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CHAPTER 108

INTERNATIONAL CONTRIBUTION TO THE HIGHWAYS AGENCY'S BRIDGE RELATED RESEARCH [1]

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ABSTRACT

The objective of this paper is to summarize recent contributions of international experts to the research program sponsored by the Highways Agency. An efficient management of existing bridges requires methodology for an accurate evaluation of the actual loads and load carrying capacity and prediction of the remaining life. However, the parameters, which are involved in the evaluation process, are random variable. Therefore, a considerable research effort has been directed at the development of probability-based methodology. The research projects considered in the paper include the development of reliability models for analysis of bridges subjected to corrosion and fatigue, and reliability-based optimization of maintenance strategies for bridges.

1. INTRODUCTION

The Highways Agency has undertaken a major revision of the bridge assessment rules (Haynes 1997). There are limited resources available for structures requiring repair, rehabilitation or replacement. Therefore, there is need for efficient evaluation of the current condition and prediction of the future rate of deterioration. The parameters which represent load and resistance are random variables and the traditional assessment procedures can be inadequate. In the last 20 years a considerable research effort has been directed at the development of probability-based approaches in conjunction with the development of a new generation of bridge design codes in the US and Canada

[1] Institution of Civil Engineers Conference, October 2-3, London, UK, 2000.

(AASHTO LRFD 1998; OHBDC 1991 [13]; CHBDC 2000 [14]). The methodology is now available and can be used for other applications including assessment and maintenance of bridges.

The Highways Agency recognized the limitations of the deterministic approach to bridge assessment, and a number of projects were initiated with the objective to establish a basis for a more rational evaluation of deteriorated bridges and optimum maintenance strategies. These contributions focused on the application of probabilistic methods to improve the bridge assessment rules. The work covered the development of reliability models for bridges, establishing of the statistical database for loads and resistance parameters, development of probability-based models for corrosion and fatigue for steel and concrete bridges, and the development of optimum maintenance strategies for bridges. The reported research was performed by the UK consultants in cooperation with international experts.

2. CORROSION MODELS FOR STEEL GIRDER BRIDGES

The project was cooperation between the team at the University of Michigan and Flint and Neill Partnership. The objective was to establish time-varying reliability profiles relating reliability index to various rates of corrosion and develop a procedure for calculating probability of failure for different time stages. The corrosion patterns and rates are modeled for steel girder bridges on the basis of empirical relationships from laboratory tests and field observation of existing bridges. Relationship between reliability index and rating factor is investigated. The obtained reliability spectra can serve as a basis for the development of rational criteria for evaluation of existing bridge structures.

The reliability of bridges depends on load and resistance parameters that may vary with time. The variation is due to natural causes (loads, strength of material), deterioration (corrosion, fatigue) and other reasons (growth in legal load limits). Corrosion is one of the most important causes of deterioration for steel bridges. The major parameters include the rate (annual loss), pattern (location, concentration), and correlation with fatigue strength. A considerable research effort was focused on the development of reliability analysis procedures in the United States and Europe. A statistical data base has been established for steel girder bridges. Thus it is important to establish the relationship between reliability and the effect of corrosion on steel bridges. Also, by developing a procedure for calculating probability of failure for different time stages during the life time of the considered structure, a rational criterion for evaluation of existing bridges can be established. Ultimate limit states are considered in this study for moment and shear. Selected bridges are slab on girder type structures. Short and medium span existing bridges are included.

The load carrying capacity of a structure depends on the resistance of its components and connections. The component resistance, R, is determined mostly by material strength, dimensions and section loss due to corrosion. The resistance is a random variable. The resulting variation of resistance has been modeled by tests, observation of existing structures and by engineering judgment. The resistance model for steel girders was developed using the available material test data and by numerical simulations.

Deterioration of steel bridges depends on atmospheric environment, exposure of component (interior or exterior), protective treatment to steel, and influence of de-icing and traffic volume. Also type of material used (weathering steel, carbon steel, etc.) and construction details may affect the overall performance of the bridge. Corrosion is one of the most important causes of deterioration on steel bridges. The primary cause of corrosion is the accumulation of water and salt (marine environment and deicing salt) on steel surfaces.

