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PARTIAL SAFETY FACTORS FOR EVALUATION OF EXISTING BRIDGES ACCORDING TO EUROCODES

P. Koteš [*], J. Vičan [**]

Abstract: *In the paper, the partial safety factors for materials and load effects recommended according to Eurocode for bridge members subjected to bending are presented. In the frame of research activities of the Department of structures and bridges, the modified reliability levels for existing bridge evaluation were derived. Firstly, these levels were used for determining partial safety factors for material. Closely, the partial safety factor of steel and concrete were determined depending on the age of the bridge and on the remaining lifetime of the bridge. New modified reliability levels for evaluation of existing bridges are also affected the partial safety factors of loads.*

Keywords: ***Bridge, existing structure, evaluation, partial safety factors.***

1. Introduction

Bridges are considered to be an inseparable and strategically very important part of the transportation infrastructure and they should have such parameters not to become the limiting component of the communication capacity and traffic reliability. In the past, bridge maintenance, repair and rehabilitation activities were performed on an "as-needed" basis. This changed in the late 1960s, when a series of bridge failures focused public attention on the deterioration of existing bridges, motivating governments to initiate standardized bridge inspection and evaluation procedures. Data collected through these inspection activities formed the basis for future computer based bridge management systems (BMS) (Lauridsen et al., 1998; Thompson et al., 1998; Vičan et al., 1998).

The evaluation of existing concrete bridge structures is the most important process in the global Bridge Management System (BMS) because of providing the basic information about existing bridges required from the viewpoint of decision making process related to the optimal bridge maintenance and rehabilitation strategy. Therefore, the existing bridge evaluation should be made not only as the result of periodic inspection on the base of subjective evaluation of actual bridge condition but from the viewpoint of the bridge reliability i.e. from the viewpoint how the actual bridge condition affects the bridge reliability for remaining bridge lifetime. Thus, the bridge evaluation becomes relevant when the significant deviations from the project descriptions are found, when some relevant damage is observed or when the bridge lifetime has gone beyond planned one, etc.

The paper deals with the determination of the modified reliability levels for evaluation of existing concrete bridges. The theoretical approach taking into account the conditional probability was used. The modified levels depend on the age of the bridge and on the planned remaining lifetime and, moreover, influence the partial safety factors of materials and loads.

2. Reliability-based evaluation of existing concrete bridges

The reliability level for newly designed bridges for whole lifetime T_d ($T_d = 100$ years), which is represented by failure probability $P_{f,d}$ ($P_{f,d} = 7.2\cdot10^{-5}$) or by reliability index β_d ($\beta_d = 3.8$), is given in a Eurocode. However, the reliability level for evaluation of existing bridges for remaining lifetime t_r is not given in the Eurocodes.

[*] Ing. Peter Koteš, Ph.D.: Department of Structures and Bridges, Civil Engineering Faculty, University of Žilina, Univerzitná 8215/1; 010 26, Žilina; SK, e-mail: kotes@fstav.uniza.sk

[**] Prof. Ing. Josef Vičan, Ph.D.: Department of Structures and Bridges, Civil Engineering Faculty, University of Žilina, Univerzitná 8215/1; 010 26, Žilina; SK, e-mail: vican@fstav.uniza.sk

Generally, the process of the existing bridge evaluation has various differences in comparison with the reliability assessment of newly designed bridge. In the case of the existing bridge structure, new information concerning the actual bridge condition is available which is unknown in the design phase. The certificates of material properties, measurements of actual bridge geometry, collection load data, results of proof load testing and especially results of the periodic inspections regularly performed within lifetime of the observed bridge are the major resources of this information. The extra information unknown in the design phase can be used not only for verification of the correct bridge performance or for detection of possible mistakes concerning the computational model assumptions or calculations but also helps to reduce some uncertainty related to the bridge member resistance and load parameters entering the evaluation process.

In abroad, the problem of evaluation of existing bridges was solved in frame of the developing Bridge Management Systems based on computer-aided expert systems. The reliability-based evaluation of existing bridges is preferred in the works of American (Nowak & Gruni, 1994; Frangopol & Estes, 1997) and Canadian (Allen, 1992; Bartlett et al., 1992) authors. Theoretical outputs of these scientific studies create the background of the contemporary Canadian (Reel & Agorwal, 1997) and Ontario (OHBDC, 1991) standards for the evaluation of existing road bridges and determining their load carrying capacities in the form of Live Load Rating Factors (LLRF). Both standards are based on the probability model of the structural reliability verification with the differentiated reliability level depending on the bridge component importance in the whole bridge structure.

