Rev. Téc. Ing. Univ. Zulia. Vol. 36, Nº 2, 114 - 121, 2013

Reliability-based optimal next inspection time of prestressed concrete bridges including the effect of corrosion deterioration

David De León, Sergio Díaz Camacho, Carlos Alberto González Pérez

Facultad de Ingeniería, Universidad Autónoma del Estado de México.
Coatepec, Toluca, Estado de México, México. Phone: 52(722)2140534 Ext. 1002
daviddeleonescobedo@yahoo.com.mx

Abstract

As the traffic demands grow constantly, and some vehicle bridges deteriorate due to corrosion, bridge agencies require non expensive procedures to support decisions on cost-effective maintenance schedules. In this paper a reliability-based formulation is proposed for the prediction of the next inspection time including the corrosion deterioration and the epistemic uncertainty on the corrosion initiation time. For the identification of the bridge integrity state, where little or no follow up has been previously made, the prediction contains a great deal of epistemic uncertainty. The impact of this uncertainty on the bridge failure probability is appraised. The bridge reliability is calculated by comparing the bridge maximum moment with the bridge capacity which depends on the corrosion on the bridge beams. The bridge failure probability is compared with the acceptable failure probability which is used as the safety threshold and it is calculated from the minimization of the annual expected life-cycle cost. Finally, epistemic uncertainty is introduced in the corrosion initiation time and the optimal next inspection time is obtained.

Keywords: bridge reliability, optimal inspection, corrosion deterioration, expected life-cycle cost.

Próximo tiempo de inspección en puentes de concreto presforzado basado en confiabilidad incluyendo el efecto de deterioro por corrosión

Resumen

A medida que las demandas de tráfico crecen constantemente, y que algunos puentes vehiculares se deterioran debido a la corrosión, las agencias de puentes requieren de procedimientos no tan caros para respaldar decisiones sobre programas de mantenimiento costo-efectivos. En este artículo se propone una formulación basada en confiabilidad para la predicción del próximo tiempo de inspección incluyendo el deterioro por corrosión y la incertidumbre epistémica en el tiempo de inicio de la corrosión. Para la identificación del estado de integridad del puente, en donde se ha dado poco o ningún seguimiento, la predicción lleva una gran cantidad de incertidumbre epistémica. Se evalúa el impacto de esta incertidumbre en la probabilidad de falla del puente. La confiabilidad del puente se calcula comparando el momento máximo en el puente con su capacidad, el cual depende de la corrosión en las vigas del mismo. La probabilidad de falla del puente se compara con su valor aceptable, que se usa como umbral de seguridad y que se calcula de la minimización del costo total esperado en el ciclo de vida. Finalmente, la incertidumbre epistémica se introduce en el tiempo de inicio de la corrosión y se obtiene el próximo tiempo de inspección.

Palabras clave: confiabilidad del puente, inspección óptima, deterioro por corrosión, costo esperado en el ciclo de vida.

Introduction

Limit state functions for engineering structures are usually formulated in terms of two opposite concepts: cost and safety. Bridges are no exception and its structural safety depends on uncertain conditions of loading and deterioration. When the bridges have a significant operating time and when the traffic loads are being continuously increased, these uncertainties introduce significant concerns on the bridge safety conditions.

Bridge agencies and operators require, therefore, reliable procedures to schedule maintenance actions for a cost-effective national or regional protection program.

In order to consider these uncertainties within a probabilistic framework, the annual structural reliability is taken as the proper safety measure and the target reliability is used as the target or allowable value.

Several studies have been previously performed to formulate bridge optimal inspection schedules by Frangopol and other authors [1 to 3]. However, the epistemic uncertainty on the bridge damage state has not yet been separated. One of the advantages of treating it separately from the aleatory one is that, by doing so, it is possible to objectively propose bounds for the economical convenience to reduce it by improving the collection of information (quality and quantity). Another one is that conservative decisions may be formulated for risk-aversive owners or managers as they may prefer to handle percentile values, say 90 or 95, of the reliability index instead of just its mean value.

