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PROBABILISTIC APPROACH TO THE RELIABILITY OF STEEL BRIDGE STRUCTURES

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1. INTRODUCTION

The present state of bridge engineering development is characterised by the passage from the deterministic and semiprobabilistic methods of reliability assessment to the more perfect ones having a fully probabilistic base. The process is especially evident in design of new bridge structures. Problem of reliability assessment and evaluation of existing bridge structures on the probabilistic base is developed only a little. Because of present-day actual . technical state of existing bridge structures in traffic and the high costs for their upgrading, it is necessary to observe above mentioned problems on the scientific base and to provide the Administration with a new more accurate methodology of existing bridge evaluation.

The calculation of bridge reliability is associated with limit states. Limit states are the boundaries between safety and failure.

Let R represent the random variable resistance of a bridge member and S represent the total random variable load effects. Then, the corresponding safety margin G could be expressed by equations

$$G = R - S > 0 \text{ or } G = R/S > 1 . \qquad (1)$$

The probability of failure, P_f is equal to

$$P_f = P(R - S < 0) = P(G < 0). \quad (2)$$

It is convenient to measure structural safety in forms of a reliability index, β, defined as a function of P_f.

$$\beta = -F^{-1}(P_f) \quad (3)$$

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

2. STANDARD PRACTICE OF TIIF EXISTING BRIDGE RELIABILITY ASSESSMENT

1The generally accepted standard practice of the existing bridge reliability assessment is based on the deterministic concepts of the bridge reliability. The bridge load carrying capacity - Live Load Rating Factor (LLRF) is presented as a basic and decisive parameter of existing bridge reliability which expresses the relative ultimate strength of a structure member in view of structural response to the ideal load train UIC-71.

$$LLRF_{UIC} = (R - \Sigma S_i)/S_{UIC},$$

where SUIC are the dynamic effects of the ideal load train UIC-71, R is the material resistance of an observed member, ESi are the effects of other applied loads (dead load, wind, brake forces, lateral strokes).

Effects of the actual traffic load can be also expressed as a function of the effects of the ideal load train UIC-71 in the form

$$\lambda_{UIC} = S_T/S_{UIC}, \quad (5)$$

where S_T are the actual traffic load effects.

The passage of this actual traffic load (certain groups of railway vehicles) over a bridge structure is allowed when relation

$$LLRF_{UIC} \geq l_{UIC} \quad (6)$$

is fulfilled.

The methodology of the LLRF calculation for steel railway bridges is specified in the Slovak standard [2], that was created at Department of Building Constructions and Bridges in Žilina in 1989. However, the same approach to the existing bridges as to newly designed ones from the reliability view point means the main disadvantage of the above mentioned standard conception.

3. PROBABILITY-BASED APPROACH TO THE RELIABILITY ASSESSMENT OF EXISTING BRIDGES

Incorporating relations (4) and (5) into relation (6), the basic formula for the traffic load carrying capacity - Traffic Load Rating Factor (TLRF) can be obtained in the following form

$$TLRF = (R - \Sigma S_i)/S_T, \qquad (7)$$

It means that the TLRF can be characterised as a relative resistance of a structure member in view of actual trallic load. The decision about passage of this actual traffic load over observed bridge structure shall be defined by relation TLRF > 1.

Now, if all the parameters entering the formula (7) are taken into account as the random variables, and S_T expresses all the random variable effects of the traffic load, then TLRF represents the reliability margin on the traffic load effects. The numerator in (7) could be treated as the effective resistance of a structure member, and in the sense of a probability approach [3], it may be defined by the relation

$$R_{ef} = f_y \, \varphi_w - \Sigma S_i ,$$

where f_y is the random variable yield strength of material,

φ_w is the mean-to-nominal ratio (bias factor) of the cross-sectional characteristic-fabrication factor.