Load-carrying capacity can be affected by corrosion. Loss of material results in reduction of section area, moment of inertia, and section modulus. Furthermore, it can lead to a premature local buckling (loss of stability). Also increase in traffic volume may lead to critical conditions with regard to fatigue. Thus it is important to establish the relationship between reliability and degree of deterioration. Reduction of section net area, build up of debris, and reduction of fatigue life are the main consequences of corrosion. In a steel girder, corrosion may affect the capacity in bending, shear, and bearing. For simple span steel girder bridges, corrosion can occur at the end supports and along the bridge due to deck joint leakage, accumulation of salt and dust on the steel surface of the girders.

The type of corrosion that will most likely occur at mid-span of a steel girder bridge is a section loss on the top surface of lower flange and on the lower one quarter of the web. The maximum flexural and torsional moments occur at midspan. The maximum shear and bearing stresses occur at the supports. Typical section loss near the support for a simple span bridge is characterized by a section loss over the whole web surface and on the top surface of the lower flange.

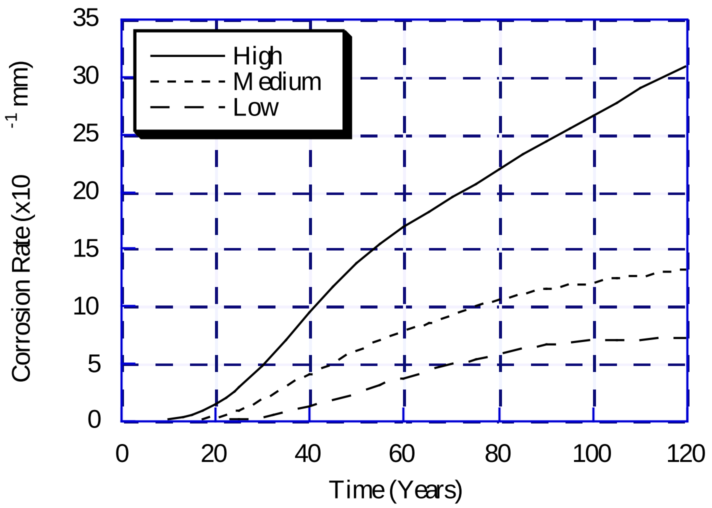

Figure 1. Assumed corrosion rate of steel girder bridges.

Rate of corrosion is a subject to considerable variation. There is some data on laboratory tests however; little is available on the actual field conditions. Based on the available literature and field observations, three deterioration rate curves (high, medium and low) are considered. Rate of corrosion is assumed to be practically zero for the first 10 to 15 years, until the paint and/or protective cover deteriorates (cracks and peels off). As the deterioration starts developing on the steel surface, an accelerated corrosion process may take place, as shown in Figure 1.

Structural performance is measured in terms of the reliability index, β. The reliability index is defined as a function of the probability of failure, P_F.

$$\beta = -\Phi^{-1}\left(P_F\right) \tag{1}$$

where Φ^{-1} is the inverse standard normal distribution function.

The time-related reliability of a bridge depends on its initial reliability and on the fractional loss of resistance. The initial reliability for different bridges under normal traffic differs considerably. Loss of resistance (load carrying capacity) depends on loss of section due to corrosion and on degree of corrosion. Fractional changes of reliability with time are closely related to fractional changes in resistance.

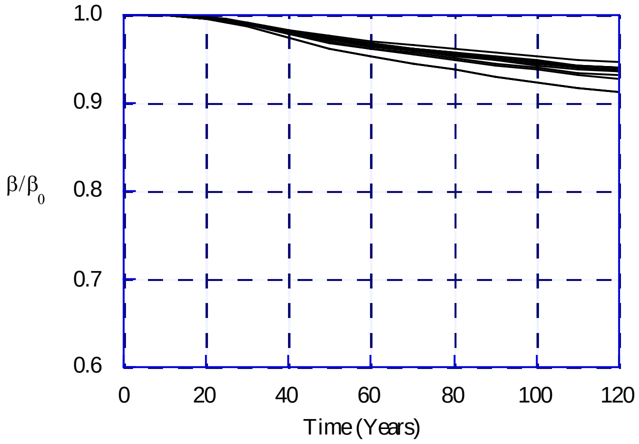

Figure 2. Expected reduction in reliability indices for moment.

