Engineering Structures 106 (2016) 137–146

Contents lists available at ScienceDirect

Engineering Structures

journal homepage: www.elsevier.com/locate/engstruct

Partial factors for loads due to special vehicles on road bridges

Roman Lenner [a,1], Miroslav Sýkora [b,*]

[a] Stellenbosch University, Department of Civil Engineering, Stellenbosch 7600, South Africa
[b] Czech Technical University in Prague, Klokner Institute, Solinova 7, 166 08 Prague, Czech Republic

ARTICLE INFO

Article history:
Received 11 November 2014
Revised 12 October 2015
Accepted 12 October 2015
Available online 11 November 2015

Keywords:
Bridge assessment
Design loading
Existing bridges
Partial factors
Reliability

ABSTRACT

The structural standards for bridge design are well calibrated to account for the general traffic loading and expected use of the structure. Consideration of special heavy vehicles on bridges is often regarded according to provisions for the general traffic model in standards that may be, however, inconsistent and problematic due to a number of reasons. This contribution identifies the characteristics of special vehicles and proposes the methodology for calibration of the related partial factors. Key steps of the methodology consist of assessing static load effect, dynamic amplification, model uncertainty, sensitivity factors and target reliability. Careful consideration of these influences then yields partial factors that correspond to the definition of special loading and can be modified to distinguish between design and assessment situations. It appears that the commonly accepted partial factor of 1.35 may be reduced for an increased ratio of permanent- to traffic loads, decreased target reliability and for controlled speed of the vehicle and/or its position on a bridge. When multiple crossings of the same vehicle or vehicles of the same type during a reference period are considered, it is proposed to keep the partial factor independent of the number of crossings while the characteristic value should be adjusted with respect to the expected number of crossings.

1. Introduction

The purpose of design codes is to provide a consistent framework of rules and criteria that guarantee an acceptable structural reliability with respect to ultimate, serviceability and durability limit states. Most common operational procedures include the partial factor method introduced by Eurocodes and ISO standards, and the Load and Resistance Factor Design (LRFD) as accepted in ASCE standards (American Society of Civil Engineers). This study is particularly focused on the application of the reliability verifications of road bridges exposed to loads due to special (heavy) vehicles by using the partial factor method. However, the proposed methodology can be utilised in applications of most present methods for reliability verifications.

The calibration of partial factors or design values needs to account for a number of uncertainties and to cover a wide spectrum of applications. The design procedure shall account for relevant uncertainties in a correct, consistent and operational way and shall lead to an optimal reliability with respect to human safety and economic criteria.

The assessment of existing structures is fundamentally different from structural design since it usually involves a specific structure with clearly defined scope and boundaries [1–4]. The general conservatism regarding the structural resistance and loading intensity may be replaced by a detailed approach concerning the real structural performance and expected loading [5].

The current provisions in Eurocodes provide insufficient guidance for the assessment of existing structures. Assessment of an existing bridge using techniques provided for structural design may yield unsatisfactory performance even if the bridge is able to carry the loads with sufficient reliability.

For both new and existing road bridges, it should be considered that some uncertainties, inherently affecting reliability of the bridge, may be reduced for well-defined heavy vehicles. For such vehicles axle loads and their spacing may be known and this information should be then utilised in order to obtain more accurate estimates of load effects. The actions due to a single well-defined vehicle can be described with less uncertainty when compared to the load effect of a generalised traffic flow that aims to cover the real traffic and its possible trends, dynamic interaction of bridges with different types of vehicles and the influence of future political decisions with regard to new traffic concepts [6,7].

* Corresponding author at: Czech Technical University in Prague, Klokner Institute, Solinova 7, 16608 Prague, Czech Republic. Tel.: +420 2 2435 3850.

E-mail addresses: rlenner@sun.ac.za (R. Lenner), miroslav.sykora@cvut.cz (M. Sýkora).

[1] Postal address: Stellenbosch University, Dept of Civil Engineering, Private Bag X1, Matieland, 7601, South Africa. Tel.: +27 21 808 9572; fax: +27 21 808 4947.

http://dx.doi.org/10.1016/j.engstruct.2015.10.024
0141-0296/

As the loading due to special vehicles often needs to be treated on a case-specific basis, the related considerations can hardly be entirely covered by the design codes. In fact, the provisions in the bridge codes regarding special vehicles seem to be limited [8] and thus the loads due to special vehicles are commonly treated in engineering practice similarly as normal traffic.

The aim of this study is to investigate traffic loads imposed by special, well-defined vehicles for the reliability assessment of existing road bridges, focusing on the Ultimate Limit State (according to EN 1990 [9] for the basis of structural design). The particular goal is then to propose appropriate values of partial factors for such loading. In accordance with EN 1991-2 [10] for traffic loads on bridges, the characteristic load associated with a single crossing of a special vehicle is taken as a nominal value. The distinction between single crossing and repeated crossings of the same vehicle is made. In the latter case it is considered that a permit or an authorisation is issued for a period of one year.

In this contribution it is assumed that weights and axle loads of special vehicles are guaranteed by measurements, or can be precisely determined on the basis of calculations and experience with similar transports. That is why the proposed approach takes no account for possible overloading as particularly intentional overloading needs to be classified as a malevolent action or human error that is commonly deemed beyond the scope of design standards. In most cases human errors are eliminated by quality control rather than reliability elements such as partial factors that normally disregard actual failure frequencies significantly affected by human errors [9]. The quality control is in this case represented by weight measurements.

2. Loads due to special vehicles

Traffic load models for special vehicles are included in EN 1991-2 [10] in order to introduce a possibility to design and size the bridge for potential exceptionally heavy loading. This may be necessary for industrial areas or selected routes where extremely heavy vehicles may travel frequently. The basic design models of special vehicles are listed in an increasing manner according to axle loads and number of axles. Specific design models should cover abnormal loading considering country-specific conditions. However, the models describe a specified loading intensity rather than real vehicles [8]. The load values may be given individually for each newly designed bridge according to the local conditions. This warrants a sufficient structural resistance when the permission to cross a bridge for a special vehicle is needed. Loads due to heavy civil vehicles can be described by the LM 3 models in Annex A to EN 1991-2 [10] whilst military vehicles are defined according to the NATO standardisation agreement STANAG 2021 [11].

