SAFETY ANALYSIS OF STEEL BOX GIRDER BRIDGES WITH PITTING CORROSION

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ABSTRACT: Steel bridges corrode due to environmental exposure. The consequence is a reduction in both the load-carrying capacity and safety of a bridge. Therefore, it is needed to evaluate procedures for an exact prediction of the load-carrying capacity and reliability of bridges, in order to make reasonable decisions about repair, rehabilitation and renewal. The aim of this study is to develop and demonstrate a procedure for the assessment of steel box girder bridge ultimate strength reliability that takes the degradation of plate members due to pit corrosion into account. The present paper treats the effect of pitting corrosion on the load-carrying capacity and reliability of steel box girder bridges and the results are compared with the uniform corrosion effect. The procedure and results of this study can be used for the better prediction of the service life of deteriorating steel box girder bridges and the development of optimal reliability-based maintenance strategies.

Keywords: Bridges, pitting corrosion, reliability analysis, probabilistic model, steel box girders

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1. INTRODUCTION

Bridges are important to everyone. The majority of steel bridges in the United States were constructed after World War II from 1950 to 1980. Therefore, bridges are aging and their probabilities of failure are increasing. Bridge elements deteriorate with time due to corrosion, wear, fatigue and other forms of material degradation. Moreover, lately, the legal load on bridges has increased and the problem is that the majority of the old bridges fail to satisfy this requirement. In addition, the maintenances cost of a bridge is of great importance. Deficient bridges are either repaired, or replaced. The efficient maintenance, repair and rehabilitation of existing bridges require the development of a methodology that allows for the accurate evaluation of load-carrying capacity and the prediction of remaining life. Corrosion is one of the most important causes of deterioration in steel bridges (Cheung and Li [1]; Czarnecki and Nowak [2]; Melchers [3]; Melchers and Jeffrey [4]; Sharifi [5]; Sharifi and Paik [6-8]; Sharifi and Tohidi [9,10]; Tohidi and Sharifi [11-14]).

Many of the factors that determine the performance of deteriorating structures are characterized by a high degree of uncertainty. Probability and statistics provide a framework for dealing with such uncertainty. As the methods and concepts of structural reliability developed over the last few decades, they have become increasingly better understood and approved by engineers. At present, reliability can be considered as a rational evaluation criterion for bridge performance. The reliability methods allow for consideration of uncertainties associated with material properties, geometry and dimensions, loads, and environmental conditions, and they can be used for a better estimation of the failure probability (Melchers [3]; Melchers and Jeffrey [4]; Sarveswaran and Roberts [15]; Sommer et al. [16]).

Probabilistic models make it possible to establish a reliability time profile for a bridge. The engineer then has to 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. Maybe system models are accurate for reliability analysis of the strength failure of bridges (Sarveswaran and Roberts [15]). However, because the rehabilitation and repair of the bending or shear failure of a steel bridge are not usually required because of structure collapse, but rather because of local limit state failure, element-level reliability analysis may be more suitable than system-level reliability analysis in cases of the ultimate strength of steel or concrete bridges. Since the flexural failure is one of the most important of failure mode for steel girder bridges, the ultimate moment resistance has been considered in this paper. Time-dependent reliability analyses such as those by Mori and Ellingwood [17], Thoft-Christensen [18], Enright and Frangopol [19] and others can be used as decision-making tools, or provide additional information on which to base inspection, maintenance and repair strategies.

The results of this study will be useful for practicing engineers who need to employ reliability analysis in practical applications. The example presented herein demonstrates the procedures that are required to calculate the latest time to repair intervention for a number of deteriorating steel box beams that support a bridge. The viewpoint adopted is that of the practicing engineer. Employing a bridge-specific deterioration model, this study highlights the problems associated with determining the latest such intervention for a sample bridge substructure. The experience gained and the difficulties faced by practicing engineers when using this method of analysis are also discussed.

2. CORROSION MECHANICS

Deterioration due to corrosion is considered in this study. To predict likely corrosion damage tolerance a priori, it is necessary to estimate the corrosion rate for each type of structural member. Theoretical predictions of these corrosion rates have been attempted, but they represent a difficult task. An easier alternative is to base the rate prediction on the statistical analysis of past data for comparable situations. There are four corrosion-related questions that ideally need to be answered for the structural components in a space:

1. Where is corrosion likely to occur?
2. When will it start?
3. What is its likely extent?
4. What are the likely corrosion rates?

The first question can usually be answered through some form of historical data, e.g., the results of previous surveys. Similarly, the information required to answer the second question can be gleaned from prior surveys of the given structure. Assumptions as to the start of the corrosion can of course be made, depending on the use of a protection system, the characteristics of the coating and the anode residence time. The extent of the corrosion will presumably increase with time, although our ability to predict corrosion progress remains very limited. Thus, the only real alternative for answering the third question is to pessimistically assumes a greater corrosion extent than is really likely, which is how nominal design corrosion values are usually arrived at (Paik et al. [20]). A potential damage due to corrosion is an important consideration in the design of steel bridges. The corrosion effects can vary from nonstructural maintenance problems to a local failure or an overall collapse. Four major categories of corrosion effects are identified: loss of section, creation of stress concentration, introduction of unintended fixity and introduction of unintended movement (Czarnecki and Nowak [2]). The most common is loss of material. The loss of material can be either uniform, when corrosion affects large areas of a bridge component, or localized in a form of pits. Likewise, the loss of section of some components may have little or even no effect on the overall capacity of a bridge, whereas deterioration of other members can have a significant effect. Therefore, it is very important to make a distinction between a localized corrosion, related to the behavior of a member and deterioration of a component that affects the structural performance of the whole bridge. The loss of material can result in a smaller net cross-section and it may lead to a reduction of fracture and buckling resistances of a member (Kayser [21]). This study addresses the most common types of corrosion that cause a reduction in strength, and develops a probabilistic rate model.

