BASIS FOR RISK MANAGEMENT ON BRIDGES ON SEISMIC ZONES

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ABSTRACT :

In this paper, the acceptable failure probability for important bridges is 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 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.
Bridge reliability is an essential component of risk and, for a given structural type is estimated in a simplified way. Monte Carlo simulation techniques are used to explicitly account for the uncertainties. Initial and failure cost curves are determined for all possible soil accelerations and expected costs conditional to these accelerations are obtained. The unconditional life-cycle cost is calculated by convolution considering the seismic hazard curve of the site.
The procedure is illustrated throughout a reinforced concrete bridge on the soft soil of Mexico City.
The results may be extended to get risk management policies to improve the current Mexican codes and to enhance the practices on bridge design and maintenance on seismic zones.

KEYWORDS: **SEISMIC RISK OF BRIDGES, LIFE-CYCLE COST, BRIDGE RELIABILITY**

1. INTRODUCTION

Several authors have presented their views about the purpose of designing a structure (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” 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 a failure occur. Therefore, if C_t is the total cost of the structure, C_i the initial cost C_d the cost of failure consequences and P_f the failure probability:

$$C_t = C_i + C_d \cdot P_f \tag{1.1}$$

2. ACCEPTABLE FAILURE PROBABILITY

By recognizing the uncertainties inherent in the design process, especially the seismic hazard, it has been proposed, (Frangopol et al, 2001), to appraise bridge performance by using the expected life-cycle assessment.
In the offshore technology (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 $E[C_d]$.

$$E[C_t] = C_i + E[C_d] \tag{2.1}$$

Where:

$$E[C_d] = PVF(P_f)C_d \tag{2.2}$$

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. Also, the average damage cost C_d is composed by the costs of consequences:

$$C_d = C_r + C_f + C_e \tag{2.3}$$

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:

$$PVF = \frac{1 - \exp(-rT)}{r} \tag{2.4}$$

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

$$E\left[C_t\right] = C_1 - C_2 \ln(P_f) + PVF(P_f)C_d \tag{2.5}$$

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

$$\frac{\partial E[C_t]}{\partial P_f} = 0 \tag{2.6}$$

$$P_f = \frac{0.434 C_2}{PVF[C_d]} \tag{2.7}$$

The acceptable failure probability depends inversely of the cost of consequences which means that, according to the bridge importance, the safety requirement should be stricter as those consequences increase. Also, the requirement may be expressed in terms of the bridge reliability index

$$\beta_a = \Phi^{-1}(1 - P_f) \tag{2.8}$$

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 = 0.08 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 bridge considered here, it has been estimated that the costs of consequences is 800 times (because of the high traffic volume) the initial cost and, therefore, the acceptable bridge reliability β_a is approximately 3.84.

3. BRIDGE RELIABILITY

From well known structural reliability theory, the bridge reliability may be calculated, (Ang and Tang, 1984):

$$\beta = \frac{E(G)}{\sigma_G} \tag{3.1}$$

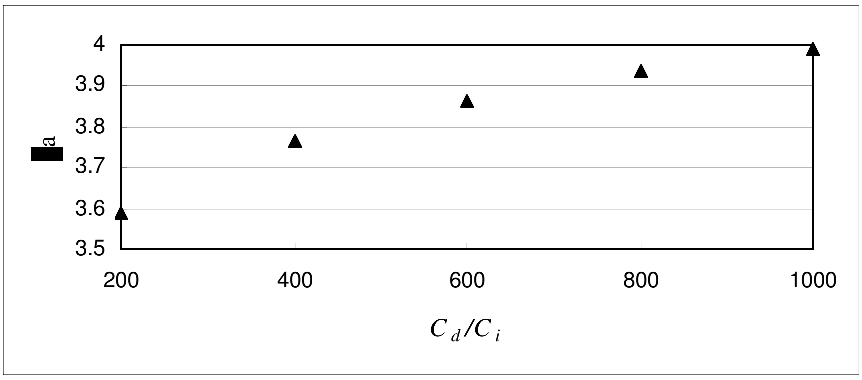

Fig. 1 Bridge acceptable reliability as a function of the ratio C_d/C_i

Where G is the bridge limit state considering its exposure to seismic loads, $E(G)$ the expected value of such limit state and σ_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:

$$G = 1 - \left[\frac{P_A}{P_R} + \frac{M_A}{M_R}\right] \tag{3.2}$$

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 effects 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:

$$\sigma_G \approx \sqrt{\sum_{i=1}^{4}\left(\frac{\partial G}{\partial X_i}\right)^2 \sigma_{x_i}{}^2} \tag{3.3}$$

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:

$$\sigma_G \approx \sqrt{\frac{\sigma_{P_A}{}^2}{\overline{P}_R{}^2} + \left(\frac{\overline{P}_A}{\overline{P}_R{}^2}\right)^2 \cdot \sigma_{P_R}{}^2 + \frac{\sigma_{M_A}{}^2}{\overline{M}_R{}^2} + \left(\frac{\overline{M}_A}{\overline{M}_R{}^2}\right)^2 \cdot \sigma_{M_R}{}^2} \tag{3.4}$$

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.