Optimal inspection times may be proposed by comparing the bridge actual reliability with its acceptable value in terms of the failure probability.

The acceptable failure probability is calculated for bridges in terms of the cost of failure consequences for a minimum expected life-cycle cost.

Similarly to what is being done to assess the fatigue life of tubular joints in offshore marine platforms [4], a time variation of the bridge reliability index is obtained through the calculation of the bridge reliability under varied scenarios of corrosion attack.

In this paper, a reliability formulation based on the calculation of maximum acting and cracking moment capacity in the reinforced concrete beams that constitute bridge structural system is proposed. The cracking moment is due to the evolution of the bridge corrosion over time [5-7]. And the estimations of the cracking moment capacity are obtained from annual variations on the beams corrosion evolution on 15 consecutive years.

Corrosion is a major damaging factor for reinforced or pre-stressed concrete bridges as studied by several researchers [8 to 12]. One of the model parameters with most of the epistemic uncertainty is the initiation corrosion time. In this paper, epistemic uncertainty is considered for the bridge performance and its impact on the evolution of bridge reliability is estimated.

Acceptable failure probability

According to previous studies [13-14], the expected life-cycle cost, $E[C_L]$, includes the structure initial cost, C_i, and the *expected cost of the consequences* to exceed the limit state within the *nominal structure operating life*, $E[C_F]$:

$$E[C_L] = C_i + E[C_F] \qquad (1)$$

where:

$$C_i = C_1 - C_2 \ln(P_f) \qquad (2)$$

C_1 and C_2 are constants which depend on the structural type and P_f is the probability to exceed the limit state. However, the conventional cost-benefit model considers a no bridge replacement within the bridge lifetime, in case of the bridge damage. The conventional formulation is modified to distinguish the temporary economic loss from the permanent economic loss of the case without replacement. The clarification of the replacement consideration is important because the benefits, from the bridge economic operation, are not lost in the event of bridge failure, but just deferred after its replacement. Therefore, the only loss, from the revenues viewpoint, is the one from the deferral and money value which changes with time. Actually, for typical values of

P_f about 0.003, the assumption of single replacement is adequate for practical purposes [15].

If the failure consequences are: the cost of bridge substitution and contents C_B, the losses related to human lives C_H, the user costs C_U and the loss of income due to deferral of the revenues R after the bridge replacement:

$$E[C_F] = PVF[C_B + C_H + C_U]P_f + R * PVF' * P_f \quad (3)$$

where C_B, C_H and C_U are calculated from standard procedures, *PVF* is the present worth factor required to update future costs to present value:

$$PVF = [1 - e^{-rT}] / r \quad (4)$$

$$PVF = [PVF - T * e^{-rT}] * [1 - e^{-r\Delta T}] / r \quad (5)$$

where R is the annual revenues, r is the net annual discount rate, T the bridge nominal operating life and ΔT the bridge reconstruction time.

From the minimization of the expected life-cycle cost,

$$\partial E[C_L] / \partial P_f = 0, \quad (6)$$

the acceptable annual bridge failure probability is obtained through:

$$P_f = C_2 / \left[(C_B + C_H + C_U) * PVF + R * PVF'\right] \quad (7)$$

The curve, for several costs of consequences, is shown in Figure 1.

On the other hand, the calculation of the probability to exceed the critical limit state in the particular bridge considered here is performed in the next section in terms of the corrosion evolution and beam cracking moment capacity.

Cracking moment capacity without epistemic uncertainty

According to Tuutti [12], the structure lifetime (T_{VU}) is given by the corrosion initiation time (T_1) plus the propagation time (T_2):

$$T_{vu} = T_1 + T_2 \quad (8)$$

The T_{vu} value is fixed by the requirements or necessities of each infrastructure facility (a common target service life for vehicular bridges is about 50 years [16]); therefore, this value is known and T_1 can be calculated as follows:

$$T_1 = T_{vu} - T_2 \quad (9)$$

From Torres and Martinez [17]:

$$T_2 = \frac{X_{CRIT}}{i_{CORR}} \quad (10)$$

where:

X_{CRIT}: corrosion size needed to crack the concrete cover, in mm, calculated from [11]

$$X_{CRIT} \approx 0.011\left(\frac{C}{\phi}\right)\left(\frac{C}{L}+1\right)^{1.95} \quad (11)$$

where

C : concrete cover, in mm.