2.1 Corrosion Damage Idealization

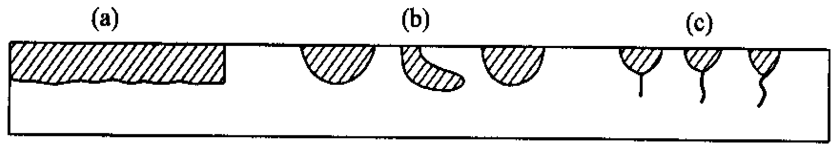

Figure. 1. Typical Modes of Corrosion Damage: (A) General (Uniform) Corrosion; (B) Localized Corrosion; (C) Fatigue Cracks Arising from Localized Corrosion

Figure 1 shows some of the more common types of corrosion-related damage that affect the strength of steel structures to a greater extent than other types. General corrosion (also called uniform corrosion) uniformly reduces the thickness of structural members, as shown in Figure 1(a), whereas localized corrosion (e.g., pitting or grooving) causes degradation in local regions, as shown in Figure 1(b). Fatigue cracks may sometimes arise from localized corrosion, as shown in Figure 1(c). In the present study, two types of corrosion damage idealization are considered, namely, general and pit corrosion. For the former, it is assumed that the thickness of the entire plate is uniformly reduced by the corrosion. The latter, in contrast, is assumed to reduce the plate thickness in localized regions. To assess the scale of breakdown due to pit corrosion, a parameter denoted DOP (degree of pit corrosion intensity) is often used, where DOP is defined as the ratio percentage of the corroded surface area to the original plate surface area, namely (Paik et al. [22, 23]),

$$DOP = \frac{1}{ab}\sum_{i=1}^{n} A_{pi} \times 100(\%), \tag{1}$$

where n is the number of pits, A_{pi} is the surface area of the *ith* pit, a is the plate length, and b is the plate breadth.

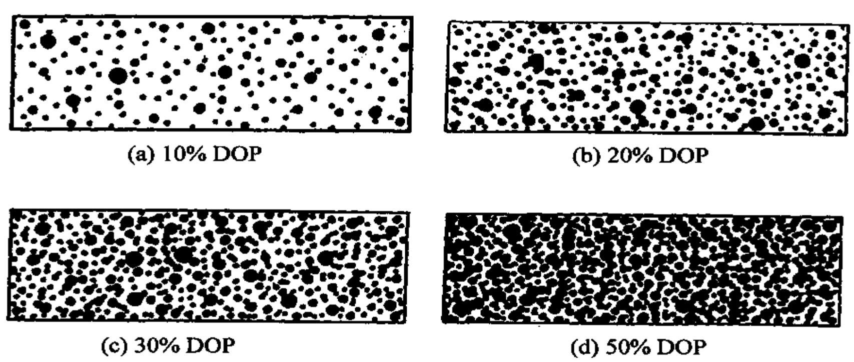

Figure. 2. Pitting Intensity Diagrams (Paik et al. [22, 23]) (DOP = Degree of Pit Corrosion Intensity as a Ratio of the Pitted Surface Area to the Original Plate Surface Area): (A) 10% DOP; (B) 20% DOP; (C) 30% DOP; (D) 50% DOP

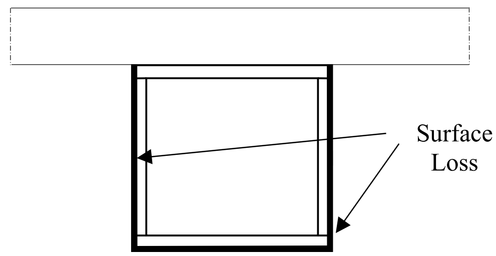

Figure 3. Model of Corroded Non-Composite Steel Box Girder Cross-Section

Figure 2 shows samples of pit corrosion damage distribution in plates (Paik et al. [22, 23]). Although the distribution of the pit corrosion on the plates is scattered, it can be seen that the shape of the corrosion is typically circular (Paik et al. [22, 23]; Nakai [24]). The maximum diameter of localized corrosion may be in the range of 25-80 mm for the marine immersion corrosion of steel (Daidola et al. [25]), with the lower values more likely.

Our investigation of the box girder section assumes that all sides of the girder will corrode uniformly and in a localized manner, 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 3. Figure 3 shows that general corrosion uniformly decreases the plate thickness of both sides and the bottom flange of the box girder section. In the same way, it is also assumed that pitting corrosion, which is uniformly distributed, affects the strength of the following plates by the different degrees of intensity shown in Figure 4. Finally, it is assumed that the pit diameter varies from 10 to 80 mm, and the distance between the adjacent pits centers is constant. The depth of pit corrosion will of course vary by time.

2.2 Probabilistic Corrosion Rate Modeling

In reliability analysis based on the ultimate steel box girder strength of corroded bridges, a probabilistic corrosion rate estimation model needs to be established in advance. Kayser and Nowak [26] collected data on corrosion performance of actual steel bridges.

There is considerable variability in corrosion losses, it is therefore appropriate to consider these quantities as random variables with parameters that change with time. A distinguishing feature of the models developed recently for general corrosion loss and for maximum pit depth under marine immersion conditions is that they are based on fundamental concepts, both in corrosion science and in marine bacteriology. Both these fields are important because it has been recognized that the corrosion process changes from being controlled by the rate of metal oxidation by oxygen in the early period of exposure to being controlled by the rate of metabolism of anaerobic bacteria under longer-term exposures. The models have been calibrated in terms of their parameters using data available in the literature. The models, both for corrosion loss and pit depth, are shown in Figure 5 together with the parameters used to describe them. These parameters are summarized in Table 1. Note that the initial corrosion rate parameter r_0 does not appear for pitting since pitting of considerable depth (around 100 *mm*) occurs usually within days of exposure (Melchers [3]; Melchers and Jeffrey [4]) (for further information, refer to (Melchers and Jeffrey [4])).

