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Procedures for reliability assessment of existing bridges

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Subject review

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Procedures for reliability assessment of existing bridges

Many bridges in the world have been built a long time ago and, because of social and economic needs for their continued use, it is necessary to assess their reliability. An overview of recent studies of these now highly topical issues is presented in the paper. Proposed advanced multi-level methods for assessment of existing bridges are based on the probabilistic theory of reliability, and involve typical material testing, assessment of condition and redundancy of structural systems, and study of actual traffic load. A review of foreign codes with their comparison is presented.

Key words:
existing bridges, reliability, assessment procedures, standards

Pregledni rad

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Postupci ocjenjivanja pouzdanosti postojećih mostova

Budući da su mnogi mostovi u svijetu davno izgrađeni, a iz društvenih i ekonomskih razloga potrebno ih je i dalje upotrebljavati, nužno je provesti ocjenjivanje njihove pouzdanosti. U radu su opisana najnovija istraživanja u svezi s tom danas vrlo aktualnom problematikom. Predložene napredne proračunske metode za ocjenjivanje postojećih mostova u više koraka utemeljene su na probabilističkoj teoriji pouzdanosti, a razmatraju se karakteristična ispitivanja materijala, ocjene stanja i zalihe nosivosti konstrukcijskog sustava i stvarna prometna opterećenja. Dan je pregled inozemnih normi i napravljena njihova usporedba.

Ključne riječi:
postojeći mostovi, pouzdanost, postupci ocjenjivanja, norme

Übersichtsarbeit

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Verfahren zur Beurteilung der Zuverlässigkeit bestehender Brücken

Da weltweit zahlreiche Brücken vor langer Zeit erbaut wurden, aber aus gesellschaftlichen und wirtschaftlichen Gründen weiterhin benutzt werden, ist die Beurteilung ihrer Zuverlässigkeit notwendig. In dieser Arbeit sind die neuesten Untersuchungen zu diesem aktuellen Thema beschrieben. Gegebene fortgeschrittene Berechnungsmethoden zur schrittweisen Beurteilung bestehender Brücken beruhen auf der probabilistischen Zuverlässigkeitstheorie. Verschiedene typische Materialversuche, Beurteilungen des Zustands und der verbleibenden Tragfähigkeit des Tragwerks, sowie die wirkliche Verkehrsbelastung werden betrachtet. Eine Übersicht internationaler Normen ist gegeben und ein Vergleich ist aufgestellt.

Schlüsselwörter:
bestehende Brücken, Zuverlässigkeit, Beurteilungsverfahren, Normen

1. Introduction

Many bridges in the world have been built a long time ago and, due to social and economic needs for their continued use, it is indispensable to assess their current level of reliability. These existing bridges were designed and constructed in accordance with the standards and technical regulations applicable at the time of their construction, which do not correspond to current much stricter requirements. Nominal traffic loads of road bridges are nowadays much higher, and introduction of a higher rail category (new categorisation) is often required for railway bridges. The "nominal" load-carrying capacity of existing bridges can be reduced by various influences during their service life, by inappropriate details, by neglect of the durability problem and construction errors, and by inadequate maintenance. Advantages of existing bridges as related to the design of new bridges lie in the fact that geometrical dimensions, material properties, some load values, structural behaviour, deterioration level, etc. can be measured on the bridge structure itself.

Current standards (Eurocodes) for the design of new bridges are based on conservative assumptions regarding the intensity of the applied actions, and structural response of bridges to such actions. During analysis of a new bridge, it should be checked whether the structure is appropriate for a specific intended use during the design service life of the bridge (50 years for bridges of usual size or normal level of importance, i.e. 100 years for large size bridges or highly important bridges - HRN EN 1990/NA [1]), which implies fulfilment of special durability requirements and, in addition, the reference period for design actions is related to the design service life of the bridge. These requirements, implicit or explicit, cannot be applied to existing bridges. Although current standards for design of new structures result in creation of safe and cost-effective bridges, the use of these standards for assessment of existing bridges may show that many of these bridges need to be strengthened or even replaced. However, direct costs and user costs for upgrading or replacement of an existing bridge are generally very high, and costs for upgrading of all bridges along a given traffic route would be excessive. That is why the assessment of existing bridges should include the reassessment and possibly loosening of conservative design requirements that are on the safe side and that have been introduced into Eurocodes for simplification purposes. This can be achieved through:

- reduction of target reliability index values for existing bridges as compared to new bridges;
- use of advanced calculation procedures and assessment methods as compared to simplified calculation procedures on the safe side that are used for the analysis of new bridges,
- update and adjustment of traffic load models based on specific (real-life) data for a particular bridge and a reduced service life,
- collection of additional information about properties of bridge materials and their response to actions through structural monitoring,
- load testing to estimate with a greater level of accuracy the actual load-carrying capacity of the bridge.