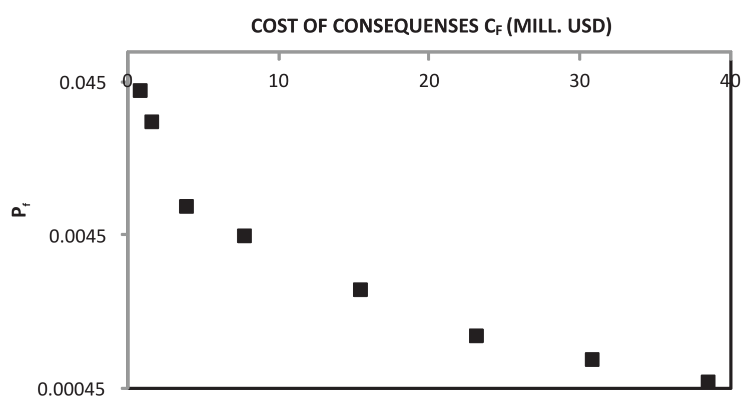

Figure 1. Acceptable annual failure probability for several consequences costs.

ϕ : steel rebars diameter, in mm.

L : length of corroded bar, in mm.

i_{CORR}: average corrosion rate, in mm/year, based on RILEM [7]

$$i_{CORR} = C_T k_0 i_0 \tag{12}$$

where,

i_0 : corrosion velocity estimated for 20°C, in μ/year.

k_0 : coefficient that considers the concrete water/cement ratio (a/c).

C_T: coefficient that considers the temperature effect.

From [17], a parabolic function is approximated to the *erf* to determine the maximum effective coefficient of chloride diffusion (D_{EFmax}), in cm^2/s:

$$D_{EF_{\max}} = \frac{1}{12T_1}\left[\frac{C}{1-\sqrt{C_{crit}/C_s}}\right]^2 \tag{13}$$

where,

T_1 : corrosion initiation time expressed in years

C_{crit} : critical chloride concentration at the depth of prestressed steel as a percent of the cement weight.

C_s : chloride concentration at the surface, as a percent of the cement weight.

And the effective coefficient of chlorine diffusion is:

$$D_{EF} = \frac{0.68(a/c)^{0.73}}{(C_f)^{2.8}(t)^{0.4}(1+f_a)^{2.6}} \tag{14}$$

where,

D_{EF} : effective coefficient of chlorine diffusion, in cm^2/s

a/c : water/cement ratio.

C_f : cementing material, in kg/m^3

f_a : amount of fly ash, in the case of puzolanic cement, as a percent of the cement weight

t : service time of the structure, in years.

Once the value of T_1 was estimated from eq. (9), the variation of the annual reliability index (β) was calculated after corrosion beginning, taking into consideration the lost of pre-stressed steel cross section area due to pitting corrosion as described below.

The steel reduction area due to the bridge corrosion has been previously proposed [18]:

$$A_r(T) = \begin{cases} \dfrac{pD_0^2}{4} - A_1 - A_2, & p(T) \le \dfrac{\sqrt{2}}{2}D_0 \\ A_1 - A_2, & \dfrac{\sqrt{2}}{2}D_0 < p(T) \le D_0 \\ 0, & p(T) > D_0 \end{cases} \tag{15}$$

where:

$$A_1 = \frac{1}{2}\left[\theta_1\left(\frac{D_0}{2}\right)^2 - \alpha\left[\frac{D_0}{2} - \frac{p(T)^2}{D_0}\right]\right] \tag{16}$$

$$A_2 = \frac{1}{2}\left[\theta_2 p(T)^2 - \alpha\frac{p(T)^2}{D_0}\right] \tag{17}$$

$$\alpha = 2p(T)\sqrt{1-\left(\frac{p(T)}{D_0}\right)^2} \tag{18}$$

$$\theta_1 = 2\operatorname{arcsen}\left(\frac{\alpha}{D_0}\right) \tag{19}$$

$$\theta_2 = 2\operatorname{arcsen}\left(\frac{\alpha}{2p(T)}\right) \tag{20}$$

$$p(T) = 0.0116(T - T_1)i_{CORR}P_{\max}/P_{av} \tag{21}$$

where:

