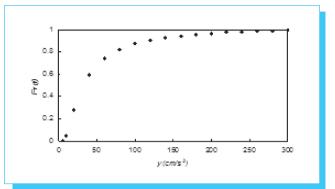

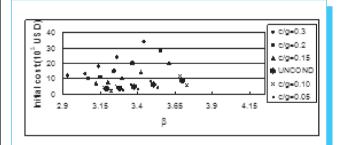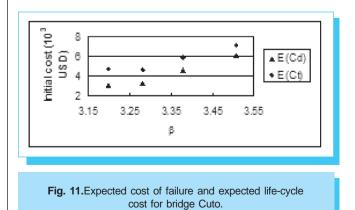**Fig. 9.** Annual cumulative probability of seismic intensities for bridge Cuto.

Fig. 10. Family of conditionals and unconditional initial cost curves for bridge Cuto.

Fig. 11. Expected cost of failure and expected life-cycle cost for bridge Cuto.

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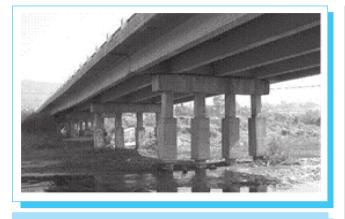*Científica*, Vol. 13 Núm. 3 (2009), pp. 135-143. ISSN 1665-0654, ESIME-IPN.
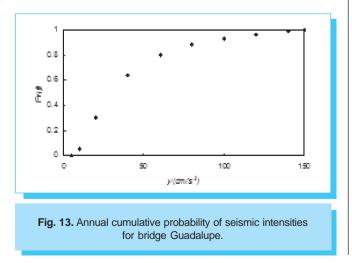
Fig. 12. Bridge Guadalupe.
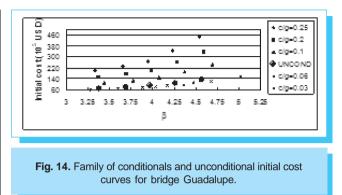
6.3 Bridge Guadalupe

This is also a reinforced concrete bridge with a structural system composed by a flat reinforced concrete slab supported by squared reinforced concrete piles. It has 6 spans with a total length of 169.5m and it is located on a low-seismic activity zone.

The annual cumulative distribution of seismic intensities, for the Bridge Cuto, is shown in Fig. 13.

The bridge traffic demand is intermediate as compared to the last 2 bridges and, as a result, the cost of consequences has been estimated to be 200 times the initial cost. The acceptable annual reliability is therefore 3.65.

Again, by following the same procedure as for the bridge in Mexico City, shown in section 4.1, the curves for the initial,
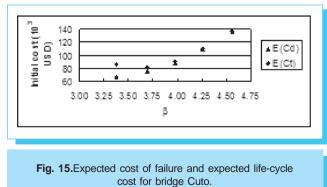

Fig. 13. Annual cumulative probability of seismic intensities for bridge Guadalupe.

Fig. 14. Family of conditionals and unconditional initial cost curves for bridge Guadalupe.

expected cost of failure and expected life-cycle cost are obtained for this bridge. The results may be seen in Figs. 14 and 15.

7. Discussion

The actual design, for the bridge in Mexico City, the one at the middle of the 5 alternative designs, has a reliability index of 3.98 which is practically equal to the optimal of 3.99, according to Figures 1 and 7. Also, it is noted that the unconditional curve resulted between the conditionals for 0.15g and 0.3g showing that the optimal seismic design coefficients is somewhere between these intensities. The influence of the above mentioned intensities is explained by the incremental occurrence probabilities that appear in the annual cumulative probability curve shown in Fig. 5. The high value of the optimal reliability index is due to the very high failure consequences for the bridge, located on a highly populated area with an almost permanent heavy traffic. It is observed that the optimal reliability index, as indicated by the minimum of the expected life-cycle curve in Fig. 7, is very close to the one derived from Fig. 1. The bridge was carefully designed and built and, given that is new, does not need any maintenance. However it will be interesting to see its performance once the next significant earthquake strikes Mexico

Fig. 15. Expected cost of failure and expected life-cycle cost for bridge Cuto.

City. The reliability curves allow the estimation of rates like, for example, with an additional 4 or 5% of initial investment (increasing the initial reliability) the failure probability may be reduced up to 1/3 of its original value. It is possible, therefore, to identify the ranges of the reliability curves, according to its slope, where the cost of reducing failure probability is not so expensive and economical cost/benefit recommendations may be derived for a specific seismic environment, like in this example the considered bridge under seismic loading.

The annual reliability index for the bridge Cuto is 3.28, exactly the acceptable 3.28. That means that this bridge is also in acceptable conditions. Although some attention is needed, because it is located on a zone with intermediate seismicity, the failure consequences are not so high as the bridge in Mexico City.

The annual reliability index for the bridge Guadalupe is 3.69, slightly over the acceptable 3.65. That means that this bridge is also in acceptable conditions. Some attention is needed because, although it is not located on a seismic zone, the failure consequences are as low as the ones for the bridge Cuto.

It is observed that both conditions: seismic activity of the bridge location and traffic and importance of the failure consequences are important to protect the bridge, as reflected in the reliability and cost items of the formulation.

8. Conclussions

Some risk and reliability calculations have been performed for three typical reinforced concrete bridges in Mexico under different seismic demand and different traffic volume. Because of the heavy traffic and the large human lives at risk, the cost of consequences is very large. The bridges may be classified according to the importance of failure consequences. The optimal reliability index depends, therefore, on the level of failure consequences. It was found that the three bridges have annual reliability indices slightly over the optimal ones. For design of new bridges similar to the ones considered here, cost-effective recommendations may be derived, as for example, the 5% initial cost increment to significantly reduce the bridge failure probability. In a large scale, these cost/benefit rates may lead to optimal maintenance strategies for the whole bridge inventory of Mexico. The study may be extended to all types of bridges, considering materials, age, current condition, span and traffic volume.

The analyses were simplified by considering only the most critical member. Further studies should be performed to measure the actual redundancy and all the other potential failure modes.

Also, additional research should be undertaken to generalize the results and update the current Mexican bridge design code. The risk-based formulation may be used to study other infrastructure works and other hazards in Mexico.

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