In the area of Europe, several authors and institutions have focused their research activities on this problem last years (Wong ar al., 2005; Draft BD 79, 2000). A publication of the Joint Committee on Structural Safety (JCSS) (Diamantidis, 2001) is being developed. This publication contains some practical and operational recommendations and rules for the assessment of existing structures.

3. Reliability analysis

From the bridge reliability view point, the reduction of the load and resistance parameter uncertainties decreases failure probability of existing bridge structure that means the possibility to admit lower reliability level for evaluation of existing bridge than it is in the case of newly designed one.

In the theoretical analysis, it is assumed that the bridge structural element was designed for total lifetime T with corresponding reliability index β given by formula

$$\beta = \left(m_R - m_S\right) \Big/ \sqrt{s_R^2 + s_S^2} \tag{1}$$

where m_R, s_R are the basic parameters of the normally distributed random variable resistance R of a bridge structural elements,

m_S, s_S are the basic parameters of the normally distributed random variable load effects S of the same bridge element.

The bridge inspection was performed at the time $t_{insp} < T$ during which the observed structural element was found to be without relevant failure due to overcrossing its limit states. This positive information expresses that resistance R of the observed structural element satisfies the following relation

$$R > max\ (S_i) \quad for\ \ i = 1\ \ldots\ldots\ N(t). \tag{2}$$

The load effects S_1, S_2 …. S_n are mutually independent normally distributed and occur in succession but randomly in time and N(t) means the random number of them within time interval (0, t). N(t) is considered as the random variable having Poisson distribution with parameter λ(t) (intensity of load effects) which is constant or linearly dependent on time t according to relation

$$\lambda(t) = \lambda_0 + \left(\lambda_{insp} - \lambda_0\right) \cdot t \Big/ t_{insp} \ , \tag{3}$$

where λ_o is the value of parameter λ at the time t = 0,

$\lambda_{t,insp}$ is the value of parameter λ at the time $t = t_{insp}$ of the periodic inspection.

If the following formula is considered

$$L(t) = \int_0^t \lambda(\tau) d\tau \ , \tag{4}$$

then time occurrence of individual sets of load effects S_i satisfies the following dependence

$$P(N(t) = n) = L(t)^n \cdot e^{-L(t)} / n!\, , \textit{for } n = 0,1 \ldots k. \tag{5}$$

If the parameter $\lambda(t)$ is constant in time, the following relation may be obtained using the relation (3)

$$L(t) = \lambda \cdot t\ , \tag{6}$$

and if the parameter is linearly dependent on time, using substitution (3) to (4) is obtained

$$L(t) = \lambda_0 \cdot t + \left(\left(\lambda_{insp} - \lambda_0 \right) \cdot t^2 \right) / \left(2 \cdot t_{insp} \right). \tag{7}$$

As has been shown in Ditlevsen & Madsen (1996), the updated failure probability P_{fu} of the observed structural element at the time period (t_{insp}, T) should be obtained by means of the conditional probability according to the formulae

$$P_{fu} = \left(P_f(T) - P_f(t_{insp}) \right) / \left(1 - P_f(t_{insp}) \right). \tag{8}$$

The corresponding updated reliability index β_u of the observed structural element for the remaining time period (t_{insp}, $T = T_d$) can be determined in accordance with

$$\beta_u = -\Phi^{-1}\left(P_{fu} \right), \tag{9}$$

where Φ^{-1} is the inverse distribution function of standardized normal distribution N(0,1).

The failure probability $P_f(T)$, $P_f(t_{insp})$ can be obtained for normally distributed bridge element resistance R and normally distributed load effects S_i using the following formulae for complete probability (Ditlevsen & Madsen, 1996)

$$P_f(T) = P\left[\max(S_i)(i = 1 \ldots N(T) > \mathrm{R} \right] = \int_{-\infty}^{\infty} \left(1 - e^{-L(T)\Phi\left(-\frac{x - m_S}{s_S} \right)} \right) \cdot \varphi\left(\frac{x - m_R}{s_R} \right) \cdot \frac{1}{s_R} dx\ , \tag{10}$$

where φ is the probability density function of standardized normal distribution N(0,1).

