

COMPARISON OF APPROACHES TO RELIABILITY VERIFICATION OF EXISTING STEEL RAILWAY BRIDGES

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Abstract. Many existing steel railway bridges are exposed to degradation and increasing traffic loads. Their reliability assessment may contribute to sustainability in construction particularly when these bridges are kept in service. The sustainability aspects mainly relate to reducing waste material and to unnecessary replacements of existing bridges. The key aspects of assessment include specification of the target reliability and selection of an appropriate assessment method. The most efficient verification methods are based on advanced probabilistic approaches. The application of the adjusted partial factor method seems to be reasonable as it provides the balance between demands on the input information, computational complexity, and achieved improvements in reliability assessment. In this contribution, the advanced reliability approach is illustrated by an example of the assessment of a 90-years old steel railway bridge. This case study demonstrates that advanced reliability approaches may significantly improve assessment and allow to increase the estimated load-bearing capacity of the bridge by 10 to 20%.

Keywords

Existing railway bridges, adjusted partial factors, probabilistic approaches, reliability.

1. Introduction

Many existing steel railway bridges are exposed to degradation due to corrosion or fatigue and increasing traffic loads. Their reliability assessment is urgently needed. The key question is whether a particular bridge can be preserved, or needs to be strengthened, or whether it needs to be replaced. In general, the sustainability aspects relate to reducing waste production and to the recycling of structural materials [1].

The assessment is positively contributing to

sustainability in construction, particularly when existing bridges are maintained and kept in service. Unnecessary replacements of existing bridges should be avoided. The assessment of existing steel bridges may be improved by:

- Using advanced reliability verification techniques – main focus of this study,
- Optimizing target reliability – discussed briefly,
- Obtaining data for a specific site or structure – discussed in previous studies [2, 3].

Lower reliability levels can be accepted for existing structures in comparison to structural design as follows from the general principles of structural reliability provided in ISO 2394:2015; see also [1, 4]. Improving target reliability by implementing cost optimization procedures and criteria for human safety are presented in [5]. The guideline of SZ (Czech railways network operator) [6] gives target reliability indices β_t for load-bearing members of existing bridges considering current age of the bridge and the required remaining service life; see also [7, 8].

At present, existing structures are mostly verified using simplified procedures based on the partial factor method commonly applied in the design of new structures. Such assessments are often conservative for existing structures and can lead to expensive upgrades. A more realistic verification of the actual performance of existing structures can be achieved by using:

- Adjusted partial factors where the assessment values are obtained as fractiles of updated probabilistic distributions corresponding to probability defined on the basis of sensitivity factor and a selected target reliability level.
- Probabilistic methods considering all basic variables describing loads and resistances as random variables using appropriate probabilistic models based on available experimental data.

The submitted study is aimed at improvements methods of reliability assessment gained by applying

advanced procedures in reliability assessment of an existing steel railway bridge; a comparison with results obtained by application of the partial factor method recommended for structural design is provided. In the generic case study, a resistance of the cross-section under bending (sufficiently braced to restrain instability; thus without buckling effects) is analysed.

2. Fixed Partial Factors

2.1. Eurocodes

EN 1990 [9] is the basic document that suggests the load combinations and relevant partial factors. The following partial factors are recommended for structural design:

- Permanent loads: $\gamma_G = 1.35$ and $\xi = 0.85$
- Rail traffic load represented by load model LM71 ($\gamma_{Q,LM71} = 1.45$ and $\psi_0 = 0.8$) and/ or by a real train ($\gamma_Q = 1.45$ and $\psi_0 = 1.0$)

2.2. Guideline of SZ

In addition, SZ S5 [6] provides guidance for adjusting partial factors for existing bridges. According to clause 4.3 [6], partial factors are assigned depending on the age of the bridge, making a distinction between oldness under 30 years or over 30 years; the former is deemed to be designed according to the principles of Eurocodes. The partial factors for permanent loads are differentiated for steel and precast concrete members or members from other materials, and with or without measuring the dimensions in-situ (see Section 3 for further details). The value of reduction factor ξ for permanent load is 0.95.

