RELIABILITY OF DETERIORATING STEEL BOX-GIRDER BRIDGES UNDER PITTING CORROSION

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ABSTRACT: Deteriorated bridges are subjected to time-variant changes of resistance. Corrosion is one of the most important types of deterioration in steel bridges. The consequence is a reduction in safety of a bridge. Therefore, it is needed to evaluate procedures for an accurate prediction of the load-carrying capacity and reliability of corroded bridges, in order to make rational decisions about repair, renewal or rehabilitation. This paper presents a highway bridge reliability-based design formulation which accounts for pitting corrosion effects on steel box girder bridges. The study involves the idealization of pitting corrosion, development of resistance models for corroded steel box girders, development of load models, formulation of limit state function, development of reliability analysis method, and development of the time-dependent reliability for corroded steel girders. Numerical example illustrates the application of the proposed approach. The 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, Steel box girders, Pitting corrosion, Load-carrying capacity, Time-dependent reliability, Repair and rehabilitation

1. INTRODUCTION

Bridges deteriorate over the time. The main causes of deterioration in bridges are fatigue and environment. For steel bridges one of the most important forms of deterioration is corrosion (Sharifi and Paik [1, 2]; Sharifi and Rahgozar [3-5]; Kayser and Nowak [6]). The major effect of corrosion is the loss of material section resulting in a reduction of structural load-carrying capacity and reliability. Deficient bridges are needed to be repaired or replaced. In addition, the repair or maintenances cost of a bridge is of great importance. Therefore, it is essential to develop a procedure in order to accurate prediction of load-carrying capacity for existing bridges (Sharifi and Paik [1, 2]; Czarnecki and Nowak [7]; Cheung and Li [8]). Several approaches to quantify the increasing dependency of our life and economy on civil infrastructures indicate that the service loss due to their malfunctioning would be extremely expensive. Therefore, this type of loss should be included in assessment of civil infrastructures. Performing maintenance interventions on civil infrastructures is a requirement for maintaining the safety and functionality of the ground transportation system (Kong and Frangopol [9]).

Load effect and resistance of a bridge are random variables. Therefore, it is common to measure bridge structural performance in terms of reliability and probability. The reliability methods allow for consideration of uncertainties associated with material properties, geometry and dimensions, loads and environmental conditions (Melchers and Jeffrey [10]; Melchers [11]; Kayser and Nowak [6]).

Kayser and Nowak [6] developed a damage model which evaluates the reliability of a corroded steel girder bridge over time. This model was used for a uniform corroded I-shape steel girder in different environments. It was found that a rural environment, with exposure to pure water, negligibly affects a bridge. An urban environment, with automobile and industrial pollutants, moderately environment, with salt from sea water or deicer use, significantly reduces safety over a

50-year life. In another research by Czarnecki and Nowak [7] the system and member reliability of deteriorated steel girder bridges with uniform corrosion has been evaluated, and it was found that the member reliability give conservative indices. Cheung and Li [8] investigated the serviceability reliability of damaged steel I-girder bridges that affected under uniform corrosion. It has been found that the serviceability of a corroded steel bridge may deteriorate drastically during its service life, even though the nominal maximum deflection only shows little increase in the same period. Thoft-Christensen [10] defined bridge reliability states and proposed a reliability-based approach to bridge maintenance planning.

The objective of this paper is to present an approach to quantifying the reduction in load-carrying capacity and safety for corroded steel box girder bridges. The discussion will concentrate on the pitting corrosion of simple span steel box girder bridges. A member reliability model is developed based on the bending failure mode. To illustrate the approach, reliability analysis is carried out for a steel box girder bridge. The calculations are performed for various lifetimes in a marine environment. 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 MODELING

There are several forms of corrosion. In this paper, pitting corrosion is considered. Corrosion affects various structural parts differently. It seriously reduces the serviceability of a bridge. In the result, there is also an increased exposure of the superstructure to a corrosive environment. The dynamic loads may also be increased. Corrosion of the superstructure may not only cause fracture, but also yielding or buckling of members. In particular, three possible changes to a steel girder bridge can be considered, an increase in stress, a change in geometric properties, and a buildup of corrosion products (Czarnecki and Nowak [7]; Kayser and Nowak [6]).

2.1 Pitting Corrosion Idealization

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, pitting corrosion damage idealization is considered.

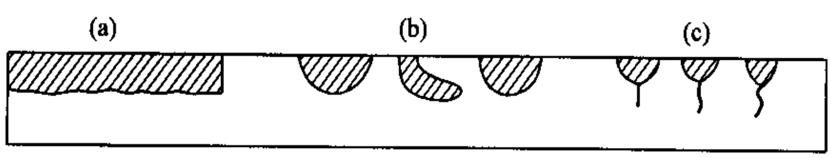

Figure 1. Typical Modes of Corrosion Damage: (a) General (Uniform) Corrosion; (b) Localized Corrosion; (c) Fatigue Cracks Arising from Localized Corrosion

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,

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

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 shows samples of pit corrosion damage distribution in plates. 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. [13, 14]; Nakai et al. [15]). 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. [16]), with the lower values more likely.

