JOURNAL OF THEORETICAL
AND APPLIED MECHANICS
2, **39**, 2001

RELIABILITY PROFILES FOR STEEL GIRDER BRIDGES WITH REGARD TO CORROSION AND FATIGUE

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Reliability-based models are developed for time-varying changes in structural performance due to corrosion and fatigue. The fatigue limit state is one of the important considerations in the design of bridges. Accumulated load cycles can cause cracking or even failure. Reliability analysis is performed for the fatigue limit state function of flexure. Prediction of the remaining fatigue life for steel and concrete beams is illustrated on examples of existing bridge girders. Corrosion causes a loss of the section and reduction of the load carrying capacity. The rate of corrosion is a random variable. Three levels of corrosion are considered. Changes in the reliability are evaluated as a function of time, covering the life-time of the bridge.

Key words: bridges, corrosion, fatigue, reliability, steel girders, limit states

1. Introduction

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 statistical data base has been established for steel girder bridges. Thus it is important to establish the relationship between the reliability and the effect of corrosion on steel bridges. Also, by developing a procedure for calculating the probability of failure for different time stages during the life time of the considered structure, rational criteria for the 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 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.

Fatigue performance depends on strength and load spectra. The most important load parameters are amplitude and frequency of a loading. To investigate the fatigue of bridges loaded with heavy trucks, it is convenient to use the load model based on weigh-in-motion (WIM) measurements. WIM can be used to calculate statistical stress parameters for girders. The results indicate that the magnitude and frequency of a truck loading are strongly site-specific. The material response has been studied by many researchers. For steel girders, the so called S-N curves were developed for various categories of details in steel structures. 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 the concrete strength under a 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 the given stress history, and the number of applied cycles. Both are random variables and they can be described by their cumulative distribution functions.

Accumulated damage in a composite material such as concrete results in micro-cracks and 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 over-reinforced beam). The fatigue model for concrete is based on the theory of plasticity for composite materials and reflects the real behavior of concrete subjected to cyclic compression. It is a convenient way to predict the remaining life of existing concrete bridges, specially those which were designed without regard to cyclic character of the live load and rheological changes in the material. Reliability analysis is performed for fatigue limit state in steel bridge girders. The limit state function for the fatigue is formulated in terms of number of load cycles. The parameters of the load (number of cycles applied) and resistance (number of cycles to failure) are derived from the available test data. However, the available data base is rather limited and there is a need for further verification.

Recently, a considerable research effort has been devoted to bridge design and evaluation in Europe and North America. Therefore, this paper focuses on the reliability models developed in conjunction with calibration of the AASHTO LRFD code in the USA, OHBDC and CHBDC in Canada, and BS 5400 in the United Kingdom. The reliability is calculated for selected structures.

The obtained reliability spectra can serve as a basis for the development of rational criteria for the evaluation of existing bridge structures.

2. Deterioration model for corrosion

Deterioration of steel bridges depends on atmospheric environment, exposure of component (interior or exterior), protective treatment to steel, 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 de-icing 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 premature local buckling (loss of stability). An increase in traffic volume may also lead to critical conditions with regard to fatigue. Thus it is important to establish the relationship between the reliability and degree of deterioration. The reduction of section net area, build up of debris, and reduction of the 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 the mid-span of a steel girder bridge is a section loss on the top surface of the lower flange and on the lower one quarter of the web. The maximum flexural and torsional moments occur at the midspan. The maximum shear and bearing stresses occur at the supports. The typical section loss near the support for a simple span bridge is characterized by the section loss over the whole web surface and on the top surface of the lower flange (Kayser and Nowak, 1989a,b).

The 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. The 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 Fig.1 (Park et al., 1999).

