Optimal inspection scheduling with alternative fatigue reliability formulations for steel bridges

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ABSTRACT: A reliability-based method for inspection scheduling of steel bridges is proposed to yield the optimal (most economical) inspection strategy that meets an acceptable safety level through the planned service life. Two fatigue reliability formulations that can be applied for most details in steel bridges are presented. For details classified according to AASHTO fatigue categories, a limit state function related to the number of stress cycles to failure based on Miner's rule is used to evaluate the fatigue reliability; for details not classified according to AASHTO fatigue categories, a limit state function related to crack size and growth rate is used to evaluate the fatigue reliability. The inspection scheduling problem is modeled as an optimization problem with an objective function that includes the total expected cost of inspection, repair, and failure formulated using an event tree approach, appropriate constraints on the interval between inspections, and a specified minimally acceptable (target) structural reliability. An optimal inspection-scheduling plan can thus be developed for any specified fatigue details or fracture-critical sections in steel bridges. Examples presented demonstrate the advantage of the reliability-based optimal inspection scheduling in cost saving and structural reliability control over alternative inspection plans. Two case studies related to steel bridges – one for a plate girder and the other for a box girder – are presented to demonstrate the proposed reliability-based optimal inspection scheduling procedure.

1 INTRODUCTION

A reliability-based inspection scheduling procedure that can yield an optimal inspection schedule and can maintain a specified safety level for fracture-critical members in steel bridges through their planned service lives is presented. This procedure is based, in sequence, on a stress range analysis, a fatigue reliability analysis, and an optimization analysis. In the stress range analysis, the "effective" stress range for the identified member or detail may be obtained from a stress spectrum analysis, an assumed stress probability distribution (e.g., Rayleigh) based on data, or a fatigue truck analysis. Once this effective stress range distribution representative of the actual traffic on a bridge is obtained, a fatigue reliability analysis of the member or detail of interest may be performed. For all details classified in specific AASHTO fatigue categories, a limit state function related to the number of stress cycles to failure that is based on Miner's Rule may be used; for all other details (i.e., not classified in specific AASHTO fatigue categories), a limit state function based on crack growth rates, as proposed by Madsen (1985), may be used. An optimization problem for inspection scheduling that incorporates fatigue reliability calculations (for details of interest) along with an event tree approach, is formulated with an objective function includes costs and appropriate constraints on the inspection intervals and on acceptable minimum levels of structural safety. Solution of the optimization problem yields the optimal inspection schedule. Numerical examples from two case studies on steel bridges – one including a plate girder, the other including a box girder – are presented to demonstrate the proposed reliability-based optimal inspection scheduling procedure.

2 STRESS RANGE ANALYSIS OF FATIGUE LOADINGS IN STEEL BRIDGES

The operating stress range for a steel member or a detail in a bridge is a key factor that directly affects its fatigue performance. Therefore, obtaining an accurate description of the effective stress range, S_{RE}, applied on the identified detail is very important in fatigue reliability analysis. Any one of three general approaches may be employed to establish the effective stress range: stress spectrum analysis, fatigue truck analysis, and assumed stress distribution analysis. While the former two approaches may be used when data are available (see Chung et al. (2003)), often a Rayleigh distribution (see, for example, Schilling (1978)) is used as a parametric stress distribution to model the stress range spectrum in steel bridges. The effective stress range based on a Rayleigh distribution analysis is easily written as follows:

$$S_{RE} = \left[E\left(S_{RE}^{B}\right)\right]^{1/B} = \sqrt{2}S_{R0} \cdot \Gamma\left(\frac{B}{2}+1\right)^{1/B} \tag{1}$$

where $S_{R0} = \sqrt{\frac{2}{\pi}}E\left(S_{RE}\right)$ and $B = 3.0$ for steel.

3 FATIGUE RELIABILITY ANALYSIS FOR FRACTURE-CRITICAL MEMBERS

The objective here is to apply reliability theory to evaluate the safety of fracture-critical members (or details) under fatigue loadings in their service lives. For details classified according to AASHTO fatigue categories, a limit state function based on Miner's Rule (with an empirical S-N curve relation based on fatigue test results) is used to evaluate the fatigue reliability. For details not classified according to AASHTO fatigue categories, a limit state function related to crack size and based on a damage accumulation function proposed by Madsen (1985) is used to evaluate the fatigue reliability. After defining a (target) minimum acceptable level for structural safety, the actual reliability of the chosen fracture-critical detail may be compared with this target to yield information that can help in scheduling of inspections.