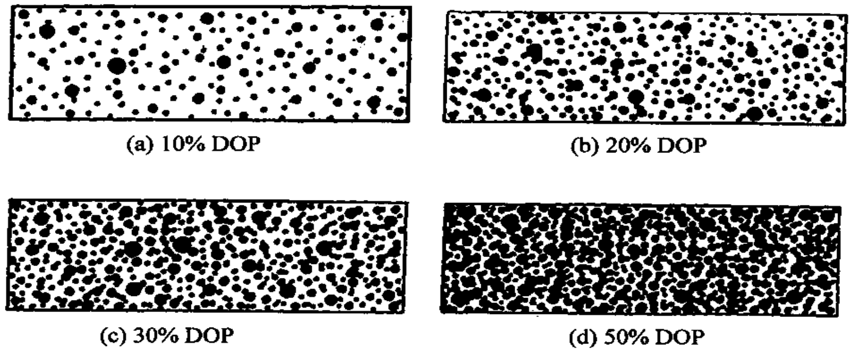

Figure 2. Pitting Intensity Diagrams (Paik et al. [13, 14])
(DOP = Degree of Pit Corrosion Intensity as a Ratio of the Pitted Surface Area to the Original Plate Surface Area): (a) 10% DOP; (b) 20% DOP; (c) 30% DOP; (d) 50% DOP

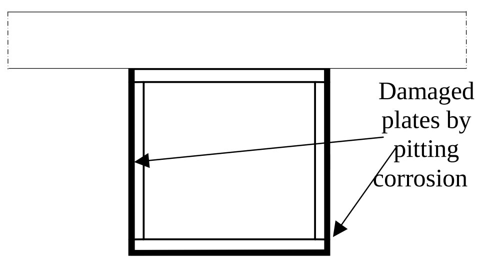

Figure 3. Model of Corroded Non-composite Steel Box Girder Cross-section

It is assumed that all sides of the girder will corrode 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 pitting corrosion which is uniformly distributed decreases the strength of both sides and the bottom flange of the box girder section 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.

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. Data on corrosion performance of actual steel bridges have been collected by Kayser [17]. As expected, corrosion occurs where water is accumulated. For steel-girder bridges, this happens at leaking deck joints. Moreover, Corrosion is influenced by the environment, i.e., the amount of moisture in the air and the presence of salt. Therefore, the geographic location is of vital importance when planning the maintenance of a steel bridge. It has been observed that the rate of corrosion can be different in different environments, and for different girders. For example, in highway overpass, girders are exposed to a mixture of salt, snow and water splashed by trucks. The highest concentration of this aggressive medium is on the exterior girder and the concentration of salt and/or water decreases in the direction of traffic (Kayser and Nowak [6]).