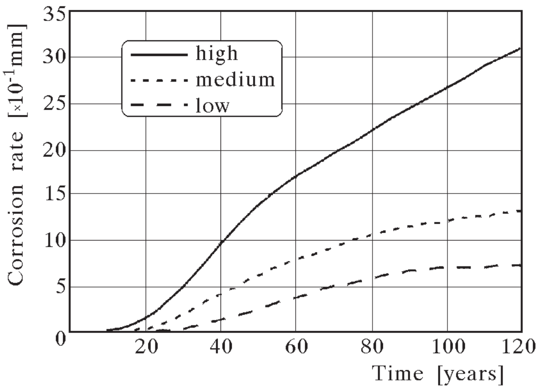

Fig. 1. Considered rates of corrosion

3. Deterioration model for fatigue

Bridge components are subjected to repeated load cycles. Each passage of a heavy truck can be considered as an application of two and sometimes even more time-varying forces. In their service life, bridges are exposed to traffic loads, sometimes very heavy, in particular, on high volume roads. Multiple application of a dynamic load may lead to fatigue-specific changes in the structure and materials. In particular, the fatigue damage can affect short span bridges, with a dominating live load and a relatively low level of dead load. The available data, used to develop the fatigue load model, show that the number of trucks in the slow lane of highways can reach 5,000 per day, which is equivalent to over 180 million vehicles during the life time of 100 years. Taking into account, that one vehicle usually produces more than one cycle, the number of load cycles during the service life of a bridge can be large enough to cause fatigue problems. The bridge performance depends on the structural durability and load spectrum. Both parameters can be treated as random variables. Durability is a function of strength of materials and connectors. The important load parameters are the amplitude and frequency of a loading.

Evaluation of the fatigue performance of bridges involves a considerable degree of uncertainty in both the load and resistance models. In general, Miner's law is proposed to establish the relationship between the variable-amplitude fatigue behavior and constant-amplitude behavior. The equivalent stress range concept or equivalent fatigue vehicle to represent the regular truck traffic are based on this rule. There are procedures available to determine the allowable stress for the traffic volume and design life. The number of load cycles depends on truck types. However, there is a need to include the future traffic growth.

The evaluation procedure, suggested by Moses et al. (1988), includes the calculation of the remaining life and the remaining safe life of a detail (the difference is in the probability of failure). The basic evaluation procedure does not cover the effects of secondary bending, the evaluation and repair of cracked members, and the effects of corrosion and mechanical damage. It does apply to members that have received normal repairs during fabrication.

In general, in the fatigue evaluation procedure, there are three steps involved:

Step 1. Calculation of the variable-amplitude stress spectrum caused by the actual loading.

Step 2. Development of a relationship between this stress spectrum and the equivalent constant-amplitude stress, for example using the cumulative damage approach.

Step 3. Comparison of the resulting applied stress and the fatigue strength (obtained from the S-N curve). If the applied stress is below the allowable stress value for the desired design life then calculate the fatigue life.

Some of the available procedures consider a variable-amplitude fatigue limit that is a fraction of the constant-amplitude fatigue limit. If the equivalent or applied stress level is below this variable-amplitude fatigue limit, the fatigue life of the bridge is assumed to be infinite. Otherwise, there is a finite fatigue life, although some researchers assume that the fatigue life is still infinite, if all the variable-amplitude stress cycles are in compression.

Because load and resistance parameters are random variables, probabilistic methods are the most convenient way to provide the required safety level for various design cases. The stress range is the main parameter in design and evaluation procedures, but the effect of the mean stress can also be considered. Safety factors are usually applied to the stress level, strength parameter, or design life.

In general, fatigue models predict changes in material characteristics such as fatigue strength or modulus of elasticity under a cyclic loading. The material response has been studied by a number of researchers. The material degradation can be described using the S-N curves based on the regression lines obtained during fatigue tests. For steel girders, the S-N curves were developed for various categories of details. The number of cycles to failure can be considered as a normal random variable, with the coefficient of variation decreasing for decreasing stress levels. In case of steel, the relationship between the applied stress level and the number of cycles to failure, using a logarithmic scale, can be described by a linear equation. It is generally accepted, that the fatigue strength of steel (the endurance limit), which depends on the stress level and amplitude/frequency of the loading, can be established after 2 million cycles.

