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Environmental Effects on Ultimate Strength Reliability of Corroded Steel Box Girder Bridges

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Abstract: This study develops an assessment procedure for the ultimate strength reliability of steel box girder bridges that takes into account the plate member degradation that results from uniform corrosion in different environmental conditions. This paper is a sequel to the authors' previous paper (Sharifi and Paik 2010), which deals with the effects of pit and general corrosion on the load-carrying capacity and reliability of these bridges. In contrast to that paper, the effects of different environmental conditions on such capacity and reliability are considered herein. Probabilistic corrosion rate parameters based on the available data are provided, and an analytical formula for predicting time-dependent ultimate strength is developed. The results of this study can be used in accurately predicting the service life and earliest repair time of corroded steel box girder bridges constructed in different environmental conditions.

Keywords: Bridges, Corrosion, Reliability analysis, Probabilistic model, Steel box girders, Loads, Environment conditions.

1 Introduction

Bridges constitute important components of a transportation network. If a steel highway bridge is not regularly cleaned, inspected and repaired, then corrosion will occur, which reduces the net cross section and leads to a decrease in load-carrying capacity. The deterioration of bridge networks due to aging and vehicular load growth, in terms of both magnitude and volume, is now a worldwide problem. Preserving the load capacity and service performance of these bridges requires repair and rehabilitation. Highway bridges, in particular, need assessment to identify those that are structurally deficient and to allow state, local and federal policymakers to determine which are in need of immediate attention (Cheung and Li 2001,

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Melchers 2005, Melchers and Jeffrey 2008, Czarnecki and Nowak 2008).

Structural safety is traditionally described and quantified in terms of safety factors. Structural reliability theory, in contrast, quantifies it using a risk measurement that takes into account the uncertainty involved. Structural safety is also time-variant, that is, the load demand and capacity of a structure may change over time (Sommer et al. 1993, Sarveswaran and Roberts 1999, Melchers 2005, Melchers and Jeffrey 2008). Aging and deteriorating bridge networks are now common problems in many developed countries. The growth of loads, both in magnitude and volume, also raises concerns over the safety of these structures. Evaluation, repair and rehabilitation are thus necessary for the preservation of their load capacity and service performance. To minimize the cost of replacement or repair, evaluation must accurately reveal the current load-carrying capacity of bridges, as well as consider additional future loads and further changes in capacity.

Probabilistic models make it possible to establish a reliability time profile for a bridge. The engineer must then decide the point at which the structure becomes unsafe. To do so, he or she must first establish a reliability index that can be used as the acceptable level below which the structure is considered to be unsafe. System models are generally employed in reliability analysis of the strength failure of bridges. However, because the rehabilitation or repair of bending or shear failure is usually made necessary by local limit state failure, rather than structure collapse, element-level reliability analysis may be more suitable than system-level reliability analysis in cases of the ultimate strength of bridges. As flexural failure is one of the most common failure modes in steel girder bridges, ultimate moment resistance is considered in this paper.

An example is presented to demonstrate the proposed method's application in determining the earliest time for the repair of steel box girder bridges in different environmental conditions, followed by a discussion of the experience gained and difficulties faced by practicing engineers in using this method of analysis.

2 Corrosion mechanics and rate modeling

As previously noted, corrosion is one of the main causes of deterioration in steel bridge superstructures. Corrosion may result not only in fracture, but also in the yielding or buckling of members. A reduction in the net area of one member results in the overall increase of member stress for a given load. One consequence of surface corrosion is deterioration in certain properties of the member cross section, such as the section modulus or slenderness ratio. Such properties are critical in a member's ability to resist bending moments or axial forces. Corrosion affects box plate members, thus decreasing the load-carrying capacity of the plates. As a result, buckling may occur with a load lower than the design load. In our previous paper (Sharifi and Paik 2010), it was found that uniform corrosion is the dominant type of corrosion in bridges. Accordingly, in this paper, the effects of uniform corrosion are considered in our investigation of the remaining life of steel box girders in different environmental conditions. As bridges may be constructed in areas with different environmental conditions, it is important to demonstrate the environmental effects on the load-carrying capacity and safety of bridges as they age. Our investigation assumes that all sides of the box girder section will corrode uniformly, except for the upper plate, which is protected from corrosion attack by the concrete deck. It is also assumed that the interior of the box girder is protected from environmental exposure and corrosion attack, as can be seen from Figure 1, which shows that general corrosion uniformly decreases the plate thickness of both sides and the bottom flange of the box girder section.

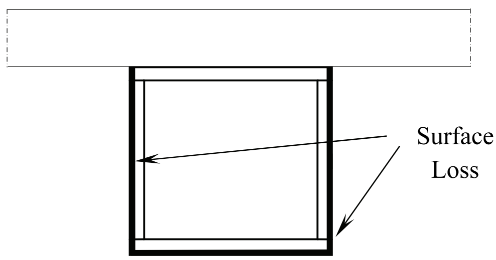

Figure 1: Model of corroded steel box girder cross-section.