When using the LM71 model to determine the load capacity of structural members, the following partial factors are considered (unless they are adjusted as described in Section 3):

- $\gamma_{Q,LM71} = 1.4$ for bridges with the age less than 30 years,
- $\gamma_{Q,LM71} = 1.3$ for older bridges,

disregarding required remaining service life.

The rules for combining the effects of wind load with the effects of rail traffic load are considered in accordance with EN 1990 [9] but combination factor ψ can be adjusted according to [6].

3. Adjusted Partial Factors

General guidelines for adjusting and updating partial factors are provided by the basic Eurocode EN 1990 [9]. Partial factors may be adjusted considering structure itself

(information about materials, dimensions, permanent actions, system behaviour etc.) and site-specific conditions (e.g. information about variable loads).

The assessment values of the partial factors are obtained as fractiles of the relevant basic variable corresponding to the reduced probability given by the product of generalised values of sensitivity factors and a selected target reliability level.

This procedure makes it possible to explicitly consider specified target reliability and variability of basic variables. The main deficiency of this approach may be attributed to the need to assume values of the sensitivity factors for resistance and load effect variables.

More detailed guidelines for determining the values of the partial factors for railway bridges are presented in Annex F of the guideline SZ [6]. In contrast to Section 4.3 in the guideline, these values are now calculated specifically for a selected remaining service life and a corresponding target reliability index. The partial factor for the permanent load is determined assuming a normal distribution:

$$\gamma_G = \gamma_{sd} \cdot (1 - \alpha_E \cdot \beta_t \cdot V_G), \quad (1)$$

where:

α_E is the sensitivity factor of the FORM method for load effects ($\alpha_E = -0.7$ for the dominating load parameter; considered for the permanent load formula 6.10a in EN 1990, or $\alpha_E = -0.28$ for other load parameters; for G in formula 6.10b);

γ_{sd} is a partial factor taking account of the uncertainties of the load model and load effect model, approximated as $\gamma_{sd} = 1.05$;

V_G is the coefficient of variation of the permanent load, determined on the basis of measured geometry and densities; if the measured densities are not available, then $V_G = 0.10$ is considered for load-bearing and secondary members with uncontrolled measurements and reduced $V_G = 0.05$ for members with dimensions checked by measurements.

The partial factor for variable actions due to rail traffic load effect is determined assuming a Gumbel distribution:

$$\gamma_{Q,LM71} = \gamma_{sd} \{1 - V_Q [0.449 + 0.778 \ln(-\ln \Phi(-\alpha_E \beta_t))]\} / \{1 - V_Q [0.449 + 0.778 \ln(-\ln(0.95))]\} \quad (2)$$

where V_Q is the coefficient of variation of the rail traffic load whose recommended value is 29%. Note that no reference period is indicated in this formula.

SZ S5 [6] recommends specification of the partial factors considering the Gamma distribution for resistances of structural steel members manufactured up to 1968, or to use tabularised values for members produced after 1968. The Gamma distribution makes it possible to reflect the changes in the mechanical properties of steels and the variation of geometric dimensions of structural members.

Note that this assumption is slightly inconsistent with

the general recommendations of EN 1990 and ISO 2394. These standards indicate lognormal, normal or Weibull distributions as appropriate models for material properties [9, 10] while a normal distribution is commonly applied for dimensions.

The Tab. 1 provides a comparison of the partial factors indicated in different documents considered in this study.

Tab.1: Overview of partial factors indicated in different documents

	EN	SZ, Section 4.3		SZ Annex F	
		<30 ^(a)	>30 ^(b)	$\beta = 3.5$	$\beta = 3.0$
γ_G	1.35	1.25 ^(c) 1.3 ^(d)	1.2 ^{(c),(e)} 1.25 ^{(c),(f)} 1.2 ^{(d),(e)} 1.25 ^{(d),(f)}	1.18 ^(e) 1.31 ^(f)	1.16 ^(e) 1.27 ^(f)
ζ	0.85	0.95	0.95	0.93 ^(e) 0.88 ^(f)	0.94 ^(e) 0.90 ^(f)
γ_O	1.45	1.45	1.3	1.35	1.21
ψ_0	0.8 ^(g) 1.0 ^(h)	0.6	0.5 ^(g) 0.75 ^(h)	0.64	0.68
γ_{M0}	1.0	1.0	1.1	1.12 ⁽ⁱ⁾ 1.16 ^(j)	1.08 ⁽ⁱ⁾ 1.11 ^(j)
γ_{M1}	1.0	1.1	1.2 ⁽ⁱ⁾ 1.25 ^(j)	=1.1 γ_{M0}	=1.1 γ_{M0}

