Reliability based lifetime maintenance of aging highway bridges

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ABSTRACT

As the nation's infrastructure continues to age, the cost of maintaining it at an acceptable safety level continues to increase. In the United States, about one of every three bridges is rated structurally deficient and/or functionally obsolete. It will require about $80 billion to eliminate the current backlog of bridge deficiencies and maintain repair levels[1]. Unfortunately, the financial resources allocated for these activities fall extremely short of the demand. Although several existing and emerging NDT techniques are available to gather inspection data, current maintenance planning decisions for deficient bridges are based on data from subjective condition assessments and do not consider the reliability of bridge components and systems. Recently, reliability-based optimum maintenance planning strategies have been developed. They can be used to predict inspection and repair times to achieve minimum life-cycle cost of deteriorating structural systems. In this study, a reliability-based methodology which takes into account loading randomness and history, and randomness in strength and degradation resulting from aggressive environmental factors, is used to predict the time-dependent reliability of aging highway bridges. A methodology for incorporating inspection data into reliability predictions is also presented. Finally, optimal lifetime maintenance strategies are identified, in which optimal inspection/repair times are found based on minimum expected life-cycle cost under prescribed reliability constraints. The influence of discount rate on optimum solutions is evaluated.

Keywords: Bridge Deterioration; Bridge Maintenance; Inspection; Life-Cycle Cost; Optimization; System Reliability.

1. INTRODUCTION

The application of structural reliability methods to bridge maintenance planning decisions is becoming increasingly popular in the US. Although the federally mandated biennial bridge inspection program centers on visual condition states, it is becoming increasingly recognized that maintenance needs should focus on safety and serviceability[2]. Many state transportation departments are becoming aware of this new focus. In fact, the New York State Department of Transportation now performs safety assessments which measure risk associated with potential bridge failure modes[3]. When structural reliability methods are applied to design it leads to structures that have a more consistent level of risk[4].

It is well known that the US infrastructure is in need of repair. Available funds are insufficient to maintain the existing infrastructure[5]. About one of every three bridges is rated structurally deficient and/or functionally obsolete. It will require about $80 billion to eliminate the current backlog of bridge deficiencies and maintain repair levels[1]. Although some bridges are functionally obsolete and must be replaced, it is usually more cost effective to extend bridge life as long as possible[6]. Life-cycle cost methods can be used to determine the timing of maintenance activities to prolong bridge life at the lowest possible cost[7-10].

The primary purpose of structural codes and standards is to manage and control risk to socially acceptable values[11]. Codes are well established for the design of new bridges[12], but do not apply to deteriorating structures. Reliability-based life-cycle cost analysis methods can be used to identify minimum cost maintenance strategies that satisfy risk constraints. Also, the cost savings associated with probabilistic/reliability assessment may be significant. For example, Enevoldsen[13] recently reported that the use of probabilistic methods in the assessment of the Vislund bridge (Denmark) saved over US$3.3 million compared to the deterministic approach.

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In this study, a reliability-based methodology which takes into account loading randomness and history, and randomness in strength and degradation resulting from aggressive environmental factors, is used to predict the time-dependent reliability of aging highway bridges. Load occurrence rate and distribution effects are both taken into consideration. A methodology for incorporating inspection data into reliability predictions is also presented. Finally, optimal lifetime maintenance strategies are identified, in which optimal inspection/repair times are found based on minimum expected life-cycle cost under prescribed reliability constraints. The influence of discount rate on optimum solutions is evaluated.

2. TIME-DEPENDENT STRENGTH MODELING

2.1 Time-dependent random strength

In theory, strength degradation can be modeled as a stochastic process dependent on multiple undeterministic variables. In practice, however, the remaining capacity of bridges is often predicted using deterministic strength degradation models. As shown in Fig.1, strength is generally random. To incorporate inspection results into time-dependent capacity predictions, probabilistic methods have to be used to account for inspection uncertainties. In addition, the initial strengths of steel reinforcement and concrete are random variables with significant variability which must also be considered in the analysis[14]. Other variables (e.g., dead load, geometry) may also have some contribution to the variability of the remaining capacity. To capture the uncertainties associated with resistance random variables and inspection results, a probabilistic approach must be used for remaining capacity predictions.