3.1 *Reliability analysis for details categorized in AASHTO fatigue categories*

In the AASHTO Specifications, empirical S-N curve relations were established from fatigue tests conducted in the 1970s to prevent design details in steel bridges from fatigue failure. Eight categories, designated as A to E' in the specifications, are tabulated to classify commonly occurring details in steel bridges and to provide information for the S-N curve relation that can be expressed as:

$$N = A \cdot S_R^{-3} \tag{2}$$

where N is the number of constant-amplitude cycles of stress range, S_R, applied on the specified detail that cause failure, and A is a fatigue strength coefficient than can be obtained from fatigue tests (for each fatigue category, an empirical estimate for the value of A is provided).

Combining the S-N curve relation with Miner's rule (1945), a fatigue limit state function, $g(\mathbf{X})$, for reliability analysis, similar to that used by Zhao et al. (1994), is defined as:

$$g(\mathbf{X}) = N_c - N(Y) \le 0 \tag{3a}$$

$$N_c = \frac{A \cdot \Delta}{S_{RE}^{3}} \tag{3b}$$

$$N(Y) = 365 \cdot C_S \cdot ADTT \cdot Y \tag{3c}$$

where N_c = critical number of stress cycles to fatigue failure under the variable-amplitude loading with effective stress range S_{RE}; $N(Y)$ = the total number of stress cycles experienced in Y years; Δ = a Miner's critical damage uncertainty parameter; C_s = stress cycles per truck passage; ADTT = average daily truck traffic.

Except for the variable, Y, all the variables in Eq. (3) can be treated as random when sufficient data are collected. For examples, Wirsching and Chen (1988) studied the test data reported by Miner (1945) and found that Δ (which represents uncertainty associated with the use of Miner's rule) may be modeled by a lognormal distribution with a mean value of 1.0 and a coefficient of variation (COV) of 30%. Chung et al. (2003) examined the results of regression analysis for S-N curves proposed by Keating and Fisher (1986) and derived the mean and COV for the assumed lognormal fatigue strength coefficient, A, for each AASHTO fatigue category.

The probability of fatigue failure for the detail can be related to a reliability index, β, as follows:

$$P_F = P\left(g(\mathbf{X}) < 0\right) = \Phi(-\beta) \tag{4}$$

3.2 *Reliability analysis for details not categorized in AASHTO fatigue categories*

To analyze the fatigue reliability of details that are not categorized in the AASHTO fatigue categories, a fatigue limit state function, $g(\mathbf{X})$, related to crack size, as proposed by Madsen (1985), is applied. This limit state function, $g(\mathbf{X})$, is expressed as:

$$g(\mathrm{X}) = \psi(a_c) - \psi(a_N) \le 0 \tag{5a}$$

$$\psi(a_c) = \int_{a_0}^{a_c} \frac{da}{\left(Y(a)\sqrt{\pi a}\right)^B} \tag{5b}$$

$$\psi(a_N) = C \cdot S_{RE}^B \cdot N \tag{5c}$$

where a_c = the critical crack size associated with failure; a_N = the crack size corresponding to N stress cycles; a_0 = initial crack size; $\psi(a)$ = a damage accumulation function resulting from change in crack size from a_0 to a; C = a material property; B = an equivalent-damage material property consistent with Eq. (1); $Y(a)$ = a geometry function accounting for the shape of the specimen and mode of fracture.

As before, the probability of failure and associated reliability index may be evaluated using Eq. (4). Solution for P_F or β may be obtained by FORM, SORM, or Monte Carlo simulation once all the random variables and their distributions are defined.

4 OPTIMAL INSPECTION SCHEDULING

An event tree approach similar to that used by Thoft-Christensen and Sorensen (1987), Madsen (1989), Sorensen et al. (1991), and Frangopol et al. (1997) is employed for our optimization problem of inspection scheduling for steel bridges. A Markov approach as part of a decision process may also be employed as has been demonstrated by Ellis et al (1995) in a study of corrosion in steel girders of a highway bridge and by Madanat (1993) in a study of the optimal management of pavements. Both these studies also consider measurement error which is not included in the present study.

An example of the event tree used here, which is very similar to that of Frangopol et al. (1997), is shown in Fig. 1.

The decision on whether or not to repair that needs to be made after every inspection of a detail can be interpreted in a probabilistic form. The probability of repair, P_R, will be employed to describe this decision. Because different definitions of limit state functions for "AASHTO type" details and "non-AASHTO" type details are used, P_R is defined differently for the two types of details.