Using the information (2), the updated reliability index β_u can be greater than designed index β_d. Next, we are able to solve back the adjusted target failure probability P_{ft} (target reliability index β_t) for which the element should be evaluated for remaining lifetime (T-t_{insp}) so that we can achieve the required value of the target failure probability P_{ft} with minimal one inspection. The change of the updated reliability index β_u and the target reliability index β_t in time is shown in Fig. 1 in dependence on time of inspection and on the values of the parameter λ. From the Fig. 1 can be seen that the updated reliability index β_u is increasing in time (markedly in the end of lifetime). From this reason, the target reliability index β_t is decreasing in time.

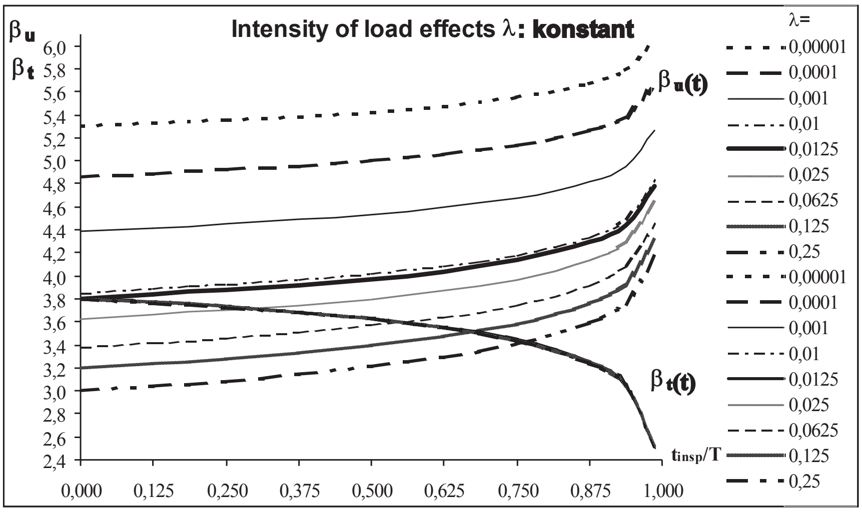

Fig. 1: Updated reliability index β_u and the target reliability index β_t in dependence on inspection time.

The reliability level given by failure probability P_{ft} or by reliability index β_t shown in Fig. 1 depends just on the full remaining lifetime (T - t_{insp}) – from time of inspection t_{insp} to the end of the lifetime T. But practically, it is usually to evaluate the structure for shortening lifetime – us selected time interval. For example, it can be time between two inspections or if the structure does not satisfy for full remaining lifetime (T- t_{insp}). In this case, the structure can be evaluated on shortening remaining lifetime – planned remaining lifetime t_r.

The theoretical approach is the same as above mentioned. But, the lifetime T of the member should be shortening to determine the required reliability of observed member for planned interval t_r. It means that the whole lifetime is not T = 100 years, but it is equal to sum t_{insp}+t_r.

This approach is important for bridge owner, because it gives to owner ability to save the funds. The results are shown in Fig. 2.

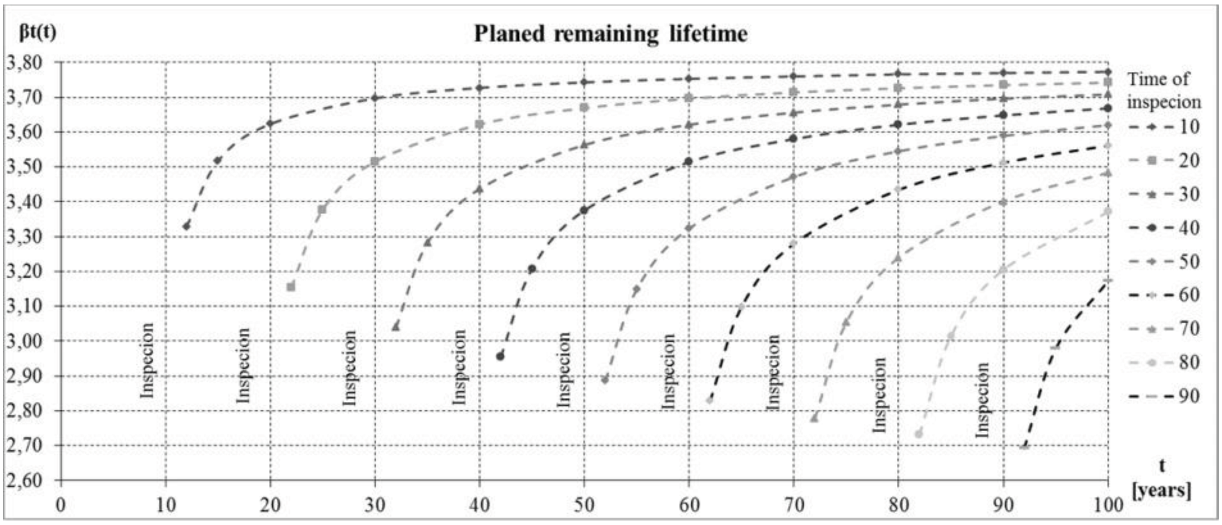

Fig. 2: Target reliability index β_t in dependence on time of inspection and planned remaining lifetime.