4. APPLICATION TO SELECTED BRIDGE

The structure is a vehicles bridge built on the Benito Juarez International airport area, in the transition seismic zone III, in order to improve the traffic conditions. The bridge has a 400 m total span 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 images of the main structural members are shown in Figs. 2 and 3.
Essentially, the main structural components of the bridge are: the transverse cap, two piers, the footing and the piles. Fig. 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.

Fig. 2 Main supports of the bridge

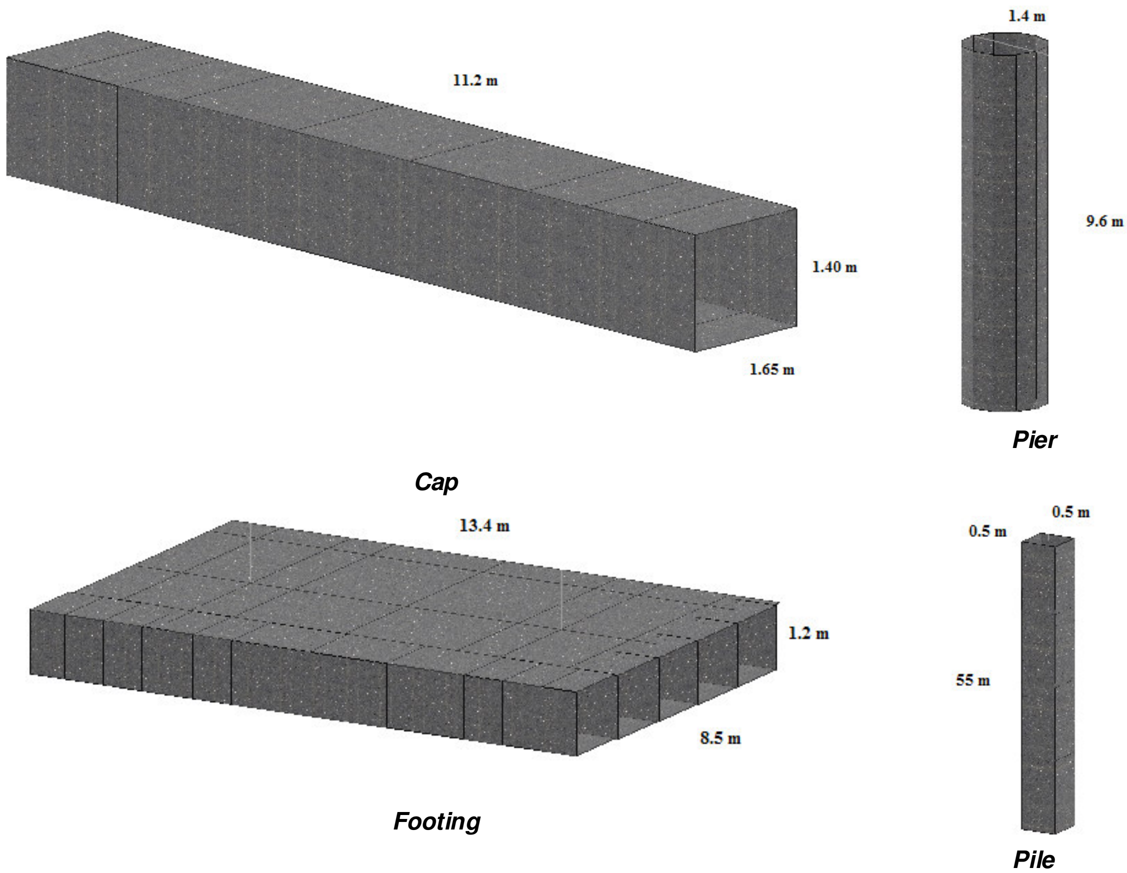

Fig. 3 Main bridge components

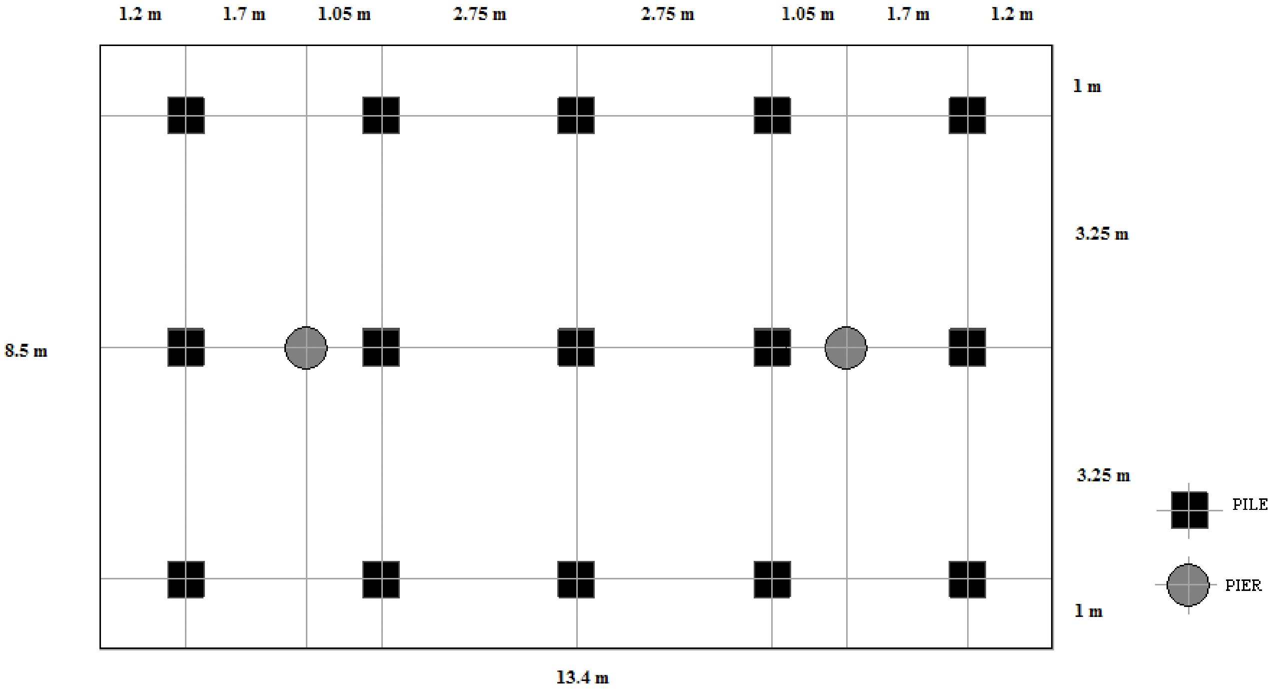

Fig. 4 Main bridge components

4.1 BRIDGE RELIABILITY

A family of bridge designs were obtained (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 4.1 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. The Table contains the seismic coefficient, the rebars size, number of rebars, mean values of maximum axial load and moment and axial and moment resistances, reliability index β and the initial costs obtained.