For the "AASHTO type" details, P_R may be considered as the probability of first observation of a crack in the identified detail. Multiplying N_c in Eq. (3a) by a reduction factor (say, 0.75), an associated limit state function to be used to evaluate P_R may be defined. Selecting 75 percent of the critical number of stress cycles to correspond to observation of the first crack in fatigue tests was found to be acceptable by Fisher et al. (1970).

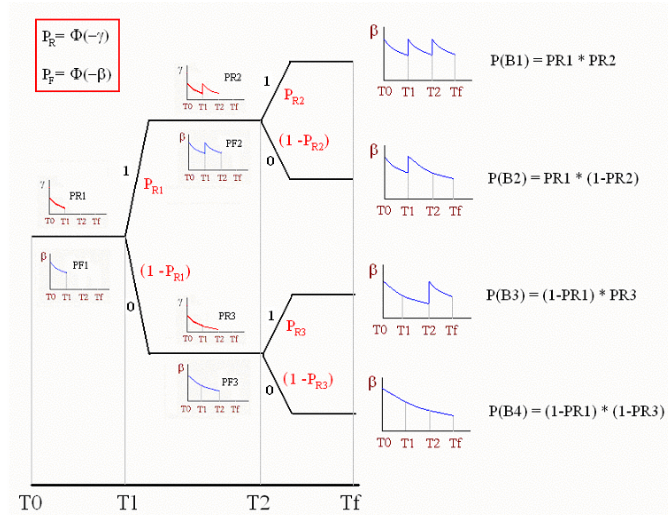

Figure 1. Representative event tree showing inspection and repair realizations (where 1 = repair; 0 = no repair; P_F = probability of failure; P_R = probability of repair).

For the "non-AASHTO" type details, P_R may be considered as the probability of detecting a crack with a predefined size, a_R, that warrants repair. Replacing $\psi(a_c)$ in Eq. (5a) by $\psi(a_R)$, an associated limit state function to be used to evaluate P_R may be defined.

The optimization problem for the inspection scheduling may be formulated as follows:

$$\min_{T_1 \ldots T_n, n} C_T = \sum_{i=1}^{n} C_I + \sum_{i=1}^{n} C_R \cdot E[R_i] + \frac{1}{T_f - T_0} \int_{T_0}^{T_f} C_F \cdot \Phi(-E[\beta])\, dT \tag{6a}$$

s.t. $T_0 < T_1 < \ldots < T_n < T_f$

$T_{min} \le T_i - T_{i\text{-}1} \le T_{max}$, $\quad i = 1 \ldots n$

$E[\beta(T_j)] \ge \beta_{min}$, $\quad j = 1 \ldots (n+1)$

where $$E[R_i] = \sum_{j=1}^{2^{i-1}} P(R_i \cap B_j^i) \tag{6b}$$

$$E[\beta] = -\Phi^{-1}\left(\sum_{j=1}^{2^n} P(F \mid B_j) \cdot P(B_j)\right) \tag{6c}$$

n = number of inspections; $(T_1, \ldots T_n)$ = times of inspections; T_{max} = maximum interval between inspections; T_{min} = minimum interval between inspections; C_I = the cost of a single inspection; C_R = the cost of a single repair; $E[R_i]$ = the expected number of repairs at time T_i; R_i = the repair event at time T_i; B_j^i = the branch j of the event tree at time, T_i; C_F =

the potential costs associated with failure of a fatigue detail; $(T_f - T_0)$ = the period from the present time (T_0) up to the planned life, T_f, over which inspection scheduling is being studied; $E[\beta]$ = the expected value of the reliability index; F = the event that the detail in question fails; B_i = the branch i of the event tree; β_{min} = the target reliability index.

The number of inspections, n, and the inspection times, T_i, may be found that result in minimum total cost – this entails solution of the optimization problem as formulated.

Note that the effect of discounted rates on all costs is not considered here. However, Eq. (6a) can include consideration for discounted rates in a straightforward manner as has been demonstrated by Madsen (1989) and Frangopol et al. (1997). In addition, the detail after repair is assumed to be as good as new, which means that the reliability after repair is raised to the same level as $\beta(T_0)$. This "as good as new" assumption may be modified for situations where either the detail is "not as good as new" or is "better than new" when sufficient data are available for the repair procedure and the altered reliability of the repaired detail. Both, the subsequent time-dependent reliability curve following the repair and the associated costs, might in general change for assumptions other than the "as good as new" case but such changes are easy to implement.

5 NUMERICAL EXAMPLES

Two examples are presented next involving details of both kinds – one that is of an AASHTO-type for a plate girder, the other that is of a non-AASHTO type for a box girder.