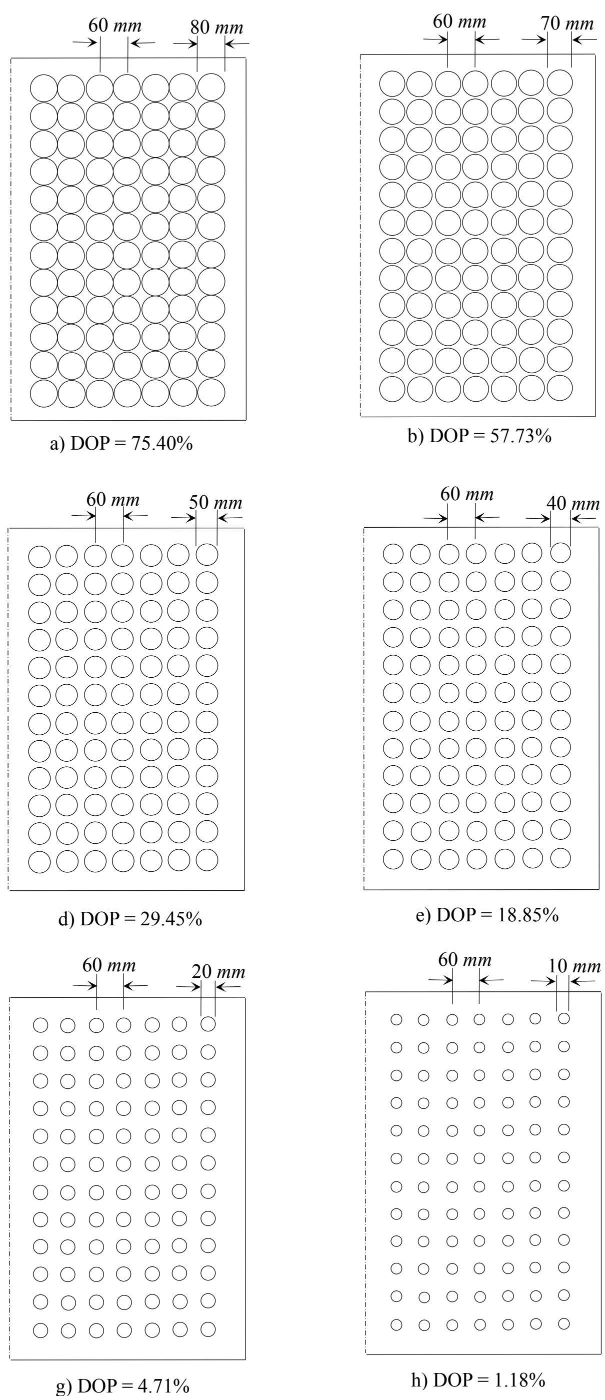

Figure 4. Typical Models of Pit Distribution

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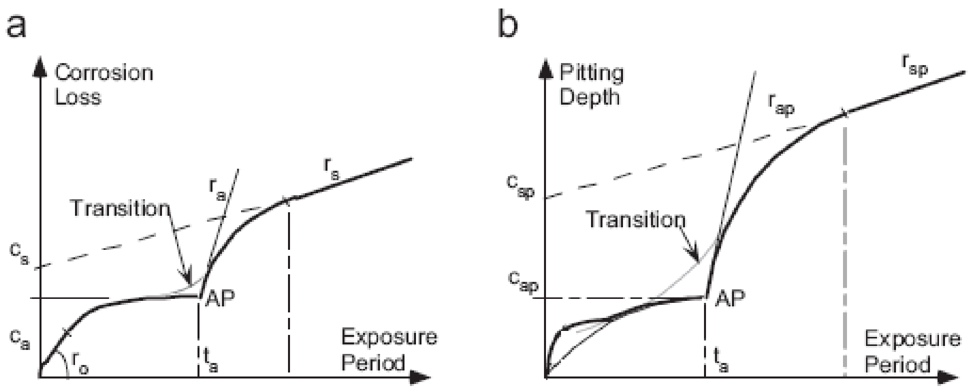

Figure 5. Model for the Mean Value Function for (A) General Corrosion Loss (in mm) and (B) Pit Depth (in mm) Versus Exposure Time (in year) (Melchers [3]; Melchers and Jeffrey [4])

Table 1. Calibration of Model Parameters for General and For Pitting Corrosion under Immersion Conditions In Unpolluted Low Velocity Shallow Waters As a Function of Annual Average Seawater Temperature *T* (Melchers and Jeffrey [4])

Model parameter	General corrosion	At 10 °C	Pitting corrosion	At 10 °C
r_0 (mm/year)	$r_0 = 0.076\exp(0.054T)$	0.13	—	
c_a (mm)	$c_a = 0.32\exp(-0.038T)$	0.22	$c_{ap} = 0.99\exp(-0.052T)$	0.59
t_a (year)	$t_a = 6.61\exp(-0.088T)$	2.75	$t_a = 6.61\exp(-0.088T)$	2.75
r_a (mm/year)	$r_a = 0.066\exp(0.061T)$	0.12	$r_{ap} = 0.596\exp(0.0526T)$	1.01
c_s (mm)	$c_s = 0.141 - 0.00133T$	0.13	$c_{sp} = 0.641\exp(0.00613T)$	0.68
r_s (mm/year)	$r_s = 0.039\exp(0.0254T)$	0.05	$r_{sp} = 0.353\exp(-0.0436T)$	0.23

3. BOX GIRDER ULTIMATE STRENGTH MODELING

To identify a performance function it is needed to estimate an appropriate resistance formula. This study employed the analytical approach suggested by Paik and Mansour [27] to calculate the ultimate strength of a box under bending conditions. It is often observed in nonlinear finite element calculations that a box 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 will also often fail, although the material around the final neutral axis will remain essentially in an elastic state.

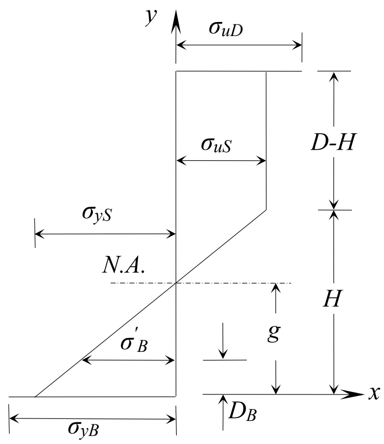

Figure 6. Longitudinal Stress Distribution over the Box Cross-section at the Overall Collapse State suggested by Paik and Mansour [27]

Based on these observations, Paik and Mansour [28] assumed a credible distribution of the longitudinal stresses in the box section at the overall collapse state shown in Figure 6. On the basis of this distribution, they then derived an explicit analytical formula for the corresponding resistive moment. The accuracy of the formula was then verified by comparison with both experimental and numerical results. The resulting expressions for the ultimate bending strength of a double-bottomed box are given by the following Paik and Mansour [28].