It is important to note that the design process may ensure a required structural resistance for normal traffic, but it does not guarantee any authorised crossing by a specific special vehicle. The load effect of such a vehicle must be assessed individually and compared to the resistance levels. The authorised special loading may then take form of a vehicle transporting heavy freight, or a military vehicle in an emergency or crisis situation when response to a threat is necessary.

Generally, three Traffic Situations are distinguished in this study:

1. Special traffic load along with normal traffic.
2. Special traffic load only with no other traffic allowed on the bridge.
3. Special traffic load under strictly defined conditions in terms of the vehicle's position and speed with no other traffic allowed.

Should a specific vehicle be authorised to cross a bridge, properties of the vehicle need to be known in advance for estimation of the load effects. That is why it is hereafter accepted that the axle loads and axle spacing of the vehicle under consideration are known and the load effect can be calculated with enhanced accuracy.

Traditional semi-probabilistic safety concept is based on the use of partial factors for both resistance and loading. EN 1990 [9] lists the partial factors for actions commonly considered in verifications of limit states. The provisions for military vehicles including the Military Load Classification system were recently investigated and the partial factors calibrated in [12]. It may be understood from EN 1991-2 [10] that special vehicles under the LM 3 provisions are treated with partial factor γ_Q for traffic loading that is generally applied to normal traffic conditions described by the design model LM 1.

A comparison of the LM 1 and LM 3 models reveals the following:

- LM 1 attempts to represent the characteristic load of real traffic with its predicted trends, while the special loading relates to a single, well-defined vehicle.
- The characteristic value of the LM 1 model in EN 1991-2 [10] corresponds to a 1000-year return period. For special vehicles the mean value is commonly accepted as a characteristic value. Consequently, a considerable reliability margin is thus included in the characteristic value for normal traffic compared to special vehicles.
- Dynamic effects are included in models for normal traffic in current bridge codes; they need to be assessed separately in the case of special vehicles.

For these reasons using the same partial factors for the LM 1 model and for well-defined special vehicles is inconsistent and calibration is needed in order to specify more appropriate partial factors for the load effects due to special vehicles.

3. Load combinations and partial factors

3.1. Load combinations

Partial factors γ_Q derived in this work are intended to be applied in conjunction with the load combination rules given in ISO 22111 [13] that are consistent with ASCE, AS/NZS (Australian/New Zealand Standard) standards and Eurocodes. As an example the load combination rule (6.10a,b) of EN 1990 [9] is shown here; with no prestressing applied, the reliability verification format can be written as (a less favourable expression is decisive):

$$
\begin{aligned}
R_d &\geqslant E_d = \sum_j \gamma_{G,j} G_{k,j} \text{ “}+\text{” } \sum_i \gamma_{Q,i} \psi_{0,i} Q_{k,i}, \quad j \geqslant 1,\ i \geqslant 1 \\
R_d &\geqslant E_d = \sum_j \xi_j \gamma_{G,j} G_{k,j} \text{ “}+\text{” } \gamma_{Q,1} Q_{k,1} \text{ “}+\text{” } \sum_i \gamma_{Q,i} \psi_{0,i} Q_{k,i}, \quad j \geqslant 1,\ i > 1
\end{aligned}
\tag{1}
$$

where R denotes resistance; E load effect; γ partial factor; G permanent action effect; Q variable action effect; ξ reduction factor for unfavourable permanent actions; and ψ_0 factor for combination value of a variable action. The subscripts “d” and “k” denote design and characteristic values, respectively. The symbol “+” implies “to be combined with” and $\sum$ “the combined effect of”. Note that favourable variable actions are not considered in structural verifications based on the partial factor method.

In principle the partial factors γ_X and the characteristic values X_k shall be based on real material properties and actions; see for instance EN 1990 [9] and ISO 2394 [14]. Values of the factors ξ

and ψ_0 are to be accepted from a relevant standard such as EN 1990 [9].

In this study the load effect due to a special vehicle Q_{spec} is assumed to be a leading variable action and thus other variable actions (normal traffic, wind, thermal actions, etc.) are always considered by their combination values $\psi_{0,i}\,Q_{k,i}$. The effect of a special vehicle should consistently dominate over the effect of normal traffic for the bridges of short to medium spans. For long span bridges the special vehicle will likely affect reliability of local structural members rather than reliability of main girders.

The partial factor γ_G should be assessed considering available information regarding permanent actions (taking into account possible geometrical measurements and material tests); in particular the distinction between structural design and assessment of an existing bridge should be made. Detailed guidance is to be provided in a foreseen *fib* bulletin [15], the procedure was elaborated in more details in [1,2]. For simplification the partial factor γ_G is considered here as follows:

- A reduced value 1.15 ($\xi\,\gamma_G$) may be accepted for the assessment of an existing bridge when measurements of geometry and volume densities are available and uncertainties in the effect of permanent actions are significantly reduced.
- In other cases, including the design phase or assessment with insufficient data on permanent actions, a value of γ_G recommended in a relevant standard should be accepted.

Eq. (1) can then be simplified to:

$$R_d \geqslant E_d = \sum_j (\xi_j)\gamma_{G,j}G_{k,j} \text{ “+” } \gamma_{Q\text{spec}}Q_{k,\text{spec}} \text{ “+” } \sum_i \gamma_{Q,i}\psi_{0,i}Q_{k,i}, \quad j \geqslant 1,\, i > 1 \tag{2}$$

where (ξ) indicates that the reduction factor may be applied. With respect to the three Traffic Situations indicated in Section 2, the following remarks are made:

1. Special vehicle along with normal traffic – when using Eurocodes, the load model LM 1 associated with the partial factor γ_Q is typically combined with the special vehicle and its partial factor γ_{Qspec}; other actions covered by the term $\sum_i \gamma_{Q,i}\psi_{0,i}Q_{k,i}$ may include wind or thermal actions, etc.
2. Special traffic load only with no other traffic allowed – the partial factor γ_{Qspec} is consistent with Traffic Situation 1 since the uncertainties related to the load effect due to a special vehicle are essentially unchanged.
3. Special traffic load under strictly defined conditions in terms of position and speed of the vehicle; no other traffic allowed – in this situation the partial factor γ_{Qspec} may be different from those accepted in Traffic Situations 1 and 2 due to the controlled crossing where uncertainties in the load effect may be significantly reduced.

Transient design situations with respect to temporary conditions of the bridge due to crossing(s) of a special vehicle are considered in this contribution. Accidental design situations like crisis or emergency situations (due to e.g. natural catastrophes or the state of war) are not treated here; for more details see [12].