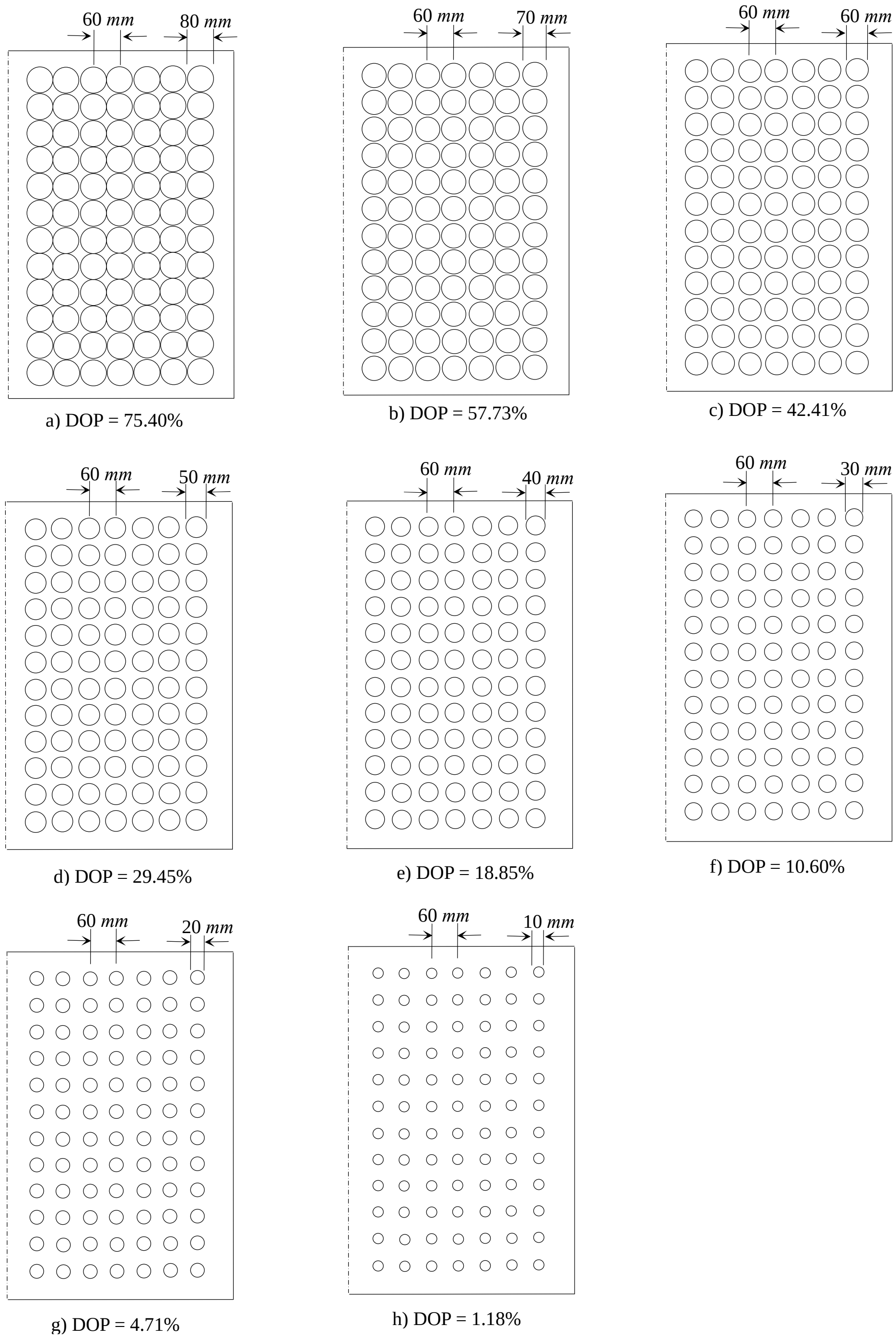

Figure 4. Typical Models of Pit Distribution

Table 1. Statistical Parameters for *A* and *B* (Sommer et al. [18])

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	

Research has shown that corrosion propagation can be modeled with a good degree of approximation with the following exponential function (Komp [19]).

$$C(t) = A t^{B}, \tag{2}$$

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 gives the mean values, coefficients of variation, and coefficients of correlation for A and B. It shows, as expected that the parameters for weathering steel are smaller than for carbon steel, since corrosion develops slower in weathering steel, and it shows that in most cases the parameters are the lowest in rural environments and highest in urban environments. It was agreed that corrosion of steel is affected by several factors such as type of steel, surface protection, environmental effects, and presence of pollutants, crevices and stress (Czarnecki and Nowak [7]).

3. BOX GIRDER ULTIMATE STRENGTH MODELING

There are three possible approaches to the development of a simple formula for the prediction of box girder ultimate strength, and this formula can also be applied to estimates of the ultimate moment of box girders.

The first is an analytical approach that is based on an assumed stress distribution over the box section, from which the box's moment of resistance is theoretically calculated by taking into account buckling in the compression flange and yielding in the tension flange. The second is an empirical approach in which an expression is derived on the basis of experimental or numerical data from scaled box models. The third is a linear approach in which the behavior of the box up to the collapse of the compression flange is assumed to be linear, and the ultimate moment capacity of the box is basically expressed as the ultimate strength to yield strength ratio of the compression flange multiplied by the first yield moment of the box girder (Paik and Thayamballi [20]).

The third approach is quite simple, but its degree of accuracy is often relatively poor, because the after-buckling of the compression flange causes the neutral axis of the box to change position. Empirical formulas (the second approach) may provide reasonable solutions for a conventional box, but care must be exercised in using them for new box types or outside the limits of the data on which they are based. Analytical formulations (the first approach), in contrast, can be applied to most situations of interest.

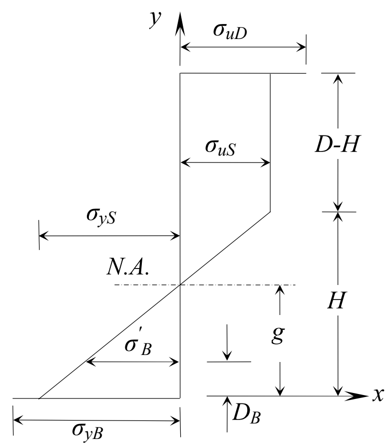

Figure 5. Longitudinal Stress Distribution Over the Box Cross-section at the Overall Collapse State Suggested by (Paik and Thayamballi [20])

This study employed the analytical approach 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. Based on these observations a credible distribution of the longitudinal stresses in the box section at the overall collapse state derived and shown in Figure 5 (Paik and Thayamballi [20]). 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.