5.1 *Plate Girder Bridge:*

The example bridge studied here is the 680-ft long Brazos River Bridge in Texas, which was built in 1972. Fig. 2 shows various section views of the bridge as well as a magnified view of the selected fatigue detail, which is classified as a Category E detail (fatigue strength coefficient of Eq. (2), A: μ_A = 2.01×10^9 ksi^3, COV = 0.24) per AASHTO Specifications. A Rayleigh distribution as described by Eq. (1), with parameter, S_{R0}, equal to 6.13 ksi, is assumed for the effective stress range on the detail. A target reliability index, β_{min}, equal to 3.7 is employed. Two sets of relative costs of inspection, repair and failure: (i) $C_I : C_R : C_F = 1 : 1.3\times10^2 : 4\times10^5$; and (ii) $C_I : C_R : C_F = 1 : 2.6\times10^2 : 4\times10^5$ are considered for illustration. The number of stress cycles per truck passage, C_s, and the Average Daily Truck Traffic, ADTT, are taken to be 1 and 84, respectively. A service life of 50 years is considered for the bridge.

The time-dependent fatigue reliability, β, for the specified detail (AASHTO Category E') over the service life is shown in Fig. 3. It can be seen that, without intervention or repair of some sort, the fatigue reliability of the chosen detail would fall below the target reliability of 3.7 by the thirteenth year.

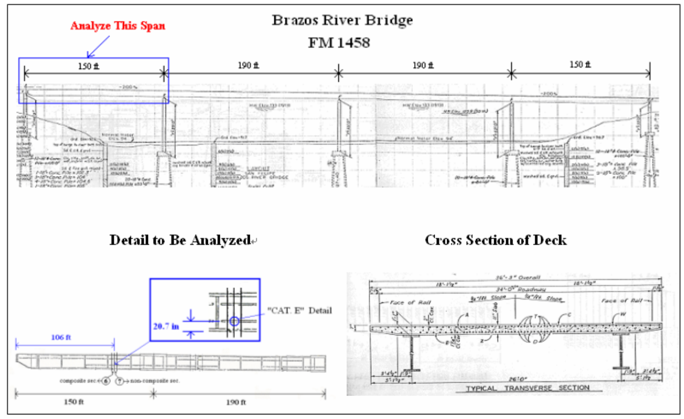

Figure 2. Brazos River Bridge in Texas.

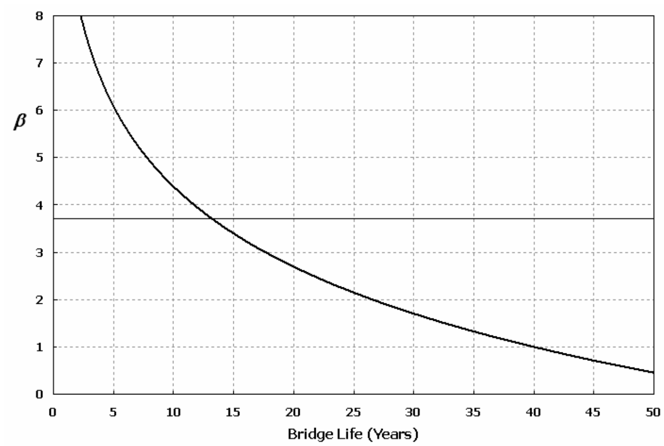

Figure 3. Fatigue reliability of the chosen detail in 50 years.

First, we will assume that in year 2002 (i.e., 30 years after 1972), no crack was found or that the crack in the detail was repaired to its original condition. To avoid too many inspections and to simultaneously meet the 2-year inspection interval required by the Federal Highway Administration (FHWA), the constraints on inspection intervals, T_{min} and T_{max}, are taken to be 0.5 and 2 years, respectively. With the "as good as new" repair policy, and for the relative costs of $C_I : C_R : C_F = 1 : 1.3\times10^2 : 4\times10^5$, it is found in Fig. 4 that the optimal number of inspections for the next twenty years is eleven and the associated optimal inspection schedule is as shown in Fig. 5 where, for comparison, an *ad hoc* periodic inspection schedule is also shown. The optimal inspection times in years are $\boldsymbol{T}$ = (2.0, 4.0, 6.0, 8.0, 10.0, 12.0, 13.5, 14.0, 14.5, 15.0, 15.5) + 30. On comparing the optimal inspection schedule with the periodic two-year interval schedule, the total relative cost (162.7) of the optimal schedule is found to be less than the total cost (168.9) of the periodic schedule. Though the optimal schedule requires two more inspections than the periodic schedule, these additional inspections and the short interval between inspections after the bridge reaches 42 years of age reduces the risk of the detail's failure. This fact can be confirmed by the reduced cost associated with failure in the total cost for the optimal schedule. Therefore, the optimal schedule clearly represents the preferred choice for inspecting this detail over its planned service life.