Environmental factors, i.e., the amount of moisture in the air and the presence of salt, affect the amount and degree of corrosion (Kayser and Nowak 1989). To estimate the loss of plate thickness, probabilistic corrosion rate modeling needs to be carried out in advance. Such modeling has to include time as a basic parameter and other random variables that describe the environmental effects on the corrosion rate. Kayser (1988) has collected data on the corrosion performance of actual steel bridges, and research has demonstrated that corrosion propagation can be modeled with a good degree of approximation using the following exponential function (Komp 1987, Paik et al. 1998).

$$C(t) = At^B, \tag{1}$$

where $C(t)$ = the average corrosion penetration in micrometers (10^{-3} mm); t = the time in years; and A and B = parameters to be determined from regression analysis of the experimental data.

Table 1: Statistical parameters for A and B(Kayser 1988).

Parameters	Carbon Steel		Weathering Steel	
	$A(\times 10^{-3}$ mm)	B	A ($\times 10^{-3}$ mm)	B
Rural Environment				
Mean value, μ	34.0	0.65	33.3	0.498
Coefficient of variation, σ/μ	0.09	0.10	0.34	0.09
Coefficient of correlation, ρ_{AB}	Not available	-	-0.05	-
Urban Environment				
Mean value, μ	80.2	0.593	50.7	0.567
Coefficient of variation, σ/μ	0.42	0.40	0.30	0.37
Coefficient of correlation, ρ_{AB}	0.68	-	0.19	-
Marine Environment				
Mean value, μ	70.6	0.789	40.2	0.557
Coefficient of variation, σ/μ	0.66	0.49	0.22	0.10
Coefficient of correlation, ρ_{AB}	-0.31	-	-0.45	-

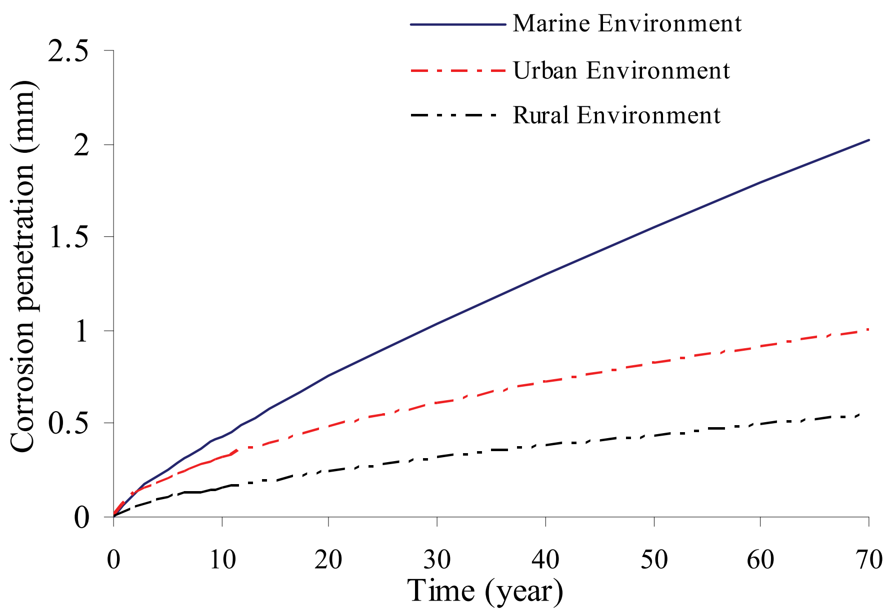

Figure 2: Corrosion penetration versus time for carbon steel in various environments.

Table 1 gives the mean values, coefficients of variation, and coefficients of correlation for A and B, and corrosion penetration versus time is plotted in Figure 2. It should be noted that the determination of A and B involves a considerable degree of uncertainty. These parameters are used to demonstrate the effects of different environmental conditions on the reliability of box girder bridges. Table 1 presents the rate probabilistic parameters for two types of steel. As carbon steel is widely used in existing bridges, and the degree of corrosion propagation in this type of steel is greater, carbon steel parameters are applied here.

3 Box girder ultimate strength modeling

Although three ultimate limit states, that is, moment, shear, and combined moment and shear, are considered important in deteriorating girders (Sarveswaran and Roberts 1999), this study considers only moment failure in its illustration of the reliability assessment. To develop a limit state function, a resistance random variable must be presented in advance. Analytical formulations can be applied to most situations of interest (Paik et al. 1997 and 1998), and a number of hull girder ultimate strength formulations have been suggested (Caldwell 1965, Valsgard and Steen 1991, Paik and Mansour 1995, Paik et al. 1996, Paik and Thayamballi 2003).