Table 4.1 Sample of the calculations for cost-reliability curve

c/g	*Rebar size ("/8)*	*No Rebars*	P_A *(T)*	P_R *(T)*	M_A *(T*m)*	M_R *(T*m)*	β	C_i *(10^3 USD)*
0	22	11	228.97	817.77	529.78	541.29	3.28	6338.4
0.15	26	11	227.4	947.43	575.51	618.62	3.68	6918.6
0.3	28	11	225.82	1026.46	621.24	656.23	3.44	7193.6
0.45	30	11	225.82	1075.38	666.97	689.22	3.37	7468.7
0.6	32	11	222.67	1171.87	712.7	722.25	3.33	7743.7

Five alternative designs and the five maximum intensities shown in Table 4.1, were considered and the corresponding reliability indices and initial costs were calculated. For the standard deviations, it was used $CV_A = 0.25$ and $CV_R = 0.1$ and the following simplifications were made (De León et al, 2007):

$$CV_A = \frac{\bar{P}_A}{\sigma_{P_A}} = \frac{\bar{M}_A}{\sigma_{M_A}} \tag{4.1}$$

$$CV_R = \frac{\bar{P}_R}{\sigma_{P_R}} = \frac{\bar{M}_R}{\sigma_{M_R}} \tag{4.2}$$

All the curves in the family shown in Fig. 6 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 Fig. 5.