2.2 Corrosion in reinforced concrete bridges

Reinforced concrete (RC) bridges are often located in aggressive environments and subjected to a variety of strength degradation mechanisms (e.g., corrosion, sulfate attack, alkali-silica reaction, freeze-thaw cycles, among others). Corrosion is the most commonly reported degradation mechanism for RC bridge members [15-17]. Chloride ion ingress is the most commonly reported cause of corrosion for RC bridges[14]. As shown in Fig. 2, chloride-laden water from deicing salts leaks onto bridge members, and chloride ions enter the members via diffusion. When the concentration of chloride ions at the level

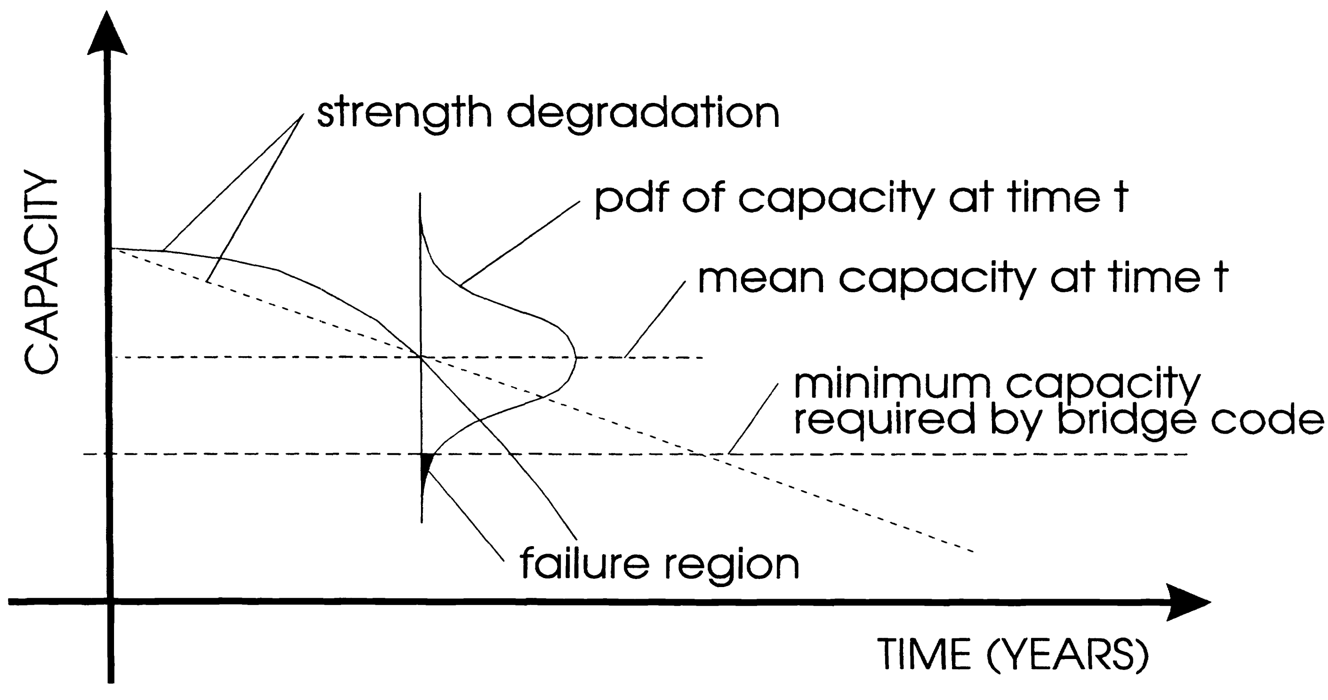


Fig. 1: Bridge Load Capacity Deterioration

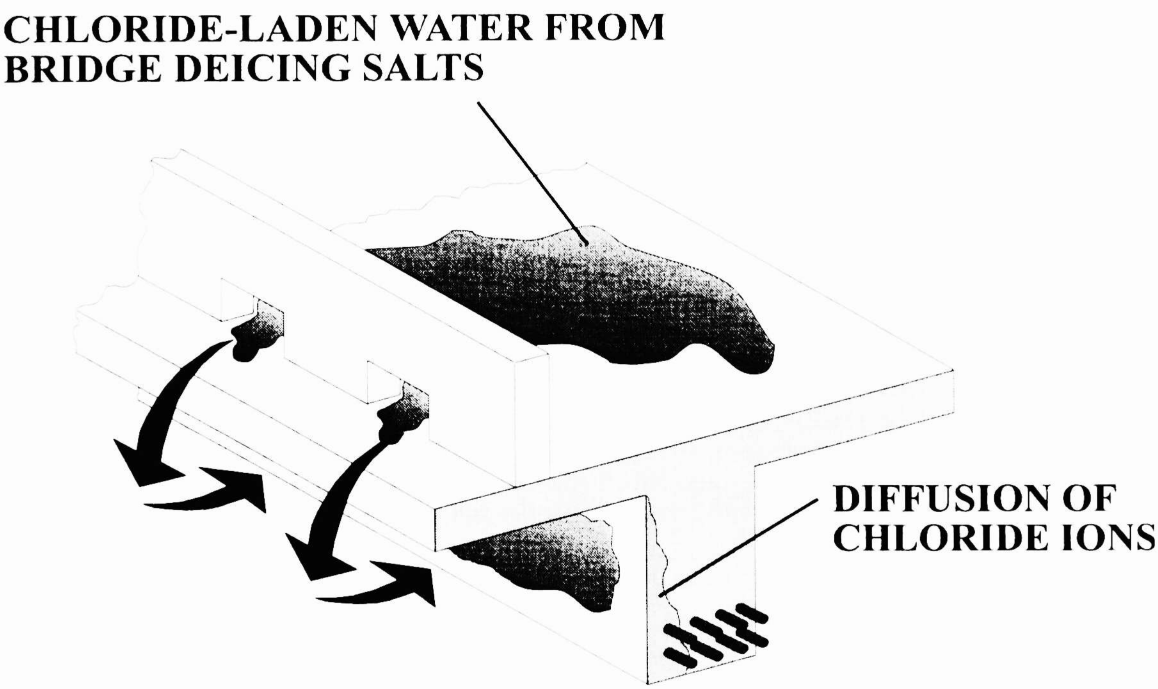


Fig. 2: Reinforcement Corrosion Due to Chloride Ions

of reinforcement reaches a critical value, corrosion begins.

In the last few decades, several methods have been developed to curtail reinforcement corrosion such as rebar coatings, hydrophobic concrete surface treatments, and corrosion inhibitors[18]. These methods can be used to delay the initiation time of corrosion, but generally do not decrease the corrosion rate.

2.3 Strength degradation function