The use of such advanced probabilistic procedures has shown that in many cases a bridge that does not meet usual safety requirements can in fact safely carry actual service loads, without requiring any strengthening or replacement [2-4].

An efficient and cost-effective maintenance, repair, rehabilitation or replacement of bridge structure (if needed) can be achieved only by assessing reliability of existing bridges, based on detailed testing. Significant differences between assessment of existing bridges and analysis of new bridges lie in the fact that existing bridges involve higher strengthening costs, more complex analysis, possibilities for conducting on-site inspection and testing, and possibility for reducing the reference period (shorter remaining service life).

Intensive research efforts have been made over the past three decades to develop new procedures for the assessment, rehabilitation, and management of existing bridges. This research has resulted in publication of a number of documents including the basic standard for probabilistic modelling [5], ISO-2394 [6] providing basic principles for reliability of structures, ISO-13822 [7] providing basis for design in assessment of existing structures, and RILEM documents for probabilistic assessment of existing structures [8]. Some recent European research projects such as BRIME [9], COST345 [10], F08a [11], and SB-LRA [12-14], have resulted in development of guidelines for current procedures for assessing reliability of existing bridges. According to Lind's postulate [15] stating that the present practice (Eurocodes) results in the sufficiently safe and cost-effective structures, all these projects show that:

- high reliability indices are always used when reliability can be ensured at a relatively low cost,
- reliability requirements become higher if brittle fracture may occur,
- reliability requirements become lower if deterioration or fatigue are present and so the failure, if it occurs, occurs at a later time,
- reliability requirements are often higher for (significant) details [6, 7].

New standards and manuals for the assessment of existing bridges have been approved in some countries including Austria [16], Canada [17], the Netherlands [18], Nordic countries [19], Germany [20, 21], USA [22], Switzerland [23], and Great Britain [24]. Such standards do not exist in Croatia and so engineers in practice most often use current codes for new bridges for assessment of existing bridges, which leads to unnecessary and expensive repairs and rehabilitations. That is why this paper proposes guidelines for the preparation of a Croatian standard for assessing reliability of existing bridges, the principal objective being to regulate this economically and socially highly important area.

2. Multi-level procedure for assessment of existing bridges

The reliability assessment of an existing bridge to determine the load-carrying capacity, the capability to assume higher loads or to extend its service life, is a generally adaptive multi-level process that enables the refinement of an initial engineering estimate of the present and future state of the bridge and its behaviour [25].

At the initial level 1 the assessment is performed using standard procedures based on currently valid standards (Eurocodes), in the same way as for new bridges. If the bridge meets the initial level requirements, then no additional calculations or measures are necessary, and the bridge can remain in service, without any additional checks.

However, bridges that do not meet initial level requirements are checked using intermediate level procedures that are most often still semi-probabilistic, usually with the prescribed reduced partial factors for actions γ_F, taking into account detailed examination and testing results so as to enable a better load-carrying capacity assessment in calculating resistance. Advantages of these procedures lie in the cost-effective analysis and ease of use and, in many cases, these procedures may suffice to confirm or dispute the results obtained at the initial level.

Advanced higher level procedures comprise safety considerations at the level of the basic structural system, "parallel" structural systems with multiple loadpaths and robustness criteria, and direct use of the reliability analysis methods. The basic premise is that the existing bridge under consideration does not have to meet all requirements of standards for the design of new bridges, but that the general level of reliability defined in these standards has to be respected. The probabilistic analysis is used in which a particular existing bridge is considered. In other words, a "standard" is defined that relates specifically to the existing bridge under consideration for both load models and resistance models. Traffic actions based on the real traffic are considered in the analysis, and the resistance values are obtained by direct analysis of results gained by detailed testing of the bridge (testing of materials, e.g. measurement of the mean compressive strength and standard deviation of concrete), and the material properties do not need to be converted into design values.

In principle, the reliability assessment of existing bridges is conducted only for the ultimate limit state, but not for the serviceability limit state, as this state is considered as already checked through appropriate structural condition inspections. However, if the serviceability of the structure has been altered, e.g. if higher traffic load is applied to the bridge, then the serviceability must also be checked through analyses based on updated actions and serviceability parameters, using the usual verification format (Eurocodes).

3. Assessment based on probability and target level of reliability

3.1. General

The basic standard for the analysis of new bridge structures in the EU countries is the Eurocode (HRN EN 1990) [26], which is based on the concept of limit states and the use of the partial factor method (PFM). The probabilistic model rules indicated in [5, 8] were used in the preparation of the code.