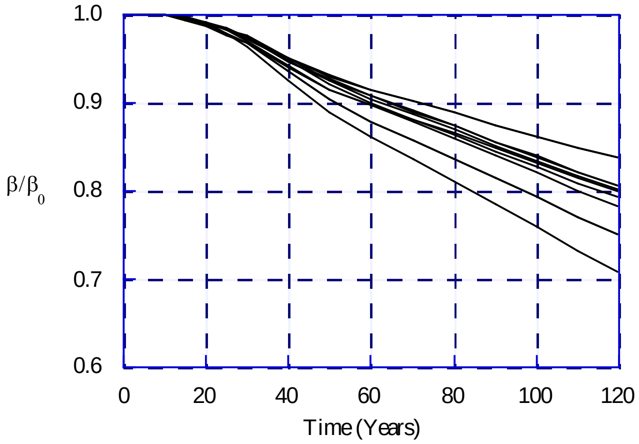

Figure 3. Expected reduction in reliability index for shear.

At any time stage, loss of section and loss in load carrying capacity (resistance) due to corrosion can be calculated. The reliability index is calculated using an iterative procedure. It is assumed that the total load effect is a normal random variable. The resistance is considered as a lognormal variable. The results are shown in Figure 2 for moments and Figure 3 for shear forces.

3. CORROSION MODELS FOR CONCRETE SLABS

CSRconsult performed this project with assistance from Cambridge University; see Thoft-Christensen et al. [1]. Corrosion initiation period refers to the time during which the passivation of steel is destroyed and the reinforcement starts to corrode actively. Fick's law of diffusion may represent the rate of chloride penetration into concrete, as a function of depth from the concrete surface and of the time, as follows:

$$\frac{dC(x,t)}{dt} = D_c \frac{d^2C(x,t)}{dx^2} \tag{2}$$

where $C(x,t)$ is the chloride ion concentration, as % by weight of cement, at a distance of x cm from the concrete surface after t seconds of exposure to the chloride source. D_c is the chloride diffusion coefficient expressed in cm^2/sec. The solution of the differential equation (2) is

$$C(x,t) = C_0 \left\{ 1 - \text{erf}\left(\frac{x}{2\sqrt{D_c \cdot t}} \right) \right\} \tag{3}$$

where C_0 is the equilibrium chloride concentration on the concrete surface, as % of the weight of cement, *x* is the distance from the concrete surface in cm, t is the time in sec, erf is the error function, and $C(x,t)$ is the chloride concentration at any position *x* at the time *t*.

In a real structure, if C_{cr} is assumed to be the chloride corrosion threshold and d is the thickness of concrete cover, then the corrosion initiation period, T_I, can be calculated. The time T_I to initiation of reinforcement corrosion is

$$T_I = \frac{d^2}{4D_c}(erf^{-1}(\frac{C_{cr} - C_0}{C_i - C_0}))^{-2} \tag{4}$$

On basis of equation (4) outcomes of the corrosion initiation time T_I has been performed on basis of the following data by simple Monte Carlo simulation (1000 simulations):

Initial chloride concentration:	0%
Surface chloride concentration:	Normal (0.650; 0.038)
Diffusion coefficient:	Normal (30; 5)
Critical concentration:	Normal (0.3; 0.05)
Cover:	Normal (40; 8)

The corresponding histogram is shown in Figure 4. The data is approximately Weibull distributed W(x; μ, k, ε), with μ = 63.67, *k*=1.81 and ε =4.79

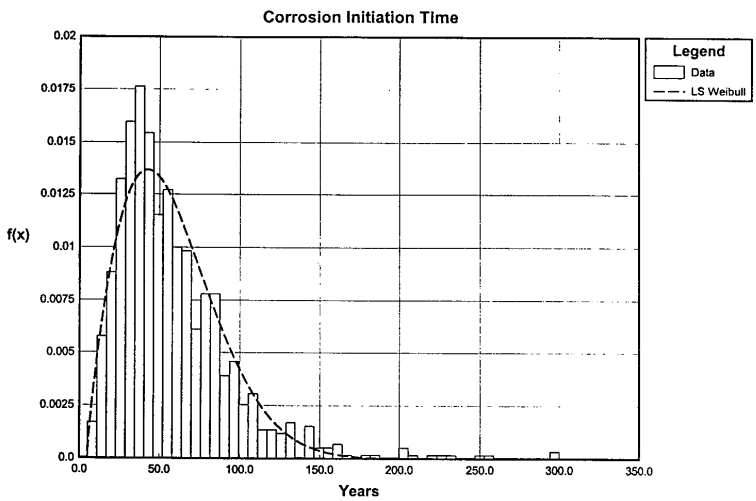

Figure 4. Density function of the corrosion initiation

When corrosion is initiated, the diameter $D(t)$ of the reinforcement bars at the time t is modeled by

$$D(t) = D_0 - C_{Corr} i_{corr} t \qquad (5)$$

where D_0 is the initial diameter, C_{corr} is a corrosion coefficient, and i_{corr} is the rate of corrosion. Based on a comprehensive survey, three models for chloride penetration have been proposed (the initial chloride is assumed to be zero): low deterioration, medium deterioration and high deterioration.