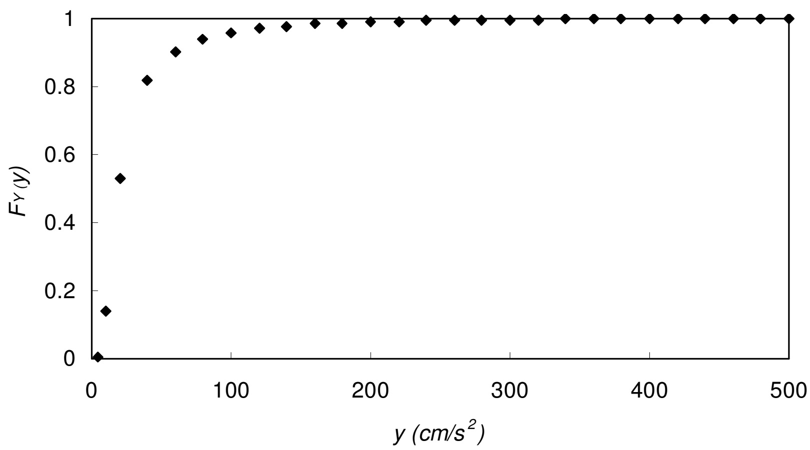

Fig. 5 Annual cumulative probability of seismic intensities in Mexico City

The unconditional curve is shown in Fig.6.

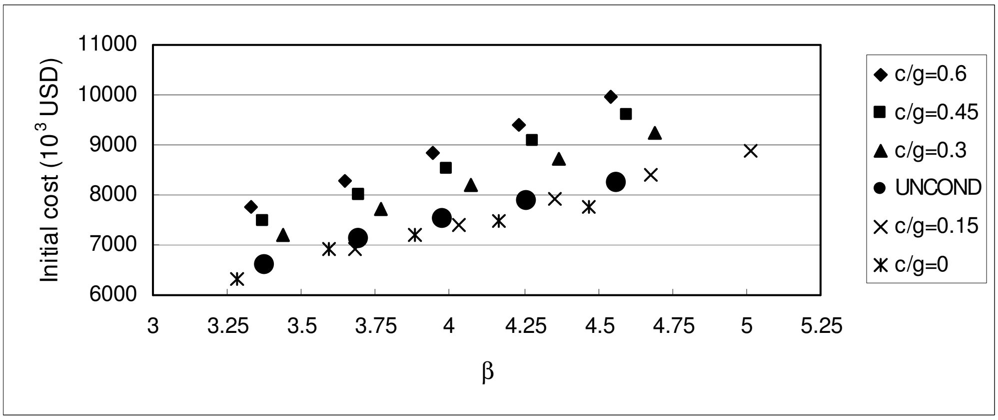

Fig. 6 Family of conditionals and unconditional initial cost curves for a bridge on the zone III, Mexico City

4.2EXPECTED FAIURE COST AND EXPECTED LIFE-CYCLE COST CURVES

By considering that the damage/failure cost is 800 times the initial cost, the expected failure cost and expected life-cycle cost are calculated. The results are shown in Fig. 7.

5 DISCUSSION

The actual design, the one at the middle of the 5 alternative designs, has a reliability index of 3.98 which is slightly over the optimal of 3.93, according to Figs. 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.

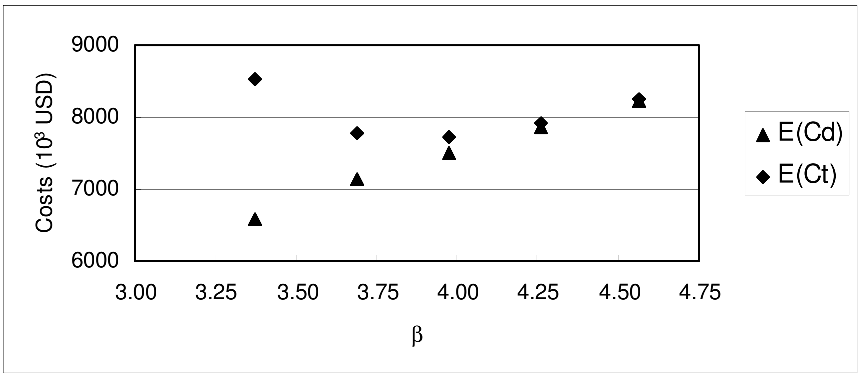

Fig. 7 Expected cost of failure and expected life-cycle cost

The high value of the optimal reliability index is due to the very high failure consequences for the bridge, located on a heavily populated area with an almost permanent strong 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.

6 CONCLUSIONS

Some risk and reliability calculations have been performed for a typical reinforced concrete vehicles bridge in Mexico City. Because of the heavy traffic and the large human lives at risk, the cost of consequences is very large. The bridge may be classified as one with very important failure consequences. The optimal reliability index is, therefore. 3.9. It was found that the bridge has a reliability index slightly over the optimal one.
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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