In the current study, the analytical formula suggested by Paik and Mansour (1995) is employed to estimate the ultimate bending moment of a steel box girder. It is often observed in nonlinear finite element calculations that a hull will reach its ultimate limit if both the collapse of the compression flange and the yielding of the tension flange occur. The side shell in the vicinity of the compression and tension flanges also often fails, although the material around the final neutral axis remains essentially in an elastic state. Based on these observations, Paik and Mansour (1995) assumed a credible distribution of the longitudinal stresses in the hull section at the overall collapse state shown in Figure 3. On the basis of this distribution, they then derived an explicit analytical formula for the corresponding resistive moment. Finally, they verified the accuracy of the formula by comparison with experimental and numerical results. The resulting expressions for the ultimate bending strength of a double-bottomed hull are given by the following.

$$\begin{aligned} M_u = {} & -A_D\sigma_{uD}(D-g) - \frac{A_S}{D}(D-H)(D+H-2g)\sigma_{uS} \\ & -A_B g\sigma_{yB} + \frac{A'_B}{H}(g-D_B)\left[D_B\sigma_{uS} - (H-D_B)\sigma_{yS}\right] \\ & - \frac{A_S H}{3D}\left[(2H-3g)\sigma_{uS} - (H-3g)\sigma_{yS}\right] \end{aligned} \quad (2)$$

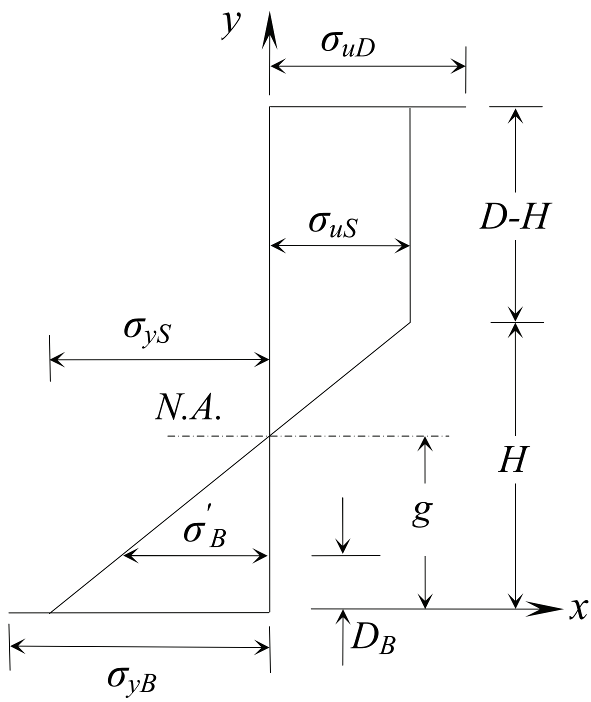

Figure 3: Longitudinal stress distribution over the box cross section at the overall collapse state suggested by Paik and Mansour (1995).

where

$$H = \frac{1}{2}\left\{\left(\frac{A_D\sigma_{uD}+2A_S\sigma_{uS}-A_B\sigma_{yB}-A'_B\sigma_{ys}}{A_S\left(\sigma_{uS}+\sigma_{ys}\right)}\right)D + \left[\left(\frac{A_D\sigma_{uD}+2A_S\sigma_{uS}-A_B\sigma_{yB}-A'_B\sigma_{ys}}{A_S\left(\sigma_{uS}+\sigma_{ys}\right)}\right)^2 D^2 + \frac{4A'_BD_B}{A_S}D\right]^{1/2}\right\}$$

$$g = \frac{\sigma_{ys}}{\sigma_{us}+\sigma_{ys}}H.$$

For a single box girder, the formula can be simplified to

$$M_u = -A_D\sigma_{uD}\left(D-g\right) - \frac{A_S}{D}\left(D-H\right)\left(D+H-2g\right)\sigma_{uS} - A_Bg\sigma_{yB} - \frac{A_SH}{3D}\left[\left(2H-3g\right)\sigma_{uS} - \left(H-3g\right)\sigma_{yS}\right] \quad (3)$$

where

$$H=\left(\frac{A_D\sigma_{uD}+2A_S\sigma_{uS}-A_B\sigma_{yB}}{A_S\left(\sigma_{uS}+\sigma_{ys}\right)}\right)D$$

$$g=\frac{\sigma_{ys}}{\sigma_{us}+\sigma_{ys}}H.$$

The ultimate stress of the compression flange and sides (σ_{uD}, σ_{us}) of the steel box must be known to compute the ultimate bending moment of the box using equation (2) or (3).

The ultimate strength of an imperfect plate under compression stress may be predicted as a function of the plate slenderness ratio, as follows (Paik and Thayamballi 2003, Paik et al. 2004).

$$\sigma_u/\sigma_y=\begin{cases}-0.032\beta^4+0.002\beta^2+1.0 & \text{for } \beta\le 1.5\\ 1.274/\beta & \text{for } 1.5<\beta\le 3.0\,.\\ 1.248/\beta^2+0.283 & \text{for } \beta>3.0\end{cases} \tag{4}$$

For convenience, the illustrative calculations presented in this study employ equation (4) to predict the ultimate compressive strength of the representative plate at the compressive flange or side structure of the box.