Corrosion affects the performance of reinforced concrete members in two primary ways: (a) loss of steel section, and (b) deterioration of the steel-concrete bond[19]. Recent experimental results[20] indicate that the steel-concrete bond has much more influence on strength degradation of reinforced concrete beams as compared to steel section loss. The following strength degradation function has been proposed for reinforced concrete beams subjected to reinforcement corrosion based on experimental results[20]:

$$B \text{ percent} = \left\{1 - \sin^2\left(2.312\frac{T}{D} i \bullet ln(i)\right)\right\}100 \tag{1}$$

where B = percent of flexural strength of control beam, T = time (elapsed in years after corrosion initiation), D = initial diameter of reinforcing bar, and i = rate of corrosion.

In this study, the following strength degradation function $g(t)$ is used based on probabilistic analysis of reinforced concrete bridge beams under corrosion [21,22]:

$$g(t) = 1 - k_1 t + k_2 t^2 \tag{2}$$

where k_1 and k_2 are damage rate parameters, T_I = damage initiation time, $t > T_I$, and $0 \le g(t) \le 1$.

3. TIME-VARIANT RELIABILITY PREDICTIONS

The influences of strength degradation and loading randomness, history and distribution to members is illustrated for an existing reinforced concrete girder bridge (for additional information, see Enright[14] and Enright and Frangopol [21-24]).