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

where

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

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

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

$$\begin{aligned} M_u = & -A_D \sigma_{uD} (D - g) - \frac{A_S}{D} (D - H)(D + H - 2g) \sigma_{uS} \\ & - A_B g \sigma_{yB} - \frac{A_S H}{3D} \left[(2H - 3g) \sigma_{uS} - (H - 3g) \sigma_{yS} \right], \end{aligned} \tag{4}$$

where

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

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

To calculate the ultimate moment capacity of the box using Eq. 3 or 4, the ultimate strength of the compression flange and the side structure in the vicinity of the compression flange must be known. 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. [21]).

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

It should be noted that the foregoing formula implicitly includes the effects of initial imperfections at a moderately large level. For convenience, the illustrative calculations presented in this study employ Eq. 5 to predict the ultimate compressive strength of the representative unstiffened plate at the compressive flange or side structure of the box.

4. PLATE ULTIMATE STRENGTH UNDER PIT CORROSION WASTAGE

Corrosion wastage can reduce the ultimate strength of bridge section's plates. 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 nonlinear analytical and Experimental studies by Paik et al. [13] demonstrate that the ultimate strength of a plate with pit corrosion can be estimated with a strength reduction factor that can be calculated using the following formulation for axial compressive loading.

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

where

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 and
A_r = cross-sectional area involved in pit corrosion at the smallest cross-section (see Figure 6).

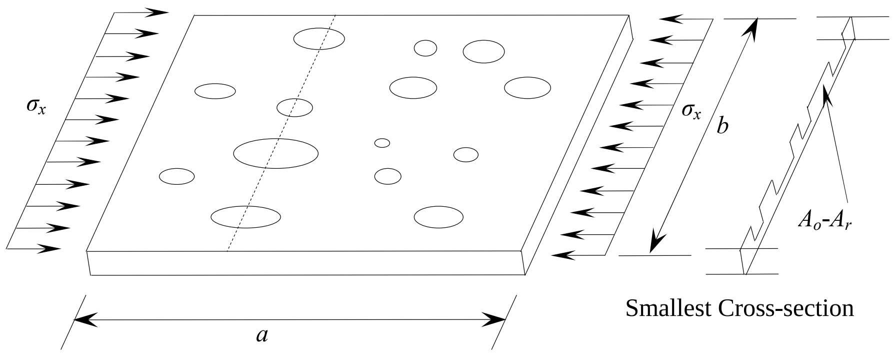

Figure 6. Schematic of Localized Pit Corrosion and Definition of the Smallest Cross-sectional Area (A_0 = Cross-sectional Area of the Intact Plate)

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 [22]; Nowak [23, 24]; Nowak and Szerszen [25, 26]).

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 [22]; Nowak [23, 24]; Nowak and Szerszen [25, 26]). This study employed the load model developed by AASHTO LRFD [27] (Figure 7).

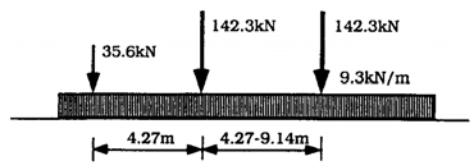

Figure 7. Proposed Nominal Live Loading (HL-93 Live Load in AASHTO LRFD [27])

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 [28]). 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 trucks (Nowak and Collins [22]; Nowak [23, 24]; Nowak and Szerszen [25, 26]). Here, the dynamic load factor (*IM*) is selected on the basis of the AASHTO LRFD specifications [27]. The design live load in AASHTO LRFD [27] is specified as the effect of the design truck shown in Figure 7 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 [28]).

6. RELIABILITY CALCULATION

In all engineering structural system design, uncertainties are unavoidable due to stochastic nature of materials and loads, and imperfect nature of mathematical model. These uncertainties can be accounted only through a reliability analysis. The aim is to calculate the probability of failure, and hence its complement, the reliability related to the ultimate strength of the bridge acted upon by the extreme total bending moment during its lifetime. The bridge strength will reduce with time because of corrosion. Thus the reliability measure will reduce with time. In probabilistic assessment, any uncertainty about a variable, which is expressed in terms of its probability density function, is explicitly taken into account. Reliability analysis begins with the formulation of a limit state function that represents the performance of a structure or element in terms of several basic random variables. Since the theory of reliability analysis is discussed in many references (Nowak and Collins [22]; Mansour [29]; Achintya and Mahadevan [30]; Lemarie [31]; Melchers [32]), only a very brief description is given here. Generally the probability of failure can be calculated as follows:

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

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, defined such that negative values imply failure.

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

$$P_f = \phi(-\gamma)\,, \qquad (8)$$

where ϕ is the standard normal distribution function.

The limit state function for the steel box girders in this example is defined as follows.