In addition to ultimate strength, it is also worthwhile to calculate the section modulus of the corroded box girder by varying the time. For this purpose, the section modulus of a single- or double-hull structure can be simply estimated as follows (Paik and Mansour 1995).

Section modulus at outer bottom flange:

$$SM=\frac{A_D\left(D-g'\right)^2+A_Bg'^2+A_B'\left(g'-D_B\right)^2+A_SD\left(2D-3g'\right)/3}{g'}, \tag{5}$$

where

$$g'=\frac{D\left(A_D+A_S\right)+A_B'D_B}{A_D+A_B+A_B'+2A_S}.$$

For a single box girder, the formula can be simplified to

$$SM=\frac{A_D\left(D-g'\right)^2+A_Bg'^2+A_SD\left(2D-3g'\right)/3}{g'}, \tag{6}$$

where

$$g' = \frac{D\,(A_D + A_S)}{A_D + A_B + 2A_S}.$$

It can be seen from equations (5) and (6) that the section modulus formula is expressed as a function of the plate thickness of primary members, as the individual sectional areas are functions of plate thickness. To calculate the mean value of the section modulus with corrosion, the reduction in plate thickness that is due to corrosion is deducted from the original plate thickness.

4 Load modeling

In the following, the two types of loads that may be applied to bridges, namely, dead and live loads, are considered.

4.1 Dead load model

The extensively used fundamental statistical parameters are the bias factor, λ, which is the ratio of the mean to nominal value, and the coefficient of variation, V. Dead load includes the weight of the girders, deck slab, wearing surface, barriers, diaphragms and sidewalk, where applicable. Here, λ = 1.03 and V = 0.08 for factory-made components (girders, diaphragms), λ = 1.05 and V = 0.10 for cast-in-place components (deck, barriers, sidewalk), and λ = 1.10 and V = 0.25 for asphalt wearing. Dead load is treated as the normal random variable (Nowak 1993, Nowak 1995, Nowak and Szerszen 1998, Nowak and Szerszen 2000, Nowak and Collins 2000).

4.2 Live load model

The live load on a bridge is the result of vehicular traffic. It can be considered as the sum of the static and dynamic components. The latter can be represented by an equivalent static load that is defined as the dynamic load factor. The effects of live load depend on a number of parameters, including the span length, axle load, axle configuration, gross vehicle weight, position of the vehicle on the bridge (transversely and longitudinally), the traffic volume, the number of vehicles on the bridge (multiple presence), girder spacing and the mechanical properties of the structural members (Nowak 1993, Nowak 1995, Nowak and Szerszen 1998, Nowak and Szerszen 2000, Nowak and Collins 2000). This study employs the load model developed by AASHTO (2004) (Figure 4). The design live load in AASHTO (2004) is specified as the effect of the design truck shown in Figure 4 superimposed with a uniformly distributed load of 9.3 kN/m.

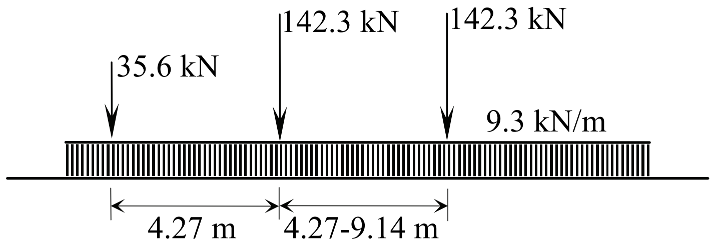

Figure 4: Proposed nominal live loading.

The bias factor, λ, for the live load distribution specified in the design code is between 1.10 and 1.20, and the coefficient of variation, V, is 0.18 (Barker and Puckett 2007). In this study, λ = 1.15 and V = 0.18. The dynamic load factor is defined as the ratio of the dynamic load to the static live load, and field measurements show it to decrease for heavier trucks (Nowak 1993, Nowak 1995, Nowak and Szerszen 1998, Nowak and Szerszen 2000, Nowak and Collins 2000). Here, the dynamic load factor (*IM*) is selected on the basis of the AASHTO specifications (2004), which can also be employed to estimate the live load distribution for the interior and exterior girders (for further information, see AASHTO (2004) or Barker and Puckett (2007)).

5 Reliability calculation

The theory of reliability analysis is discussed in many studies (e.g., Melchers 1999, Nowak and Collins 2000, Achintya and Mahadevan 2000, Lemarie 2009), and thus only a very brief description is given here. The probability of failure can generally be calculated as follows.

$$P_f = \int_{g(X)\leq 0} p_x(X)\, dx, \tag{7}$$

where $p(X)$ is the joint probability density function of the random variables, $X = (x_1, x_2, \ldots, x_n)$, which are associated with loading, material properties, geometrical characteristics, etc., and $g(X)$ is the limit state function, which is defined such that negative values imply failure. Because $g(X)$ is usually a complicated nonlinear function, performing the integration of equation (7) directly is difficult, and the equation is thus normally solved using approximate procedures (Melchers 1999, Nowak and Collins 2000, Achintya and Mahadevan 2000, Lemarie 2009).