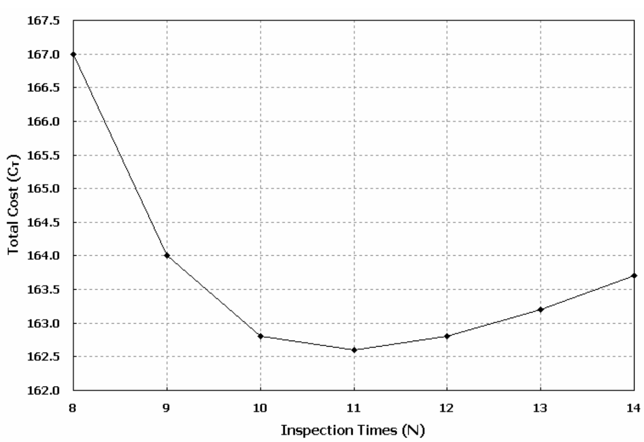

Figure 4. Optimal total cost as a function of the number of inspections for the chosen detail (C_I : C_R : C_F =1 : 1.3×10^2 : 4×10^5).

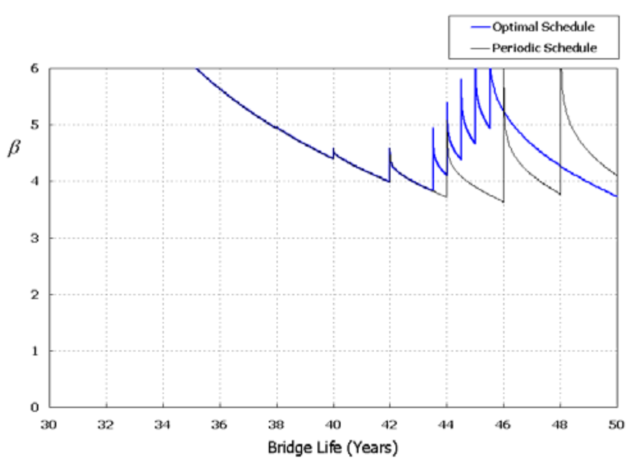

Figure 5. Optimal inspection schedule (T_{min} = 0.5 yrs, T_{max} = 2 yrs) for the case of C_I : C_R : C_F =1 : 1.3×10^2 : 4×10^5, C_T = 162.7.

Upon releasing the constraints on T_{max}, it is found as shown in Fig. 6 that only five inspections are required to achieve the optimal schedule with an associated total cost of 157.6, which is less than the total cost (162.7) of the previous optimal schedule with T_{max} = 2 yrs. The inspection times in years are $\boldsymbol{T}$ = (13.2, 14.1, 14.6, 15.1, 15.6) + 30 and note that the reliability index, β, is equal to exactly 3.7 at T_1 (13.2 yrs) and T_f (20 yrs). No inspections are needed before the reliability curve first hits the target reliability level at 13.2 yrs; also, no inspections are needed after the bridge has completed 45.6 yrs of its planned life. Because of this, the total cost is lower than for the case where the constraint on T_{max} is included.

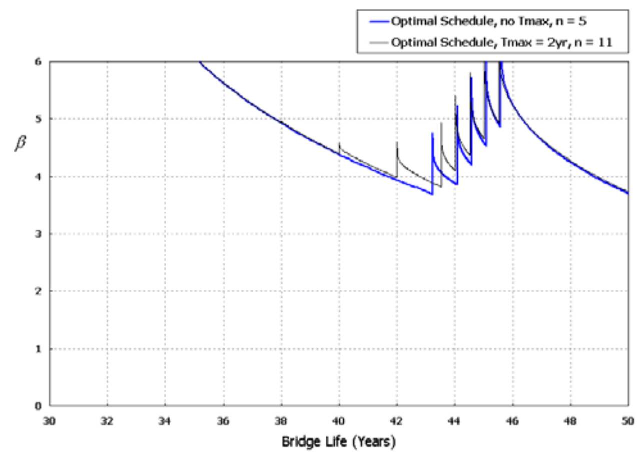

Figure 6. Optimal Inspection Schedule (T_{max} unbounded) for the case of C_I : C_R : C_F = 1 : 1.3×10^2 : 4×10^5, C_T = 157.6.