^(a) Members or structural parts less than 30 years old, ^(b) Members or parts more than 30 years old, ^(c) Steel and precast concrete members, ^(d) Members from other materials, ^(e) Dimensions checked by measurements, ^(f) Without checking dimensions, ^(g) LM71, ^(h) real train, ⁽ⁱ⁾ for steel class S235, ^(j) for steel class S355.

4. Probabilistic Reliability Analysis

A generic limit state function for members of railway steel bridges may be written as follows:

$$g(\mathbf{x}) = K_R R - K_E [G + (1 + \varphi) Q_{\text{stat}}] \quad (3)$$

where K_R and K_E are random variables characterizing the uncertainty in resistance and load effect models respectively, R is a random variable characterizing the resistance of the cross-section or of structural member, G is a random variable characterizing the permanent load, $(1 + \varphi)$ is the dynamic amplification factor (φ being the dynamic enhancement), Q_{stat} is the static component of the train load.

4.1. Resistance

The statistical parameters of the yield stress of steels produced in the European Union, deemed to be relevant for the assessment of existing steel bridges built in 1950s and later, are presented in [11–14]. Some of the latest data on the statistical parameters for steels currently available on the European market satisfying the relevant European product standards are presented in [15]. In the assessment of existing structures, it is possible to measure material and geometrical properties of steel members that may considerably vary for different steel grades, profiles and production processes adopted by various producers. For existing steel bridges, the main source of uncertainty is a within-batch (within-rolling) variability. CSN

730038 [16], the Czech standard for assessment of existing structures, recommends coefficient of variation of yield stress $V_{fy} = 3\text{--}5\%$. Based on these assumptions, Lenner et al. [2] considered $\mu_{fy} / f_{yk} = 1.09$ and $V_{fy} = 5\%$ for historic steel bridges; these characteristics are adopted in this study.

Data on variability of the geometric characteristics of rolled I-sections produced in the Czech Republic and in Great Britain are presented in [12, 13], respectively. For main girders of bridges, high (deep) welded sections are often used. Variability of high cross sections is relatively less important for structural reliability than that of small profiles (even though variability of dimensions of welded sections is commonly slightly higher than for rolled sections). The variability of the geometric characteristics for steel structures is small, the coefficient of variation is 2–5% [15, 17]. When dimensions are verified in-situ, unbiased values and a lower coefficient of variation can be considered [2], $V_{geo} = 3\%$ is taken for further analysis. When combined with V_{fy} , the following coefficient of variation of resistance (see also Tab. 2 in the following text):

$$V_R = \sqrt{(V_{fy}^2 + V_{geo}^2)} = 5.8\% \quad (4)$$

It should be noted that probabilistic description of uncertainties in resistance models is insufficiently addressed in the scientific literature. In the general case, the statistical parameters of the model uncertainties are determined empirically by comparing the test results with theoretical calculations. The basis for assessing uncertainties in design models is a database of test results, which should include all information necessary for repeating the tests and for determining resistance values according to the models.

The generalization of the statistical characteristics of the resistance model uncertainties is complicated by continuous development of resistance models. As a rule, resistance models for cross-sections are similar across a range of documents as they are based on the fundamental principles of the representation of material resistances. In contrast, resistance models for structural members, typically those for stability verifications, are often more complex, partly empirical, and differ significantly between normative documents as they may be based on various experimental data. Additionally, it can be assumed that for historic steel members resistance model uncertainty might be larger due to the fact that some resistance models consider imperfections (welding stresses, initial curvatures, eccentricity, etc.).