Proposed partial factor values for actions γ_F and combination factors ψ were defined on the basis of calibration to a long experience of building tradition, and on the basis of statistical evaluation of experimental data and field observations using the probabilistic theory of reliability.

The general expression for verification of the ultimate load-carrying capacity is defined using the known expression:

$$R_k / \gamma_M \geq \sum_{j\geq1} \gamma_{G,j} G_{k,j} \text{"+"} \gamma_P P \text{"+"} \gamma_{Q,1} Q_{k,1} \text{"+"} \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \qquad (1)$$

The design concept for the ultimate load-carrying capacity in linear-elastic analysis is presented in Figure 1.

Partial factors for the assessment of existing bridges must also be calibrated using the procedures based on probability, with the partial factor method format, similar to expression (1). At that, unlike in the analysis of new bridges, the partial factor γ_R is calibrated for various reliability index values depending on the limit state considered, taking into account the ductility level and the consequence of an element failure on the structural system.

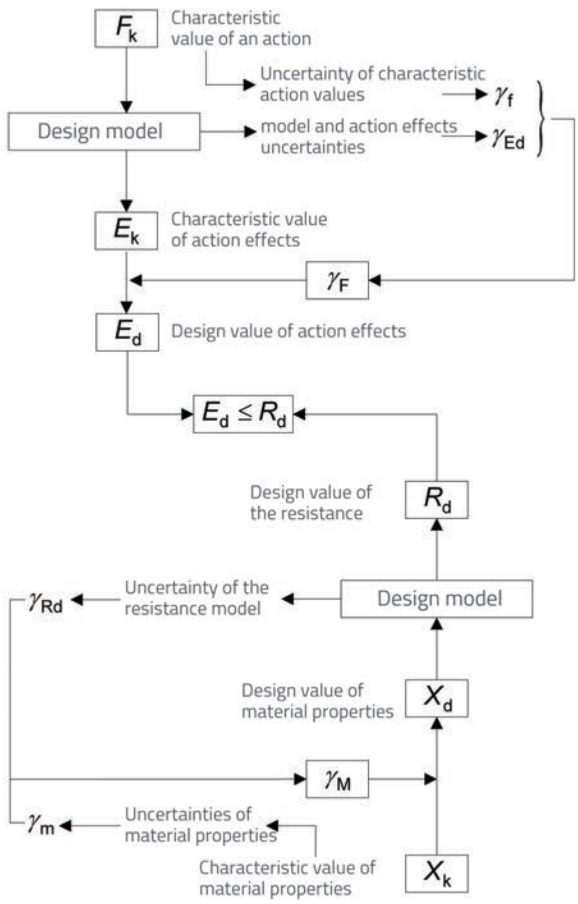


Figure 1. Design concept for ultimate limit state in linear-elastic analysis

Although methods utilised for calibration of partial factors in assessment of existing bridges are similar to those used for new bridges, partial factors are lower because of the lower target reliability levels and shorter service life (Canada [17], Netherlands [18], Nordic countries [19], USA [22]), or because of removal of uncertainties in these factors while retaining the same level of reliability (Austria [16], Germany [20], Switzerland [23], Great Britain [24]). Differences in the level of reliability can be justified by the cost–benefit analysis. A higher level of reliability for new bridges requires only higher material costs, while other costs, including design and construction practically remain unchanged. On the other hand, many existing bridges would have to be replaced due to a higher reliability level which would involve, in addition to the cost for materials and construction of a new bridge, other considerable direct costs for the demolition and removal of the existing bridge, and most often higher indirect costs arising from disturbance to traffic, traffic jams and the related road user costs, economic losses and environmental impacts. Lower reliability level and shorter service life can only be justified by requiring regular inspections and maintenance activities, and if the positive experience exists about behaviour of bridges assessed using a lower target reliability level.

3.2. Assessment based on probability

If a bridge does not meet requirements during standard verifications based on the partial factor method (PFM) concept, then a direct probabilistic method can be used so as to ensure that structural elements of the bridge meet appropriate requirements:

$$\beta = -\Phi^{-1}(P_f) \geq \beta_{target} \tag{2}$$

where β_{target} is the target index of reliability depending on the limit state under consideration and the consequences of failure of one element on the entire structural system, Φ^{-1} is the inverse function of a cumulative standard normal distribution, and P_f is the probability of failure for the failure mode considered within an appropriate reference period, defined by the expression:

$$P_f = P\,(Z = R - S < 0) \tag{3}$$

where Z is the ultimate state function, a R and S are the generalized resistances and action effects. The reliability index β can roughly be calculated using the first order reliability method (FORM) or the Monte Carlo simulation.