For the second case with T_{max} = 2 yrs and the relative costs of C_I : C_R : C_F = 1 : 2.6×10^2 : 4×10^5, nine inspection times for the next twenty years are needed and the optimal inspection times in years are $\boldsymbol{T}$ = (1.7, 3.4, 5.4, 7.4, 9.4, 11.4, 13.4, 15.1, 17.1) + 30 as shown in Fig. 7 where again a two-year periodic inspection interval schedule is also shown. Though the number of inspections (nine) is the same as with the periodic schedule, the total cost (200.5) for the optimal schedule is still lower than the total cost (211.7) for the periodic schedule. After removing the constraint on T_{max}, again, it is found in Fig. 8 that fewer (four) inspection times, $\boldsymbol{T}$ = (10.8, 13.5, 15.1, 17.1) + 30, are needed to reach the optimal schedule with the total cost of 195.0, which, again, is lower than the total cost of 200.5 for the optimal schedule that uses the constraint, T_{max} = 2 yrs.

5.2 *Box Girder Bridge:*

We consider a newly built box girder bridge with a center crack in the bottom flange (width = 42 in) as shown in Fig. 9. This example bridge is adapted from one described by Zhao et al. (1994) for which we seek an optimal inspection schedule for the next twenty years.

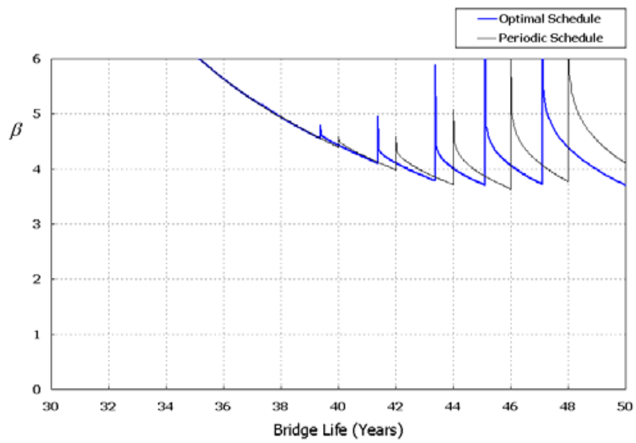

Figure 7. Optimal inspection schedule (T_{min} = 0.5 yrs, T_{max} = 2 yrs) for the case of $C_I : C_R : C_F = 1 : 2.6\times10^2 : 4\times10^5$, C_T = 200.5.

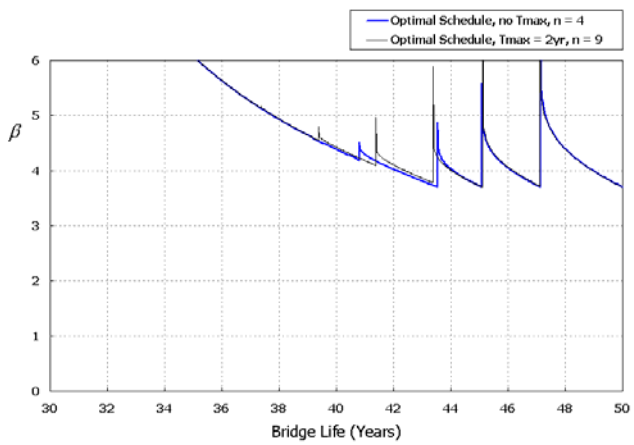

Figure 8. Optimal Inspection Schedule (T_{max} unbounded) for the case of $C_I : C_R : C_F = 1 : 2.6\times10^2 : 4\times10^5$, C_T = 195.0.

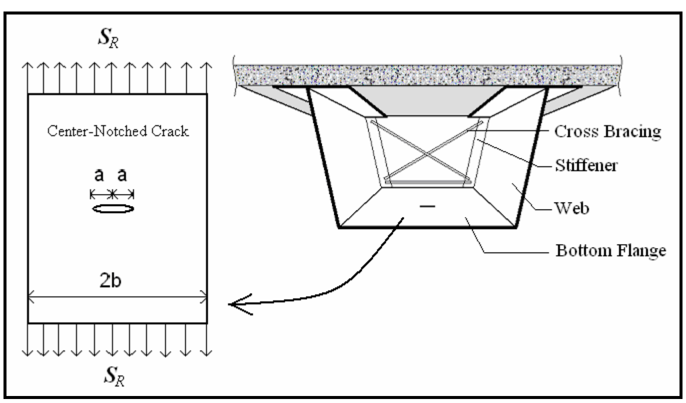

Figure 9. Box girder.