The JCSS Probabilistic Model Code [17] contains the general recommendations regarding statistical parameters for resistance model uncertainties, which are often adopted in probabilistic analyses. Some more detailed information about resistance model uncertainty characteristics for steel members is provided in [18]. In this study, a resistance model of the cross section under bending (sufficiently braced to restrain instability; thus without buckling effects) was adopted with the following statistical parameters $\mu_{KR} = 1.1$ and $V_{KR} = 5\%$ [18].

4.2. Load Effects

Bridges may be exposed to effects of permanent loads, rail traffic loads, climatic actions, differential settlements, water and earth pressures, earthquakes, accidental actions etc. The following analysis is focused on two key load types for railway bridges – permanent loads and rail traffic loads.

1) Permanent load

Permanent loads are caused by self-weight of structural and non-structural members connected to the bridge, including surfacing and other coatings. As a rule, it is useful to divide the total permanent load into components, considering different variability of material densities and geometric dimensions. The permanent loads may be commonly described by the normal distribution with the unbiased mean and coefficient of variation 3-10% [15]. To simplify the following analysis, the permanent load is assumed here to be a single-source, unbiased with respect to a nominal value and with coefficient of variation of 5% (considering the possibility of measurements during the assessment).

2) Rail traffic load

The rail traffic load effect consists of static and dynamic components. The resulting load effect depends on a number of parameters including traffic intensity, bridge span length, vehicle weight, train loads, axle configuration, the position of a train on the bridge, stiffness of structural members etc.

According to [19] the ratio of the mean of rail traffic load effect over the characteristic value (based on LM71) is 0.82 and coefficient of variation of 10% might be considered. Moreira et al. [20] assumed a Gumbel distribution with a unity mean and coefficient of variation of 9% (relatively to LM71 effects). In the presented study, the mean value and coefficient of variation are taken equal 0.82 and 10% respectively. As an alternative for real heavy trains with measured axle loads, unbiased mean and coefficient of variation 5% are assumed.

Distribution of the maximum of independent identically distributed variables (denoted as q_{stat} in (3)) was used for the description of time-variant components of rail traffic load for different reference periods [21].

O'Connor et al. [22] considered for dynamic enhancement following parameters $\mu_\varphi / \varphi_{nom} = 0.33$ and $V_\varphi = 100\%$. James [23] assumed a coefficient of variation for dynamic enhancement in the range of 20-50%. According to UIC 776-1 [24], the nominal value of the dynamic enhancement, φ , 'covers about 95% of values studied'. This is understood here to represent a 95% fractile of φ . In this study, $V_\varphi = 50\%$ is accepted taking into account [22, 23]; then $\mu_\varphi / \varphi_{nom} = 0.62$ is obtained for the nominal value corresponding to a 95% fractile.

It is emphasised that an auxiliary variable can be introduced to describe uncertainties (simplifications) in the rail traffic load model. Here it is assumed that uncertainties

associated with the idealization of the rail traffic load model are rather small since, for instance:

- The position of load is determined by the position of the rails.
- The load models are standardized depending on the used types of trains.
- Weights of heavy trains are measured and their speed is controlled, etc.

3) Load effect model uncertainty

The uncertainty in the calculation of internal forces and displacements (load effects) is considerably dependent on the idealizations adopted in structural and load modelling and their correspondence with real structural conditions and behaviour. The important assumptions in reliability analysis may be related e.g. to joints, supports, initial imperfections, the behaviour of material and sections under different load levels and related effects (development of cracks, changes in stiffness), spatial aspects in structural behaviour and distribution of loads or to the interaction between structural members (load redistribution). In general, material and physical nonlinearity may significantly affect load redistribution and resulting load effects on steel structures.

In some cases, the accepted idealizations and models can substantially differ from real structural behaviour and it might be difficult to decide whether the simplification should be classified as uncertainty or error. For example, stiffness of joints or stiffness of secondary elements (such as bracing) in the real structure may largely deviate from that assumed in structural analysis. Such error then obviously affects structural behaviour and load redistribution. In such cases, parametric studies can be useful to identify the most important parameters and a subsequent detailed analysis e.g. of joints behaviour may improve results of structural analysis.