The advantage of using the direct reliability analysis in comparison with the use of partial factors calibrated according to the probabilistic theory of reliability lies in the fact that verifications according to standards based on partial factors fulfil or exceed target reliability levels, while the direct analysis of reliability is used to check whether a particular bridge, with all its properties and probable types of failure, will meet the target level of reliability (probability of failure).

However, these reliability methods are approximate only and their results greatly depend on assumptions related to the functions of distribution of resistance and actions, and on complexity of the function of limit state under consideration. This is why the assessment of the level of safety using comparisons between reliability indices and their target values must be followed by a subsequent assessment that will contain a detailed analysis of sensitivity, and the comparison with results obtained during preliminary assessment.

3.3. Target levels of reliability

The selection of the target index of reliability for assessment purposes generally depends on specific features of the bridge under consideration, such as the cause and type of failure, consequences of failure, cost of safety measures to be implemented to reduce the failure risk, and also on the economic and social conditions, and environmental conditions. All these factors cannot be considered in a simple way and so the target indices of reliability for standard bridges are defined in advance, based on the experience of experts, and according to political, social and economic constraints, construction practices and quality control measures applied in a particular country, effects of environment on the deterioration of bridges, and historic bridge behaviour data. Failure types can thus be classified as follows:

- ductile failure with remaining capacity due to strengthening,
- ductile failure without remaining capacity,
- brittle failure.

In accordance with the above considerations, structural parts in which the failure can occur without warning must be designed for the higher reliability level than those in which some kind of warning occurs prior to failure, which enables undertaking of appropriate measures to avoid serious consequences. An example of such approach according to requirements given in the standard of Nordic countries is given in Table 1 [19].

Table 1. Reliability indices β and associated values of failure probability P_f for ultimate limit states and one-year reference period [19]

Consequence class (Reliability class)	Failure type		
	Failure type I Ductile failure with remaining capacity	**Failure type II** Ductile failure without remaining capacity	**Failure type III** Brittle failure
CC1 (RC1)	$\beta \geq 3{,}09$ ($P_f \leq 10^{-3}$)	$\beta \geq 3{,}71$ ($P_f \leq 10^{-4}$)	$\beta \geq 4{,}26$ ($P_f \leq 10^{-5}$)
CC2 (RC2)	$\beta \geq 3{,}71$ ($P_f \leq \approx 10^{-4}$)	$\beta \geq 4{,}26$ ($P_f \leq 10^{-5}$)	$\beta \geq 4{,}75$ ($P_f \leq 10^{-6}$)
CC3 (RC3)	$\beta \geq 4{,}26$ ($P_f \leq 10^{-5}$)	$\beta \geq 4{,}75$ ($P_f \leq \approx 10^{-6}$)	$\beta \geq 5{,}20$ ($P_f \leq \approx 10^{-7}$)

Table 2. Reliability index values β and associated failure probability values P_f for ultimate limit states, extended Table B.2 [26]

Reliability class (Consequence class	Lowest values β	
	1-year reference period	50-year reference period
RC1 (CC1)	$\beta \geq 4{,}2$ ($P_f \leq 10^{-5}$)	$\beta \geq 3{,}3$ ($P_f \leq 5 \cdot 10^{-4}$)
RC2 (CC2)	$\beta \geq 4{,}7$ ($P_f \leq 10^{-6}$)	$\beta \geq 3{,}8$ ($P_f \leq 10^{-4}$)
RC3 (CC3)	$\beta \geq 5{,}2$ ($P_f \leq 10^{-7}$)	$\beta \geq 4{,}3$ ($P_f \leq 10^{-5}$)

Table 3. Target reliability index β and associated failure probabilities Pf for ultimate limit states and one-year comparison period [5]

Relative cost of interventions (remedial measures)	Consequences of failure		
	Small	Medium (moderate)	Serious
High	$\beta = 3{,}1$ ($P_f \approx 10^{-3}$)	$\beta = 3{,}3$ ($P_f \approx 5 \cdot 10^{-4}$)	$\beta = 3{,}7$ ($P_f \approx 10^{-4}$)
Usual (average)	$\beta = 3{,}7$ ($P_f \approx 10^{-4}$)	$\beta = 4{,}2$ ($P_f \approx 10^{-5}$)	$\beta = 4{,}4$ ($P_f \approx 5 \cdot 10^{-5}$)
Low	$\beta = 4{,}2$ ($P_f \approx 10^{-5}$)	$\beta = 4{,}4$ ($P_f \approx 5 \cdot 10^{-5}$)	$\beta = 4{,}7$ ($P_f \approx 10^{-6}$)

The comparison of reliability index values β from Table 1 with Eurocode values [26] specified in Table 2 confirms that standards for new structures are conservative and that they implicitly cover all types of failure.
Target values of the index of reliability β and associated failure probabilities P_f for the ultimate limit state and one-year reference period, dependent on the relative cost of safety measures and consequences of failure according to JCSS Probabilistic Model Code [5], are presented in Table 3.