$A_r(T)$: net cross section area of a steel bar corroded at $T>T_1$, in cm^2 (see Fig. 2)

D_0 : initial diameter of steel bar, in cm

P_{max}/P_{av}: maximum versus average penetration ratio.

For the reduction of tension steel due to the prestressing of the strands at time $T > T_1$, an empirical model was used to determine the behavior of tension stress as a function of time as proposed by Carrion *et al.* [9]:

$$\sigma(T) = \sigma_0(1-bT)^m \tag{22}$$

where,

σ : tension stress in a given time, en kg/cm^2

σ_0 : 14 250 kg/cm^2 initial tension stress.

T : time in which reducing of σ is measured after the onset of corrosion in years.

b : 0.01538

m : 0.41 constants of material influenced by environmental conditions (for a corrosive solution of NaCl).

According to standard procedures [5-7], the cracking moment capacity is calculated and the statistical properties of random variables are shown in Table 1.

Finally, the bridge is modeled as a single component, being the beam under corrosion the main element, the limit state is described by the event where the beam cracking moment capacity M_{CR} is exceeded by the maximum acting moment M_{max}, note that the cracking moment M_{CR} is ruled by the pre-stress force, that is directly proportional to the steel surface [5-7][1];

$$M_{CR} = M_1 + M_2 \tag{23}$$

where $M_1 = M_{sw} + M_{slab}$; M_{sw} is the moment due to the girder selfweight and M_{slab} is the moment due to the road concrete slab.

$$M_2 = \left[2\sqrt{f'c} - \frac{M_1}{S_{is}} + \frac{(P_f)(e)}{S_{is}} + \frac{P_f}{A}\right] S_{ic} \tag{24}$$

where $f'c$, is the compressive strength of concrete, S_{is}, S_{ic}, A, and e are geometric properties of the girder cross section and P_f is the total pre-stressed force that depends of the pre-stressed steel transversal section area as pointed out in reference [6].

The M_{CR} is also associated with the presence of fissures and the reduction of the bending capacity; therefore, the bridge annual failure probability is expressed as:

$$P_f = P(M_{\max} \geq M_{CR}) \tag{25}$$

where;

M_{max}: maximum acting moment, calculated for a simple supported beam by the influence lines method for beams [19], using a vehicle T3-S2-R4 as a design live load [19] (see Table 1).

M_{CR}: beam cracking moment, calculated by standard procedures (eq. 23) in accordance with recommendations of ACI [5], ANIPPAC [6] and RILEM [7].

Epistemic uncertainty on corrosion initiation time for bridge damage condition

Corrosion initiation time (T_1) is a very difficult to predict variable and its relevance for the bridge capacity is crucial. One way to deal with this modeling uncertainty is introducing a correction factor, to account for epistemic uncertainty, on the available expressions to predict this time. In this paper, and in a simplified way, a random factor is introduced to modify the corro-

Table 1
Mean (μ) and coefficient of variation (V_D) of random variables assuming normal distribution

Random Variable		μ	V_D
Concrete resistance	f'c (kg/cm^2)	350	0.180
Prestressed steel area	Ap (cm^2)	67.32	0.016
Live load from design vehicle T3-S2-R4[20]	FR (tn)	71.15	0.319

1 Since the estimation of M_C implies a lot of calculations where the target value is related with very specific geometrical properties of girder cross section, area of pre-stressed steel, etc., the authors of this research work encourage the readers to review references 5-7 for a better understanding of the calculation of M_C.