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

where

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

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

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

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

where

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

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

To calculate the ultimate moment capacity of the box using equation 2 or 3, the ultimate strength of the compression flange and the side structure in the vicinity of the compression flange, both of which are either stiffened panels or lacking in stiffeners, must be known. Theoretically, the idealized failure modes of a stiffened panel under compressive loads can be categorized into six classes (Paik and Thayamballi [29]):

- Mode I: Overall collapse of plating and stiffeners as a unit
- Mode II: Biaxial compressive collapse without failure of support members
- Mode III: Beam – column type collapse
- Mode IV: Local buckling of the stiffener web (after failure of the plating between the stiffeners)
- Mode V: Tripping of the stiffener
- Mode VI: Gross yielding

The collapse of a stiffened panel may be said to occur at the lowest value of the ultimate load calculated from one of these six collapse patterns. As an alternative, a number of simplified formulas for predicting the ultimate compressive strength of stiffened panels are available in the literature (Paik and Thayamballi [28]), but the realistic calculation of ultimate strength considering all possible modes and their interactions remains a relatively complicated task. In this regard, Paik and Thayamballi [29] derived an empirical formula to predict the ultimate compressive strength of stiffened panels on the basis of data from a total of 130 collapse tests on stiffened plates with usual levels of initial imperfection. The formula is expressed as a function of plate slenderness ratio β' and column (stiffener) slenderness ratio λ' (for further information, refer to Paik and Thayamballi [29]):

$$\sigma_u / \sigma_y = \left[0.995 + 0.936\lambda'^2 + 0.17\beta'^2 + 0.188\lambda'^2\beta'^2 - 0.067\lambda'^4 \right]^{-0.5} . \qquad (4)$$

It should be noted that the foregoing formula implicitly includes the effects of initial imperfections at a moderately large level. In addition, the ultimate strength of an imperfect unstiffened plate under compression stress may be predicted as a function of the plate slenderness ratio, as follows (Paik *et al*. [30]).

$$\sigma_u / \sigma_y = \begin{cases} -0.032\beta'^4 + 0.002\beta'^2 + 1.0 & \text{for} \quad \beta' \le 1.5 \\ 1.274 / \beta' & \text{for} \quad 1.5 < \beta' \le 3.0 \\ 1.248 / \beta'^2 + 0.283 & \text{for} \quad \beta' > 3.0 \end{cases} . \qquad (5)$$

For convenience, the illustrative calculations presented in this study employ equation 5 to predict the ultimate compressive strength of the representative unstiffened plate at the compressive flange or side structure of the box.

4. EFFECTS OF PIT CORROSION ON PLATE ULTIMATE STRENGTH

Corrosion wastage can reduce the ultimate strength of bridge plates. As previously mentioned, two types of corrosion damage are usually considered, namely, general (or uniform) and localized corrosion. The former reduces plate thickness uniformly, whereas the latter, such as pitting, appears non-uniformly in select regions, such as the side plates in box girder bridges. The ultimate strength of a steel member with general corrosion can be easily predicted, i.e., by excluding the plate thickness loss that results from corrosion. It is proposed here, in contrast, that the ultimate strength prediction of a structural member with pit corrosion be made using a strength knock-down factor approach. A series of experimental and numerical studies on steel-plated structures (Paik and Thayamballi [29]), however, led to the realization that the plate ultimate strength reduction characteristics that are due to general corrosion are quite different from those that are due to pit corrosion. So-called equivalent plate thickness reduction approaches, which represent a pitted plate with an equivalent plate, are sufficient for the accurate prediction of the plate's strength.

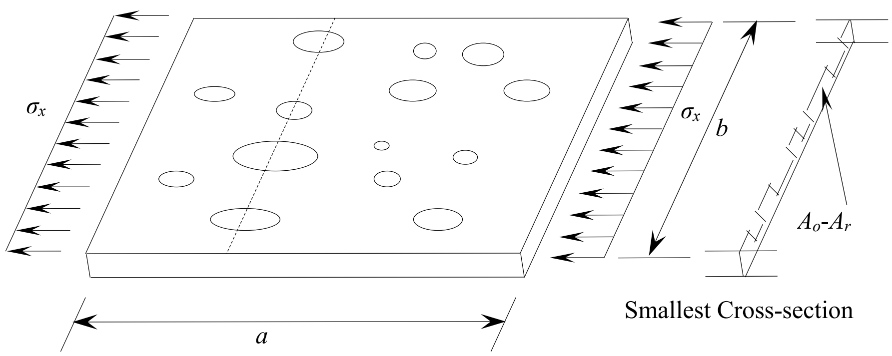

Figure 7. Schematic of Localized Pit Corrosion and Definition of the Smallest Cross-sectional Area (A_0 = cross-sectional area of the intact plate)

Experimental and numerical studies (Paik and Thayamballi [29]) demonstrate that the ultimate strength of a plate with pit corrosion can be estimated with a strength knock-down factor that can be calculated using the following formulation for axial compressive loading.

$$R_{xr} = \frac{\sigma_{xu}}{\sigma_{xuo}} = \left(\frac{A_0 - A_r}{A_0} \right)^{0.73}, \tag{6}$$

5. LOAD MODELING

Two load components are considered: dead load and live load (truck traffic).

5.1 Dead Load Model

Dead load is treated as the normal random variable. The basic statistical parameters are a bias factor, λ, which is the ratio of the mean to nominal value, and coefficient of variation V. Dead load includes the weight of the girders, deck slab, wearing surface, barriers, diaphragms and sidewalk, where applicable. Bias factor 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 the asphalt wearing surface is taken to have a mean value of 75 mm, with $V = 0.25$ (Nowak and Collins [31]; Nowak [32]; Nowak [33]; Nowak and Szerszen [34]; Nowak and Szerszen [35]).

5.2. 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 live load effects 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), traffic volume, number of vehicles on the bridge (multiple presence), girder spacing and mechanical properties of the structural members (Nowak and Collins [31]; Nowak [32]; Nowak [33]; Nowak and Szerszen [34]; Nowak and Szerszen [35]). This study employed the load model developed by AASHTO (AASHTO LRFD [36]) (Figure 8).

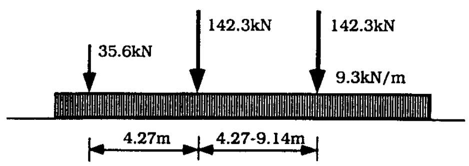

Figure 8. Proposed Nominal Live Loading (AASHTO LRFD [36])

It is assumed that the bias factor, λ, for the live load distribution factors specified in the design code is between 1.10 and 1.20, and that the coefficient of variation, V, is 0.18 (Barker and Puckett [37]). This study adopted a bias factor of 1.15 and a coefficient of variation of 0.18. The dynamic load factor is defined as the ratio of the dynamic load to the static live load. Field measurements show that the dynamic load factor decreases for heavier. Here, the dynamic load factor (*IM*) is selected on the basis of the AASHTO specifications. The design live load in AASHTO (AASHTO LRFD [33]) is specified as the effect of the design truck shown in Figure 8 superimposed with a uniformly distributed load of 9.3 kN/m. The live load distribution for interior and exterior girders can be estimated using the AASHTO specifications (for further information, see (Barker and Puckett [37])).