4. Reliability levels

The obtained reliability levels depend on the age of the bridge and on the planned remaining lifetime. The results of the reliability levels for bridge element not respecting the degradation due to regularly performed maintenance are shown in Tab. 1.

Tab. 1: Reliability levels for existing bridge evaluation not respecting degradation.

Remaining lifetime [years]	10. years		20. years		30. years		40. years		50. years	
	β_t	P_{ft}	β_t	P_{ft}	β_t	P_{ft}	β_t	P_{ft}	β_t	P_{ft}
2	3.328	$4.38\cdot10^{-4}$	3.153	$8.09\cdot10^{-4}$	3.039	$1.19\cdot10^{-3}$	2.954	$1.57\cdot10^{-3}$	2.886	$1.96\cdot10^{-3}$
5	3.517	$2.19\cdot10^{-4}$	3.377	$3.67\cdot10^{-4}$	3.282	$5.16\cdot10^{-4}$	3.208	$6.68\cdot10^{-4}$	3.149	$8.21\cdot10^{-4}$
10	3.623	$1.46\cdot10^{-4}$	3.515	$2.20\cdot10^{-4}$	3.437	$2.94\cdot10^{-4}$	3.375	$3.70\cdot10^{-4}$	3.323	$4.46\cdot10^{-4}$
20	3.697	$1.09\cdot10^{-4}$	3.622	$1.46\cdot10^{-4}$	3.563	$1.83\cdot10^{-4}$	3.514	$2.21\cdot10^{-4}$	3.471	$2.59\cdot10^{-4}$
30	3.727	$9.70\cdot10^{-5}$	3.669	$1.22\cdot10^{-4}$	3.621	$1.47\cdot10^{-4}$	3.58	$1.72\cdot10^{-4}$	3.545	$1.97\cdot10^{-4}$
40	3.743	$9.08\cdot10^{-5}$	3.696	$1.09\cdot10^{-4}$	3.656	$1.28\cdot10^{-4}$	3.621	$1.47\cdot10^{-4}$	3.589	$1.66\cdot10^{-4}$
50	3.753	$8.72\cdot10^{-5}$	3.714	$1.02\cdot10^{-4}$	3.679	$1.17\cdot10^{-4}$	3.648	$1.32\cdot10^{-4}$	3.62	$1.47\cdot10^{-4}$
60	3.76	$8.48\cdot10^{-5}$	3.726	$9.72\cdot10^{-5}$	3.696	$1.10\cdot10^{-4}$	3.668	$1.22\cdot10^{-4}$		
70	3.766	$8.31\cdot10^{-5}$	3.735	$9.38\cdot10^{-5}$	3.708	$1.05\cdot10^{-4}$				
80	3.77	$8.18\cdot10^{-5}$	3.742	$9.12\cdot10^{-5}$						
90	3.773	$8.07\cdot10^{-5}$								

Remaining lifetime [years]	60. years		70. years		80. years		90. years	
	β_t	P_{ft}	β_t	P_{ft}	β_t	P_{ft}	β_t	P_{ft}
2	2.828	$2.35\cdot10^{-3}$	2.777	$2.75\cdot10^{-3}$	2.732	$3.15\cdot10^{-3}$	2.692	$3.56\cdot10^{-3}$
5	3.098	$9.75\cdot10^{-4}$	3.053	$1.13\cdot10^{-3}$	3.014	$1.29\cdot10^{-3}$	2.978	$1.45\cdot10^{-3}$
10	3.279	$5.22\cdot10^{-4}$	3.239	$6.00\cdot10^{-4}$	3.204	$6.78\cdot10^{-4}$	3.172	$7.57\cdot10^{-4}$
20	3.434	$2.97\cdot10^{-4}$	3.401	$3.35\cdot10^{-4}$	3.371	$3.74\cdot10^{-4}$		
30	3.512	$2.22\cdot10^{-4}$	3.483	$2.48\cdot10^{-4}$				
40	3.561	$1.85\cdot10^{-4}$						

5. Partial safety factors

New modified reliability levels for evaluation of existing bridges given in Tab. 1 affect the values of partial safety factors for material resistance and for loads, also. In the practical design, the reliability levels are transformed to the design values of the material resistance and loads. In the partial safety factors method, the design values of material resistance and loads are determined by means of characteristic values and appropriate partial safety factors. Loads and resistance are treated as random variables and are described by bias factors λ (expressing ratio of mean value to nominal value) and by coefficient of variation v.