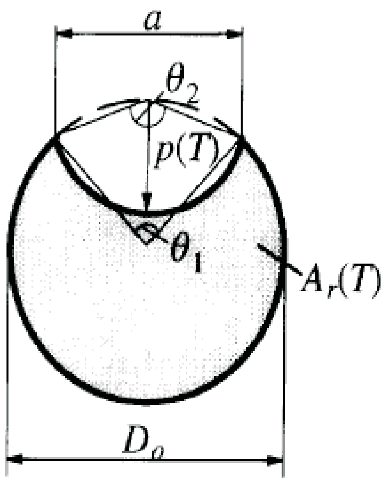

Figure 2. Pit corrosion configuration [16].

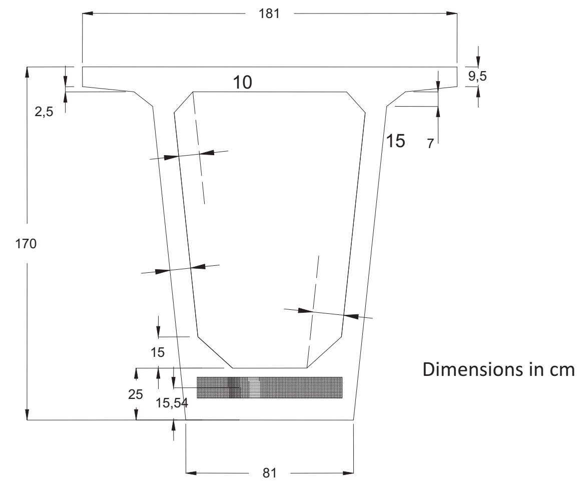

Figure 3. Cross section of bridge girder.

sion initiation time T_1. This factor is assumed to be lognormal with median 1 and proposed coefficient of variation $CV^e{}_{T1}$

$$T_1^{ep} = LN(1, CV_{T1}^E) * T_1 \tag{26}$$

A series of 10,000 corrosion initiation times $T_1{}^{ep}$ is generated by using $CV^e{}_{T1}$ =0.3, and a series of annual failure probabilities and annual reliability indices are calculated for each year after corrosion initiation. Numerical distributions for M_{max} and M_c were built for every year after the corrosion initiation time, and its failure probability is calculated. A Monte Carlo simulation process is used for that purpose.

Application to La Joya bridge in Mexico

La Joya bridge is pre-stressed concrete vehicular bridge located in Mexico State, close to Mexico City. Its main span is 30 m and the structural type consists on box-shaped prestressed concrete girders (Figure 3) supported by reinforced concrete piers that rest on reinforced concrete piles. The bridge reconstruction time is ΔT = 2 years.

If the consequences cost is 7.7 US million Dollar, C_2 = 0.7 US million Dollar, for the specific bridge type considered, r = 0.08, for Mexico and T = 50 years, the acceptable annual failure probability is 0.0027 and its corresponding annual reliability index is 2.78. By applying the above described procedure, the results are shown in Figures 4 and 5.

Discussion

After calculating $T_1{}^{ep}$, the variation of the pre-stressed cross section area and the value of the reliability index were calculated annually for 15 years, each line showed in Figures 4 and 5 represents a realization conducted (for simplicity only 200 experiments are showed in each Figure). The construction of both Figures, involves the effect of the corrosion process; when time after initiation is nearest to T_1, the pre-stressed cross section area is nearer to its original value; therefore, the largest the time after T_1, the lowest the cross section area and the reliability index.

The bridge failure probability evolves as the corrosion develops in the girder and, as the bridge is under heavy and frequent vehicles loading, the tension stresses trend to accelerate the cracking process in the girder. The evolution of the bridge reliability index, as observed in Figure 4, may be used to establish a time to visually inspect, at least, the bridge condition. The acceptable value of the annual reliability of 2.78 is reached by the bridge reliability at the time range between 5.5 and 10.5 years, after corrosion initiation, from Figure 5, for the coefficient of variation of corrosion initiation time, of 0.3. Being conservative, one may select to inspect the bridge at the time of 5.5 years. However, other option may be to select the time corresponding to the percentiles 5 or 10, which would lead to the