6. UNCERTAINTY ASSESSMENT AND RELIABILITY CALCULATION

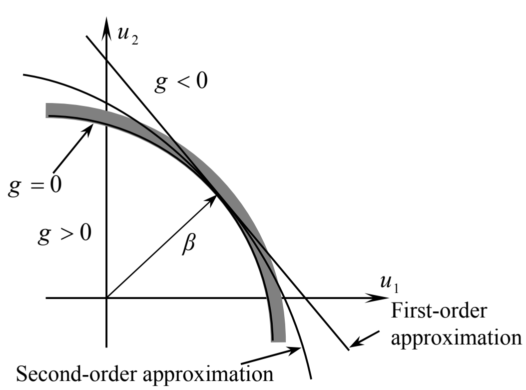

Figure 9. First- and Second-order Reliability Methods

The aim here is to calculate the probability of failure, and hence its complement, reliability, that is related to the ultimate strength of a box girder bridge acted upon by an extreme total bending moment during its lifetime. Box girder strength declines over time because of corrosion. Thus, the reliability measure also reduces with time. As the theory of reliability analysis is discussed in many studies [e.g., (Nowak and Collins [31]; Mansour [38]; Achintya and Mahadevan [39]; Lemarie [40])], only a very brief description is given here. The probability of failure can generally be calculated as follows.

$$P_F = \int_{f(X)\le 0} p_x(X)dx \,, \tag{7}$$

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 $f(X)$ is the limit state function, defined such that negative values imply failure. Reliability analysis can be performed through numerical integration, the simulation technique or approximate methods. Because $f(X)$ is usually a complicated nonlinear function, it is not easy to perform the integration of equation (7) directly. Although the simulation technique may be time-consuming because of the small probabilities involved in the analysis, it has become popular in recent years due to the development of such variance reduction techniques as importance sampling (Sarveswaran and Roberts [15]). Therefore, the equation is normally solved through simulation techniques or approximate procedures Nowak and Collins [31]; Mansour [38]; Achintya and Mahadevan [39]; Lemarie [40]).

In the approximation methods (Figure 9), the limit state surface is usually approximated at the design point by either a tangent hyper plane or hyper parabola, which simplifies the mathematics related to the calculation of failure probability. 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. The reliability analysis in this study was performed using FORM analysis.

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

$$P_F = \phi(-\beta) \,, \tag{8}$$

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):

$$f(x) = Z_1 M_u - M_D - M_L \le 0 \,, \tag{9}$$

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

Z_1 = a variable modeling the uncertainty in estimating the moment capacity.

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

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

It seems that there are six types of random variables to be characterized.

7. APPLICATION EXAMPLE

To demonstrate the application of the proposed procedure, a hypothetical steel box girder bridge is selected from an extensive parametric study aimed at the design of box girder bridge components.

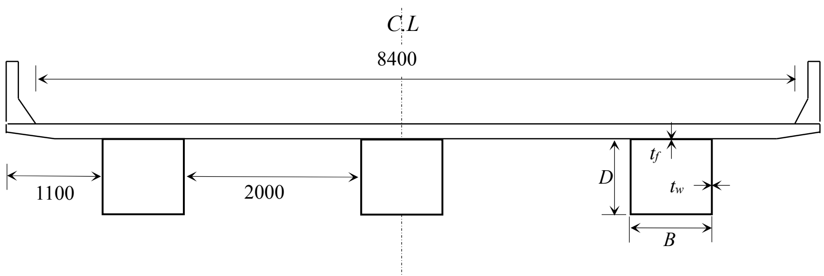

Figure 10. Typical Cross-Section of Box Girder Bridge (dimensions in mm)

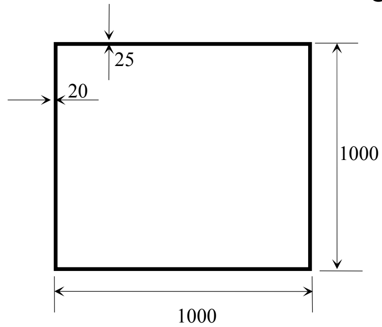

Figure 11. Dimensions (in mm) of Cross-Section of Box Girder

Table 2. Values Used in Calculations (lognormal distributions)

Parameters	Mean μ	Standard deviation σ
Modulus of elasticity for steel, E	2.1×10^5 N/mm^2	2.1×10^4 N/mm^2
Yield stress in steel, σ_y	350 N/mm^2	35 N/mm^2
Model uncertainty variable, Z_1	1.0	0.1

It is assumed that the bridge is not protected against corrosion. It has a simple span of 20 *m* and two lanes with traffic flowing in the same direction. The cross-section is shown in Figs. 10 and 11. The material parameters are assumed to be log-normally distributed, and the mean values and standard deviations are shown in Table 2. The thicknesses of the deck and asphalt are 250 mm and 75 mm, respectively. The lifetime, T, chosen is 75 years. Deterministic analysis showed that, for each girder, prior to any corrosion, the nominal moment capacity is M_n = 9725 kN-m. In the probabilistic analysis, M_n is calculated using the statistical parameters shown in Table 2.

7.1 Dead Load

The mean value of the design dead load bending moment of the steel box girder is calculated, with the results shown in Table 3 for interior and exterior girders. To calculate the dead load for each girder, 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 the components, as shown in Table 3.

Table 3. Dead Load for Girders (normal 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

7.2 Live Load

Based on the specifications provided in Section 5.2, 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

7.3 Results

Probabilistic analysis was carried out to calculate the ultimate strength, reliability and probability of failure of the corroded box girders as the bridge ages. Figure 12 shows the trends of variation in the ultimate moment strength versus time. It can be seen that the ultimate bending strength of the corroded box girders is reduced with an increase in the age of the bridge. In addition, as expected, the ultimate moment decreases with an increase in the DOP at the same time.