For reinforced concrete components, the fatigue-caused degradation applies to reinforcing steel, concrete and bond properties. It was observed that the strength of concrete under a cyclic loading can be drastically reduced. The process depends on the amplitude and frequency of the applied load, the mean stress value and max/min stress ratio. In fact, there is no fatigue limit for concrete and the degradation process is nonlinear. Nevertheless, it can be considered as linear up to a certain stress limit, which is about 70% of the ultimate concrete strength. Above this limit, the degradation process is highly nonlinear (nonlinear creeping) and leads to unconditional failure.

In the fatigue analysis, resistance is the ability of the structure to resist cyclic loads. For steel girders, the number of load cycles to failure N_f 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, a cyclic loading can also result in a higher damage rate compared with a sustained load. This should be taken into account, in the prediction of the remaining life of a concrete bridge. Fatigue changes in concrete may be difficult to monitor because they are inside of the material, but they can lead to a decrease in strength of the 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.

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

$$dD_n = \frac{dn}{N_f} \tag{3.1}$$

The number of cycles N_f corresponds to pure fatigue failure. For higher levels of the 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 the creep damage D_t, at any time t is (Szerszen et al., 1995)

$$\dot{D}_t(t) = \frac{\beta}{r+1}\Big(\frac{\sigma}{f_c'}\Big)^k (1 - D_t)^{-r} \tag{3.2}$$

where β, k and r are material coefficients (Szerszen et al., 1994).

A convenient load model for fatigue investigation can be based on weigh-in-motion (WIM) measurements. The WIM can be used to calculate statistical stress parameters for girders. Also, the frequency of load cycles is associated with the stress history. The field measurement results indicate that the magnitude and frequency of the truck loading are strongly site-specific. The limit state function for fatigue can be expressed in terms of two random variables, the number of applied cycles, and the number of cycles to failure under the given stress history. The fatigue limit state function, in the case of bridge girders, can also be presented as a difference between the moment carrying capacity of the section and the applied load moment.

4. Reliability analysis procedure

Structural performance is measured in terms of the reliability index β (Nowak and Collins, 2000). The reliability index is defined as a function of the probability of failure P_F

$$\beta = -\Phi^{-1}(P_F) \tag{4.1}$$

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

It is assumed that the total load Q, is a normal random variable. The resistance R, is considered as a log-normal random variable. The formula for the reliability index can be expressed in terms of the given data $(R_n, \lambda_R, V_R, m_Q, \sigma_Q)$ and the parameter k as follows (Nowak, 1995)

$$\beta = \frac{R_n\lambda_R(1-kV_R)[1-\ln(1-kV_R)] - m_Q}{\sqrt{[R_nV_R\lambda_R(1-kV_R)]^2+\sigma_Q^2}} \tag{4.2}$$

where

R_n – nominal (design) value of resistance
λ_R – bias factor of R
V_R – coefficient of variation of R
m_Q – mean load
σ_Q – standard deviation of the load.

The value of the parameter k depends on the location of the design point. In practice k is about 2.

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 the reliability with time are closely related to fractional changes in the resistance.

At any time stage, the loss of section and the loss in load carrying capacity (resistance) due to corrosion can be calculated. The reliability index is calculated using an iterative procedure (Nowak and Collins, 2000). It is assumed that the total load effect is a normal random variable. The resistance is considered as a lognormal variable.

The 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. The limit state function for fatigue in steel girder bridges can be expressed in terms of two variables

$$N_f - N_n = 0 \tag{4.3}$$

where

N_f – number of cycles to failure under the given stress history
N_n – number of applied cycles.

Both N_f and N_n are random variables, and they can be described by their cumulative distribution functions (CDFs).

The other way to express the limit state function for fatigue is to calculate the difference between the moment carrying capacity $M_n(t)$ and loading moment M_{ap}

$$M_n(t) - M_{ap} = 0 \tag{4.4}$$

This function can be used to predict the remaining life of concrete beams because the moment carrying capacity can be based on the damage function for concrete under the fatigue loading.