Uncertainty in load effects often depends on the level of loading; for example, bolted connections may work linearly at low load levels while slips and deformations at higher load levels increase modelling uncertainty.

Quantification of load effect uncertainty is complicated as generalised guidelines for creating and idealizing structural models are missing due to a wide range of design situations. Therefore, in general, it should be understood that the approach to structural and load modelling strongly affects load effect uncertainty. For existing structures, techniques to reduce uncertainty in load effects are available. As an example, the model can be validated by deflections measured during a proof load test.

In accordance with the generally accepted practice, load effect model uncertainty is described here by a unity mean and coefficient of variation of 10% [17].

The probabilistic models considered in the case study are presented in Tab.2.

Tab.2: Probabilistic models of basic variables considered in the case study

Basic variable	X	Dist.	μ_X / X_k	V_X
Resistance (including variability of yield strength and geometry)	R	LN0	1.09	5.8%
Resistance model uncertainty	K_R	LN0	1.1	5%
Permanent load	G	N	1.0	5%
LM71 (1-year maxima)	$Q_{stat,1}$	N	0.82	10%
LM71 (10-year maxima)	$Q_{stat,10}$	Gum	0.97	5.9%
Real train (1-year maxima)	$Q_{stat,1}$	N	1.0	5%
Real train (10-year maxima)	$Q_{stat,10}$	Gum	1.08	2.9%
Dynamic enhancement	φ	LN0	0.62	50%
Load effect model uncertainty	K_E	LN0	1.0	10%

μ_X - mean, V_X coefficient of variation, N - normal distribution, LN0 - lognormal distribution with the lower bound at the origin, Gum - Gumbel distribution (max. values), X_k characteristic value of basic variable.

5. An Example

In this section, reliability requirements following from the fixed partial factors (*FPF*) provided in EN 1990 and SZ S5 (Section 2), adjusted partial factors (*APF*) according to SZ S5 (Section 3), and probabilistic method (*PM*) (Section 4) are critically compared. An existing steel railway bridge is considered. The bridge has been in service for 90 years and its reliability need to be verified. Structural survey reveals no defects affecting structural reliability at Ultimate Limit States. Reliability assessment should verify whether or not the bridge can remain in service for the next 10 years. Target reliability index of 3.0 is considered according to Table F.1 in the SZ guideline [6]. To calculate the partial factors according to [6], it is assumed that the dimensions were measured.

Using the First Order Reliability Method FORM, partial factors are derived to provide for the target reliability index of 3.0. The values of the partial factors are presented in Tab.3 for the load ratio between 0.4 and 0.8 (see below).

Tab.3: Comparison of partial factors

	EN <i>FPF</i>	SZ <i>FPF</i>	SZ <i>APF</i>	<i>PM</i>
γ_G	1.35	1.2	1.16	1.2
ξ	0.85	0.95	0.94	-
γ_Q	1.45	1.3	1.21	1.17 ⁽¹⁾ or 1.27 ⁽²⁾
ψ_0	0.8 ⁽¹⁾ or 1.0 ⁽²⁾	0.5 ⁽¹⁾ or 0.75 ⁽²⁾	0.68	-
γ_{M0}	1.0	1.1	1.08	1.0

⁽¹⁾ LM71, ⁽²⁾ real train

Values of partial factors for loads decreased by 10-15% in comparison with EN. At the same time, SZ partial factors for resistance increased by 10% in comparison with to EN and FORM.

To cover a wide range of load combinations, load ratio χ is introduced. The load ratio χ denotes the ratio of

characteristic variable loads to the total characteristic load given as:

$$\chi = Q_k / (G_k + Q_k) \quad (5)$$

The load ratio may vary within the interval from nearly 0 (underground structures, foundations) up to nearly 1 (local effects on bridges, crane girders). For steel structures, $0.5 \leq \chi \leq 1$ is expected [25, 26]. For main girders of steel bridges, a range from 0.4 up to 0.8 is considered here to cover most of the practical cases.