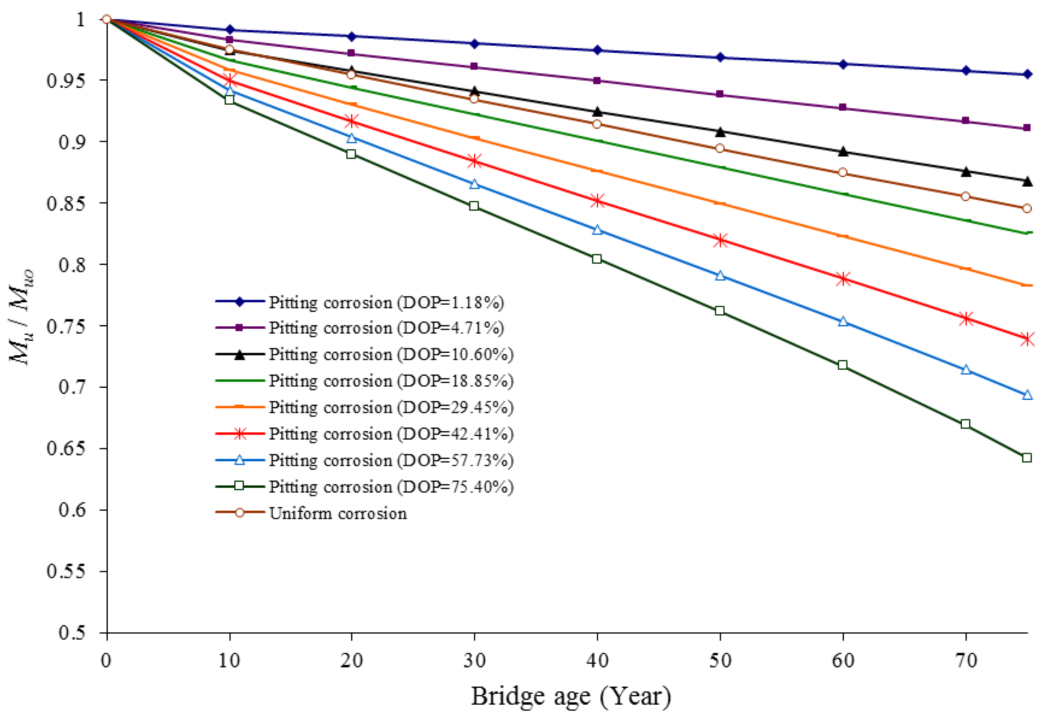

Figure 12. Variation of Box Ultimate Strength Mean Values with Age

Reliability analysis is performed based on FORM analysis. The reliability indices for two girders (interior and exterior) of a highway bridge that is assumed to be uniformly corroded and localized with different DOP are demonstrated using reliability software, with the results shown in Figs. 13 and 14.