3.1 Time-dependent maximum live moment

Stochastic live load is modeled as a Poisson point process with an initial normal distribution applied at each of six truck wheel groups as shown in Fig. 3. Maximum live moment for a critical girder is shown for each of three load conditions in Fig. 3. It is interesting to note that although load condition L3 has the largest initial mean moment, load condition L2 has the largest 75 year mean maximum moment. This illustrates the influence of load occurrence rate on the critical demand for deteriorating bridges.

3.2 Lifetime failure probability

Both loading and resistance are time-variant variables for bridges under aggressive conditions. Assume that resistance and loads are independent, and stochastic live load S_1 is modeled as a Poisson point process. In this case, the cumulative-time failure probability of a series system of *m* deteriorating members subjected to a live load process with intensity S_1 can be expressed as[25]:

$$P_f(t_L) = \underbrace{\int_0^\infty \cdots \int_0^\infty}_{m\text{-}fold} \left\{ 1 - \exp\left[-\lambda_{S_1} t_L \left\{ 1 - \frac{1}{t_L} \int_0^{t_L} F_{S_1}\left[\min_{i=1}^{m} \left(\frac{r_i \cdot g_i(t)}{c_i} \right) \right] dt \right\} \right] \right\} f_{\underline{R}_0}(\underline{r}) d\underline{r} \tag{3}$$

where $P_f(t_L)$ is the cumulative-time failure probability; S_1 is time-variant (live) load; λ_{S1} and F_{S1} are the mean load occurrence rate and the cumulative distribution function of S_1, respectively; $g_i(t)$ is the resistance degradation function for member *i* (i.e., fraction of initial strength of member *i* remaining at time *t*); c_i is the structural action coefficient for member *i*; and $f_{\underline{R}o}(\underline{r})$ is the joint probability density function of the initial strength of the members in the system.

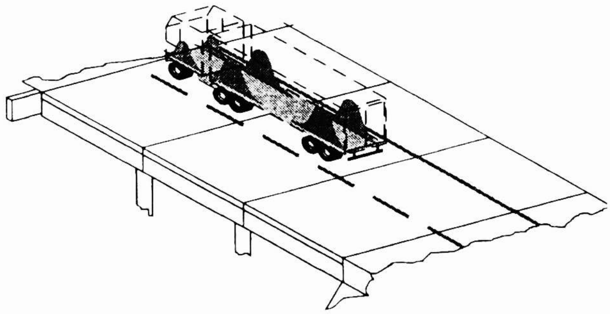
3.3 Influence of live load distribution to girders on lifetime failure probability predictions

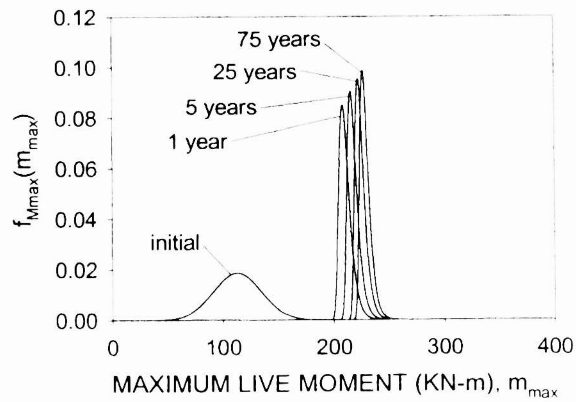
Lifetime failure probability predictions are shown in Fig. 4 for $g(t) = 1 - 0.005t$ and $E(T_I) = 5$ *years* for flexural failure of a critical girder, where $E(T_I)$ = mean corrosion initiation time. When AASHTO girder distribution factors[12] (GDF) are used to predict load distribution to girders, the failure probability of a critical (interior) girder is about 10^{-3} at t = 75 years. However, when finite element results are used to compute girder loads (Fig. 3), the maximum lifetime failure probability is about 10^{-8}. Consequently, the bridge load model and the type of structural analysis selected (i.e., simplified, finite element-based, nonlinear) can have a significant influence on maintenance planning decisions for deteriorating concrete bridges.