3.4. Definition of redundancy and robustness of structural systems

Bridge structures are systems of interconnected elements and so the usual procedure for checking reliability of a critical element using the linear-elastic analysis does not necessarily provide a correct assessment of the real safety of structural system of a bridge. The level of real existing reliability can only be determined by nonlinear analysis, which can follow a partial or full failure of each structural element, and consider the redistribution of effects of actions within the structural system once an element has reached the nonlinear behaviour or failure. The usual analysis of new structures is usually conducted at the level of individual elements and redundancy of structural system of the bridge is not considered for reasons of simplicity. Three types of redundancy are differentiated: load path redundancy due to multiple (three or more) load paths, structural redundancy for statically indeterminate continuous structural systems that enables force redistribution, and internal redundancy when a bridge element contains three or more elements that are mechanically connected to form multiple independent load paths. The existing bridges whose structural systems possess redundancy are unjustly discredited, if the presence of redundancy is not taken into account. "Parallel" structural systems in which individual elements are connected parallel in relation to their function, are redundant. The entire system's failure occurs only in case of failure of several elements. Statically indeterminate systems are parallel systems if the elements are sufficiently ductile. If a "parallel" system also contains ideally brittle elements, the failure can occur just like in a "serial" system. The probability that such a system will fail can be calculated using the "intersection" of limit states of all elements of the system.
"Serial" structural systems, in which elements are linked in series in relation to their function, are non-redundant. The failure of an individual element causes failure of the entire system ("the weakest link"). Statically determinate systems are serial systems. If elements are brittle, the structural system collapses by brittle failure and, if they are ductile, the collapse is due to excessive yielding. The probability of failure can be calculated using the "compilation" of limit states of all elements. The linear-elastic analysis is obviously sufficient for the verification of reliability of such structural systems.
According to its definition, robustness is the ability of a load-carrying system to withstand events such as the explosion, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause, i.e. it is the capacity to withstand damage, and to retain its basic significant function [1]. Robustness does not necessarily eliminate or reduce known risks, and it principal value is in the fact that it reduces risks from unknown causes, and limits impact of local failure resulting from undetermined accidental actions. Bridge structure assessment as related to robustness is directly mentioned in the Swiss standard only [23]. It requires investigation of the consequences of damage due to accidental actions or environmental impact on the load-carrying capacity and stability of the structure, with possible risks and adequate failure scenarios.

3.5. Real traffic loads on bridge

According to HRN EN 1991-2 [27], traffic loads on bridges are the maximum expected loads for a long-lasting reference period (1000 years for road bridges).

Data from Europe [28, 29] and North America [30] point to large variability in live loads for road bridges, depending on the economy and other factors relevant for individual countries or regions. Traffic loads for railway bridges are defined at the European level [27] and also include real trains and so that the classification of loads can easier be made [31]. In addition, it should be taken into account that the service life of an existing bridge is usually much shorter than the design service life.

It is therefore quite clear that the real traffic loads on the bridge must be measured for advanced higher level assessments as, in many cases, these loads are the basic source of uncertainty and hence relevant variables for probabilistic reliability assessment of existing bridges.

The real traffic load on bridges is measured using the so called weigh-in-motion (WIM) system that is applied to both road [32] and railway bridges [33]. In case of steel bridges, the WIM measurements can be replaced or complemented with deformation measurements of critical details. In Croatia, annual publications "Traffic count on roads in the republic of Croatia" [34] contain many useful data, including the percentage of heavy vehicles on Croatian roads, but they do not provide their total weight and load distribution. Although it is mentioned in the data for 2002 that the WIM system testing was initiated at some segments of three national roads, the results of these measurements are unfortunately not available.

4. Overview of standards for assessment of existing bridges

4.1. General

If standards for assessing reliability of existing bridges are not available in a particular country, structural engineers shall use appropriate standards for the design of new bridges. That can lead to wrong decisions and hence to un-economical and much more costly maintenance of existing bridges, which is why many countries have in fact adopted such standards.

Basic characteristics of the selected foreign reliability-assessment standards are presented below. Out of these standards, the Canadian standard [17], the US standard [22], the Dutch standard [18], and the German guideline [20] cover road bridges only, while the Austrian standard ONR 24008 [16] is applied for the assessment of existing road and railway bridges, and the Swiss standards from the SIA 269 series [23] cover the entire area of civil engineering structures, including road and railway bridges.

4.2. Standards based on reduced reliability indices

The concepts for assessing reliability of existing bridges, as given in Sections 2 and 3, are the most consistently implemented in the Canadian standard CAN/CSA-S6-06 [17], US standard MBE [22] and Dutch standard NEN 8700 [18].

