Probabilistic Fatigue Life Prognosis for Steel Railway Bridges after Local Inspection and Repair

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ABSTRACT

Steel railway bridges are exposed to repeated train loads which often cause fatigue failure. To guarantee the target fatigue life, bridge maintenance such as local inspection and repair should be properly provided based on accurate fatigue life prognosis, but it is a challenging task because there are various sources of uncertainty associated with bridges, train loads, environment, and maintenance work. For the optimal risk-based maintenance, it is thus essential to predict the probabilistic fatigue life of a steel railway bridge and update the life prognosis information based on the results of local inspection and repair. In this research, a probabilistic approach is proposed to estimate the fatigue failure risk of steel railway bridges and update the prior information of fatigue life prognosis after bridges are inspected and repaired. The proposed method is applied to a generic steel railway bridge, and the effects of local inspection and repair on the probabilistic fatigue life prognosis is discussed through parametric studies.

1. INTRODUCTION

Steel bridges which are important nodes in a railway transportation network are prone to the risk of fatigue failure. It is thus necessary to predict the fatigue life accurately so that appropriate decisions on optimal bridge maintenance can be made for a target period. However, such fatigue life prognosis is a challenging task because it requires considering the impact of inspection and repair, which are affected by various sources of uncertainty.

Recently, Lee & Song (2014) proposed a new method for fatigue life prognosis of structures. In their research, however, the proposed method was applied to a structure where the loading could be simplified to have a constant amplitude. Furthermore, it was possible to update the prior life prognosis information after structural inspection only, but not after repair. Therefore, in this research, the formulation in Lee & Song (2014) is further developed so that it can handle varying-amplitude loads and can update the life prognosis information of steel railway bridges after inspection and repair. To demonstrate the proposed method, it is applied to a numerical example of a generic steel railway bridge, and the fatigue life of the bridge is evaluated under various scenarios of inspection and repair.

2. PROPOSED METHODOLOGY

After a short review of the fatigue life prognosis method proposed by Lee & Song (2014), this paper explains how the method is further developed.

First, consider the following crack propagation model (Paris and Erdogan 1963):

$$\frac{da}{dN} = C(\Delta K)^m \tag{1}$$

where a denotes the crack length, N is the number of load cycles, C and m are the material parameters, and ΔK denotes the range of the stress intensity factor. Using Newman's approximation (Newman & Raju 1981), this stress intensity factor range can be calculated as

$$\Delta K = \Delta S \cdot Y(a) \cdot \sqrt{\pi a} \tag{2}$$

where ΔS and $Y(a)$ denote the stress amplitude and the geometry function, respectively.

By substituting Eq. (2) into Eq. (1), the following equation can be obtained:

$$\frac{1}{\left[Y(a)\sqrt{\pi a}\right]^m} da = C \cdot \Delta S^m dN \tag{3}$$

If the load amplitude ΔS is a constant, Eq. (3) can be developed to an equation for fatigue life prognosis, as described by Lee & Song (2014). However, when a train passes over a bridge, the stress fluctuates and has various amplitudes. With various stress amplitudes ΔS_i and the corresponding number of cycles n_i, the time duration T required for a crack propagation from the initial crack length a^0 to the crack length a^c can be estimated as

$$T = \frac{1}{C\nu_0 \sum_{i=1}^{N_{amp}} \left[n_i (\Delta S_i)^m \right]} \int_{a^0}^{a^c} \frac{1}{\left[Y(a)\sqrt{\pi a} \right]^m} da \tag{4}$$

where ν_0 is the frequency of train loading, and N_{amp} is the total number of stress amplitudes.

Using Eq. (4), the failure of a structural member within a given time interval [0, T_s] can be expressed by a limit-state function shown below.

$$g(\mathbf{X}) = T - T_s \le 0 \tag{5}$$

where **X** denotes the vector of random variables representing uncertainties.

For the fatigue failure event of interest E, the probability can be updated after multiple events of inspection and repair, by use of conditional probability. According to Jiao & Moan (1990), these inspection and repair events can be classified into two types: *equality* and *inequality* types, depending on whether a crack is detected (and measured) or not. Considering inspections and repairs are generally made at multiple locations, there are three possible combinations: *inequality*, *equality*, and *mixed* cases. The detailed formulations of the limit-state functions and probability update for these three cases are provided in Lee & Song (2014).

3. NUMERICAL EXAMPLE

The proposed method of fatigue life prognosis is tested through its application to a generic steel railway bridge. The bridge is assumed to consist of over thirty members, and it is not feasible to monitor all the members. Therefore, in this research, only 5 structural members that show the highest levels of axial stress under a generic train are selected for life prognosis. The 5 members are named Members 1, 2,…, 5 in the decreasing order of their maximum stresses, and their maximum stresses are estimated to be 32.8, 32.5, 21.3, 20.7, and 20.7 MPa, respectively.

The statistical information of random variables is determined through a comprehensive literature survey (Lee & Song, 2014, Lee & Cho 2016, Jiao & Moan 1990), and it is summarized in Table 1. Moreover, it was confirmed through a preliminary analysis that the correlation between random variables gave a negligible impact to the result of fatigue life prognosis. Thus, for the sake of simplicity, all random variables are assumed to be independent in this example.

In addition, the following deterministic parameters are used: Paris law parameter (m) = 3.344, L-bracket width = 650 mm, critical crack length (a^c) = 12.7 or 19.05 mm, and average train traffic = 20/day. For the geometry function $Y(a)$ in Eq. (2), an equation for I-beams described in Lee & Cho (2016) is introduced.