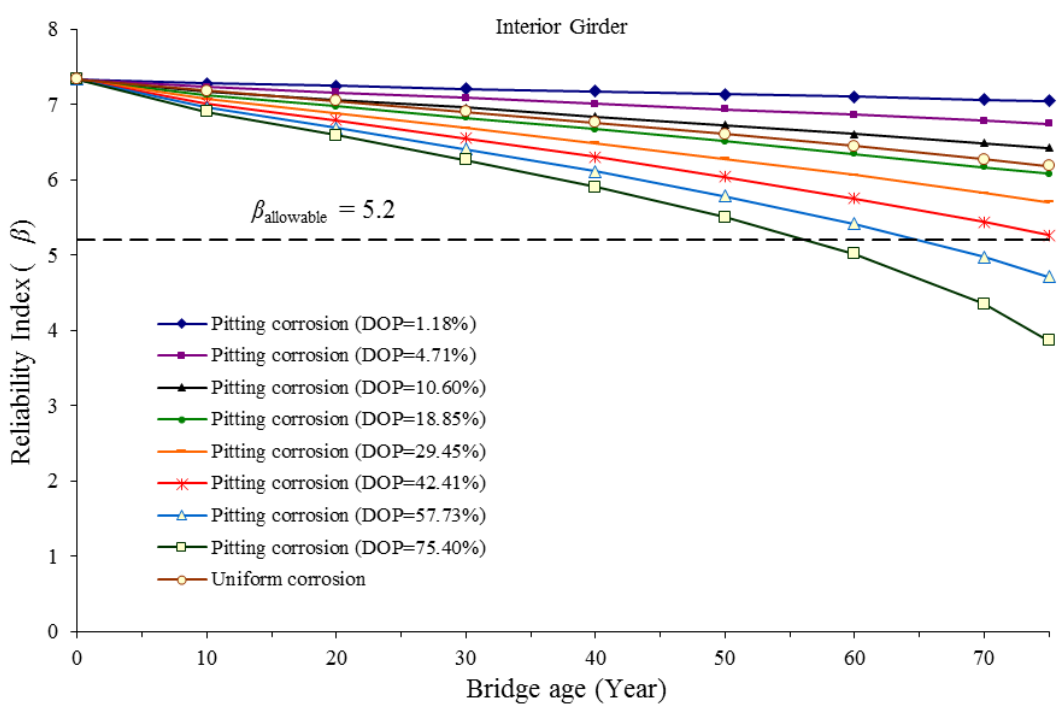

Figure 13. Variation in Reliability Index with Age for Interior Girder

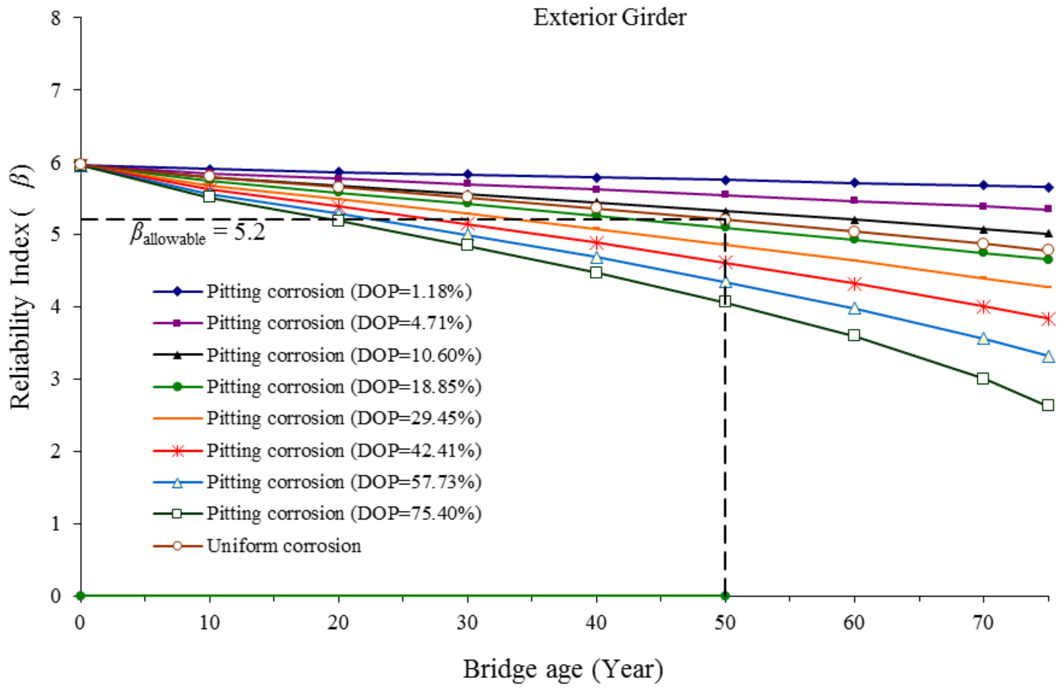

Figure 14. Variation in Reliability Index with Age for Exterior Girder

The corresponding probabilities of failure for the two girders (interior and exterior) for different pitting corrosion intensities (DOP) and uniform corrosion are also presented in Figs. 15 and 16.