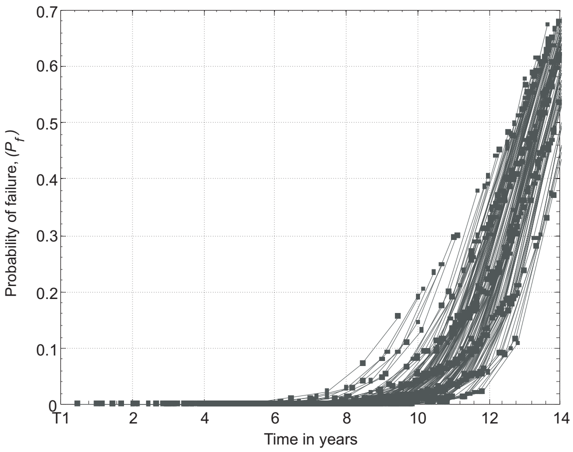

Figure 4. Annual failure probability for epistemic uncertainty = 0.3.

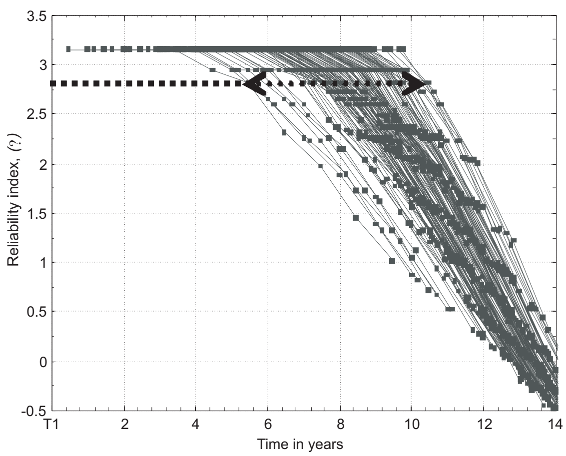

Figure 5. Annual reliability index for epistemic uncertainty = 0.3.

inspection time of 6 or 8 years, respectively, depending on the manager conservative degree or the bridge importance in the national road network.

Cost/benefit studies may be used to weight the cost of narrowing this range against the benefit of making more precise predictions of the bridge inspection time.

The bridge importance will move the acceptable reliability index to higher or lower thresholds. Also, the corrosiveness of the location environment, and stresses demand on the bridge, may produce that the girder deteriorates faster and the recommended time to inspect and potentially maintain the bridge may be shorter.

Improvements and upgrading works to the bridge are not considered here.

The bridge age may play an important role as the corrosion may be initiated faster than for other newer bridges. Further studies, with other spans, ages, and importance level, may help to produce guidelines for bridge maintenance, where corrosion is the major damaging factor, in Mexico.

Conclusions

A procedure to calculate the optimal time for next inspection on bridges has been proposed based on yearly bridge corrosion which reduces its bending capacity.

The optimal schedule considers the acceptable bridge reliability which is calculated as a function of the failure losses, which are a measure of the bridge importance.

The procedure accounts for epistemic uncertainty on the corrosion initiation time and this produces a range of potential times to inspect, which allows for conservative decisions by picking the minimum time.

For the example shown here, the next inspection time should be 5.5 years after the corrosion initiation time, in a conservative scheme.

References

1. Frangopol, D., Furuta, H., Kameda, T., Ankara, K. and Takahashi, Y. “Optimal Bridge Maintenance Planning using Improved Multi-Objective Genetic Algorithm”, Structure and Infrastructure Engineering, Ed. Taylor & Francis, Vol. 2, N° 1, (2006), 33-41.
2. Frangopol, D. and Biondini, F. “Collapse Reliability and Lifetime Prediction of Reinforced Concrete Framed Structures,” Structure and Infrastructure Engineering, Ed. Taylor & Francis, Vol. 4 (2008).
3. Petcherdchoo, Aruz; Neves, Luis A. C. and Frangopol Dan M. “Optimizing Lifetime Condition and Reliability of Deteriorating Structures with Emphasis on Bridges” J. Struct.