3.2. Load effect of the special vehicle

It is proposed to address the design load effect of a special well-defined vehicle according to its characteristics that differ from traditional traffic loading. It is hereafter assumed that the load effect due to the passage of a special vehicle Q_{spec} can be obtained as follows:

$$Q_{\text{spec}} = \theta\,\delta\,Q_{\text{stat}} \tag{3}$$

where θ denotes the model uncertainty in estimation of the load effect from the load model, δ is the dynamic amplification factor and Q_{stat} is the static load effect including uncertainties in measurements of weights and spacing. In principle all the variables in Eq. (3) are to be treated as random variables.

Therefore, the characteristic value $Q_{k,spec}$ is the product of characteristic values of the three basic variables that can be realistically taken equal to their mean values, $Q_{k,spec} = \mu_{Qspec} = \mu_\theta\,\mu_\delta\,\mu_{Qstat}$. The design load effect is then expressed using an appropriate partial factor:

$$Q_{\text{d,spec}} = \gamma_{Q\text{spec}}\,Q_{\text{k,spec}} \tag{4}$$

Assuming lognormally distributed θ and δ, and a normal distribution of Q_{stat} (see Section 4), a lognormal distribution can be considered for the load effect Q_{spec} since greater variability is associated with both θ and δ rather than with a well-described Q_{stat}. Based on these assumptions the partial factor γ_{Qspec} is obtained as:

$$\begin{aligned}\gamma_{Q\text{spec}} &= Q_{\text{d,spec}}/Q_{\text{k,spec}} = \lfloor \mu_{Q\text{spec}} \exp(-\alpha_E\,\beta\,V_{Q\text{spec}})\rfloor/\mu_{Q\text{spec}} \\ &= \exp(-\alpha_E\,\beta\,V_{Q\text{spec}})\end{aligned} \tag{5}$$

where α_E denotes the FORM sensitivity factor, β the target reliability index discussed in Section 7 and V_{Qspec} the coefficient of variation of Q_{spec}, estimated as follows:

$$V_{Q\text{spec}} \cong \sqrt{V_\theta^2 + V_\delta^2 + V_{Q\text{stat}}^2} \tag{6}$$

where V_θ, V_δ and V_{Qstat} are the coefficients of variation of model uncertainty, dynamic amplification and of static load effect, respectively.

4. Stochastic models for load effects due to special vehicles

Stochastic models for load effects due to special vehicles are developed considering two possible crossing conditions:

- Special traffic load unrestricted – special loading with no restriction in terms of velocity or position on the bridge – Traffic Situations 1 and 2.
- Special traffic controlled – special loading with reduced velocity (<5 km/h) and restricted transverse position on the bridge – Traffic Situation 3.

4.1. Static load effect

The stochastic model for well-defined special loading can be assessed in terms of uncertainty related to axle loads and spacing of axles. Assuming no bias in estimating Q_{stat}, the coefficient of variation of the static load effect is of particular interest. Conclusions regarding variability of the static load are drawn from [16] where numerical simulations were employed in order to quantify the expected static load effect due to passage of military vehicles over bridges. It established the effect of an uncertainty tied to the axle loads and geometrical properties of a military vehicle. Only a single vehicle on a span was simulated. The static system was represented by the relevant influence lines for simply supported, fixed and continuous beams, thus encompassing most of common structural systems. The definition of military vehicles was similar to that of special vehicles accepted here.

Using Monte Carlo simulations, random properties of vehicles were generated according to specified mean values and coefficients of variation for both axle loads and axle spacings [16]. The mean values were selected according to [11]. Coefficients of variation were selected more broadly, so as to cover a wide range of situations relevant also for the variability of heavy civilian vehicles. The simulations also reflected possible uneven, random distribution of vehicle weight to axle loads and the differences between axle loads of a vehicle in motion and their static values. However, it is emphasised that significant unexpected distribution of axle loads should be mitigated during transport by measures such as restraining movements of the load on a trailer, or appropriate increase of characteristic values of axle loads in reliability verification.

As a conclusion, when a vehicle defined by its axle loads and axle geometry is considered, then the variation of its static load effect V_{Qstat} is largely dependent on the particular coefficient of variation of axle load while the coefficient of variation of axle spacing is of minor importance. The span length L is an additional decisive factor in estimating the coefficient of variation of static load V_{Qstat}. The extensive analysis revealed the range of V_{Qstat} from 0.03 to 0.07. It was additionally shown that a value of V_{Qstat} in the given range has only a marginal influence on the reliability as indicated by the probabilistic reliability and sensitivity analysis. Therefore, it is sufficient to accept $V_{Qstat} = 0.05$ for static load effect due to the passage of special vehicles.

Note that normal distribution of the loading is deemed appropriate since the aforementioned properties, such as axle load and axle spacing, are normally distributed variables. This is supported by the evaluation of traffic loading and measurements of axle loads where all the applied vehicular loading followed a normal distribution [17].

4.2. Dynamic amplification

In general, the dynamic effect of traffic load is influenced by a number of factors, such as maximal bridge span length, bridge natural frequency, vehicle weight, axle loads, axle configuration, vehicle suspension properties, position of a vehicle on the bridge, quality of pavement, and stiffness of structural members. Considerable differences exist between various approaches and no consensus has been reached among the scientific community. However, a large contribution may be attributed to vibrations of the vehicle induced by the road profile roughness, depending on the velocity and surface unevenness between the approach and the bridge deck [18,19].

An increase of static loading leads to a decrease of mean value of the dynamic amplification μ_δ (in some cases approaching unity) and also results in a reduction of its coefficient of variation [20,21]. Therefore, the dynamic component attains a minimum at the maximum loading level. A similar pattern can be observed for articulated vehicles and increasing number of axles interfering with the bridge response. Recent testing and calibration of variable loading was accomplished within the project Assessment and Rehabilitation of Central European Highway Structures ARCHES. For heavy loads and smooth roadways the amplification factors typically remained below 1.1.