The stochastic variables used in the deterioration modeling are: chloride concentration on surface, initial chloride concentration in concrete, diffusion coefficient for the concrete, cover on rebar, critical chloride concentration, and rate of corrosion. The above-mentioned modeling has been used on an existing UK bridge build in 1975, Thoft-Christensen et al. [1]. The bridge was designed for 45 units HB load, Department of Transport [15]. The bridge has a span of 9.755 m, the width is 2×13.71 m, and the slab thickness is 550 mm. The general traffic highway load in the Eurocode 1, Part 3 (ENV 1991-3:1995 [16]) for lane and axle load is applied. The load effects produced by the Eurocode model (lane and axel load) are multiplied by a static loading factor (extreme type 1 distributed) and a dynamic load factor (normally distributed).

The reliability profile for a collapse (yield line) limit state is shown in Figure 5. The safety margin is

$$Z = VE_D - W_D \qquad (6)$$

where V is a model uncertainty variable, E_D is the energy dissipated in yield lines, and W_D is the work done by the applied loads. The Reliability index at time t = 0 is 11.4. Due to the size of the concrete cover (mean value 60 mm) the deterioration does not have any effect until year 70.

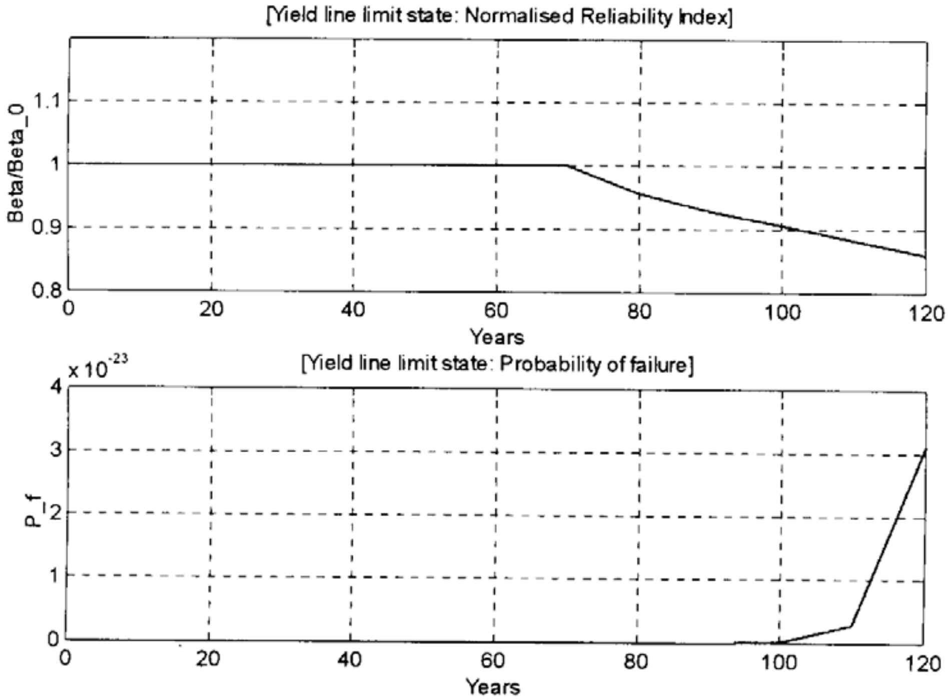

Figure 5. Reliability profile and failure of probability profiles.

The results from a sensitivity analysis with regard to the mean values for t = 0 years and t = 120 years are shown in Figure 6. The most important variables are as expected the thickness of the slab (stochastic variable no. 2) and the yield strength (stochastic variable no. 4) of the reinforcement. Observe that the magnitude of sensitivity with regard to the cover changes from negative at time t = 0 years to positive at time t = 120 years due to corrosion.