Table 1. Statistical properties of random variables.

Random variable	Mean	c.o.v.	Distribution type
Paris law parameter C	1.537×10^{-12} m/cycle/(MPa·mm)^m	0.226	Lognormal
Initial crack length	0.11 mm	1.0	Exponential
Initial crack length in repaired member	0.11 mm	1.0	Exponential
Detectable crack size	1.0 mm	1.0	Exponential
Crack measurement error	0	*0.1	Normal
Live load scale factor	1	0.1	Lognormal

(*: standard deviation)

4. ANALYSIS RESULTS

Fig. 1 shows the reliability indices of the 5 selected members at various periods of service time. Overall, the reliability indices of the bridge decrease as the service life increases, which means that the probability of failure increases as the use of the bridge increases.

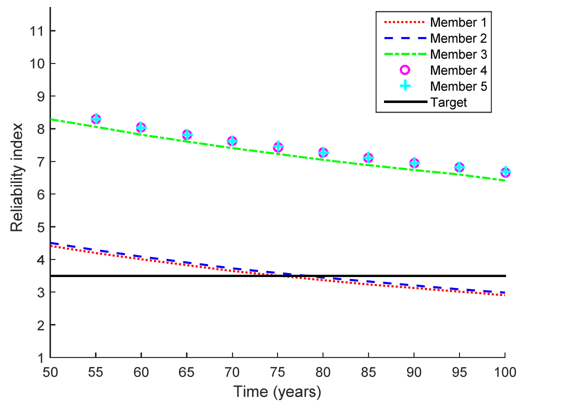

Figure 1. Reliability indices of the 5 selected members.

According to Lee & Cho (2016), the American Association of State Highway and Transportation Officials (AASHTO) Bridge Design Code recommends a target reliability index of 3.5 (i.e., a failure probability of 2.33×10^{-4}) with a service life of 75 years for steel members. With this target reliability index (a black line in Fig. 1), the fatigue lives of all the 5 members are calculated. The fatigue lives of Members 1 and 2 are evaluated as 75.4 and 78.2 years, respectively, whereas the fatigue lives of the other members are estimated to be longer than 100 years. This is mainly because the stresses of Members 1 and 2 are much larger than those of the other members. Therefore, the analysis results of only Members 1 and 2 are presented hereafter.

For maintenance events, in this study, hypothetical scenarios of local inspection and repair are assumed, as listed in Table 2 (T_I: inspection time & a^d: detectable crack size).

Table 2. Assumed scenarios of inspection and repair.

Scenario number	Scenario description
1	No crack is detected in Member 1 (T_I = 50 years & mean of a^d = 1.0 mm)
2	No crack is detected in Member 1 (T_I = 50 years & mean of a^d = 0.5 mm)
3	No crack is detected in Member 1 (T_I = 75 years & mean of a^d = 1.0 mm)
4	No crack is detected anywhere (T_I = 50 years & mean of a^d = 1.0 mm)
5	0.1 mm crack is found in Member 1 (T_I = 50 years)
6	0.5 mm crack is found in Member 1 (T_I = 50 years)
7	1.0 mm crack is found in Member 1 (T_I = 50 years)
8	1.0 mm crack is found in Member 1, but nowhere else (T_I = 50 years & mean of a^d = 1.0 mm)
9	0.5 mm crack is found in Member 1, but nowhere else, and Members 1 and 2 are repaired (T_I = 50 years & mean of a^d = 1.0 mm)

Using the proposed method, the fatigue lives of Members 1 and 2 can be estimated. Table 3 summarizes the result.

Table 3. Fatigue life prognosis results

Scenario	Fatigue life (years)	
	Member 1	Member 2
Design life	75.4	78.2
1	88.8	84.2
2	97.9	88.1
3	92.5	85.8
4	94.2	88.8
5	73.3	75.3
6	63.6	70.7
7	54.2	65.3
8	61.6	65.4
9	> 100	> 100

In Scenario 1, the fatigue lives of the two members increase, because no crack is detected even after 50 years. The updated lives increase further in Scenario 2, because no crack is detected even with a better crack-detecting device (i.e., a smaller mean of the detectable crack size a^d). In Scenario 3, no crack is detected even though an inspection is made at a later time than in Scenario 1, which increases the fatigue lives. In Scenario 4, the fatigue lives more increase than in Scenario 1, because no crack is detected at any of the members.

From Scenario 5 to Scenario 7, the measured crack size increases from 0.1 to 1.0 mm. It is clearly observed that the longer the crack size measured, the more likely the members are to fail. In addition, the fatigue lives of Members 1 and 2 in Scenarios 5–7 are shown in Table 2.

In Scenario 8, it is clearly seen that the updated fatigue lives becomes larger in Scenario 8 than in Scenario 7 even though the same crack size is observed at the same inspection time point. This is due to the additional inequality events (i.e., no more cracks being detected at the other locations) observed in the mixed case for Scenario 8. However, after Members 1 and 2 are repaired, the lives are significantly increased.

5. CONCLUSION

This study presents a new method for the fatigue life prognosis of steel railway bridges. The proposed method can handle a varying-amplitude load and can update the risk of fatigue failure for steel railway bridges after inspection and repair. To demonstrate the proposed method, it is applied to a numerical example of a generic railway bridge. As a result, the effects of various inspection and repair scenarios on the reliability updating are investigated in the numerical example, such as the number of inspections, crack-detecting resolution, inspection interval, inspection location, measured crack length, and repair

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