The dynamic response is influenced by the unevenness at the bridge approach or a damaged roadway surface. A rough road profile or a small bump can in some cases produce a significant increase of dynamic effects. Figs. 1 and 2 show the results adapted from ARCHES [22] where statistical properties of dynamic amplification were numerically investigated. In this example, the *Truck* represents the normal traffic conditions while the *Crane* is selected for exceptionally heavy loading. It is observed that the mean values of the dynamic amplification factor due to the heavy loading exhibit insignificant variability for different situations A–C (A – smooth surface, B – 2 cm bump and C – 4 cm bump). Coefficient of variation is less than 0.10 at shorter span lengths and below 0.05 at span

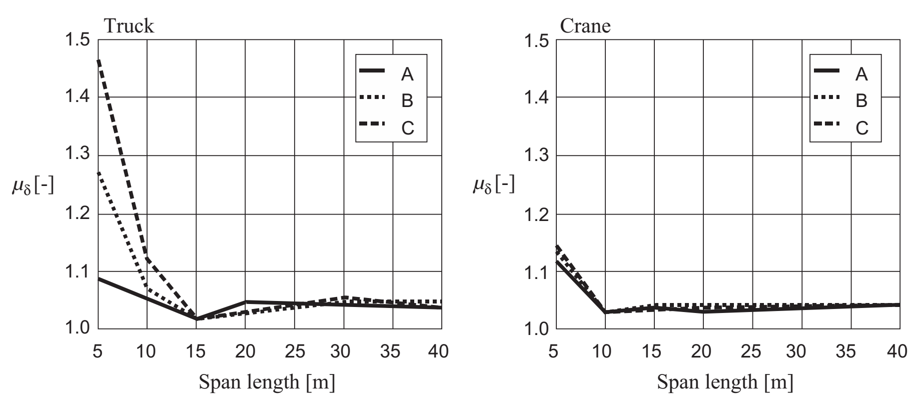

Fig. 1. Mean value of dynamic amplification for bending in midspan: A – smooth surface, B – 2 cm bump and C – 4 cm bump [22].

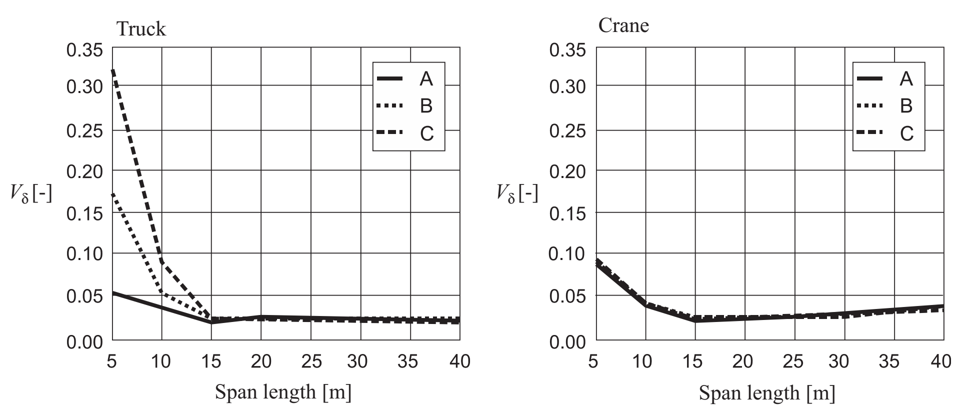

Fig. 2. Coefficient of variation of dynamic amplification for bending in midspan: A – smooth surface, B – 2 cm bump and C – 4 cm bump [22].

lengths over 7 m. For shear response, dynamic amplification exhibits similar patterns.

Besides road profile roughness, the dynamic amplification stochastic properties are to a certain extent dependent on the considered span length. The characteristic (mean) value of dynamic amplification δ is clearly tied to the bridge length or natural frequency. The largest amplification of the static load and its variation V_δ is generally observed at short span lengths. However, it should be noted that the mean value of dynamic amplification can be actually reduced for long vehicles crossing short spans. In this case, only few axles contribute to the loading and the resulting effect is positively influenced by rigidity of the vehicle or trailer. Considering a typical pavement settlement in front of a short bridge, some axles of long rigid vehicles may actually jump over the bridge and thus reduce the transfer of forces between axles and the deck; or the interference between the axles of articulated vehicles results in considerable damping effects. The mean value in such a case must be assessed with caution. Further studies related to these effects are needed.

It is important to realise that the coefficient of variation V_δ affects the value of the partial factor, while the expected mean value influences the total load effect (Eqs. (5) and (6)). EN 1991-2 [10] offers limited guidance in this respect. It is suggested to accept the values proposed by ARCHES for the *Crane* as relevant for special, well-defined heavy vehicles. From Figs. 1 and 2 the following values of the mean (μ_δ) and coefficient of variation (V_δ) of the dynamic amplification factor may be recommended for crossings by heavy vehicles:

$$\begin{aligned} &\text{For } 5\text{ m} \leqslant L \leqslant 10\text{ m}: \quad \mu_\delta = 1.15 - 0.02(L - 5\text{ m}); \\ &\quad V_\delta = 0.1 - 0.01(L - 5\text{ m}) \\ &\text{For } L > 10\text{ m}: \quad \mu_\delta = 1.05; \quad V_\delta = 0.05 \end{aligned} \tag{7}$$

These values take into account variation due to profile roughness or a bump between the approach and the deck. However, site-specific conditions should be carefully evaluated. Any seriously adverse conditions for dynamic amplification, such as rough profile or exceptionally short span lengths, can be then mitigated by demanding a controlled crossing. For exceptionally smooth profiles μ_δ = 1.05 and V_δ = 0.05 can be regarded for any span length.

According to Eq. (7) the mean value of the dynamic amplification ranges from 1.05 to 1.15. It is assumed that an appropriate unbiased value is considered when determining the characteristic load effect and the mean value is thus not reflected in estimating the partial factor.

Note that for the controlled crossing conditions, vehicular speeds of 5 km/h are sufficiently low to consider a quasi-static loading [21]. In accordance with Annex A to EN 1991-2 [10] the dynamic amplification factor need not be applied for the controlled crossing.

4.3. Model uncertainty

According to the Probabilistic Model Code of the Joint Committee on Structural Safety [23], the model uncertainty is generally a random time-invariant variable accounting for effects neglected in the models and simplifications in the mathematical relations. Model uncertainty in the load effect θ should cover numerous aspects including idealisation of supports, composite actions of structural members, computational options (e.g. in FE analysis), description of input data and other effects not covered by a load effect model such as deviations from expected load distributions. The Model Code [23] provides limited guidance regarding the selection of mean values and coefficients of variation.

A unit mean and V_θ = 0.07 for permanent action and V_θ = 0.10 for traffic load are recommended for the development of partial factors [24]. Danish Reliability-Based Classification [25] lists the classes of uncertainties for variable loading regarding the level of confidence in modelling. The classes depend on a structural system, geometric properties and crossing mode where conditional (controlled) passage is usually associated with a higher level of confidence and thus low uncertainty in the loading model.