Taking into account the effective resistance of a structure member according to (8), the relation (7) expresses the safety margin in a ratio form. The determination of TLRF could be introduced in two approaches. The first approach is semiprobabilistic one having a probabilistic base but a deterministic form. It means that all the parameters in relation (7) are treated as random variables, however, their values entering formula (7) are determined separately for corresponding probability occurrence. The second approach to TLRF determination is fully probabilistic. The actual value of TLRF can be obtained by their complete simulation according to relation (7) for certain probability occurrence corresponding to the recommended failure probability P_f. Because of difficulties in TLRF determination according to relation (7), it is convenient to apply the numerical procedures based on the simulation methods, for example Monte-Carlo [4] or LHS methods. However, both approaches to the TLRF calculations require a knowledge of the basic statistical characteristics of the actual traffic load. Recently, a considerable work has been done in conjuction with the development of live load model for newly designed bridges [5]. However, there is a need for the verification of these results from the point of view of present actual traffic load effects and present-day reliability theory. Therefore, the attention of our present research activities is paid to the numerical analysis of the dynamic response of steel railway bridge members to the actual traffic load and its verification by field measurements.

The process of the reliability assessment of existing bridge structure in traffic has some differences compared to the assessment of newly designed ones. Existing bridges are evaluted to determine their actual load carrying capacity (LLRF or TLRF) and predict the remaining life. From this point of view, the major differences between evaluation of existing bridges and design of new ones are as follows.

3.1 Different Reference Time Period

New bridges are usually designed for 80-year lifetime. In the case of existing bridge structure, it is possible to appreciate knowledge about its present technical state by inspections. It means that inspections help to reduce some uncertainty related to the resistance and load parameters. The results obtained during inspections may be included not only in the determination of load response and resistance of observed structure member but also in the base of reliability concept in the form of different failure probability level compared with newly designed bridges. Therefore, the interval between two following inspections may be recommended as the most convenient reference time period for the evaluation of existing bridges: The standard inspection interval is 4 or 5 years. Thus, the 4-year reference time period was recommended for evaluation of existing bridges.

3.2 Different Level of Failure Probability

In the case of newly designed bridges, the common design failure probability, P_{fd}, corresponding to the 80 year lifetime is 7.10^{-5} and target reliability index β_{td} of 3,81 in accordance with [1]. Taking into account the inspection results, the failure probability level, P_{ft} , may be higher and corresponding target reliability index, β_t, lower, respectively for the existing bridges inspected and evaluated at the regular 4-year reference period. The adjusted value of failure probability, P_{ft}, valid for 4-year period can be obtained using the formula

$$P_{ft} = 1 - (1 - P_{fd})^{Td/T}, \tag{9}$$

where T_d is the design lifetime of bridge structure, T_d = 80 years; and T is the reference time period for evaluation of existing bridges, T = 4 years.

For values Pfd = 7.10^{-5}, Td = 80 ears and T = 4 years the adjusted failure probability, P_{ft}, equals $1{,}4.10^{-3}$ and corresponding adjusted reliability index, $\beta_t = 3{,}0$. This value of β_t corresponds to the target reliability indices recommended also in other standards for the evaluation of existing bridges [6], [7]. The corresponding probability occurrence of the extreme load and minimum resistance are in accordance with [1], $P_{f3} = 1{,}8\ .10^{-2}$ and $P_{fR} = 8{,}2.10^{-3}$ for 4-year reference period.

4. NUMERICAL APPLICATION

As a numerical example of the above described LLRF and TLRF determination, we present their calculation for the plate girder with span, L = 25 m. The considered load combination includes dead load, long-term affecting load and live load.

4.1 Standard Approach

Using the deterministic standard approach according to relation [4], the load carrying capacity, $LLRF_{UIC}$, of the examined plate girder may be expressed by the following formula

$$LLRF_{UIC}=(R_d-\sigma_{gd}-\sigma_{qd})/\ \sigma_{UIC,d}=(210-21{,}91-10{,}32)/207{,}4=0{,}854, \qquad (4a)$$

where σ_{gd}, σ_{qd}, $\sigma_{UIC,d}$ are the design value of normal stresses due to dead (σ_{gd}) long-term affecting load (σ_{qd}) and ideal load train UIC-71 including dynamic effects ($\sigma_{UIC,d}$),

Rd is the design resistance according to [1].