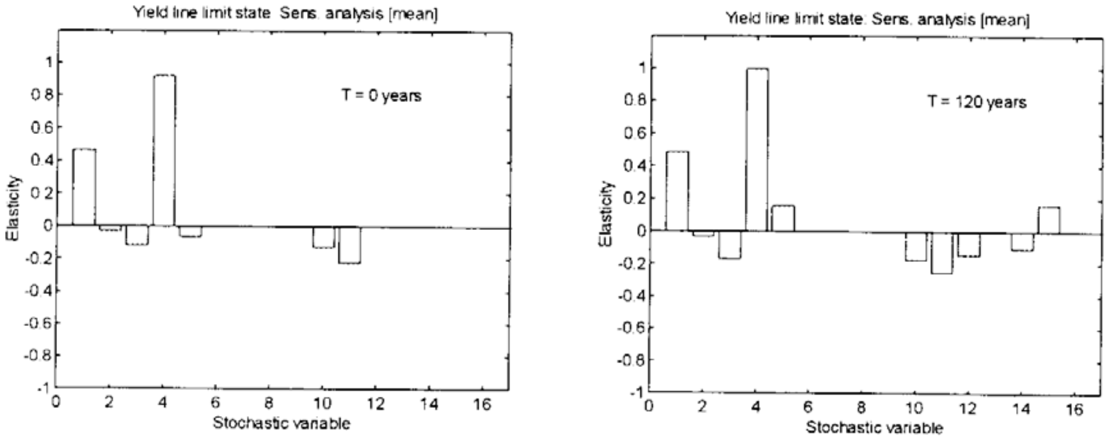

Figure 6. Sensitivity analysis.

4. FATIGUE MODELS FOR STEEL GIRDER BRIDGES

This research was performed as a joint project by a team at the University of Michigan with Flint and Neill Partnership. The objective was to develop criteria for assessment of existing steel bridges with regard to fatigue. An important part of this effort was the development of reliability analysis procedure including the fatigue related parameters.

A fatigue performance depends on strength and load spectra. The most important load parameters are amplitude and frequency of loading. The field observations indicate that magnitude and frequency of truck loading are strongly site-specific. The available data, used to develop the fatigue load models show that the number of trucks on the slow lane of highways can be very high, in some cases 8,000 per day was observed. It gives 200 million vehicles during a lifetime of 100 years. This number of trucks corresponds to many more cycles in structural elements compared to what is assumed in design codes, for example Eurocode specifies 100 million cycles.

Many researchers have studied material response. For steel girders, the so-called S-N curves were developed for various categories of details in steel structures (Fisher et al. [17]). The distribution of the number of cycles to failure can be approximated as normal, with the coefficient of variation decreasing for decreasing stress levels. For reinforced concrete components, the fatigue-caused reduction of strength applies to reinforcing steel and/or concrete. It was observed that strength concrete under cyclic loading can be drastically reduced. The limit state function for fatigue can be expressed in terms of two variables, the number of cycles to failure under given stress history, and the number of applied cycles. Both are random variables and they can be described by their cumulative distribution functions. For reinforced concrete T-beams, a procedure is presented for prediction of the remaining life with regard to fatigue. Presented model assumes degradation of concrete in compression zone caused by repeated loading. Accumulated damage in a composite material such as concrete results in micro-cracks and a reduced ultimate strength. Then, the load carrying capacity of flexural members can be governed by the ability of compression zone to carry the load (as in the case of an overreinforced beam).

Limit state function for fatigue can be written as a function of time (the time to failure should be greater than the time of desired service), or as a percentage of the remaining life (damage function reaches value 1 at the failure point). The limit state function for fatigue in steel girder bridges can be expressed in terms of two variables,

$$N_f - N_n = 0 \tag{7}$$

where N_f is the number of cycles to failure under given stress history, and N_n denotes the number of applied cycles. Both N_f and N_n are random variables.