5.1. Partial safety factors for material

Considering normally distributed random variable resistance, the partial safety factors of concrete and reinforcement (EN 1991-1-1, 2002) are given by formulae

$$\gamma_M = \frac{R_k}{R_d} = \frac{1-\beta_k \cdot v_R}{1-\alpha_R \cdot \beta_t \cdot v_R}, \tag{11}$$

where R_k is the characteristic value of the material resistance,

R_d is the design value of the material resistance,

$\alpha_R = 0.8$ is the sensitivity coefficient,

v_R is the coefficient of variation of material resistance,

$\beta_k = 1.645$ is the reliability index corresponding to probability 5 % (valid for characteristic values),

β_t is the recommended target reliability index depending on the age of the bridge and on the planned remaining lifetime (Tab. 1).

To determine values of partial safety factors, the basic statistical characteristics shall be known, especially coefficient of variation v_R. In the case of partial safety factor γ_c for concrete, the value of variation coefficient v_R was backward calculated from formulae (11) for $\gamma_c = 1.50$ (for new designed bridges) and reliability index $\beta_t = 3.80$. So, the value of coefficients of variation $v_R = 0.172$ was used for determining the partial safety factors of concrete.

Using mentioned parameters, new values of partial safety factors for concrete were determined considering the planned remaining lifetime and a bridge age. The determined values are shown in Tab. 2.

Tab. 2: The partial safety factor γc for concrete strength valid for existing bridge evaluation

Planned remaining lifetime t_r [years]	γ_c - age of bridge [years]		
	< 60 years	60-80 years	> 80 years
< 2	1.21	1.20	1.15
2 – 10	1.35	1.31	1.30
10 – 20	1.40	1.36	1.34
20 – 40	1.43	1.41	
> 40	1.45		

In the case of the partial safety factor γ_S for reinforcement, it is possible to use the value of $v_R = 0.081$, which is corresponding to partial safety factor $\gamma_s = 1.15$ for reinforcement (for new designed bridges) and reliability index $\beta_t = 3.80$. The determined values of the partial safety factors for reinforcement are shown in Tab. 3. For the serviceability limit state were determined values $\gamma_c = \gamma_c = 1.0$ and $\gamma_s = 1.0$.

Tab. 3: The partial safety factors γs for reinforcement valid for bridge existing evaluation

Planned remaining lifetime t_r [years]	γ_s - age of bridge [years]		
	< 60 years	60-80 years	> 80 years
< 2	1.10	1.10	1.06
2 – 10	1.11	1.11	1.10
10 – 20	1.13	1.12	1.11
20 – 40	1.14	1.13	
> 40	1.14		

5.2. Partial safety factors for loads

The load of the bridges is given in the code STN EN 1991-2 (2006). The load models are an important part of evaluation. The basic load combination for road bridges is a simultaneous occurrence of permanent load and variable load. The code STN EN 1991-2 (2006) specifies the characteristic loads, partial safety factors for newly designed bridges and the load combinations. However, the calculation of load partial safety factors for evaluation of existing bridges requires knowledge of the statistical models of singular loads, in particular distribution function, density function, standard deviation, coefficient of variation, time variation and correlation with other load components.

Permanent loads

The partial safety factors of permanent loads respecting the recommended modified reliability levels (expressed by β_t) given in Tab. 1 considering normally distributed random variables were established using the formulae

$$\gamma_{G,i} = \gamma_{Sd} \cdot \frac{S_d}{S_k} = \gamma_{Sd} \cdot \frac{\mu_{G,i}.(1+\alpha_S \cdot \beta_t \cdot v_G)}{\mu_{G,i}} = \gamma_{Sd} \cdot (1+\alpha_S \cdot \beta_t \cdot v_{G,i}) \,, \tag{12}$$

where S_k is the characteristic value of load,

S_d is the design value of load,

$\alpha_S = 0.7$ is the sensitivity coefficient of loads,

$\gamma_{Sd} = 1.05$ is the partial safety factor of model uncertainties,

$v_{G,i}$ are the coefficients of variation of single permanent loads.