They are based on the probabilistic criterion for human safety, economic aspects and reduced remaining service life [35-39]. The maximum allowable annual probability for the loss of human lives has been adopted in accordance with ISO 2394 [6].

4.2.1. Canadian standard CAN/CSA-S6-06

In Canadian standard [17], the recommended adjusted lowest target value of the reliability index β is defined by the expression:

$$\beta = 3{,}75 - (\Delta_C + \Delta_S + \Delta_I + \Delta_R) \geq 2{,}0 \quad (4)$$

where:

- Δ_C - adjustment factor for component behaviour (0.0 for failure without warning, 0.25 for failure with little or no warning, but with retention of post failure capacity, 0.5 for gradual failure with probable warning)
- Δ_S - adjustment factor for system behaviour (0.0 if element failure leads to total collapse, 0.25 if element failure probably does not lead to total collapse, 0.5 if element failure leads to local failure only)
- Δ_I - adjustment factor for inspection level (-0.25 if component is not inspectable, 0.0 if component is regularly inspected, 0.25 if critical component was inspected by evaluator)
- Δ_R - adjustment factor for risk category (0.0 for all traffic categories except supervised overload, 0.5 for supervised overload).

The assessment procedure starts with identification of the most probable forms of failure and definition of appropriate reliability

Table 4. Maximum partial factors for permanent load and traffic load for assessment [40]

Load	Symbol	Target reliability index β						
			2,75	3,00	3,25	3,50	3,75*	4,00
Permanent load D1	α_{D1}	1,05	1,06	1,07	1,08	1,09	1,10	1,11
Permanent load D2	α_{D2}	1,10	1,12	1,14	1,16	1,18	1,20	1,22
Permanent load D3	α_{D3}	1,25	1,30	1,35	1,40	1,45	1,50	1,55
Traffic loads	α_L	1,35	1,42	1,49	1,56	1,63	1,70	1,77

D1 - prefabricated elements and cast-in-place concrete excluding decks; D2 - cast-in-place concrete decks; D3 - bituminous surfacing with assumed standard thickness of 90 mm.
*Target index of reliability and the corresponding safety factors are also used for the design of new bridges.

indices β, which are related to each particular failure mode, taking into account the behaviour of the structural element and system, and the level of inspection (expression (4)) [40].
Instead of applying direct verifications of reliability, the adjustment of partial safety factors is given based on the target reliability index, as shown in Table 4. The assessment is made using the format similar to that used in the Eurocode, but adjusted to obtain load rating parameters, i.e. the multiplier of the dominantly variable action F (required to cause failure):

$$F = \frac{U R_r - \sum \alpha_D D - \sum \alpha_A A}{\alpha_L L(1+I)} \tag{5}$$

In Expression (5) R_r is the design resistance, D is the permanent load, A are other variable actions, L is the dominant variable action (e.g. traffic load), I is the dynamic amplification factor, U is the resistance adjustment factor, while α_D, α_A and α_L are associated partial factors for the specified actions.
The design resistance adjustment factor U is given in standard [17] for various structural elements and limit states, taking into account the variation between the real element resistance obtained by testing and the design resistance calculated by simplified methods in the standard. The design resistance R_r is calculated using nominal values of material strength parameters as taken from the drawings or historic data, multiplied with the material resistance factors, such as those used in the design of new bridges, as presented in Table 5.

Table 5. Material resistance factors adopted in reliability assessment [40]

Material and critical failure mode	Material resistance factor
Concrete	Φ_c = 0,75 (1/1,33)
Reinforcing steel	Φ_s = 0,90 (1/1,11)
Prestressing steel	Φ_p = 0,95 (1/1,05)
Structural steel (bending, shear, and tension)	Φ_s = 0,95 (1/1,05)
Structural steel (compression and torsion)	Φ_s = 0,90 (1/1,11)