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

where

$g(x)$ = the limit state function,
M_u = a random variable representing the resistance ultimate strength,
M_D = a random variable representing the dead load, and
M_L = a random variable representing the live load.

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

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

It would appear that there are five types of random variables to be characterized. In fact, however, there are six, because the thickness value of the member at any particular time is also a function of the two parameters of the corrosion rate (A, B).

Reliability analysis can be performed through numerical integration, the simulation technique or approximate methods. Numerical integration is not performed in this case because of the large dimension and the complexity of the problem. 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 [33]). Therefore, the equation is normally solved through simulation techniques or approximate procedures (Nowak and Collins [22]; Mansour [29]; Achintya and Mahadevan [30]; Lemarie [31]; Melchers [32]).

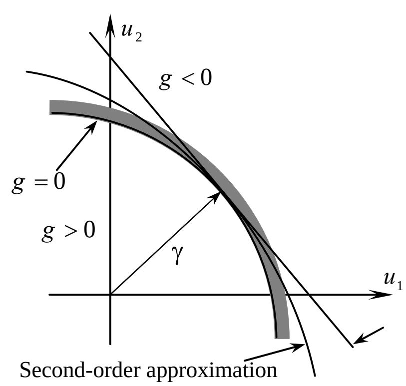

Figure 8. First- and Second-order Reliability Methods

In the approximation methods (Figure 8), 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). Simulation technique consists of obtaining cumulative distribution functions for each and every random variable and simulating the ultimate strength of box plates for combinations of random variable values. It uses randomly generated samples of the input variables for each deterministic analysis, and estimates reliability after numerous repetitions of the deterministic analysis. However for simulation technique to be successful, the sample size should be very large. Hence methods have been proposed to reduce the sample size without however sacrificing any accuracy on reliability. 'Point Estimation Method', 'Response Surface Technique', 'Importance Sampling Procedure', 'Latin Hypercube Sampling' etc., are some of these methods. In this paper, a method called 'Importance Sampling Procedure is proposed for efficient reliability estimation. Since this method is used with the Importance Sampling, the number of simulation cycles required for the analysis can be much reduced.

The approximate methods, FORM and SORM, are efficient methods and known to provide sufficiently accurate results. However, it is known that these methods can in some instances become numerically unstable for complex formulations and one can not always be sure to obtain the global minimum using these methods (Sarveswaran and Roberts [33]). Therefore, the reliability analysis in this study was performed by simulation technique using importance sampling as variance reduction.

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.

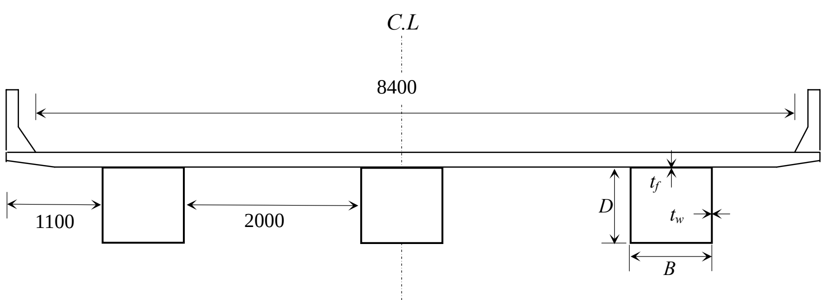

Figure 9. Typical Cross-section of Box-girder Bridge (Dimensions in mm)

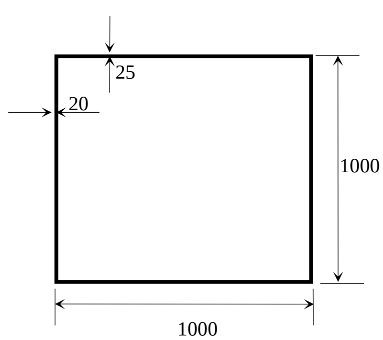

Figure 10. Dimensions (in mm) of Cross-section of Box Girder

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
Corrosion parameter, A ($A < 200 \times10^{-3}$ mm)	70.6×10^{-3} mm	46.6×10^{-3} mm
Corrosion parameter, B ($B < 1.5$)	0.79	0.39

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 Figures 9 and 10. The material and corrosion parameters are assumed to be log-normally distributed (Sharifi and Paik [1,2]; Kayser and Nowak [6]; Czarnecki and Nowak [7]; Cheung and Li [8]; Kayser [17]; Sommer et al. [18]), and the mean values and standard deviations are shown in Table 2. The corrosion parameters chosen correspond to carbon steel in a marine environment (i.e., onshore near the coast) (see Table 1). The thicknesses of the deck and asphalt are 250 mm and 75 mm, respectively. The lifetime, T, chosen is 70 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 11 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.