To investigate fatigue of bridges loaded with heavy trucks, it is convenient to use the load model based on weigh-in-motion (WIM) measurements (Laman and Nowak [2]). WIM can be used to calculate statistical stress parameters for girders. Field measurements were conducted on steel girder bridges. Strain transducers were attached to all girders at the lower, mid span flanges. Dynamic strain cycles were measured under normal traffic using the rainflow algorithm. The data was collected and recorded including stress histograms for the girders and other components. The strain history under normal traffic was collected and the stress cycle histogram was assembled by the rainflow method of cycle counting. The rainflow method counts the number, n, of cycles in each predetermined stress range, S_i, for a given stress history (Laman and Nowak [2]). Examples of cumulative distribution functions (CDFs) for a steel girder bridge are shown in Figure 7, on a normal probability scale. Strain histories were collected continuously and reduced using the rainflow algorithm. The data is presented here represents strain cycles due to 7 days of normal traffic. The girders are numbered from G1 (exterior), through G9. The extreme stress is in the exterior girder, G9. The observation of extreme loaded girder is important for focusing inspection on potential fatigue prone details and fatigue design of components near the location of maximum equivalent stresses.

As a means of comparison of fatigue live load, the equivalent stress, s_{eq}, is calculated for each girder using the following root mean cube (RMC) formula:

$$s_{eq} = \sqrt[3]{\sum \left(p_i \times S_i^3 \right)} \tag{8}$$

where S_i = midpoint of the stress interval i and p_i = the relative frequency of cycle counts for interval i. The stress, S_i, is calculated as a product of strain and modulus of elasticity of steel. The number of applied load cycles, N_n, in Eq. 7, is associated with the equivalent stress calculated using Eq. 8. The uncertainty in $s_{eq} = \sqrt[3]{\sum \left(p_i \times S_i^3 \right)}$ is expressed in terms of the bias factor, 1.0, and coefficient of variation, 0.15.

In the fatigue analysis, resistance is the ability of the structure to resist cyclic loads. For steel girders, N_f in Eq. 7 can be determined from the *S-N* curves. For each girder, the load amplitude varies. Most of the available material tests were performed for a constant load amplitude.

In concrete structures, cyclic loading can also result in a higher damage rate compared with sustained load. This should be taken into account, in prediction of the remaining life of a concrete bridge. Fatigue changes in concrete may be difficult to monitor because they are inside of material, but they can lead to a decrease in strength of concrete and reduction of the modulus of elasticity. Such changes obviously affect the load carrying capacity and deflection. In particular this applies to girders, which support the slow lane traffic.

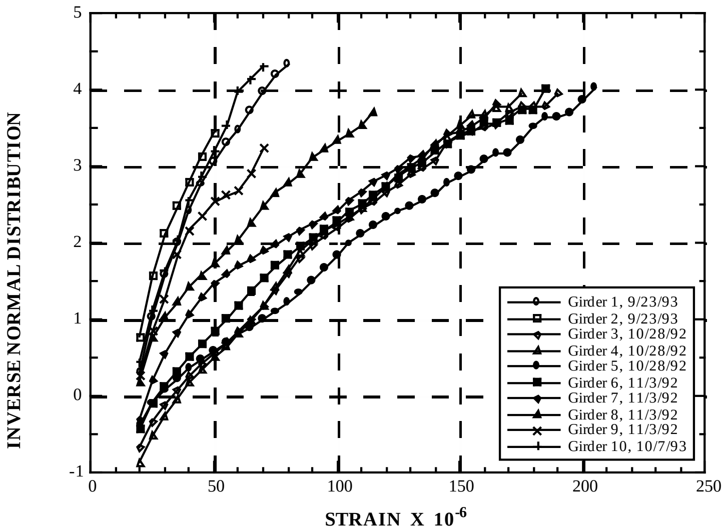

Figure 7. CDFs of Measured Girder Strains.

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Miner's rule (1945) provides a simple way to allow for the rate of cycle dependent damage. The damage factor D_n accumulates in a linear way in terms of cycle ratio to failure:

$$dD_n = \frac{dn}{N_f} \tag{9}$$

The number N_f of cycles corresponds to pure fatigue failure. For higher levels of loading, the number of cycles at failure proves to be more sensitive to the parameters of the cyclic stress. A convenient way to express the rate of creep damage, D_t, at any time t is (Szerszen et al.[18]),

$$\dot{D}_t(t) = \frac{\beta}{r+1}\left(\frac{\sigma}{f_c^{'}}\right)^k \left(1 - D_t\right)^{-r} \tag{10}$$

where β, k and r are material coefficients (Szerszen et al., [3]).