4.2.2. American standard MBE

The American standard MBE [22] employs reliability-based assessment principles and the load rating process similar to those specified in Canadian standard CAN/CSA-S6-06 [17]. The standard is based on a considerable number of investigations and on the collection of data about the load and resistance of existing bridges. Reliability is determined using the Monte-Carlo simulations based on data collected from 145 typical American bridges [41]. One million Monte Carlo simulations were made for each bridge considered [42]. The reliability indices obtained are independent from the assessment methodology as they represent the real characteristic reliability. The assessment of load is based on the reduced partial factors and the set of "legal" heavy vehicles with the weight lower than that of vehicles used in the analysis of new bridges. The rating results reflect the live load-carrying capacity at its current condition. Therefore, a recent thorough field inspection is essential for accurate load ratings [43]. The assessment procedure must be conducted for all new bridges, rehabilitated or repaired bridges, and existing bridges, and the results must be placed in the personal identity card of the bridge as contained in the data bank. The assessment must be repeated in case of any change in structural condition, such as structural damage or deterioration, change of self-weight due to renovation of wearing surface, change in traffic conditions, change of regulations, etc.
Two levels of reliability, dependent on service life, are used during assessment. During initial assessment, the target index of reliability β_{target} = 3.5 is used just as in the analysis of new bridges for the design service life of 75 years and, at higher levels of assessment, the target index of reliability is β_{target} = 2.5 for the 5-year service life [43], which corresponds to the time interval in which detailed inspections and assessments of American bridges must be made.
The assessment factor RF is defined instead of the usual verification of the ultimate limit state as given in expression (1). This factor corresponds to the traffic action multiplier F provided in expression (5). If RF is smaller than 1.0, the verification requirement has not been fulfilled.
The assessment factor RF is described by the expression:

$$RF = \frac{\phi_c \phi_s \phi R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_p)(P)}{(\gamma_{LL})(LL + IM)} \tag{6}$$

where R_n is the design nominal resistance, DC is the effect of self-weight of structural components and attachments, DW is the effect of self-weight of wearing surface and bridge equipment, P is the effect of other permanent actions (secondary effects due to prestressing in continuous structures, residual stresses due to construction procedures used), LL is the effect of traffic load, IM is the dynamic allowance, γ_{DC}, γ_{DW}, γ_P i γ_{LL} are partial factors for relevant actions in accordance with the standard [44], Φ_c is the condition factor, Φ_s is the system factor, and Φ is the resistance factor in accordance with the standard [44].
The real bridge condition at the time of assessment, defined based on bridge inspections and kept as data in the National Bridge Inventory (NBI), is taken into account by means of the condition factor Φ_c. The bridge condition assessment is made by the certified inspector and the value of 0 - 9 is allocated to each element of the bridge. It should be noted that the assessment of bridge condition only points to the deterioration of or damage to individual structural elements of the bridge, but it cannot directly provide information about load-carrying capacity of the bridge. The condition factor Φ_c amounts to 1.0 for the good or satisfactory condition of the structure (grade 6 or more), 0.95 for grade 5, and 0.85 for poor condition of the structure (grade 4 or less).
The structural system factor Φ_s is related to the redundancy of the entire superstructure, and it ranges from 0.85 to 1.0. Greater reliability levels are required for non-redundant structural

systems, in comparison to similar redundant structural systems. The objective is to achieve a uniform level of reliability of the structural system, rather than the uniform level of reliability of individual elements. The factor Φ_S generally depends on the number of plastic hinges that convert the structure into a mechanism. In case of steel bridges the factor Φ_S is dependent on the number of main girders, their spacing, and on whether they are welded or connected with rivets and, in case of prestressed concrete bridges, even on the number of tendons in a single web. For the verification of shear the value Φ_S amounts to 1.0.
The condition factor Φ_C and the system factor Φ_S are used only for verification of the ultimate limit state, while the product Φ_C Φ_S should be equal to or greater than 0.85. If the product is greater than 0.85, the reliability index β is greater than 2.5.
The reliability assessment is conducted at four levels. At the first level the nominal traffic load is used with the partial factor γ_{LL} = 1,75, and with the reliability index of β = 3,5. At the second level the in-service traffic load is used. It is equal to the nominal load, but the reliability index is β = 2.5, which gives the partial factor value of γ_{LL} = 1,35. At the third level of assessment, the reliability index is β = 2.5 and the "legal" specified heavy vehicles are used, while the partial factor γ_{LL} depends on an average daily traffic of such vehicles. If this traffic is unknown or if its exceeds 5,000 vehicles for routine commercial vehicles, then the partial factor amounts to γ_{LL} = 1.80, and it is γ_{LL} = 1,60 for specialized hauling vehicles. If an average daily traffic for heavy vehicles amounts to < 100, then the partial factor for routine commercial vehicles amounts to γ_{LL} = 1,40, while it is γ_{LL} = 1,15 for specialized hauling vehicles. Overweight vehicles for which a special traffic permit must be procured are considered separately. The analysis of each next level need not be made if the *RF* for the considered lower level is greater than 1.0. If the assessment factor *RF* is smaller than 1.0 even during the third level calculation, then advanced analytical methods, and other not specifically defined assessments, can be used.