It can be concluded that the model uncertainty is largely influenced by the static system and the level of confidence in applied loading. The mean is generally considered as 1.0. The controlled crossing conditions may be associated with a reduced coefficient of variation V_θ = 0.07, while general crossing with uncontrolled position of the vehicle on the bridge deck may be associated with V_θ = 0.10.

5. Sensitivity factors and load ratio

Sensitivity factor α indicates the influence of a particular variable in the limit state on the resulting reliability. These factors depend on the stochastic properties of both resistance and loading variables and can be in principle estimated for general use. EN 1990 [9] (Annex C) and ISO 2394 [14] allow for the following approximations for actions:

1. $\alpha_E \approx -0.70$ for the leading action,
2. $\alpha_E \approx -0.28$ for accompanying actions.

However, appropriate values of the sensitivity factors should be estimated on a case-specific basis since they have a considerable influence on the partial factor $\gamma_{Q\text{spec}}$ and subsequently on the decision about crossing. They can be calculated using the FORM analysis [26,27].

It was shown during the investigation of military vehicles representing well-defined loads [12] that the ratio κ, describing the relationship between permanent and variable loading [28], is a key parameter affecting sensitivity factors:

$$\kappa = M_G/(M_G + M_Q) \tag{8}$$

where M_G denotes the characteristic permanent load effect and M_Q the characteristic traffic load effect.

The bending limit state of reinforced concrete beam is investigated in detail and analysed using FORM. Flexural resistance is described according to EN 1992-1-1 [29] and EN 1992-2 [30]. Probabilistic models for all relevant variables given in Table 1 are adopted from [1,31]. For controlled and general traffic conditions the respective statistical parameters are listed in Table 2. $V_{Q\text{spec}}$ is calculated according to Eq. (6). The higher coefficient of variation for dynamic effects is omitted here, considering smooth profiles and/or span lengths over 10 m. Fig. 3 shows the selected resulting sensitivity factors $\alpha_{E,G}$ and $\alpha_{E,Q}$ for permanent and variable actions, respectively, in relation to the load ratio κ.

It appears that the influence of each random variable considerably depends on the load ratio. Clearly, an absolute value of the sensitivity factor for variable action $\alpha_{E,Q}$ attains its maximum at a low load ratio indicating a dominant variable action whilst absolute $\alpha_{E,G}$ is minimal.

Table 1
Stochastic models of basic variables for bending limit state [1,31].

Variable	X	Distr.	μ_X/X_k	V_x
Yield strength reinforcement	f_y	LN	490/560	0.054
Reinforcement area	A_s	N	1	0.02
Concrete comp. strength	f_c	LN	30/40	0.15
Uncertainty in resist. model	θ_R	LN	1.1	0.1
Geometry	b, d	N	1	0.02

Table 2
Coefficients of variation for different traffic situations considered in the sensitivity analysis.

Traffic situation	V_{Qstat}	V_δ	V_θ	V_{Qspec}
Special traffic load – controlled (traffic situations 2 and 3)	0.05	–	0.07	0.09
Special traffic load – unrestricted (traffic situation 1)	0.05	0.05	0.10	0.12

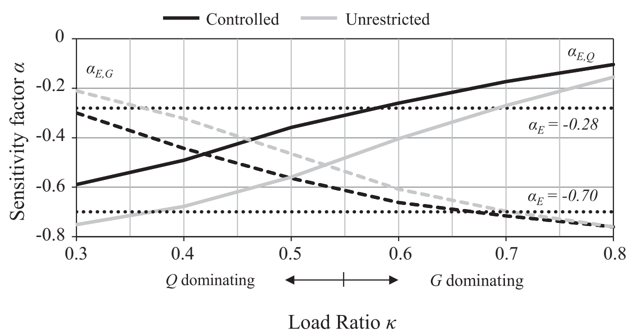

Fig. 3. Variation of the sensitivity factors $\alpha_{E,Q}$ and $\alpha_{E,G}$ with the load ratio κ.

Generally, the results in Fig. 3 indicate that decreasing the variability of traffic loading decreases its associated sensitivity factor and increases the sensitivity factor for permanent loading. The numerical investigation confirms that the leading action is likely to have a larger influence on the reliability although better described variables are likely to yield lower values of α. Furthermore, flexural resistance is associated with relatively small uncertainties and thus the importance of load effects as expressed by the α-factors increases. The exact value of α is principally important as overly conservative approximations result in unnecessarily high partial factors. However, the α-factor needs to be selected with caution to warrant adequate performance under various loading scenarios.

Depending on the level of involvement and required level of reliability verification, the following approaches to estimation of α_E-factors can be accepted:

1. The approximate values given in standards (i.e. $\alpha_E = -0.70$ for the leading action and $\alpha_E = -0.28$ for accompanying actions) can be adopted, providing conservative values for most practical cases.
2. The values can be obtained from Fig. 3 for a given ratio κ, provided that the assumptions made in this section apply for an investigated bridge.
3. The FORM analysis can be used to estimate α_E-values.

6. Multiple crossings

The previous analysis provides the methodology for establishing parameters of the load effect due a single crossing. In this section multiple crossings of the same vehicle or vehicles of the same type during the period covered by permit are investigated. For the authorisation a typical period of one year, during which n crossings take place, is considered.

Model uncertainties θ and all resistance variables are a priori assumed as time-invariant components of the limit state function, and therefore are deemed independent of the number of crossings. On the other hand, the dynamic load effect obtained as the product of dynamic amplification and static load effect, $Q_{dyn} = \delta\, Q_{stat}$, is deemed to vary for each crossing due to inherent randomness of influencing factors such as velocity of the vehicle, its position on the bridge, bridge vibrations, and the effect of surface unevenness. In the absence of statistical data it is further assumed that Q_{dyn} can be described by identically distributed, independent random variables $Q_{dyn,i}$ ($i = 1, 2, \ldots, n$). This is a conservative assumption as static load effect and dynamic amplification are additionally affected by factors likely independent of the number of crossings, including bridge natural frequency, axle loads and configuration. Consequently, the values $Q_{dyn,i}$ are positively correlated.