According to standard STN EN 1990/A1/NA (2007), the permanent loads are divided into cast-in-place made produces with recommended value of partial safety factor $\gamma_G = 1.35$ (for new designed bridges) and factory-made produces and transported to construction with recommended value of partial safety factor $\gamma_G = 1.25$ (for new designed bridges). The value of variation coefficient v_R is equal to $v_G = 0.107$ for cast-in-place made produces and $v_G = 0.072$ for factory-made produces and transported to construction.

The new recommended partial safety factors of permanent loads depending on the age of bridges and on planned remaining lifetime are shown in Tab. 4 and Tab. 5.

Tab. 4: The partial safety factors of permanent loads - cast-in-place made produces

Planned remaining lifetime	$\gamma_{G,i}$ - age of bridge [years]		
t_r [years]	< 60 years	60-80 years	> 80 years
< 2	1.29	1.28	1.27
2 – 10	1.32	1.31	1.31
10 – 20	1.33	1.32	1.32
20 – 40	1.34	1.33	
> 40	1.34		

Tab. 5: The partial safety factors of permanent loads - factory-made produces and transported to construction

Planned remaining lifetime	$\gamma_{G,i}$ - age of bridge [years]		
t_r [years]	< 60 years	60-80 years	> 80 years
< 2	1.21	1.20	1.20
2 – 10	1.23	1.23	1.22
10 – 20	1.24	1.24	1.23
20 – 40	1.25	1.24	
> 40	1.25		

Variable loads

The variable loads are random variables with Gumble distribution according to STN EN 1990 (2009). The partial safety factors of variable loads respecting the recommended modified reliability levels (expressed by β_t) given in Tab. 1 considering Gumble distributed random variables were also established using the formulae

$$\gamma_Q = \gamma_{Sd} \cdot \frac{S_s}{S_k} = \gamma_{Sd} \cdot \frac{\mu_Q \cdot \{1 - v_Q[0,449 + 0,778 \cdot ln(-ln\Phi(\alpha_S \cdot \beta_t)]\}}{\mu_Q \{1 - v_Q[0,449 + 0,778 \cdot \ln(-\ln(0,95)]\}}, \tag{13}$$

where S_k is the characteristic value of load,

S_d is the design value of load,

$\alpha_S = 0.7$ is the sensitivity coefficient of loads,

$\gamma_{Sd} = 1.05$ is the partial safety factor of model uncertainties,

v_Q is the coefficients of variation of variable load.

The variable loads are different to variable loads of road bridges and variable loads of railway bridges. In the case of road bridges according to standard STN EN 1990/A1/NA (2007), the partial safety factor is equal to $\gamma_Q = 1.35$ and in the case of road bridges, the partial safety factor is equal to $\gamma_Q = 1.40$ (for new designed bridges). The value of variation coefficient v_Q is equal to $v_Q = 0.1944$ for loads on road bridges and $v_Q = 0.2414$ for loads on railway bridges considering the formulae (13).

The new recommended partial safety factors of variable loads depending on the age of bridges and on planned remaining lifetime are shown in Tab. 6 and Tab. 7.

Tab. 6: The partial safety factors of variable loads on road bridges

Planned remaining lifetime	$\gamma_{Q,i}$ - age of bridge [years]		
t_r [years]	< 60 years	60-80 years	> 80 years
< 2	1.17	1.14	1.12
2 – 10	1.26	1.23	1.22
10 – 20	1.29	1.27	1.26
20 – 40	1.31	1.30	
> 40	1.32		

Tab. 7: The partial safety factors of variable loads on railway bridges

Planned remaining lifetime t_r [years]	$\gamma_{Q,i}$ - age of bridge [years] < 60 years	60-80 years	> 80 years
< 2	1.19	1.16	1.14
2 – 10	1.29	1.27	1.25
10 – 20	1.33	1.30	1.29
20 – 40	1.35	1.34	
> 40	1.37		

6. Conclusions

The paper presents the results of the research concerning the reliability levels for evaluation of existing bridges. The modified reliability levels for evaluation were determined and they depend on the bridge age and on planned remaining lifetime. The values of the levels are valid for members subjected to bending. Theoretical reliability basis for modification of partial safety factor method due to allowing for the major differences between existing bridge evaluation and design of the new ones is presented.

In final consequence, the lower reliability levels reflect into the partial safety factors of materials and loads. In the paper are shown determined partial safety factors for concrete γ_c, partial safety factor for reinforcement γ_s and partial safety factors for permanent loads $\gamma_{G,i}$ and variable loads $\gamma_{Q,i}$.

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