Engrg, ASCE., Volume 134, Issue 4, (2008), 544-552.

4. Silva F. L. and Heredia E. “Effect of Uncertainties on the Reliability of Fatigue Damaged Systems”. 23th International Conference on Offshore Mechanics and Artic Engineering, Vancouver, Canadá, 2004.
5. American Concrete Institute, “Code Requirements for structural concrete (ACI 318S-05) and Comments. From Committee 318, 2005.
6. ANIPPAC and Institute of Engineering UNAM, “Manual de Diseño de Estructuras Prefabricadas y Presforzadas”. (In Spanish). 1st Edition. Betancout Ribotta, R., Reinoso Angulo, E. and Rodriguez, M. E. Editores, México, D.F., 2000.
7. RILEM Report 14, “Durability design of concrete structures”, Editors. Sarja A: and Vesikari E., London, UK: E & FN SPON, 1996.
8. Bamforth, P., “Chloride penetration and service life”, presented in Corrosion and protection of metals in contact with concrete - COST-509 Workshop, Sevilla, Spain., 1996.
9. Carrion Viramontes, F. J., Hernandez Rivera, J. and Acosta Esqueda, M. A., “Estudios de Corrosión en Puentes de Concreto Pre-esforzado. Cables de Presfuerzo”, (In Spanish) Secretary of Communications and Transportation. Mexican Institute Transportation. Technical Publication 122, Sanfandila, Qro., 1999, 79 pp.
10. Collins M., Mitchell, D. “Prestressed concrete structures”. First Edition. Prentice Hall, 1991.
11. Torres Acosta, A. A. “Cracking induced by localized corrosion of reinforcement in chloride contaminated concrete”, Ph D dissertation, University of South Florida, USA, 1999.
12. Tuutti, K., “Corrosion of steel in concrete”, Stockholm, Sweden: Swedish Cement and Concrete Research Institute, 1982.
13. Ang, Alfredo H-S. and De Leon, D., (2005), “Modeling and analysis of uncertainties for risk-informed decisions in infrastructures engineering”, Journal of Structure and Infrastructure Engineering, Vol. 1, Nº 1, (2005) 19-31.
14. De Leon, D. “Integrating socio-economics in the development of criteria for the aseismic design of reinforced concrete structures”. Ph. D. Thesis, Department of Civil and Environmental Engineering, UCI, Irvine, California, 1996.
15. Stahl, Bernhard, Edited by McClelland, B. and Reifel, M. D. “Reliability Engineering and Risk Analysis”, Chapter 5 from Planning and Design of Fixed Offshore Platforms. Van Nostrand Reinhold Co. New York, 1986.
16. Marsh, P. S., and Frangopol D. M. “Reinforced concrete bridge deck reliability model incorporating temporal and spatial variations of probabilistic corrosion rate sensor data”. Reliability Engineering and Systems Safety, Vol. 93, (2008) 394-409.
17. Torres Acosta, A. A. and Martinez Madrid, M. “Diseño de Estructuras de Concreto con Criterios de Durabilidad”, (In Spanish) Secretary of Communications and Transportation. Mexican Institute Transportation. Technical Publication 122, Sanfandila, Qro., 2001, 87pp.
18. Kràlik, J. and Varga, T. “Probability Analysis of Reinforced Concrete Structure Considering Nonlinear Behavior and Degradation Effects”. Modelováni V Mechanice. Ostrava, Únor, 2005.
19. Hibbeler, R.C. “Structural Analysis”, 3rd edition, Prentice Hall, Upper Saddle River, New Jersey, USA, 1997.
20. Rascón-Chávez, O.A., Barouse-Moreno, M. and Ventura-Suárez, G. Secretaría de Comunicaciones y Transportes. “Análisis Normativo y Estadístico de Cargas Vivas en Puentes en México”. Instituto Mexicano del Transporte. Publicación Técnica Sanfandila, Qro. Nº 97, (1997) 152.

Recibido el 26 de Enero de 2012

En forma revisada el 27 de Mayo de 2013