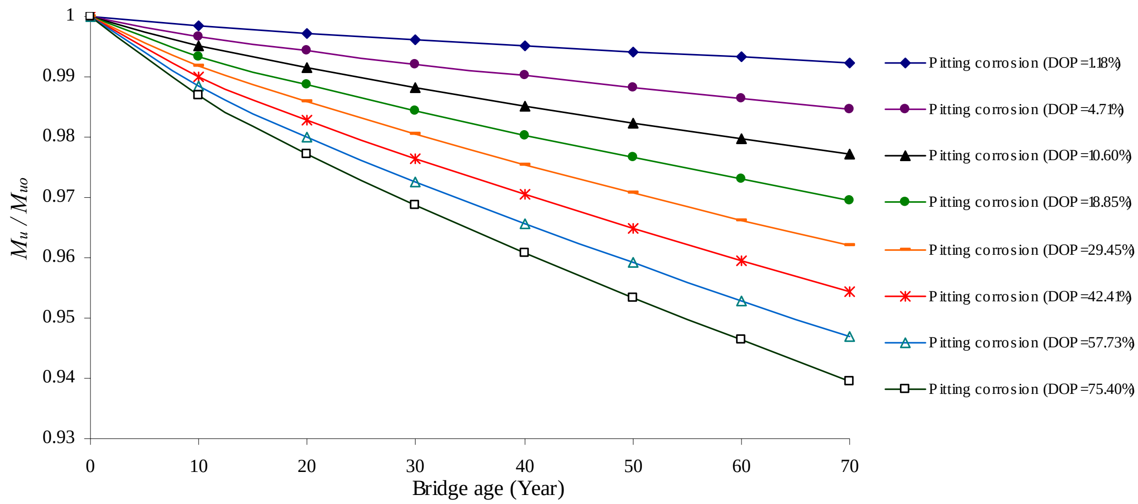

Figure 11. Variation of Box Ultimate Strength with Age

The reliability indices for two girders (interior and exterior) of a highway bridge that is assumed to be corroded localized with different DOP are demonstrated using reliability software, with the results shown in Figures 12 and 13. To derive reliability indices over time, it is usual to first find these indices for certain structural ages and then connect them together.

The reliability indices are calculated by simulation technique using the sampling analysis method for both interior and exterior girders. The results of simulation technique using sampling reliability analysis indicate that up to a bridge age of four years, there is no failure. The corresponding probabilities of failure are also calculated, and are presented in Tables 5 and 6 for interior and exterior girders, respectively. As expected, an increase in the DOP leads to an increase in the probability of failure.

Table 5. Failure Probabilities of Interior Girder

Bridge age (years)	Pitting corrosion (DOP) 1.18%	4.71%	10.60%	18.85%	29.45%	42.41%	57.73%	75.40%
Up to 4	No failure	No failure	No failure	No failure	No failure	No failure	No failure	No failure
4	No failure	No failure	No failure	No failure	No failure	No failure	0.00000451	0.000016
10	0.0000261	0.0000921	0.000137	0.000242	0.000306	0.000308	0.000431	0.000541
20	0.000313	0.000624	0.000851	0.001194	0.001507	0.002012	0.002579	0.002779
30	0.000739	0.001256	0.002038	0.002512	0.002979	0.003624	0.004041	0.004852
40	0.00118	0.002312	0.002868	0.003475	0.004388	0.005162	0.00591	0.006767
50	0.00169	0.002617	0.003779	0.004727	0.005697	0.006672	0.007571	0.008627
60	0.002077	0.003303	0.004733	0.005769	0.00699	0.008003	0.00913	0.009934
70	0.002331	0.003828	0.005283	0.006724	0.00819	0.008955	0.010222	0.011516