Based on Equation (3), a generic limit state function based on the partial factor method may be written as follows:

$$g(\mathbf{x}) = W f_{yk} / \gamma_{M0} - [\gamma_G G_k + \gamma_Q (1 + \varphi_{nom}) Q_{stat,k}] \quad (6)$$

where W is a geometric parameter such as modulus or area of the section. Figure 1 displays variation of W_i / W_{EN} with χ for the case where a rail traffic load effect is based on the LM71 model. W_i denotes the geometrical characteristic of a cross-section (such as section module) required to satisfy the limit state in accordance with a particular approach to reliability verification the selected system of partial factors and W_{EN} is the reference value based on the partial factors recommended in Eurocodes for structural design.

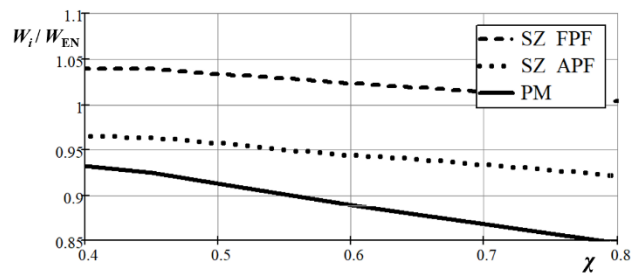
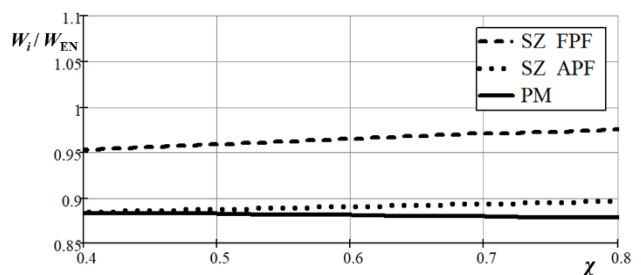
**Fig. 1:** Variation of W_i / W_{EN} with χ for LM71 model (age of bridge – 90 years, remaining service life – 10 years)**Fig. 2:** Variation of W_i / W_{EN} with χ for real train model (age of bridge – 90 years, remaining service life – 10 years)

Fig.1 shows that for the LM71 model, the fixed partial factors (*SZ FPF*) lead to slightly higher requirements than the reference *EN* level, up to 4%. In contrast, Fig. 2 shows that for the real train model, *SZ FPF* lead to slightly lower requirements than *EN*. It follows from Fig.1 and Fig.2 that adjusted partial factors (*SZ APF*) lead to the requirements by about 5–10% lower than *EN*. The *PM* leads to about 5–15% lower levels. Regarding *SZ APF* and *PM*, the decrease in requirements is attributed to the use of case-specific probabilistic distributions for basic variables that reduces

the conservativeness of fixed partial factors.

In comparison to the *PM*, the main deficiency of the *APF* is that the generalised sensitivity factors are applied. However, Fig. 2 shows that for the real train model, the assumption on sensitivity factors seems to be less important as both *SZ APF* and *PM* lead to similar results. Also, for the LM71 model, it is assumed that the sensitivity factors do not affect the results of *SZ APF* and *PM*. The difference observed in Fig. 1 is attributed to the difference in the probabilistic model used for the rail traffic load effect in *SZ APF* (see Section 3) and *PM* (Table 2).

For a lower age of the bridge and/ or longer remaining service life, the benefit of applying the advanced methods decreases, mainly due to the fact that the required reliability index increases according to [6]. As an example, when the age of the bridge is 20 years and the remaining service life is 50 years, then the target reliability index of 3.56 is recommended [6]. This leads to $W_i / W_{EN} \approx 1$; see Fig. 3.

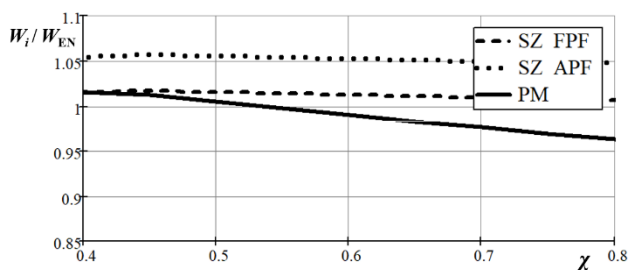


Fig. 3: Variation of W_i / W_{EN} with χ for LM71 model (age of bridge – 20 years, remaining service life – 50 years)

Fig. 4 displays the variation of the FORM sensitivity factors with the ratio χ . In this case, the α -factors are practically independent of the rail traffic load model.