In recent studies of the ultimate limit states, the structural performance was measured in terms of the reliability index (Nowak [4]). It is further assumed that the resistance and load parameters (N_F and N_n) are lognormal random variables. Therefore, the reliability index, β , is

$$\beta = \ln\left(m_{N_f} / m_{N_n}\right) / \left(V_{N_f}^2 + V_{N_n}^2\right)^{-1/2} \tag{11}$$

where m_{N_f} = mean number of cycles to failure, V_{N_f} = coefficient of variation of the number of cycles to failure, m_{N_n} = mean number of cycles applied, V_{N_n} = coefficient of variation of the number of cycles applied. The reliability analysis is performed for several values of the effective stress range. The results are presented in Figure 8.

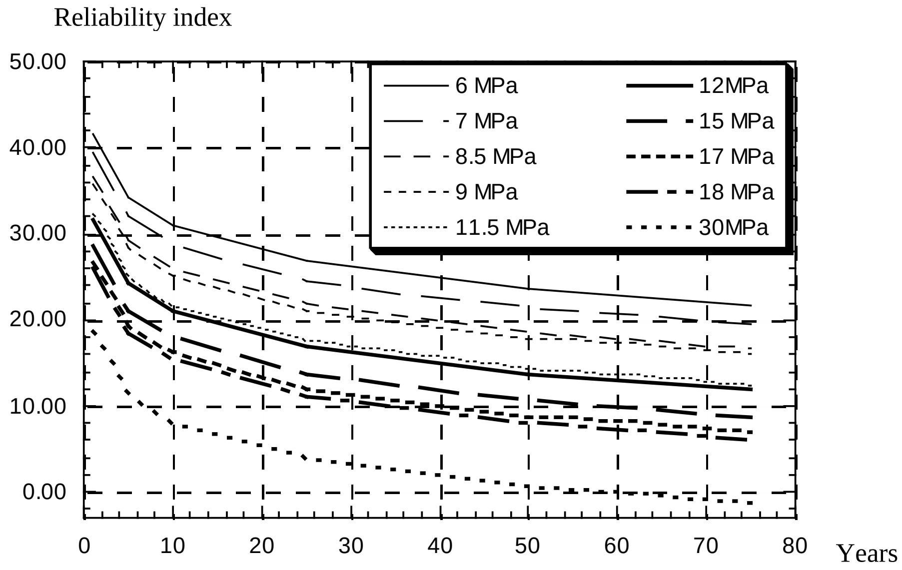

Figure 8. Reliability Indices vs. Time (Nowak et al. [5]).

5. OPTIMUM MAINTENANCE STRATEGIES FOR HIGHWAY BRIDGES

The objective of the project is to develop a procedure for optimization of maintenance, including scheduling of repairs. The project is a joint venture involving the team from the University of Colorado, CSRconsult ApS, Denmark, and Maunsell Ltd. A strategy plan was proposed by the Highway Agency.

CSRconsult ApS is responsible for the research related to reinforced concrete bridges, Thoft-Christensen [6]. The main purpose was to develop a methodology by which the so-called reliability distributions for structures can be estimated. The reliability distributions derived are used in development of optimum strategies for highway bridges see Frangopol et al. [7]. For a group of bridges the following reliability distributions are estimated using crude Monte Carlo simulation:

- The *initial reliability distribution* is the distribution of the reliability indices for all bridges at t = 0.
- The *deterioration initiation distribution* is the distribution of the deterioration (corrosion) initiation times for all bridges.
- The *deterioration rate distribution* is the distribution of the deterioration rates of all bridges.
- The *fundamental distribution at time t* is the distribution of the reliability indices for all bridges at a given point in time *t*.
- The *rehabilitation time distribution* is the distribution of the points in time by

which the considered bridges reach a critical rehabilitation reliability index. If no maintenance has taken place it is called the *first rehabilitation time distribution.* If maintenance has taken place it is called the *rehabilitation time distribution after maintenance.*

The methodology is based on a simplified reliability profile for each bridge, see Figure 9. The time t = 0 is the year when the bridge in question is build. $\beta(0)$ is the reliability index at time t = 0. $\beta(t)$ is the reliability index at the time t. Deterioration is assumed to be initiated at time t_i. The deterioration rate is α. $\beta = 4.6$ is used as the critical (target) reliability index.