3.4 Influence of inspection on lifetime failure probability

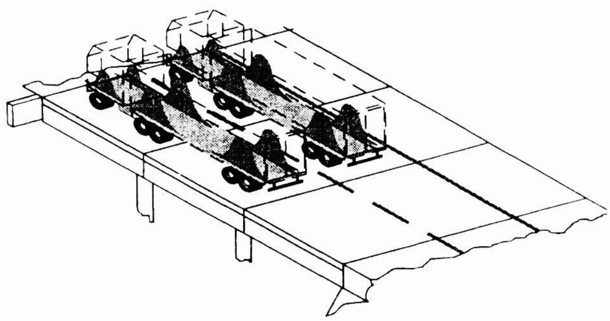
Inaccurate condition assessment has been identified by Aktan *et al.* [26] as the most critical technical barrier to the effective management of highway bridges. Suppose that the corrosion rate can be predicted for a deteriorating reinforced concrete bridge based on past performance for other bridges in similar environments. Furthermore, assume that corrosion rate inspection data are available for the existing bridge. Using Bayesian methods, an updated prediction for the corrosion rate can be obtained as follows[27]:

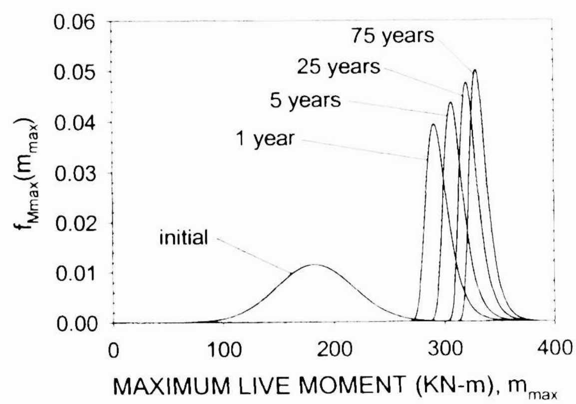
$$g(\underline{\theta} \mid \underline{x}) = \frac{f(\underline{x} \mid \underline{\theta}) \cdot g(\underline{\theta})}{\int f(\underline{x} \mid \underline{\theta}) g(\underline{\theta}) d\underline{\theta}} \tag{4}$$



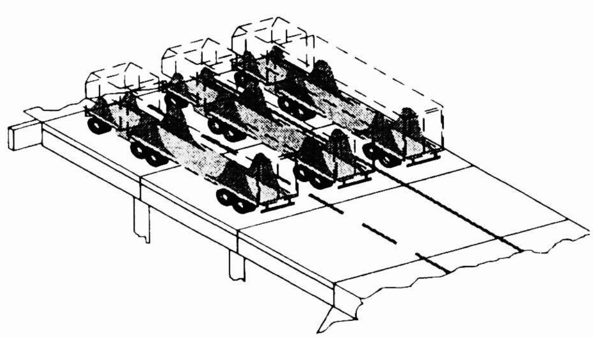


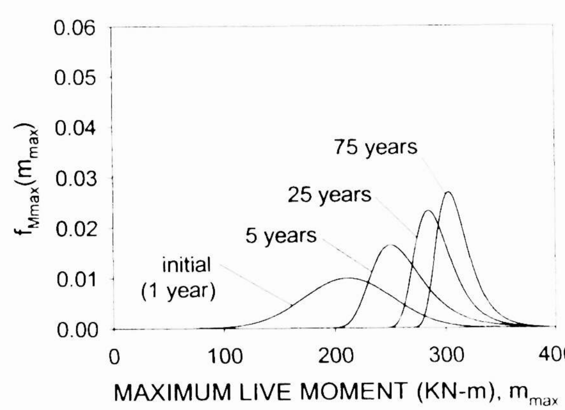
Load Condition L1
Single Truck in Lane 1
220,000 Occurrences Per Year





Load Condition L2
Trucks in Lanes 1 and 2
1,000 Occurrences Per Year





Load Condition L3
Trucks in Lanes 1 and 2 and Shoulder
One Occurrence Per Year

Fig. 3: Live Load Probability Densities Associated With Three Bridge Loading Conditions