The ratio value, λ_{UIC}, according to (5) may be usually obtained using railway guidelines. In this case, we can use the measured load response of the observed plate girder to the actual traffic load shown in Fig. 1 in the histogram form of normal stresses, σ_T. For the common probability occurrence P_{fs} = 4.10^{-3} of the extreme value of the normal stresses corresponding to the design value of the load actions, the value σ_T = 72,7 MPa can be found in Fig.1. Thus, the effect of this actual traffic load expressed as a function of the ideal load train UIC-71 effects shall be $\lambda_{UIC} = \sigma_T/\sigma_{UIC} = 0{,}35$. By a comparison of both values, λ_{UIC}, and, $LLRF_{UIC}$, ($LLRF_{UIC} > \lambda_{UIC}$), it is evident that passage of the actual traffic load over observed bridge structure is allowed.

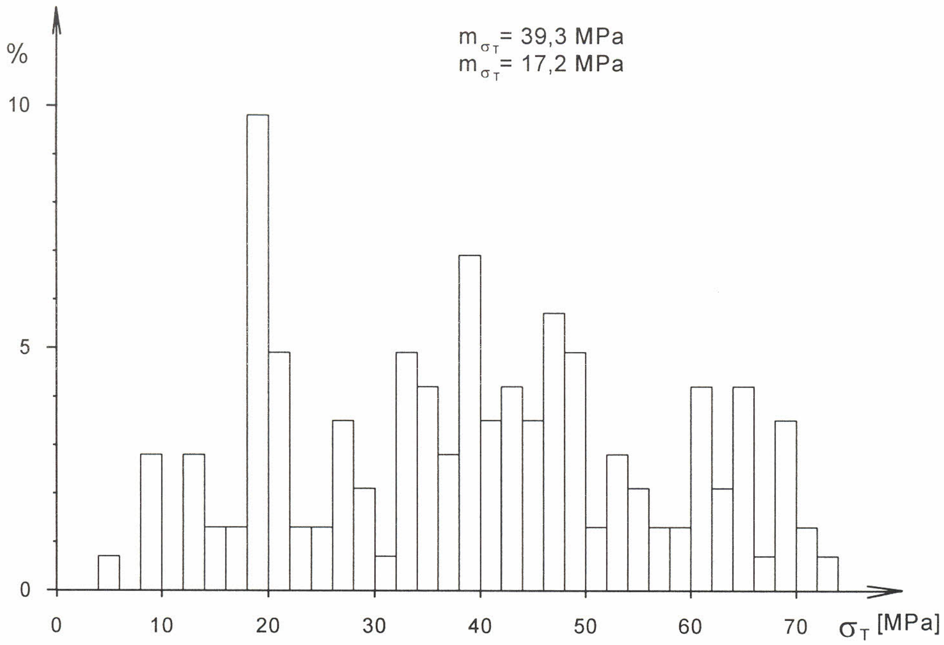

Fig.1 *The empirical distribution of the plate girder response to the actual traffic load*

4.2 Semiprobabilistic Approach

In the semiprobabilistic approach to the TLRF calculation, all design values entering formula (4) are supposed to be determined separately for certain probability occurrence corresponding to the recommended failure probability. First of all the actual bending resistance, R_b, of the observed plate girder has to be determined. In accordance with formula (8), the resistance, R_b, depends mostly on material strength, f_y and girder dimensions which are represented by fabrication factor, φ_w. To determine the actual value of the girder bending resistance, it is necessary to have statistical characteristics of the random variables, f_y, and φ_w. In this case, the following actual values of above mentioned random variables have been used:

f_y - normal distribution with basic parameters $m_f = 266{,}6$ MPa, $s_f = 21{,}7$ MPa,

φ_w - empirical distribution with the histogram according to Fig. 2,

where m_f is the mean of the random variable f_y, and s_f is its standard deviation.