In recent studies on the ultimate limit states, the structural performance was measured in terms of the reliability index (Nowak, 1995). It is further assumed that the resistance and load parameters (N_f and N_n) are lognormal random variables. Therefore, the reliability index β can be calculated as follows

$$\beta = \frac{\ln \frac{m_{Nf}}{m_{Nn}}}{\sqrt{V_{Nf}^2 + V_{nf}^2}} \tag{4.5}$$

where

m_{Nf} – mean number of cycles to failure
V_{Nf} – coefficient of variation of the number of cycles to failure
m_{Nn} – mean number of applied cycles
V_{Nn} – coefficient of variation of the number of applied cycles.

5. Reliability indices

The reliability indices β are calculated for girders designed using AASHTO Specifications (1996) and AASHTO LRFD (1998). According to AASHTO (1996), the basic design requirement is expressed in terms of moments or shears (Load Factor Design)

$$1.3D + 2.17(L + I) < \phi R \tag{5.1}$$

where D, L and I are the moments (or shears) due to a dead load, live load and impact, R is the moment (or shear) carrying capacity, and ϕ is the resistance factor. Values of the resistance factor are $\phi = 1.00$ for the moment and shear in steel girders, $\phi = 0.90$ and 0.85 for the moment and shear in reinforced concrete T-beams, respectively, $\phi = 1.00$ and 0.90 for the moment and shear in prestressed concrete AASHTO-type girders, respectively.

The results of calculations show a considerable variation in the reliability indices depending on the limit state and span length, from about 2 for a short span (10 m) and short girder spacing (1.2 m) to over 4 for larger spans and girder spacing. The target reliability index was selected $\beta_T = 3.5$.

The results of the reliability analysis served as a basis for the development of more rational design criteria for the considered girders. The load factors developed for LRFD AASHTO Code (1994 and 1998) are

$$1.25D + 1.50DA + 1.75(L + I) < \phi R_n \tag{5.2}$$

where D is the deal load, D_A – dead load due to asphalt wearing surface, L – live load (static), I – dynamic load, R_n – resistance (load carrying capacity), and ϕ – resistance factor.

In the selection of resistance factors, the acceptance criterion is the closeness to the target value of the reliability index β_T. The recommended resistance factors are $\phi = 1.00$ for the moment and shear in steel girders, $\phi = 0.90$ for moment and shear in reinforced concrete T-beams, $\phi = 1.00$ and 0.90 for the moment and shear in prestressed concrete AASHTO-type girders, respectively.

The reliability indices calculated for the bridges designed using the new LRFD AASHTO code (1994) are close to the target value of 3.5 for all materials and spans. The calculated load and resistance factors produce a uniform spectrum of the reliability indices. For comparison, the ratio of the required load carrying capacity by the new LRFD AASHTO code (1998) and the AASHTO (1996) varies from 0.9 to 1.2.