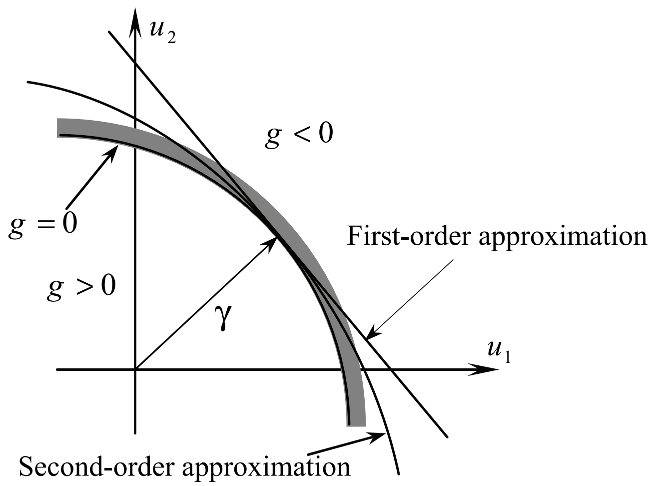

Figure 5: First- and second-order reliability methods.

In the approximation methods shown in Figure 5, the limit state surface is usually approximated at the design point by a tangent hyper plane or hyper parabola. The first type of approximation results in the use of a so called first-order reliability method (FORM), and the second type is central to the so called second-order reliability method (SORM). Such methods facilitate the rapid calculation of the probability of failure by widely available standard software packages. In addition to the individual probability distributions of the random variables involved, such calculations also allow the correlation between them to be readily accounted for. In this study, FORM is employed to assess the failure strength reliability of corroded steel box girder bridges.

The result of such a standard reliability calculation is a reliability index, γ, which is related to the probability of failure by

$$P_f = \phi(-\gamma), \tag{8}$$

where ϕ is the standard normal distribution function.

In our case, the failure condition associated with box girder collapse can be written as (limit state function):

$$g(x) = M_u - M_D - M_L \leq 0, \tag{9}$$

where
M_u = the random variable representing ultimate strength,

M_D= the random variable representing the dead load and
M_L= the random variable representing the live load.

The aforementioned failure condition uses the limit state function for box girder collapse as a function of three variables. However, recall that M_u is actually estimated using an analytical procedure that involves the individual thicknesses, yield strength and modulus of elasticity, namely, t, σ_y and E, such that

$$M_u = M_u\left(t, \sigma_y, E\right). \tag{10}$$

Although it appears that there are five types of random variables to be characterized, there are actually six, because the member thickness value at any particular time is also a function of the two parameters of the corrosion rate (A, B).

6 Application example

A hypothetical steel box girder bridge selected from an extensive parametric study of the design of steel box girder bridge components is employed to demonstrate the application of the proposed procedure.

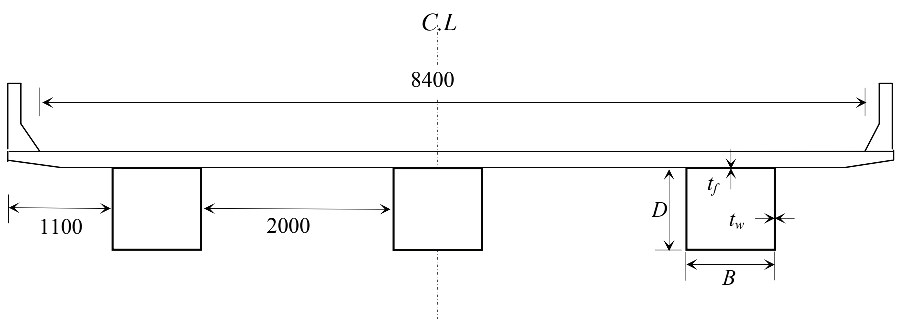

Figure 6: Typical cross-section of box girder bridge (dimensions in mm).

It is assumed that the bridge is not protected against corrosion and that it has a simple span of 20 m and two lanes of traffic in the same direction. The cross section is shown in Figures 6 and 7. The material and corrosion parameters are assumed to be log normally distributed, and the mean values and standard deviations are shown in Table 2. The corrosion parameters correspond to carbon steel in three different environmental conditions (see Table 1). The thicknesses of the deck and asphalt are 250 mm and 75 mm, respectively, and the lifetime, T, is 70 years. In the probabilistic analysis, M_u is calculated using the statistical parameters shown in Table 2. As the correlation factor between the rate parameters (A, B) is unavailable