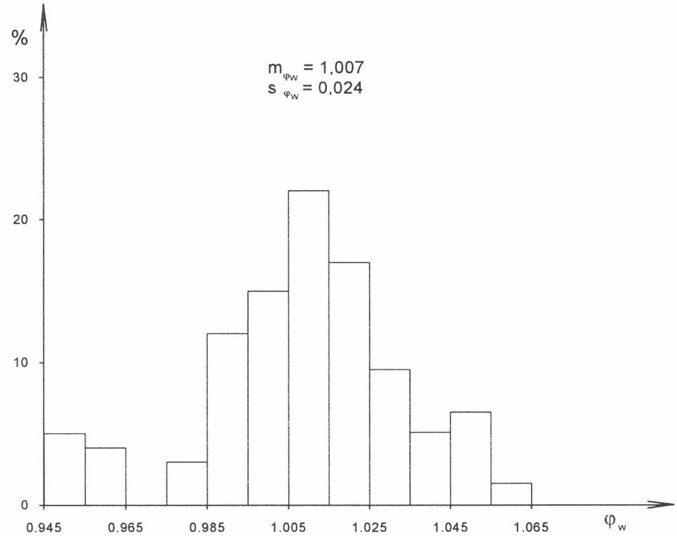

Fig. 2 *The empirical distribution of the fabrication factor, j_w.*

Using the Monte-Carlo simulation method, the histogram of actual bending resistance, R_b, has been developed. The basic statistical characteristics of the bending resistance are $m_R = 269{,}21$ MPa, $s_R = 22{,}95$ MPa, and for adjusted probability occurrence $P_{fR} = 8{,}2.10^{-3}$ of minimum bending resistance, the corresponding value of $R_b = 215{,}25$ MPa shall be found. The extreme value of the normal bending stresses due to traffic load corresponding to the adjusted probability occurrence of $1{,}8.10^{-2}$ is $s_T = 70{,}29$ MPa. Because of not time-dependent dead and long-term affecting load models, the same values of s_{gd} and s_{qd} can be used as in the case of standard approach. Thus, the TLRF can be calculated according to following formula

$$TLRF = (R_b - s_{gd} - s_{qd})/s_T = (215{,}25-21{,}91-10{,}32)/70{,}29 = 2{,}60. \quad (4a)$$

Results of calculation show that the passage of the actual traffic load over bridge structure is allowed.

4.3 Probabilistic Approach

In this case, the actual value of TLRF shall be obtained by its complete simulation in accordance with the relation (7). The same statistical characteristics of f_y, and φ_w as in previous approach have been used. For random variables, σ_{gd}, and, σ_{qd}, the following statistical characteristics have been used:

σ_{gd} - normal distribution with parameters : $m_{\sigma g}$ = 19,9 MPa, $s_{\sigma g}$ = 0,88 MPa,

σ_{qd} - normal distribution with parameters: $m_{\sigma q}$ = 8,6 MPa, $s_{\sigma q}$ = 0,74 MPa,

which are fully corresponding to the extreme values used in standard or semiprobabilistic approach. By the Monte-Carlo simulation of the random variable TLRF according to relation (7), its histogram has been developed that is shown in Fig. 3.