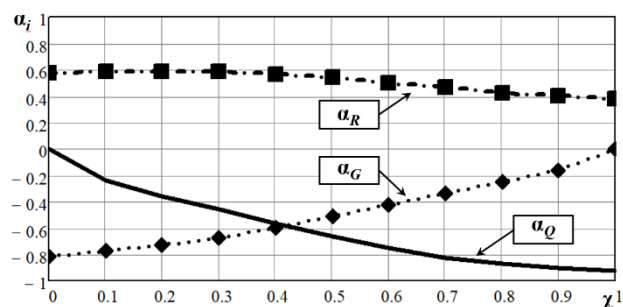


Fig. 4: Variation of sensitivity factors with χ

It appears that for steel bridges, the dominant influence on reliability can be attributed to variability of the rail traffic load effect. The sensitivity factor $\alpha_E = -0,8$ for the dominating load or $\alpha_E = -0,32$ for accompanying load, $\alpha_R = -0,6$ for resistance could be recommended. Detailed numerical studies indicate that in this case, the sensitivity factors depend insignificantly on a reference period as they are primarily affected by uncertainties in time-invariant variables including the dynamic amplification factor.

6. Discussion

6.1. Restrictions, Simplifications

The presented study provides a first insight into the performance of various approaches to reliability verifications of existing steel bridges. The main restrictions are summarised as follows:

- Linear limit state function is considered only.
- Permanent load should be divided into self-weight and other permanent loads; the latter being associated with larger uncertainty.
- Only a train load is taken into account as a variable load and the combination of variable loads is not considered in this study.
- The focus is rather on a global assessment of the main load-bearing structure while in local assessments, axle loads and dynamic enhancement with different statistical characteristics are dominating.

6.2. Future Research

Development and wider use of the adjusted partial factors seems to be reasonable considering the balance between demands on input information, computational complexity, and achieved improvements in reliability assessments. Besides certain limitations of this method, it remains to define the target reliability levels for new and specifically for existing railway bridges.

The probabilistic approaches providing a reference procedure to simplified approaches require further investigations as well. In particular, the review of available information shows incomplete empirical evidence to unambiguously justify the statistical parameters of rail traffic load effects. In a number of cases, the available data, particular those for the dynamic factors, differ significantly.

Modelling of degradation processes due to corrosion and fatigue seems to be another important challenge for further improvement of reliability studies.

Within further research, similar analyses focusing on reinforced concrete and prestressed bridges could be performed. It is expected that similar results will be obtained for reinforced concrete bridges where flexural resistance dominates. However, larger variability associated with resistance e.g. for shear is expected to change the sensitivity factors and different levels of reliability requirements may be obtained. Reliability of prestressed bridges is commonly controlled by Serviceability Limit States (crack avoidance, sections under compression etc.) and detailed investigations are needed.

7. Conclusions

- Application of probabilistic-based partial factors allows to reduce assessment requirements by 15%. It is attributed to the use of case-specific statistical parameters and probabilistic distributions of the basic variables and to definition of partial factors defined for a specified target reliability level. The use of more advanced probabilistic methods requires additional complex calculations and special experience.
- Application of adjusted partial factors allows to reduce assessment requirements by about 10%. This is possible due to the determination of partial coefficients for a particular target reliability level (possibly lower for existing bridges) and to consider structure-specific statistical parameters for basic variables. This method is easy to use. Nevertheless, the main conservatism of the method remains when the generalized values of the sensitivity factors are used.
- For all the methods, the input data include the probabilistic models of basic variables. However, further studies and standardisation of such models is important and demanding task. As uncertainty in the rail traffic load effects has the largest impact on the reliability of steel bridges, it is necessary to focus subsequent studies on a description of the models for static train loads, relevant dynamic factors and load effect model uncertainty.
- Within future research, target reliability levels for new and specifically for existing railway bridges should be specified.

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