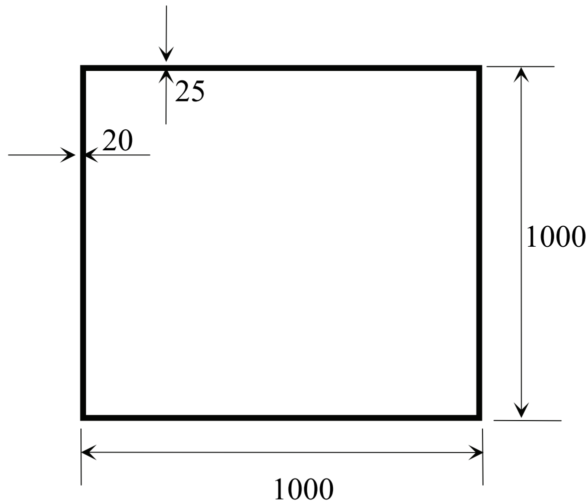

Figure 7: Dimensions (in mm) of cross-section of box girder.

for a rural environment, it is assumed that there is no correlation between them in this environment.

Table 2: Statistical parameters for A and B(Kayser 1988).

Parameters	Mean μ	Std Dev. σ
Modulus of elasticity for steel, E Yield stress in steel, σ_y	2.1×10^5 N/mm^2 350 N/mm^2	2.1×10^4 N/mm^2 35 N/mm^2
Rural Environment		
Corrosion parameter, A	34.0×10^{-3} mm	3.06×10^{-3} mm
Corrosion parameter, B	0.65	0.65
Coefficient of correlation, ρ_{AB}	0.68	
Urban Environment		
Corrosion parameter, A	80.2×10^{-3} mm	33.684×10^{-3} mm
Corrosion parameter, B	0.593	0.2372
Coefficient of correlation, ρ_{AB}	0.68	
Marine Environment		
Corrosion parameter, A	70.6×10^{-3} mm	46.6×10^{-3} mm
Corrosion parameter, B	0.789	0.3866
Coefficient of correlation, ρ_{AB}	-0.31	

6.1 Dead load

The mean value of the design dead load bending moment of interior and exterior steel box girders is calculated, with the results shown in Table 3. The dead load components for asphalt and the other components are first calculated separately, after which the equivalent dead load for each girder is calculated by estimating the mean and standard deviation of those components, as shown in Table 3.

Table 3: Values used in calculations (lognormal distributions).

Parameters	Mean μ	Standard deviation σ
Midspan dead load moment for interior girder, M_{DI}	15.50×10^8 N-mm	1.44×10^8 N-mm
Midspan dead load moment for exterior girder, M_{DE}	19.46×10^8 N-mm	1.81×10^8 N-mm

6.2 Live load

Based on the specifications provided in Section 4.2, the mean and standard deviation of the live load for each girder are calculated and shown in Table 4.

Table 4: Live load for girders (lognormal distributions).

Parameters	Mean μ	Standard deviation σ
Midspan live load moment for interior girder, M_{LI}	18.58×10^8 N-mm	3.34×10^8 N-mm
Midspan live load moment for exterior girder, M_{LE}	22.92×10^8 N-mm	4.13×10^8 N-mm

6.3 Results

Probabilistic analysis that takes different environmental conditions into account has been carried out to calculate the section modulus, ultimate strength, reliability and probability of failure of corroded steel box girders as a bridge ages. Figures 8 and 9 show the trends of variation in the section modulus and the ultimate bending strength versus time. It can be seen that the section modulus and the ultimate bending strength of corroded steel box girders decline as the bridge ages. In addition, as expected, they also decline when the environment changes from rural to marine at the same time.

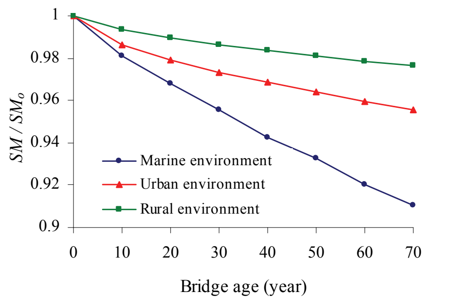

Figure 8: Variation of section modulus with age in different environments.

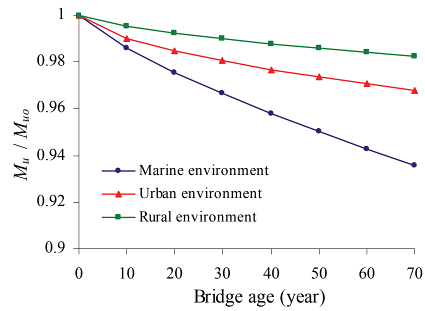

Figure 9: Variation of box ultimate strength with age in different environments.