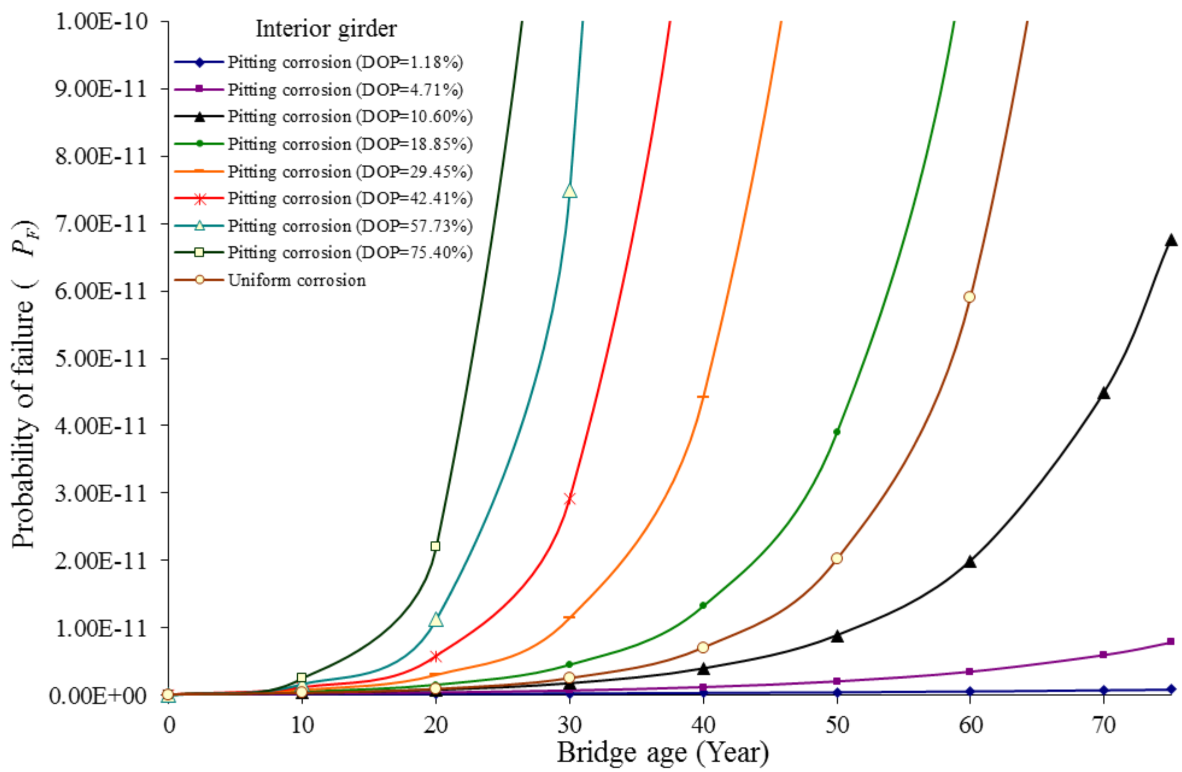

Figure 15. Variation in Probability of Failure with Age for Interior Girder

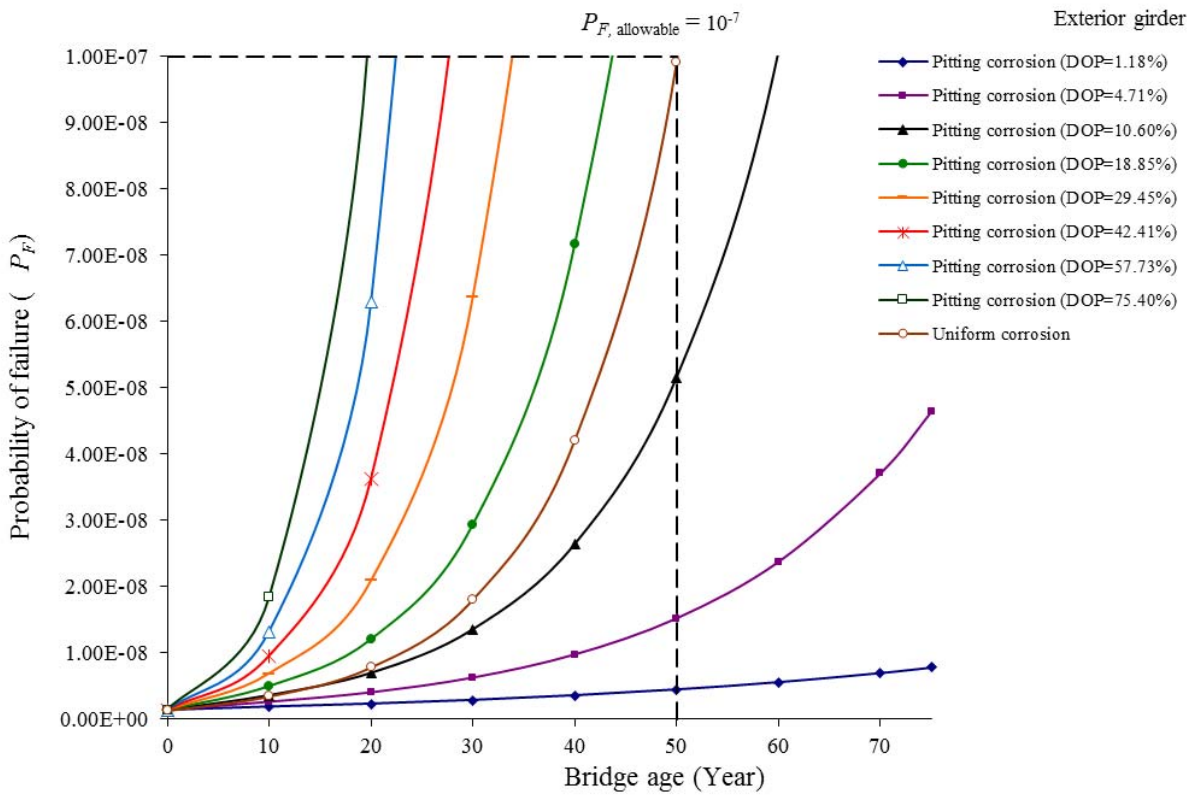

Figure 16. Variation in Probability of Failure with Age for Exterior Girder

This study considers the effects of pitting and uniform corrosion on the load-carrying capacity and reliability of a corroded steel box girder. The results of the FORM analysis method plotted in Figs. 13-16. These figures also give the minimum reliability index or maximum probability of failure for assessment of the ultimate strength reliability of corroded steel box girder bridges.

8. ACCEPTANCE LEVEL OF RELIABILITY

8.1 Approaches to Establishing Acceptance Level

To determine the latest time for the repair intervention of girders, it is first necessary to establish an acceptance level of reliability below which they may be considered unsafe. Not only the accuracy of resistance and load modeling has an influence on reliability but also there are several factors which cannot be modeled in structural reliability analysis. Practicing engineers generally prefer to quantify the reliability level that is implicit in current bridge codes and standards, which have a proven safety record, using probabilistic analysis and then employ this level as the acceptance level of reliability. This method is known as "calibration to existing codes and standards," and is widely used to establish target reliability levels for design situations. The calibration procedure is discussed in many textbooks, e.g., Melchers [41]. It has also been applied to the quantification of the reliability levels implicit in bridge design and assessment codes (Nowak and Lind [42]; Flint et al. [43]; Chryssanthopoulos and Micic [44]). It is also recommended that acceptance levels be based on the consequences of failure and the nature of the failure mode. Therefore, the allowable reliabilities shown in Table 5, which are based on the type of failure has been used in this study.

Table 5. Target (or acceptance) Reliability Levels (Chryssanthopoulos and Micic [44])

Failure consequences	Ductile failure with reserve strength	Ductile failure without 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

8.2 Determination of Latest Time to Intervention

The foregoing approaches were taken into account in selecting the time for repair intervention. Applying the allowable reliability index, for example 5.2 (brittle failure and very serious) or corresponding probability of failure 10^{-7}, and employing reliability analysis results, the earliest time for the repair of exterior girders in a marine environment is around 50 years in case of uniform corrosion (see Figs. 14 and 16). By using the abovementioned procedure it can be found that there is no failure for interior girders during their life time, 75 years, (see Figs. 13 and 15). In other words, if such a bridge is constructed now (in 2010), then its exterior girders should be repaired in fifty years' time (in 2060). If it is already in existence and is older than fifty years, then it is unsafe to use the assumptions and procedures reported here.

9. CONCLUDING REMARKS

This study has developed a probability-based procedure for selection of the critical time at which bridge girders should be repaired during their service life. Two types of corrosion are considered, namely, uniform corrosion and pitting corrosion with different degrees of intensity. Reliability indices are calculated using available load and resistance models. A probabilistic ultimate strength model is developed by employing the simple analytical formulation derived by Paik and Mansour [27]. Bridge girders are subject to a loss in capacity over time due to corrosion. The live load can be distributed to girders using the guidance formula for the highway bridge design. Time-dependent reliability indices can serve as the basis for selecting the latest time to repair or renew individual girders, with the critical components identified as those associated with the lowest reliability indices.

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

1. The results show that by calculating the DOP of a corroded bridge girder, it can be estimated its time-variant reliability profiles, and hence it can be predicted the repair or renewal time.
2. As expected, the ultimate strength of corroded box girders may decrease with time, and in a certain time it will decrease with an increase in DOP.
3. The ultimate limit analytical formula (Paik and Mansour [27]) described in this study, and applied to the prediction of the ultimate strength of box girders, is useful for evaluating the time-variant steel box girder strength reliability of corroded bridges.
4. The procedures developed herein will be useful in assessing the ultimate strength reliability of aging steel box girder bridges by taking into account the degradation of plate members due to corrosion. This procedure is not only applicable to practicing engineers, but is also presented as a scientific method for estimating the longevity of bridges.

NOTATIONS

The following symbols are used in this paper.

A_B, A'_B, A_D = sectional area of outer bottom, inner or upper
A_S = half of the sectional area of the side structure, including any longitudinal stiffeners
E = Young's modulus
$f(x)$ = ultimate limit state function
g = height of the neutral axis
P_F = probability of failure
M_u, M_{uo} = random variable representing the ultimate strength of a corroded or uncorroded box girder
σ_{xi} = standard deviation of random variable x_i
t = plate thickness of a member and time in years
$\beta' = \frac{b}{t}\sqrt{\frac{\sigma_y}{E}}$ = slenderness ratio of plating between longitudinal stiffeners
$\lambda' = \frac{l}{\pi r}\sqrt{\frac{\sigma_y}{E}}$ = slenderness ratio of a stiffener together with fully effective plating
l = length of the stiffened panel between the transverse support frames
r = radius of gyration of stiffener
β = reliability index
λ = bias factor
μ_x = mean value of random variable x_i
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
ϕ = standard normal distribution function
M_D = dead-load moment
M_L = live-load moment
M_n = nominal bending moment strength
T = lifetime of bridge
IM = dynamic live load
DOP = degree of pit corrosion intensity as a ratio of the pitted surface area to the original plate surface area
n = number of pits
A_{pi} = surface area of the *ith* pit
a = plate length
b = plate breadth
d_r = diameter of the pit
R_{xr} = a factor of ultimate compressive strength reduction due to pit corrosion
σ_{xu} = ultimate compressive strength for a member with pit corrosion
σ_{xuo} = ultimate compressive strength for an intact (uncorroded) member
A_0 = original cross-sectional area of the intact member
A_r = cross-sectional area involved in pit corrosion at the smallest cross-section (see Fig. 7)

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