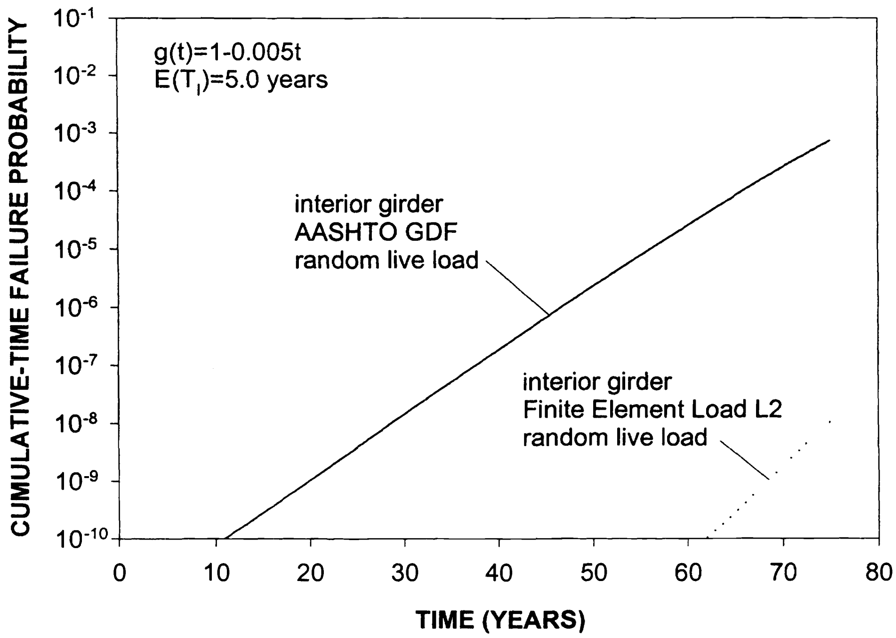


Fig. 4: Influence of Bridge Load Model and Type of Analysis on Lifetime Failure Probability

where $g(\underline{\theta}\,|\,\underline{x})$ = updated distribution, $f(\underline{x}\,|\,\underline{\theta})$ = conditional PDF of $\underline{X}$ given $\underline{\theta}$ (sampling distribution); $g(\underline{\theta})$ = PDF of $\underline{\Theta}$ (prior distribution); $g(\underline{\theta}\,|\,\underline{x})$ = posterior PDF of $\underline{\Theta}$ given $\underline{x}$ (posterior distribution); $\underline{\theta}$ = continuous parameter vector; and $\underline{x}$ = sample data.

Mean values for damage rate parameters k_1 and k_2 can be obtained using probabilistic strength degradation modeling (see Enright[14] and Enright and Frangopol [21] for details). An example of updated (posterior) values for corrosion rate and damage rate parameters based on inspection data and previous data (prior) is indicated in Table 1.

The influence of inspection updating on time-variant reliability for the shear failure mode is shown in Fig. 5 for critical interior girders, exterior girders, and a weakest-link system consisting of 5 girders. Reliability predictions for the system and the critical interior girder are very similar. This illustrates the dominant influence of a single girder on the reliability of the

Table 1. Inspection Updating Data and Influence on Mean Damage Parameters

Distribution	Corrosion Rate		Damage Rate Parameters	
	Mean (mm/yr)	COV	E (k_1)	E (k_2)
Prior (previous data)	0.15	0.30	0.017	0.00011
Inspection	0.10	0.40	0.012	0.00005
Posterior (updated distribution)	0.12	0.24	0.014	0.00007