The cumulative distribution function of the maximum dynamic load effect due to n crossings, $Q_{dyn,n}$, is obtained as follows:

$$F_{Q\mathrm{dyn},n}(x) = F_{Q\mathrm{dyn}}(x)^n \tag{9}$$

A fully probabilistic model is developed for the maximum load effect $Q_{spec,n} = \theta\, Q_{dyn,n}$. Fig. 4 illustrates the variation of its mean $\mu_{Qspec,n}$ and coefficient of variation $V_{Qspec,n}$ with n for $V_\theta = 0.1$ and $V_\delta = V_{Qstat} = 0.05$. The mean value increases with an increasing number of crossings, while the coefficient of variation is nearly independent of n. The former represents the nominal (characteristic) value while the latter affects the partial factor as follows from Eq. (5).

That is why it is proposed to keep the partial factor γ_{Qspec} independent of the number of crossings while the characteristic value should be adjusted with respect to $n \geqslant 2$ using the following ratio:

$$\frac{Q_{k,spec,n}}{Q_{k,spec}} = 1 + V_{Q\mathrm{dyn}}\left[\frac{4\ln\, n - \ln\,(\ln\, n) - 1.377}{2\sqrt{2}\ln\, n}\right] \tag{10}$$

Eq. (10) provides the mean value of the maximum of n identically distributed, independent normal variables with unity mean and coefficient of variation $V_{Q\mathrm{dyn}} \approx \sqrt{(V_\delta^2 + V_{Qstat}^2)}$ [32]. The approximation gives estimates close to those based on the fully probabilistic model as shown in Fig. 4. Additional comparisons with the fully probabilistic approach reveal that, for any n, the proposed approach is sufficiently accurate for the controlled crossing associated with $V_\theta = 0.07$ as well as for higher uncertainty in dynamic amplification associated with $V_\delta = 0.1$. The differences in design value, $\gamma_{Qspec} \times Q_{spec,k}$, are less than 5%. Consequently the proposed approach is recommended for practical applications.

7. Target reliability

The target reliability level, mostly given in terms of the reliability index β, is one of the inputs required for the modification of partial factors; see Eq. (5). Target levels used in the reliability analysis are mainly associated with the failure of components and not of the whole or main part of a new or an existing bridge. In this study

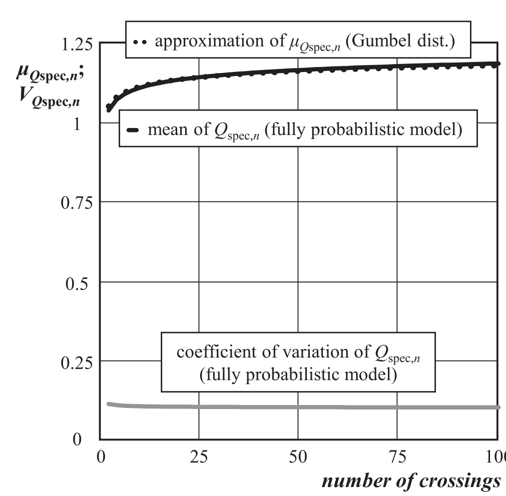

Fig. 4. Variation of $\mu_{Qspec,n}$ and $V_{Qspec,n}$ with n (for $V_\theta = 0.1$ and $V_\delta = V_{Qstat} = 0.05$).

it is assumed that the reliability index is associated with a structural member dominating a system failure mode of the bridge ('key structural member'). For other structural members lower reliability levels can reflect lower failure consequences.

7.1. Available recommendations on target reliabilities

7.1.1. Standards

Target reliabilities for structural design and assessment of existing structures are provided in many international and national standards. Recent contributions [33–36] provided an overview of the recommendations given in EN 1990 [9], ISO 2394 [14], ISO 13822 [37] and the Dutch standard NEN 8700 [38]; the last two standards are focused on the assessment of existing structures. Several difficulties were identified:

- The target reliabilities are related to different reference periods – usually one year, 50 years or working life; recalculation for different periods may be complicated by dependencies amongst failure events.
- A broad range of β-values is recommended.
- Differences between the assessment of existing structures and structural design (considering higher costs of safety measures for existing structures as a key one) are often inadequately reflected.

Taking into account also several national standards, the principal factors affecting the target reliabilities include:

1. Failure consequences, in some cases with explicit considerations of a type of failure (ductile or brittle, system behaviour considering redundancy and proneness to progressive collapse).
2. Cost of safety measures.
3. Reference period.

7.1.2. Previous studies

Specification of the target reliabilities for structural design and assessment of existing structures has recently been addressed by several studies providing the following indications:

- In accordance with ISO 2394 [14] the basis for specification of the target reliabilities is provided by two different concepts – cost optimisation and minimum levels required for acceptable human safety [24,39]; these principles apply for both new and existing structures.
- It is uneconomical to require the same target reliabilities for existing structures as for new structures [33,40–42]. This requirement is consistent with regulations accepted in nuclear and offshore industry, for buildings in seismic regions, bridges in USA and Canada, etc.
- Minimum levels for human safety are commonly decisive for existing structures while economic optimisation dominates the criteria for design of new structures [33].
- Two target levels are needed for the assessment of existing structures – the minimum level below which a structure should be repaired and the optimum level for repair.

7.2. Appropriate target levels for crossings by special vehicles

7.2.1. Economic optimisation

A risk-informed decision making (economic optimisation) should be conducted in order to justify the authorisation of individual or multiple crossings. In addition, human safety levels should also be adhered to whenever normal traffic is allowed to cross a bridge along with the special vehicle as argued in Section 7.2.3.

ISO 2394 [14] indicates that the target reliability level should depend on the balance between the consequences of failure and the cost of safety measures. The objective is to minimize the total working-life cost [43]. Cost associated with the bridge, and benefits for the whole society should be included in the optimisation.

Single or multiple crossings resulting in the load effect Q_{spec} are authorised when the benefit B associated with approved transports exceeds a risk-related cost due to the crossing:

$$B \times \lfloor 1 - P_f(Q_{spec}) \rfloor \cong B \geqslant C_f \times P_f(Q_{spec}) \tag{11}$$

where C_f denotes the failure costs and $P_f(Q_{spec})$ is the small probability of failure due to the crossing(s). B and C_f need to be expressed in the same units for a considered number of crossings. The failure probability is considered to be insignificantly affected by the number of crossings as the time-invariant variables such as resistance and geometry parameters, permanent actions, model uncertainty for load effects and some factors affecting dynamic load effect dominate bridge reliability. In view of considerable uncertainties in specifying failure costs it is acceptable to consider $P_f(Q_{spec})$ independent of the number of crossings.