Two cases of relative costs of inspection, repair and failure: (i) $C_I : C_R : C_F = 1 : 3\times10^2 : 3.6\times10^6$; and (ii) $C_I : C_R : C_F = 1 : 6\times10^2 : 3.6\times10^6$ are considered here. The variables, C_s, β_{min}, and ADTT, are taken to be 1, 3.7 and 300 respectively. The random variables related to considerations for a center crack in the bottom flange are listed in Table 1 and the geometry function (see Eq. (5b)) for this crack geometry may be expressed as:

$$Y(a) = \left[1 - 0.025(a/b)^2 + 0.06(a/b)^4\right] \cdot \sqrt{\sec\left(\frac{\pi a}{2b}\right)} \qquad (7)$$

Since the detail shown in Fig. 9 is not specifically defined in the AASHTO Specifications, the procedure described for non-AASHTO type details is applied and a fatigue reliability curve from a FORM (First-Order Reliability Method) computation leads to the time-dependent reliability curve shown in Fig. 10. It can be seen that, without intervention or repair of some sort, the fatigue reliability of the chosen detail would fall below the target reliability of 3.7 by the fifteenth year.

Table 1. Random variables for center crack in bottom flange.

Variable	Type	Mean	COV
a_0	lognormal	0.010	0.500
a_c	constant	1.000	0.000
a_R	constant	0.200	0.000
C	lognormal	2.05×10^{-10}	0.630
B	normal	3.000	0.100
S_{R0}	constant	6.334	0.000

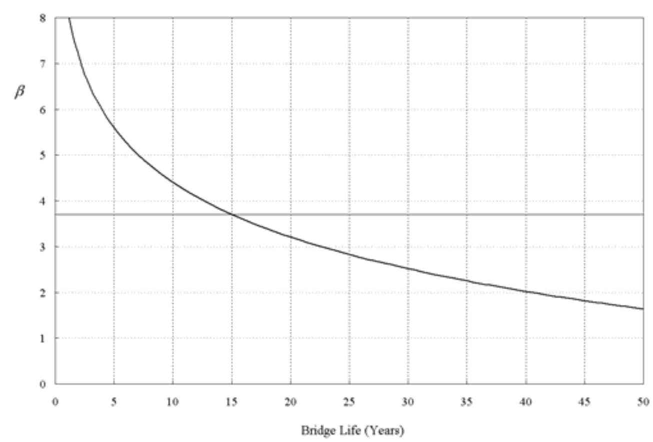

Figure 10. Fatigue reliability of the chosen detail in 50 years.

For the relative costs of $C_I : C_R : C_F = 1 : 3\times10^2 : 3.6\times10^6$, the reliability curves for optimization schedules in two cases, where T_{max} = 2 yrs as well as where no T_{max} constraint is imposed, are shown in Fig. 11. It may be seen that, for the optimal schedule constrained by T_{max} = 2 yrs, the optimal number of inspections is ten with a total cost of 322.4 and the inspection times in years, $\boldsymbol{T}$ = (2.0, 4.0, 6.0, 8.0, 10.0, 10.5, 11.0, 11.5, 12.0, 12.5). Not unexpectedly, the optimal schedule without a T_{max} constraint required fewer (six) inspections and costs less (318); the associated inspection times in years, $\boldsymbol{T}$ = (10.3, 10.8, 11.3, 11.8, 12.3, 12.8). Both optimal schedules result in lower costs than the total cost (396.9) for a two-year periodic inspection schedule that is required by the FHWA.

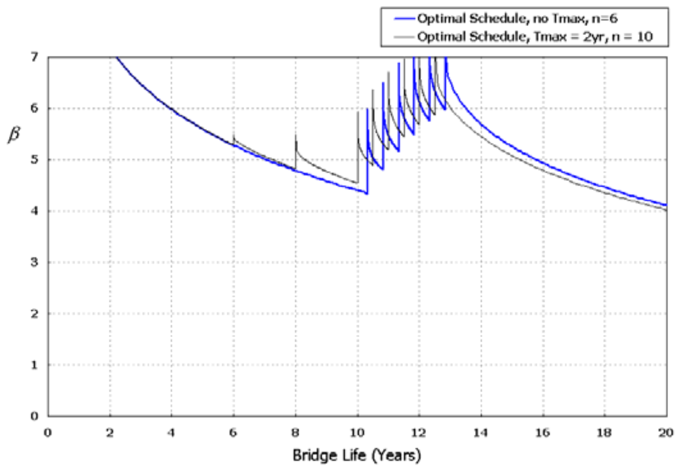

Figure 11. Optimal Inspection Schedules with T_{max} = 2 yrs and without T_{max} for the case of $C_I : C_R : C_F = 1 : 3\times10^2 : 3.6\times10^6$.