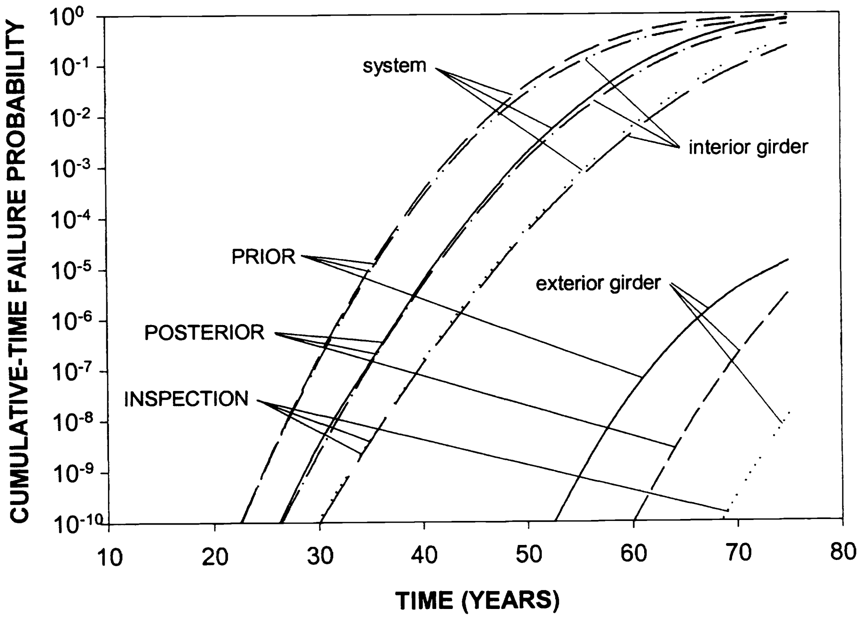


Fig. 5: Influence of Inspection on Bridge Girder Failure Probability

system. The effect of inspection updating on failure probability predictions can be significant, and is dependent on girder type (i.e., interior vs. exterior) and time. The role of non-destructive evaluation in time-dependent reliability analysis can be found in Zheng and Ellingwood[28].

4. LIFETIME MAINTENANCE OPTIMIZATION

Many methods are available for estimating the cost of maintenance and repair of existing structures, such as plant value, formula budgeting, life-cycle cost, and condition assessment, among others[29]. When life-cycle cost methods are used, the sensitivity of cost to the timing of maintenance activities can be computed. Furthermore, when life-cycle cost is formulated as an optimization problem, lifetime maintenance strategies can be identified which minimize total life-cycle cost [10,22,30,31]:

$$\min C_T = C_{INIT} + C_{INSP} + C_{REP} + C_{FAIL} \tag{5}$$

subject to

$$P_f(75) \leq P_f^o \tag{6}$$

$$t_{R_i} - t_{R_{i-1}} \geq t_{\min} \quad i = 1, n \tag{7}$$

where

$$C_{INSP} = \sum_{i=1}^{n} \frac{C_{IN_i}}{(1+r)^{t_{R_i}}}; \quad C_{REP} = \sum_{i=1}^{n} \frac{C_{R_i}}{(1+r)^{t_{R_i}}}; \quad C_{FAIL} = \frac{C_F \bullet P_f(75)}{(1+r)^{75}} \tag{8}$$

and C_T = total cost; C_{INIT} = initial cost; C_{INSP} = cost of all inspections; C_{REP} = cost of all repairs; C_{FAIL} = cost of failure; $i = i$th inspection/repair; n = total number of inspections/repairs; r = discount rate of money; t_{R_i} = time of inspection/repair i; C_{IN_i} = cost of inspection i; C_{R_i} = cost of repair i; C_F = failure cost coefficient; $P_f(75)$ = probability of failure of the system at t = 75 years; P_f^0 = target lifetime failure probability; and $t_{\min}$ = minimum time between inspections.

Using the estimated inspection and repair costs provided by the Colorado Department of Transportation[32,33], the optimum inspection/repair times associated with minimum total life-cycle cost are 37.8 and 51.1 years for discount rates of 0% and 4%, respectively[22]. The influence of optimal repair times associated with 0% and 4% discount rates on the strength degradation function and lifetime failure probabilities are shown in Figs. 6 and 7, respectively[22]. It can be observed that as the discount rate increases, the optimal repair time occurs later in the life of the bridge. Also, since the critical interior girder has a dominant influence on the reliability of the system, repair of the exterior girder has little influence on the lifetime system reliability.