Realistically assuming that the benefit is less than the failure costs, $B < C_f$, the target failure probability based on the economic optimisation, $P_{t,eco}$, is then obtained from Eq. (11):

$$P_f(Q_{spec}) \leqslant P_{t,eco} \cong B/C_f \tag{12}$$

The reliability index corresponding to the target probability is (for $B < C_f$):

$$\beta_{eco} = -\Phi^{-1}(P_{t,eco}) \approx -\Phi^{-1}(B/C_f) \tag{13}$$

where Φ^{-1} is the inverse cumulative distribution function of the standardised normal distribution. The reliability index β_{eco} is deemed independent of n and thus can be related to a short period of an individual crossing or to a longer period covered by the permit for multiple crossings such as one year.

Fig. 5 indicates the variation of the target reliability index β_{eco} with the ratio B/C_f. The target level is approximately linearly proportional to the order of magnitude of the ratio. Note that the reliability index for human safety β_{hs} is discussed in Sections 7.2.2 and 7.2.3.

While the benefit B needs to be estimated on a case-specific basis, guidance can be provided for estimation of failure consequences. They may include:

- potential societal consequences (costs of injuries and fatalities) directly caused by the failure (collapse),
- cost of upgrade or replacement,
- economic losses and potential societal consequences caused by bridge closure due to repair works taken after the failure (possibly including also losses due to damage on detour routes),
- possible other consequences such as unfavourable environmental or psychological effects.

It is emphasised that the societal consequences should be considered in terms of compensation costs or increased insurance.

Estimation of the failure cost is a very important, but likely the most difficult step in the cost optimisation. It is important to include not only direct consequences of failure (those resulting from the failures of individual components), but also follow-up consequences (related to a loss of the functionality of a whole bridge). Background information for the consequence analysis is provided by ISO 2394 [14], papers [44,45] and by outcomes of the SeRoN project (seron-project.eu) focused on risks of road transport networks. Limited statistical data concerning the expected number of fatalities given a bridge failure were provided in [35,46].

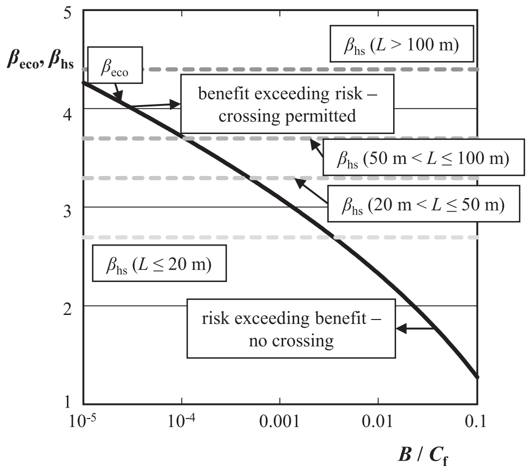

Fig. 5. Variation of the target reliability indices β_{eco} and annual β_{hs} with the ratio B/C_f.

It is noted that the proposed approach illustrates the risk-based decision making in a simplified manner, yet it should cover main aspects affecting the decision regarding the authorisation of the crossing. More detailed considerations can be accepted following [47–50].

7.2.2. Minimum levels for human safety – crossing without normal traffic

Considering crossings of an existing bridge by a special vehicle without normal traffic (Traffic Situations 2 and 3 listed in Section 2), only the safety of persons involved in the transport (a driver and possibly crew) is endangered. In this case there is no public exposure and human safety levels required by the society do not apply.

Higher risk exposure should be compensated to the persons involved in the transport. This is a common practice in various industries such as construction, mines, energy and the shipping industry [51,52]. Significant risk compensations are then provided to members of rescue or army corps. The acceptance of related risks and decisions on appropriate compensations are tasks of a company responsible for the transport and is beyond the scope of this study.

7.2.3. Minimum levels for human safety – crossing along with normal traffic

Considering individual or multiple crossings of an existing bridge by a special vehicle along with normal traffic (Traffic Situation 1), safety of users is endangered by the crossing and minimum levels accepted for human safety need to be considered. A detailed analysis can be based on the Life Quality Index approach provided in ISO 2394 [14] and requires a separate study.

When the special vehicle is crossing a bridge along with normal traffic, risks of persons not involved in the transport should not be increased as compared to normal traffic situations. For existing bridges Steenbergen et al. [35] proposed the following annual target levels β_{hs} based on societal risk criteria:

- $\beta_{hs} = 2.7$ for span length $L \leqslant 20$ m,
- $\beta_{hs} = 3.3$ for 20 m < $L \leqslant 50$ m,
- $\beta_{hs} = 3.7$ for 50 m < $L \leqslant 100$ m,
- $\beta_{hs} = 4.4$ for $L > 100$ m.

These values can be accepted as minimum requirements for multiple crossings and annual permits. Fig. 5 provides the comparison of the target reliabilities based on economic optimisation and human safety levels. For instance for medium-span bridges (20 m < $L \leqslant 50$ m), economic optimisation dominates target reliabilities for $B/C_f < 0.001$ while for higher ratios human safety criteria become decisive. In the former case the benefit is too low to accept risks related to possible failure; in the latter case lower target reliability could be justified by the benefit, but minimum human safety levels need to be adhered to. For the latter case normal traffic can be restricted during crossing when lower target reliability level needs to be considered.

These target levels should be recalculated for different reference periods t_{ref} for which permits could be issued. The guidance provided in EN 1990 [9] assumes independent failure events in subsequent reference periods. However, this assumption cannot be accepted in the analysed case as the failure events are nearly fully correlated (Section 7.2.1) and the target reliabilities tend to be independent of a reference period [53,54]. That is why it is proposed here to accept the aforementioned β_{hs}-values for the assessment of crossing irrespectively of a reference period t_{ref}. However, this approach can be accepted only for short reference periods, say one year or the period between main inspections, for which it can be reasonably assumed that conditions of the bridge with respect to e.g. deterioration of construction materials or pavement do not change significantly.

For permits related to a single crossing, it is inconsistent to require the annual targets indicated above and recalculation to very short reference periods hardly makes sense. The edition of ISO 2394 issued in 1998 indicates that target levels should be considered as an average over a reference period such as one year. In general, it is allowable to have a large failure rate in some part of the reference period and a smaller value in another part and deviations from the yearly average for a much shorter period of time can be accepted. Consequently the target reliability level for a single crossing can be based purely on economic optimisation using Fig. 5. In such cases failure consequences C_f need to explicitly account for societal consequences related to fatalities due to bridge failure; these consequences should be transformed into monetary units by multiplying:

- The expected number of fatalities.
- The Societal Value of a Statistical Life according to the Life Quality Index approach, i.e. the amount to be compensated for each fatality, as proposed in ISO 2394 [14].