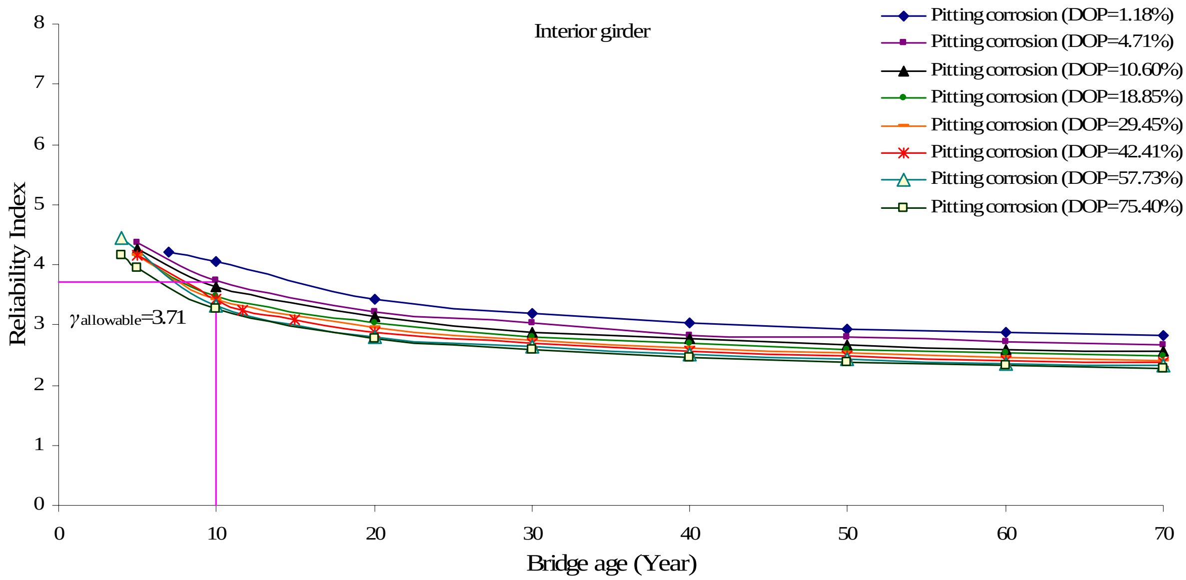

Figure 12. Variation in Reliability Index with Age for Interior Girder

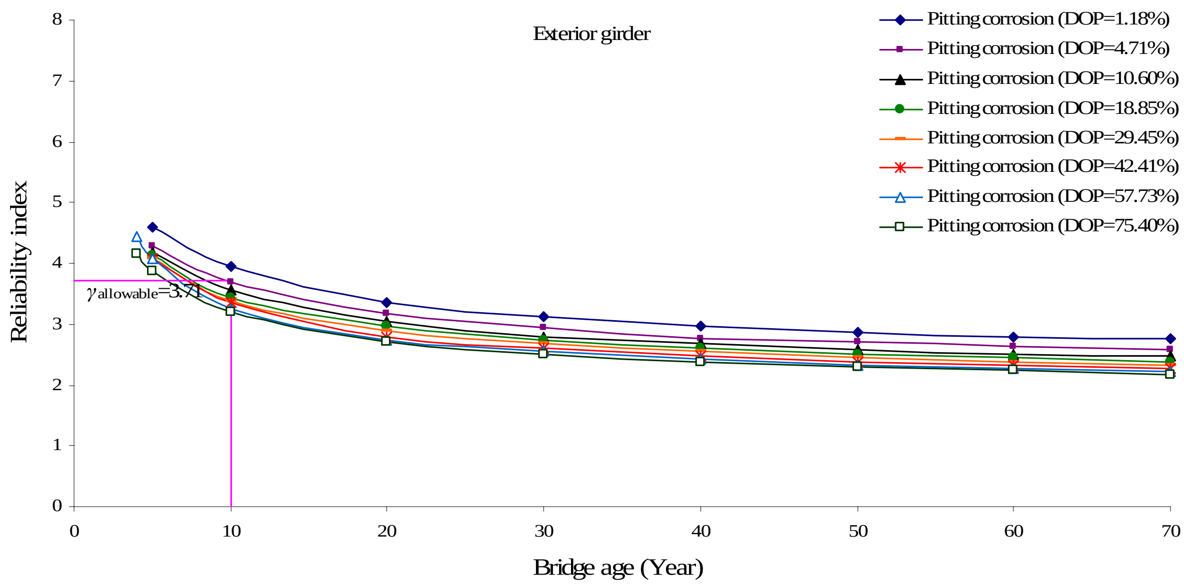

Figure 13. Variation in Reliability Index with Age for Exterior Girder

Table 6. Failure Probabilities of Exterior Girder

Bridge age	Pitting corrosion (DOP)							
(years)	1.18%	4.71%	10.60%	18.85%	29.45%	42.41%	57.73%	75.40%
Up to 4	No failure	No failure	No failure	No failure	No failure	No failure	No failure	No failure
4	No failure	No failure	No failure	No failure	No failure	No failure	0.00000451	0.000016
10	0.0000341	0.000116	0.000193	0.000309	0.000375	0.000393	0.000553	0.000699
20	0.000391	0.000768	0.001119	0.00151	0.001975	0.002589	0.003195	0.003472
30	0.000895	0.001601	0.002629	0.003084	0.003747	0.004535	0.005257	0.006015
40	0.001489	0.00291	0.003624	0.0045	0.005346	0.006444	0.007426	0.008664
50	0.002074	0.003348	0.004785	0.006077	0.007193	0.008597	0.00981	0.010807
60	0.002607	0.004307	0.005959	0.007242	0.008802	0.01022	0.011499	0.012508
70	0.002926	0.00486	0.00673	0.008617	0.010234	0.011289	0.01311	0.014607

This study considers the effects of pitting corrosion on the load-carrying capacity and reliability of a corroded steel box girder. The results of the reliability analysis plotted in Figures 12 and 13 give the minimum reliability index for assessment of the earliest time of the ultimate strength reliability of corroded steel box girder bridges.