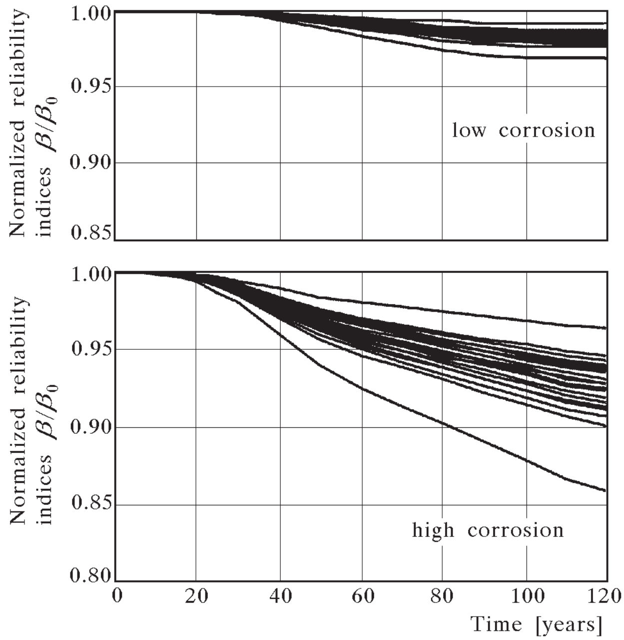

Fig. 2. Life-time normalized reliability indices for corrosion, moments

For bridges affected by corrosion, the reliability indices are calculated for the selected structures designed according to BS-5400. The resistance is calculated in terms of the moment carrying capacity and shear capacity. For the bridges under high and low corrosion rate environments, the reliability indices for the actual bridges, as a function of time, are shown in Fig. 2 for the moment and Fig. 3 for the shear. The reliability indices at each time stage were divided by the initial reliability indices. The initial reliability indices were high for the selected structures, ranging between 10 and 11.9 for the moment and between 11 and 13.6 for the shear limit state.

For fatigue, the reliability analysis is performed for the steel girder bridges designed according to BS-5400. Nine detail classes are considered for welded and non-welded components. The reliability indices calculated using the loading parameters obtained from the weigh-in-motion (WIM) measurements are presented in Fig. 4.

In addition, the reliability analysis is also performed for reinforced concrete beams. Fatigue of concrete is considered for several values of the effective stress range. The results are presented in Fig. 5.