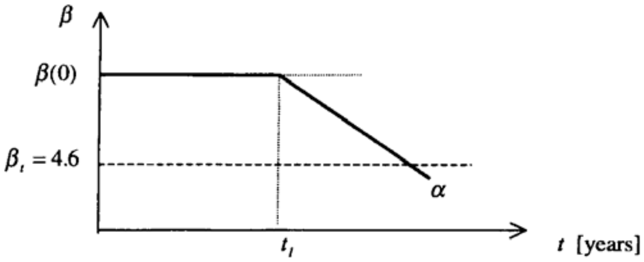

Figure 9. Modeling of the reliability profile for a reinforced concrete bridge.

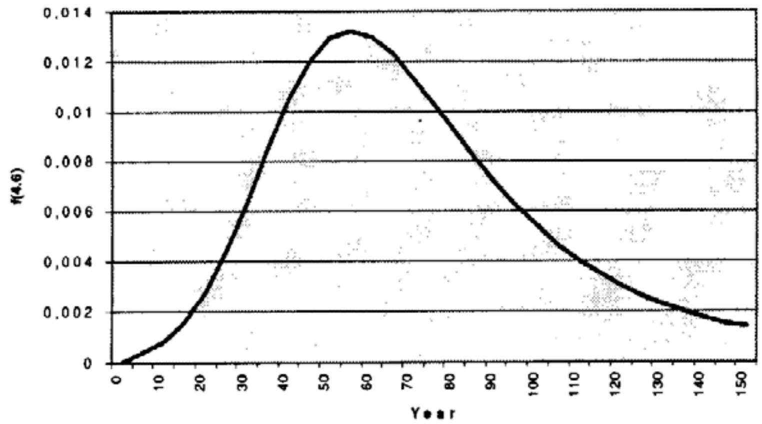

Figure 10. The first rehabilitation time distribution.

V_{N_f} The upper part of the initial reliability distribution $\beta(0)$ is based on the reliability index $\beta(0)$ for 15 “good” bridges Thoft-Christensen & Jensen [8]. The lower part is assumed to have only a small probability for reliability index values smaller than 4.6. Therefore, a log-normal distribution LN(2.0 ; 0.15) is used. The deterioration is limited to corrosion of the reinforcement. The deterioration reliability distribution is assumed to be a Weibull distribution with a mean value of 63.67 years and k=1.81, Thoft-Christensen [9]. Based on information from Thoft-Christensen & Jensen [8] a uniform distribution U[0.01 : 0.20] is chosen for the deterioration rate distribution.

On basis of the above-mentioned three distributions, fundamental reliability distributions and rehabilitation time distributions can be obtained by Monte-Carlo simulation. As an example the first rehabilitation time distribution for 970 reinforced concrete overbridges in UK is shown in Figure 10.

A similar investigation for steel/concrete composite bridges has been performed by University of Colorado. Some of the results are shown in Frangopol & Das [10]. As indicated by Das [11], the bridge rehabilitation rates can be determined using three methods:

(a) Based on expert opinions of experienced bridge engineers
(b) Based on available data
(c) Based on reliability studies of whole life performance.

The Highways Agency, London, used method (a) in 1997 and 1998. “In 1997 simple triangular distributions were used. In 1998 more representative distributions were derived on basis of discussions with bridge engineers with considerable experience of maintaining Trunk Road Structures”, Maunsell Ltd. and Transport Research Laboratory [12]. In 1998 a project was commissioned by the Highways Agency to determine optimum maintenance strategies for different bridge types. As part of that project, bridge rehabilitation rates were obtained based on reliability studies for steel and concrete bridges by Frangopol et al. [7] and Thoft-Christensen [6].

The PDF’s for rehabilitation rates for steel/concrete composite bridges obtained by the experts in 1997 (a triangular distribution), by experts in 1998 (a logistic distribution), an the one obtained through reliability analysis have been compared. It is interesting to note that:

(a) the target reliability index was not specified in 1997 and 1998
(b) in the reliability analysis carried out in 1999 the target reliability was specified as $\beta_{t\arg et}=4.6$
(c) the modes of the three distributions are quite similar.

The reliability based procedure developed for finding the rehabilitation distribution assuming no preventive maintenance can be extended to the case of preventive maintenance by adding random variables associated with the preventive maintenance action. This research is in progress.

6. SYSTEM RELIABILITY MODELS FOR BRIDGES

This is an ongoing project, cooperation between the team at the University of Michigan and University of Surrey.

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