8. ACCEPTANCE LEVEL OF RELIABILITY

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 [32]. It has also been applied to the quantification of the reliability levels implicit in bridge design and assessment codes (Nowak and Lind [34]; Flint et al. [35]; Chryssanthopoulos and Micic [36]). 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 7, which are based on the type of failure has been used in this study.

Table 7. Target (or Acceptance) Reliability Levels (Sarveswaran and Roberts [33])

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.

The foregoing approaches were taken into account in selecting the time for repair intervention. For an example, the results for the worst condition of pitting corrosion (DOP = 75.40%) used in assessing the earliest time at which to order a repair. Applying the acceptance levels of the probability of failure, or a reliability index from 3.09 to 5.2, and employing reliability analysis results (Figures 12 and 13), the earliest time for the repair of both interior and exterior girders is around 10 and 4, respectively. In other words, if such a bridge is constructed now (in 2010), then it should be repaired in ten years' time (in 2020). If it is already in existence and is older than ten years, then it is unsafe to use based on the assumptions and procedures reported here, which demonstrates that the repair intervention date.

In order to estimate the repair or renewal time in maintenance schedule of a damaged steel box girder bridge, first, the degree of intensity of pitting corrosion (DOP) must be calculated by the procedure which has been described in section 2. Then a target reliability (acceptance level of reliability below which they may be considered unsafe; e.g. Table 7) needed to be employed in order to calculate the earliest time by using the time-variant reliability profiles (Figures 12 and 13) that have been developed for the bridge.

For an instance, by a visual intervention of a corrosion damaged steel box girder plates bridge, the corroded area and hence the degree of intensity of pitting corrosion (Eq. 1) can be evaluated (e.g., DOP = 4.71%). By using Table 7, if the failure be serious and is ductile with reserve strength, the allowable reliability index will be 3.71. By employing this index for DOP = 4.71% and using Figures 12 and 13, the earliest time for both interior and exterior girders will be 10 years.

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. Pitting corrosion with different degrees of intensity is considered. Reliability indices are calculated using available load and resistance models. A probabilistic ultimate strength model is developed by employing the simple analytical formulation. Bridge girders are subject to a loss in capacity over time due to pitting 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. It should be noted that, the procedure and approach that employed here in order to treat the effect of pitting corrosion on steel box girder bridges strength and reliability could be applicable and interesting.

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

1. As expected, the ultimate strength of corroded box girders may decrease with time, although the degree of change (decrease) is greater in the first few years.

2. The ultimate limit analytical formula 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.

3. The numerical results reveal that the safety and reliability of the corroded steel box girders of a bridge may deteriorate during their service life (Figures 12 and 13), even though the nominal maximum moment may show little decrease in the same period. This phenomenon can be attributed to the high degree of variability in the slow mean corrosion. Even though the mean values of corrosion parameters A and B are moderate, large values close to the upper limits may have a considerable likelihood of occurrence, thus causing a significant reduction in the reliability of the corroded structures.

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	= corrosion parameter or cross-sectional area
A_B, A'_B, A_D	= sectional area of outer bottom, inner or top
A_S	= half of the sectional area of the side structure, including any longitudinal bulkhead
A_{pi}	= surface area of the *ith* pit
A_0	= original cross-sectional area of the intact member
A_r	= cross-sectional area involved in pit corrosion at the smallest cross-section
a	= plate length
B	= corrosion parameter
b	= plate breadth
C	= average corrosion penetration

D	= height of the box
D_B	= height of double bottom
DOP	= degree of pit corrosion intensity as a ratio of the pitted surface area to the original plate surface area
d_r	= diameter of the pit
E	= Young's modulus
g	= height of the neutral axis
$g(x)$	= the limit state function,
IM	= dynamic live load
M_D	= dead-load moment
M_L	= live-load moment
M_n	= nominal bending moment strength
M_u, M_{uo}	= random variable representing the ultimate strength of a corroded or uncorroded box girder
n	= number of pits
P_f	= probability of failure
$p(X)$	= the joint probability density function of the random variables
R_{xr}	= a factor of ultimate compressive strength reduction due to pit corrosion
T	= lifetime of bridge
t	= plate thickness of a member and time in years
V	= Coefficient of variation
β	= slenderness ratio of plating between longitudinal stiffeners
γ	= reliability index
λ	= bias factor
μ_x	= mean value of random variable x_i
ρ	= Coefficient of correlation
σ_B	= ultimate compressive strength of a representative plate at the bottom shell
σ'_B	= ultimate compressive strength of a representative plate at the inner bottom shell
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_{xi}	= standard deviation of random variable x_i
σ_{xu}	= ultimate compressive strength for a member with pit corrosion
σ_{xuo}	= ultimate compressive strength for an intact (uncorroded) member
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

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