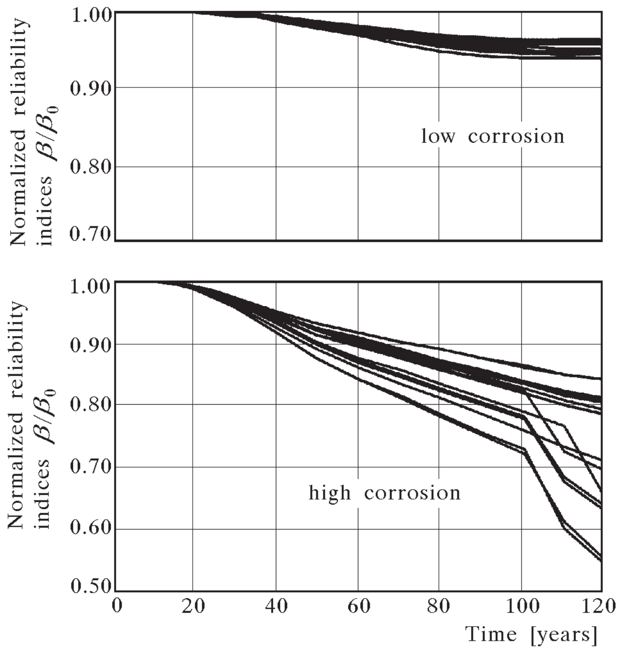

Fig. 3. Life-time normalized reliability indices for corrosion, shears

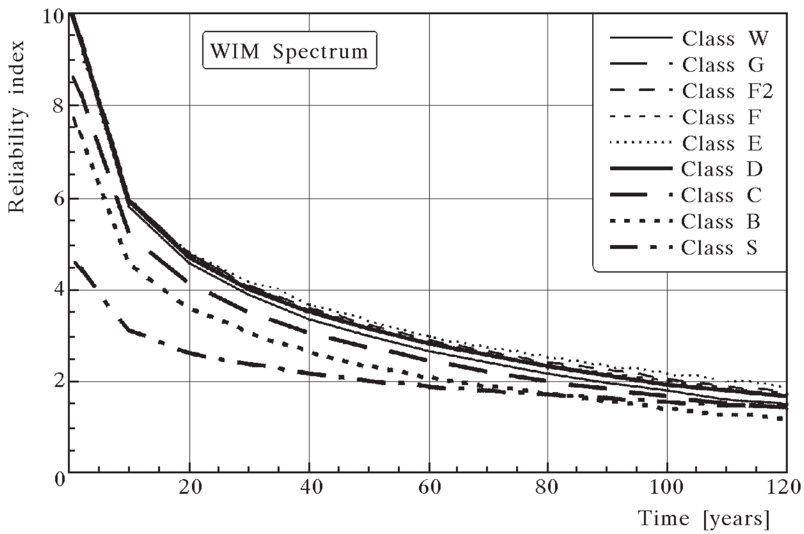

Fig. 4. Life-time reliability indices for various fatigue classes of detail

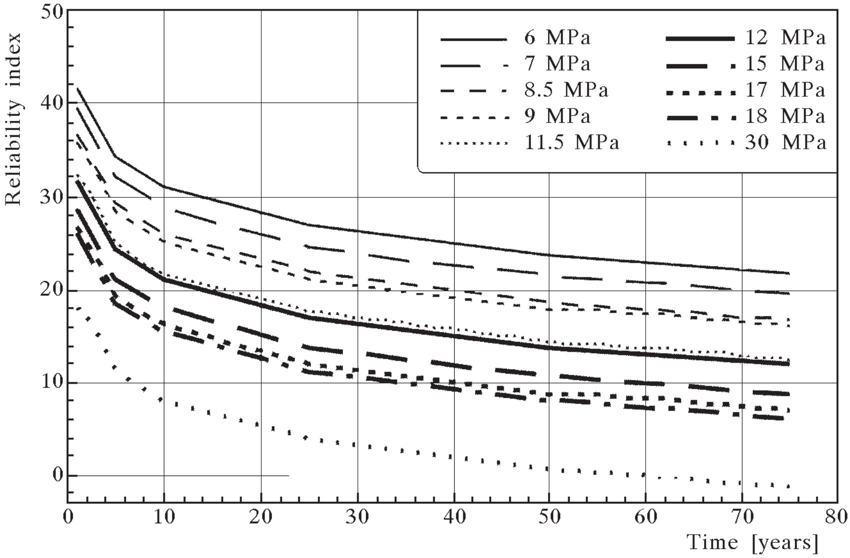

Fig. 5. Reliability Indices vs. Time for Reinforced Beams (Nowak et al., 1999)

6. Conclusions

The reliability can be used as a rational measure for quantification of the structural performance. The time dependent reliability of steel girder bridges is considered with regard to corrosion and fatigue.

The deterioration model of steel girder bridges has been developed for different rates of corrosion. The reliability indices are calculated for selected bridges. The resulting reliability indices vary depending on the original design and degree of deterioration. In general, the loss of load carrying capacity due to corrosion is less than 10 percent for the moment and larger for the shear.

The limit state function for fatigue is formulated in terms of the number of load cycles. The parameters of the load (number of cycles applied) and resistance (number of cycles to failure) are derived from the available test data. However, the available data is rather limited and there is a need for further verification. The reliability indices vary depending on class of detail.

Acknowledgments

The research presented in this paper has been partially sponsored by the National Science Foundation and the UK Highway Agency which is gratefully acknowledged. The presented results and conclusions are those of the authors and not necessarily those of the sponsors.

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Niezawodność stalowych mostów belkowych z uwzględnieniem korozji i zmęczenia

Streszczenie

Praca dotyczy modeli opartych na teorii niezawodności, uwzględniających zmiany w czasie powstałe w konstrukcji na skutek zmęczenia i korozji. Stan graniczny jest jednym z istotniejszych zagadnień w projektowaniu mostów. Akumulacja cykli obciążenia może spowodować zarysowanie, a nawet awarie mostu. Analiza niezawodnościowa jest przeprowadzona dla stanu granicznego zmęczenia spowodowanego zginaniem. Szacowanie pozostałego czasu użytkowania konstrukcji ze względu na zmęczenie stalowych i żelbetowych belek przedstawiono na przykładzie analizy istniejących mostów belkowych. Korozja powoduje utratę przekroju elementów nośnych konstrukcji i spadek nośności granicznej. Prędkość rozwoju korozji jest zmienną losową. W artykule rozpatrzono trzy poziomy występowania korozji w belkach mostowych. Zmiany w niezawodności mostu przedstawiono w funkcji czasu, obejmującej cały okres przewidywanego użytkowania mostu.

Manuscript received November 3, 2000; accepted for print November 23, 2000