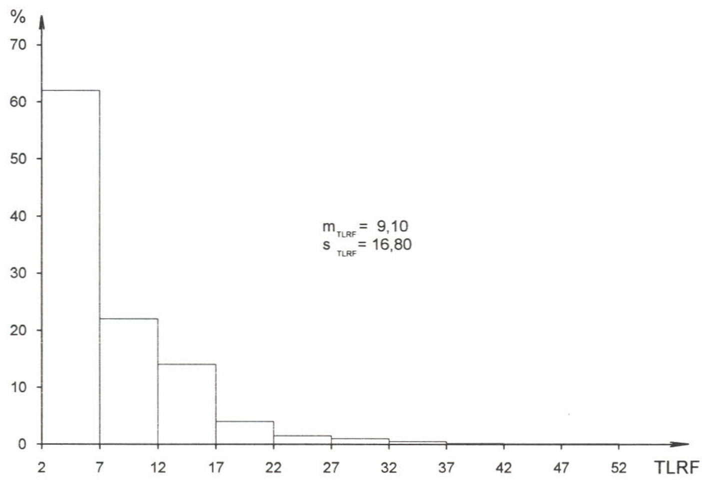

***Fig. 3** The histogram of the TLRF distribution*

The basic statistical characteristics of TLRF are m_{TLRF} = 9,10, s_{TLRF} = 16,80. In accordance with chapter 3.2, the actual value of TLRF corresponding to the adjusted failure probability, $P_{ft}=1{,}4.10^{-3}$ can be found. This actual value is TLRF = 2,56 > l.

In comparison of both determined values of TLRF (semiprobabilistic and probabilistic approach), it is evident that they are very close. In addition, it can be seen that probability P(TLRF < 1) is very small and it cannot be numerically reached.

5. CONCLUSIONS

The basic criteria and the methodology of the assessment of existing bridges are described in presented paper. The standard practice of the existing bridge evaluation in the terms of live load rating factor calculation is presented compared to the traffic load rating factor determination that is introduced in semiprobabilistic and fully probabilistic concept. Major differences between evaluation of existing bridges and assessment of newly designed ones are considered to define the adjusted failure probability level for bridge components of the evaluated and regularly inspected existing bridges. Numerical applications are presented to demonstrate described approaches.

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Resumé

PRAVDĚRPODOBNOSTNÍ VÝPOČET SPOLEHLIVOSTI OCELOVÝCH MOSTNÍCH KONSTRUKCÍ

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Článek popisuje pravděpodobnostní koncepci hodnocení existujících mostů. Je představená současná metodika výpočtu zatížitelnosti železničních mostů a je prezentovaný pravděpodobnostní přístup, založený na výpočtu provozní zatížitelnosti. Provozní zatížitelnost' je možné stanovit dvěma postupy - polopravděpodobnostním a pravděpodobnostním. V obou přístupech jsou promítnuté základní odlišnosti spolehlivostního přístupu k novênavrženým a existujícím mostům. V závěru je nový přístup dokumentovaný numerickou aplikací.

Summary

PROBABILISTIC APPROACH TO THE RELIABILITY OF STEEL BRIDGE STRUCTURES

Jiří SLAVÍK, Josef VIČAN, Hynek ŠERTLER

The paper describes a probability-based concept for the evaluation of existing bridges. Firstly the standard concept of the Live Load Rating Factor (LLRF) calculation is presented. Secondly, the probability-based concept of the Traffic Load Rating Factor (TLRF) calculation is described. The determination of TLRF is introduced in two approaches. Both approaches enable to respect the main reliability differences between evaluation of existing bridges and design of the new ones affecting the failure probability level. Numerical applications are presented to demonstrate described approaches.

Zusammenfassung

PROBABILISCHE BERECHNUNGSWEISE DER ZUVERLÄSSIGKEIT DER BRÜCKENSTAHL KONSTRUCTIONEN

Jiří SLAVÍK, Josef VIČAN, Hynek ŠERTLER

Der Beitrag befaßt sich mit einem probabilistischen Hintergrund der Beurteilung bestehender Brücken. Die Berechnung der Belastbarkeit bestehender Eisenbahnbrücken ist angeführt. Ein probabilistisches Verfahren der Beurteilung bestehender Eisenbahnbrücken ist auf der Betriebsbelastbarkeit gegründet, die durch semiprobabilistischen und völlig probabilistischen Zutritt ermittelt wird. Die grundlegende Besonderheiten in Beurteilung bestehender Eisenbahnbrücken gegenüber neu zu erstellenden Bauwerken werden vorgestellt. Die numerischen Beispiele der Berechnungen der Belastbarkeit und Betriebsbelastbarkeit sind angeführt.