8. Partial factors

The investigation and definition of all necessary stochastic parameters allows for the calculation of partial factors. According to Eqs. (5) and (6) the necessary input parameters include the target reliability, sensitivity factors and coefficients of variation that can be adjusted as shown in previous sections. It is important to note that a careful review of their applicability to each considered situation is necessary. Moreover, the suggested modification of partial factors is aimed at the global verification only. Additional local checks with modifications of the target reliability and sensitivity factors might be necessary.

As an example, partial factors are calculated and plotted in Fig. 6 for the target reliability β from 2.7 to 4.4 and for a range of the load ratio κ. Partial factor γ_{Qspec} is calculated in accordance with Eq. (5), V_{Qspec} is chosen as 0.12 for unrestricted traffic conditions and as 0.09 for the controlled crossing.

It can be observed that the load ratio has a major impact on the partial factor. As the permanent load becomes dominant the partial factor significantly decreases. The selected target reliability index has a larger influence at low κ values. The influence of both load ratio and target reliability is less apparent for the controlled traffic loading where a lower coefficient of variation is applied.

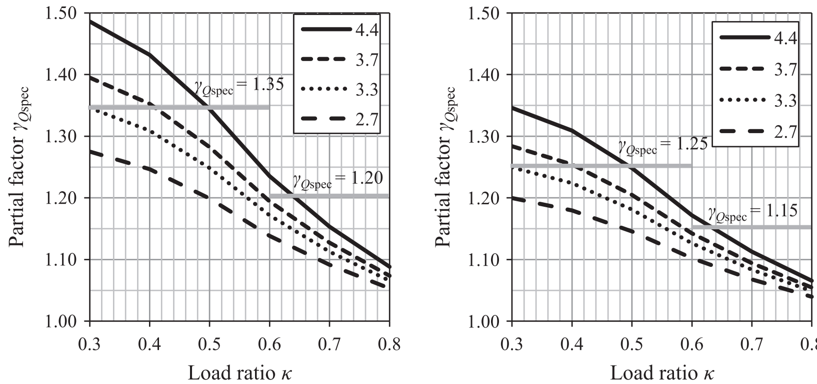

Fig. 6. Variation of the partial factor γ_{Qspec} with the load ratio κ for unrestricted traffic conditions (V_{Qspec} = 0.12, left) and for the controlled crossing (V_{Qspec} = 0.09, right).

Considering the annual target reliability index of 3.7 for long-span and 3.3 for medium-span existing bridges and sufficiently smooth profile conditions, the following values of the partial factor can be recommended for the assessment of existing bridge due to special heavy vehicles:

- For main structural members of long-span bridges ($\kappa > 0.6$) – $\gamma_{Qspec} \approx 1.20$ for unrestricted traffic conditions and $\gamma_{Qspec} \approx 1.15$ for the controlled crossing.
- For short to medium span bridges and local verifications ($\kappa < 0.6$) – $\gamma_{Qspec} \approx 1.35$ for unrestricted traffic conditions and $\gamma_{Qspec} \approx 1.25$ for the controlled crossing.

However, these simplified recommendations should be considered with caution. The main aim of this study is to propose a framework that can be readily extended to case specific situations. The proposed characteristics of basic variables and partial factors given above are intended for general use and are deemed conservative in most cases.

9. Concluding remarks

Current partial factors in Eurocodes are not optimal for reliability verifications of new or existing bridges exposed to load effects due to special heavy vehicles with known axle loads and weights. The present study indicates that:

- When specifying an appropriate partial factor for the load effect of a special vehicle, γ_{Qspec}, it is essential to recognise differences in the definition of characteristic load effects due to common traffic and the special vehicle; the characteristic value for common traffic is broadly defined with a large return period and thus with significant reliability margin whilst a mean value of the load due to a special vehicle is used.
- The partial factor γ_{Qspec} is affected by variability and possible bias in the static load effect, dynamic amplification and by uncertainties in an applied model for load effect; the factor γ_{Qspec} is further influenced by selected target reliability and by the sensitivity factor of the traffic load.
- When assessing variability of the dynamic amplification, the distinction between unrestricted- and controlled crossings (in terms of speed of a vehicle and its position on the bridge) needs to be made.
- The sensitivity factor predominantly depends on the ratio of permanent and variable actions and variability of the dynamic effect; the approximate value of -0.7 is mostly conservative.
- When multiple crossings of the same vehicle or vehicles of the same type during a reference period are considered, the partial factor γ_{Qspec} is independent of the number of crossings while the characteristic value should be adjusted with respect to the expected number of crossings according to Eq. (10).
- In the assessment of individual or multiple crossings, target reliability should be established on the basis of benefits associated with the transport and consequences of possible failure of bridge; guidance is provided in Fig. 5. When the crossing along with normal traffic is allowed, minimum human safety levels should be adhered to; indications for acceptance criteria differentiated with respect to span length are also given in Fig. 5.
- Numerical part of the study indicates that the following values can be accepted for the partial factor γ_{Qspec} (considering the annual target reliability index of 3.7 for long-span and 3.3 for medium-span existing bridges with sufficiently smooth profile conditions): γ_{Qspec} = 1.20 for main structural members of long-span bridges and unrestricted traffic conditions; γ_{Qspec} = 1.15 for the controlled crossing, – γ_{Qspec} = 1.35 for short to medium – span bridges and local verifications and unrestricted traffic conditions; γ_{Qspec} = 1.25 for the controlled crossing.

Further studies should be particularly focused on advanced probabilistic modelling of dynamic amplification due to crossing by special vehicles and on improving model uncertainties.

Acknowledgements

The presented research has been partly conducted in the framework of the research projects VG20122015089 "Safety Assessment of Transportation Structures" supported by the Ministry of the Interior of the Czech Republic, LG14012 "International Co-operation in Reliability and Risk Assessment of Structural Systems" supported by the Ministry of Education, Youth and Sports of the Czech Republic and "Investigation of the implications behind adaptation of EN 1991-2 in South Africa" supported by the Subcommittee B of Research Committee at Stellenbosch University, South Africa. Comments and suggestions provided by reviewers are appreciated.

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