For the second case with higher repair costs, similar findings as in the case with the lower repair costs are noted as shown in Fig. 12. The optimal schedule without a constraint on T_{max} requires fewer inspections (four) in twenty years and costs less (395.8) than the inspection times (nine) and associated cost (430.1) for the optimal schedule when T_{max} = 2 yrs. The inspection schedules in years for the cases with T_{max} bounded and unbounded are $\boldsymbol{T}$ = (0.5, 1.0, 1.5, 2.9, 4.9, 6.9, 8.9, 10.9, 12.9) and $\boldsymbol{T}$ = (0.5, 1.0, 1.5, 14.6), respectively.

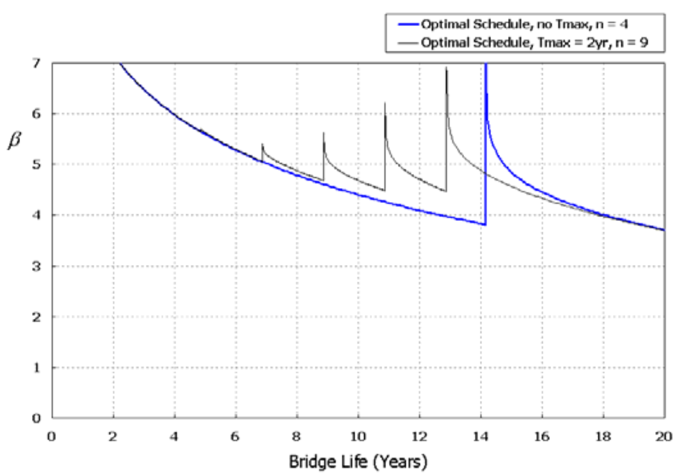

Figure 12. Optimal Inspection Schedules with T_{max} = 2 yrs and without T_{max} for the case of $C_I : C_R : C_F = 1 : 6\times10^2 : 3.6\times10^6$.

6 DISCUSSION AND CONCLUSIONS

From the results presented, it can be observed that an increase in the number of inspections, n, tends to increase inspection and repair costs but typically decreases expected failure costs. The optimal result (or lowest cost) occurs for a number, n_{opt}, of inspections where the decrease in failure costs starts to become smaller than the increase in inspection and repair costs. Comparing the optimal schedules in the two examples studied, the lower relative repair cost cases required more inspections to reach the optimal point while the higher relative repair cost cases required fewer inspections to yield the minimum costs. Clearly, the relative costs of inspection, repair, and failure all affect the optimization results in a very direct manner, regardless of whether or not a constraint on T_{max} is imposed.

The maximum time between inspections, T_{max}, is an important constraint that influences the number of inspections, the total cost, and the inspection strategy. When the inspection scheduling is constrained by T_{max}, a greater number of inspections results which raises the fatigue reliability of the detail and, thus, lowers the expected cost of failure. However, the cost of inspections and repairs increase and the total cost grows as a result. When the constraint on T_{max} is removed, the repair strategy changes so as to require inspections only when the reliability curve gets close to the target reliability; this results in lower total costs.

It is seen that a periodic two-year inspection schedule over the planned service life as is required by the FHWA for steel bridges will not be the optimal schedule for some details if one is interested in keeping costs low as well as maintaining safety. Though this periodic schedule keeps the fatigue reliability at a higher level than the optimal schedules obtained for the example bridge studied here, a larger number of inspections and repairs over the service life cause an increase in total cost. The reliability-based fatigue inspection strategy presented here yields the optimal inspection schedule maintaining prescribed safety levels for lower costs. The optimization results are affected by the time-dependent fatigue reliability of the detail in question, the imposed constraints (i.e., on the minimum acceptable reliability and on the inspection interval), and the relative costs of inspection, repair and failure. The influences of the constraints on the interval between inspections (at least on the upper bound of this interval), and of the relative costs of inspection, repair and failure were highlighted in the numerical examples presented.

Reliability-based inspection scheduling offers a rational method to arrive at inspection and maintenance strategies for steel bridges. Applying the procedure described for various types of details, bridge authorities can optimally allocate their maintenance budgets in an efficient manner without compromising the safety of their bridges. This optimal scheduling procedure may also be applied to other degrading civil infrastructure systems if the reliability, costs, and the related random variables affecting performance can be quantified.

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