5. CONCLUDING REMARKS

Reinforced concrete bridges in aggressive environments are often subjected to reinforcement corrosion that can drastically reduce the remaining strength over time. For an existing reinforced concrete bridge, it was shown that live load occurrence rate and load distribution to individual girders can have a significant influence on the critical live load condition and lifetime failure probability, respectively. A method for incorporating inspection data into reliability predictions was illustrated. Opti-

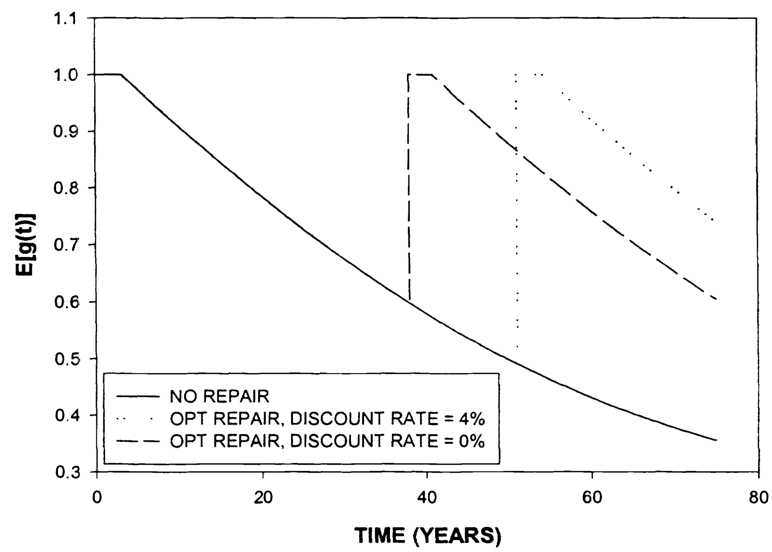


Fig. 6: Strength Degradation Function Associated with Optimum Inspection/Repair

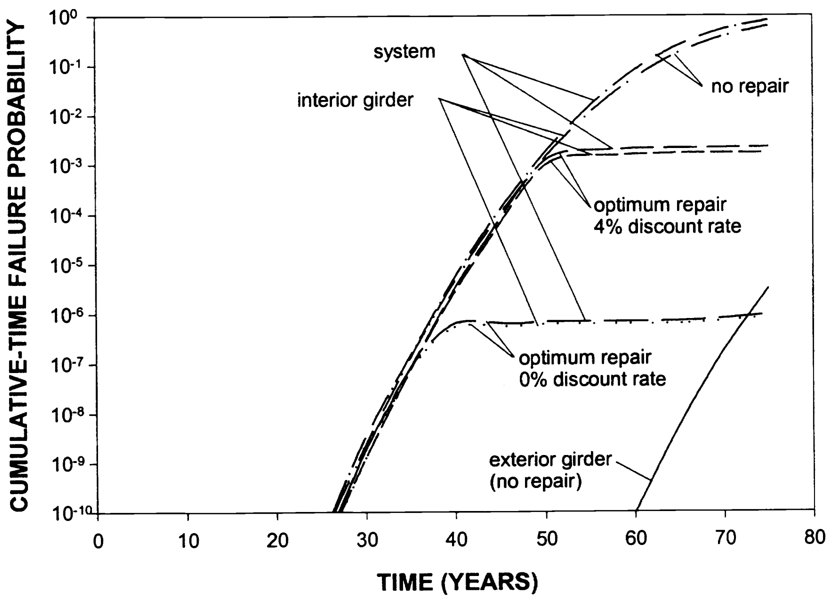


Fig. 7: Lifetime Failure Probabilities Associated with Optimum Inspection/Repair

mal lifetime maintenance strategies were also identified, and the influence of discount rate on lifetime failure probabilities associated with optimal inspection/repair times were shown for girders and systems of girders. It was also shown that the reliability of the bridge system is relatively insensitive to repair of non-critical girders. The results can be used for the further development of optimal reliability-based lifetime maintenance strategies for reinforced concrete bridges.

6. ACKNOWLEDGEMENTS

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