The reliability indices for two girders (interior and exterior) of a highway bridge that is assumed to be uniformly corroded are computed using reliability software, with the results shown in Figures 10 and 11. As can be seen from these figures, to find the exact time at which girders become unsafe (the critical time), attempts were made through trial and error to calculate the reliability indices at several time periods before and after the critical time (at which a drop occurs). Therefore, drop in the reliability indices at a certain time appears remarkable and unexpected. Furthermore, the decrease in the resistance moment due to corrosion (it can be seen in Figure 9 that the slope of the lines in the first years of age is the most dramatic) and the high degree of variability in the corrosion parameters may be important reasons for the large change values seen in the reliability indices in Figures 10 and 11.

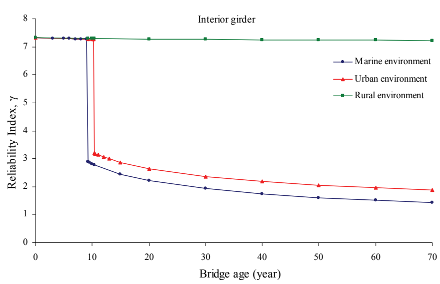

Figure 10: Variation in reliability index with age in different environments for interior girder.

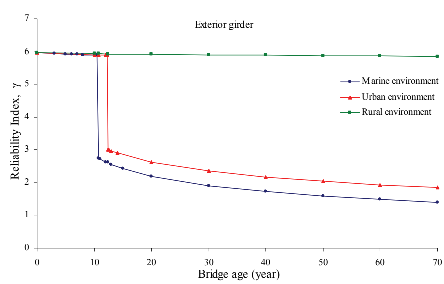

Figure 11: Variation in reliability index with age in different environments for exterior girder.

In the ultimate strength formulation introduced by Paik and Mansour (1995), the buckling of the upper and side plate members was considered to be an important factor when deriving the formula. Therefore, an important reason for the box girder collapse could be the buckling of the compressive plates. In this paper, such buckling may be a significant factor in the mutations that take place in the reliability indices. Hence, the box girder may collapse within the critical time and may be unable to sustain the applied loads afterwards. The results show that, unlike the urban and marine environments, there is no mutation in the reliability index versus the bridge age curves for a rural environment. It is found that different environmental conditions, which result in different corrosion rate parameters, have remarkable effects on load-carrying capacity and hence on the earliest time at which a corrosion-damaged bridge must be repaired.

The probabilities of failure for the two girders (interior and exterior) in different environments are presented in Tables 5 and 6, respectively. As expected, a change from a rural to a marine environment leads to an increase in the probability of failure.

Table 5: Failure probabilities in different environments for interior girder.

Time (year)	Probability of failure, P_f		
	Rural environment	Urban environment	Marine environment
0	1.16×10^{-13}	1.16×10^{-13}	1.16×10^{-13}
9.2	1.42×10^{-13}	1.70×10^{-13}	0.001860
10	1.44×10^{-13}	1.85×10^{-13}	0.002424
10.4	1.45×10^{-13}	0.000703	0.002731
20	1.64×10^{-13}	0.004022	0.013317
30	1.82×10^{-13}	0.008923	0.026972
40	2.01×10^{-13}	0.014322	0.040742
50	2.19×10^{-13}	0.019813	0.053911
60	2.38×10^{-13}	0.025226	0.066278
70	2.58×10^{-13}	0.030488	0.077839

7 Determination of the time for repair

To determine the earliest time for the repair intervention of girders that have been affected by corrosion damage, a reliability index below which they may be considered unsafe must first be established. As previously noted (Sharifi and Paik 2010), it is not only the accuracy of resistance and load modeling that has an influence on reliability, but also several factors that cannot be modeled in structural reliability analysis. Hence, the determination of an exact reliability index as the target may be

Table 6: Failure probabilities in different environments for exterior girder.

Time (year)	Probability of failure, P_f		
	Rural environment	Urban environment	Marine environment
0	1.24×10^{-9}	1.24×10^{-9}	1.24×10^{-9}
10	1.50×10^{-9}	1.82×10^{-9}	0.002424
10.7	1.51×10^{-9}	1.86×10^{-9}	0.003135
12.4	1.54×10^{-9}	0.001301	0.004525
20	1.67×10^{-9}	0.004295	0.013935
30	1.83×10^{-9}	0.009464	0.028097
40	1.98×10^{-9}	0.015125	0.042333
50	2.14×10^{-9}	0.020863	0.055890
60	2.38×10^{-9}	0.026502	0.068611
70	2.58×10^{-9}	0.031972	0.080480

impossible. Practicing engineers generally prefer to employ probabilistic analysis to quantify the reliability level implicit in current bridge codes and standards with a proven safety record, and then use this level as the acceptance level of reliability (Melchers 1999). This method has also been applied to the quantification of the reliability levels implicit in bridge design and assessment codes (Nowak and Lind 1979, Flint and Smith 1980, Chryssanthopoulos and Micic 1996). It is also recommended, however, that acceptance levels be based on the consequences of failure and the nature of the failure mode, and therefore the allowable reliabilities shown in Table 7, which are based on the failure type, are used in this study.

Table 7: Target (or acceptance) reliability levels (Sarveswaran and Roberts 1999).

Failure consequences	Ductile failure with reserve strength	Ductile failure with reserve strength	Brittle failure
Not serious	3.09 (10^{-3})	3.71 (10^{-4})	4.26 (10^{-5})
Serious	3.71 (10^{-4})	4.26 (10^{-5})	4.75 (10^{-6})
Very serious	4.26 (10^{-5})	4.75 (10^{-6})	5.20 (10^{-7})

Corresponding failure probabilities are given in parentheses.

The foregoing approaches were adopted to select the time for repair intervention. Applying the acceptance levels of the probability of failure, or a reliability index of from 3.09 to 5.2, the earliest times for the repair of interior girders are around 9 and $10^1\!/_2$ years, and those for exterior girders are around $10^1\!/_2$ and $12^1\!/_2$ years, in marine and urban environments, respectively. For example, if such a bridge is constructed now (in 2010) in a city (urban environment), then it should be repaired in $10\frac{1}{2}$ years' time (in 2020). If it is already in existence and is older than $10\frac{1}{2}$ years, then it would be unsafe to use the assumptions and procedures reported here, which demonstrates that the repair intervention date in this example is not very sensitive to the choice of acceptance level. As noted, corrosion has no serious effects on steel box girder bridges constructed in a rural environment (Figures 10 and 11). The foregoing results also show that it is very important to obtain real reliabilities in the first years of a steel box girder bridge's life.

8 Concluding remarks

A probabilistic-based method of determining the earliest time for the repair and rehabilitation of steel box girder bridges in various environmental conditions has been developed and presented herein. A resistance model is developed on the basis of an analytical ultimate strength formula. The load is distributed to the girders using the guidance formula for the highway bridge design, and different corrosion rate random variables are applied to identify the effects of three different environmental conditions on the time-variant behavior of bridges. Time-dependent reliability indices can then serve as the basis for selecting the earliest time to repair or renew individual girders, with the critical components identified as those associated with the lowest such indices.

From the developments and illustrations presented herein, the following conclusions can be drawn.

1. Environmental conditions have a great effect on the load-carrying capacity and reliability of steel box girder bridges. As expected, the section modulus and ultimate strength of corroded box girders may decrease with time, although the degree of that decrease is greater in the first few years. From existing and extensively used corrosion rate data (Table 1), it was known that a rural environmental condition has little effect on the load-carrying capacity of steel box girders during the lifetime of the bridge.

2. The analytical formula used here to consider the ultimate strength of steel box girders is suitable for evaluating the time-dependent steel box girder strength reliability of corroded bridges.

3. The reliability results presented herein demonstrate that the safety of a steel box girder may deteriorate suddenly during its service life even though the remaining ultimate moment may exhibit little decrease in the same period, which can be attributed to the nature of the resistance ultimate strength formula. As the resulting formulation is derived on the basis of compressive

plate buckling, overall collapse will occur if one of the plate members buckles. In other words, the high degree of variability in the corrosion rating parameters and the loss of resistance moment cause the compressive plate members to buckle in probabilistic analysis, and, accordingly, a large mutation occurs.

4. The reliability-based method considered in this paper for the computation of the risk and load-carrying capacity of corroded steel box girder bridges with environmental conditions taken into account should be useful in practice. It is presented not only as an applicable procedure for practicing engineers, but also as a scientific method for estimating the longevity of bridges.

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Nomenclature

A = corrosion parameter or cross-sectional area

B = corrosion parameter

C = average corrosion penetration

t = time in years and plate thickness of a member

A_B, A'_B, A_D = sectional area of outer bottom, inner or deck

A_S = half of the sectional area of the side structure, including any longitudinal bulkhead

D = box depth

D_B = height of double bottom

b = breadth of plate between longitudinal stiffeners

σ_u = ultimate compressive strength of a plate

σ_{uD}, σ_{uS} = ultimate compressive strength of a representative plate at the upper or side shell

σ_y =mean yield strength of the material

σ_{yB}, σ_{yS} = mean yield strength of the bottom or side shell

H = depth of box section in linear elastic state

M_u, M_{uo} = random variables representing the ultimate strength of a corroded or uncorroded box girder

g = height of the neutral axis (equations (2) or (3))

β =slenderness ratio of plating between longitudinal stiffeners

SM = elastic section modulus at bottom plate

g' =elastic horizontal neutral axis (equations (5) or (6))

λ = bias factor, which is the ratio of the mean to nominal value

V = coefficient of variation

M_D = dead-load moment

M_L = live-load moment

IM = dynamic live load

M_n = nominal bending moment strength

$g(x)$ = ultimate limit state function

P_f = probability of failure

$p(X)$ = joint probability density function of the random variables

$X = (x_1, x_2, \ldots, x_n)$ = random variable vector

σ_{xi} = standard deviation of random variable x_i

γ = reliability index

μ_x = mean value of random variable x_i

ϕ = standard normal distribution function

E = Young's modulus

T = lifetime of the bridge