4.2.3. Dutch standard NEN 8700

In the Dutch standard [18], the reliability index for assessment of existing bridges can be reduced to no less than $\beta_u = \beta_{new} - \Delta\beta_u$ under which level the existing bridges are considered unfit for use and urgency measures have to be taken immediately. Based on rough economic optimisation of existing structures, the value of $\Delta\beta_u$ = 1,5 has been adopted.
Another reliability level i.e. β_{repair}, was also introduced, with $\beta_u < \beta_{repair} < \beta_{new}$. As a rule, the existing structures without significant deficiencies are capable of meeting requirements for this reduced level of reliability. In this way, all structures that were considered safe according to previous standards, and that have proven to be safe in practice, do not have to be suddenly strengthened or replaced. The expression $\beta_{repair} = \beta_{new} - \Delta\beta_{repair}$, where $\Delta\beta_{repair}$ = 0,5, was proposed as the target value. The reliability indices β for new and existing structures, dependant on the consequence class, were obtained by considering of the maximum annual probability of failure (Table 6) [37, 39].
The method for the assessment of existing road bridges according to [18] has four levels [39].
The first level analysis is based on the adjusted partial factors that can be derived, for the defined reference periods, based on the value β. In [37], probabilistic methods were used to derive partial factors for the permanent and traffic loads, using distribution functions for traffic load obtained from weigh in motion (WIM) measurements on a representative Dutch motorway for the remaining 15-year service life (Table 7) [37, 38].
The actual use of the bridge, which may differ from the use anticipated in the design, is additionally considered in the

Table 6. Reliability indices β for new and existing bridges [37]

Consequence class	Reference period (number of years)	New	Repair	Unfit for use
		β_{new}	β_{repair}	β_u
CC1	15	3,3	2,8	1,8
CC2	15	3,8	3,3	2,5*
CC3	15	4,3	3,8	3,3*

* mjerodavna je maksimalna godišnja vjerojatnost sloma (sigurnost ljudi)

Table 7. Adjusted partial factors for the permanent and traffic load on bridges, as related to the consequence class (great proportion of self-weight) [37]

Classification	Reference period (number of years)	Partial factors			
		Consequence class 2 (CC2)		Consequence class 3 (CC3)	
		γ_G	γ_Q	γ_G	γ_Q
New	100	1,30	1,35	1,40	1,50
Repair	15	1,25	1,20	1,30	1,30
Unfit for use	15	1,10	1,10	1,25	1,25

second level analysis. Traffic measurements are needed in the third level analysis because the real bridge traffic has to be taken into account. The fourth level analysis is based on the full probabilistic approach.

4.3. Standards based on retained reliability indices

In Austrian standard ONR 24008 [16], Swiss standards from the series SIA 269 [23], and in the German guideline [20], the first level reliability assessment is conducted using standard procedures in accordance with standards valid for new bridges. Bridges that do not meet initial design requirements are checked by means of the second level semi-probabilistic procedure using reduced partial factors for permanent action γ_G = 1,20 and weights determined through real bridge measurements, while also taking into account results of detailed inspections and tests for updating of cross-sections and material properties. Partial factors for variable actions γ_Q and combination factors γ remain the same as in the analysis of new bridges (except in German guideline [20]). In German standard [45] and Swiss standard [46] the traffic load on road bridges does not include the double axle load TS in the third lane, but the partial factor γ_Q = 1,5 is used.
The subsequent higher level analyses are probabilistic. However, details on the reliability level used are given in Swiss standard only, as indicated below.

4.3.1. Austrian standard ONR 24008

In Austrian standard [16] the subsequent third level analysis is aimed at determining an optimum relationship between the costs and reliability of a particular existing bridge, and at reducing the maintenance and rehabilitation costs. The basic concept is that the existing bridge under consideration does not have to fulfill all requirements contained in currently valid standards for the design of new bridges, but that the general level of reliability - as defined in these standards - has to be maintained. The probabilistic assessment in which a real existing bridge is considered is used.
In subsequent fourth level analysis, a lower level of reliability is deliberately accepted, based on detailed explanations and appropriate alternative measures.
The general procedure for assessing reliability of existing bridges is presented in Figure 2.

4.3.2. Swiss standard SIA 269

Swiss standards for assessment of existing structures, designated as series 269 [23], are structured as Eurocodes. The main standard describes basic principles and procedures for the assessment of existing structures. This main standard is complemented with a series of standards which deal with specific issues. Thus the standard SIA 269/1 contains updated (revised) action models and action effects. Standards SIA 269/2 through SIA 269/6 (SIA 269/2 reinforced concrete, SIA 269/3 steel, SIA 269/4 prestressed structures, SIA 269/5 timber, and SIA 269/6 masonry structures) provide information for updating materials, structural parameters and models for various types of structures, especially with regard to materials and structural systems that were used in the past. The standard SIA 269/7 covers geotechnical aspects specific for existing structures, and the standard SIA 269/8 (only the pre-standard has so far been published) is related to seismic engineering of existing structures.

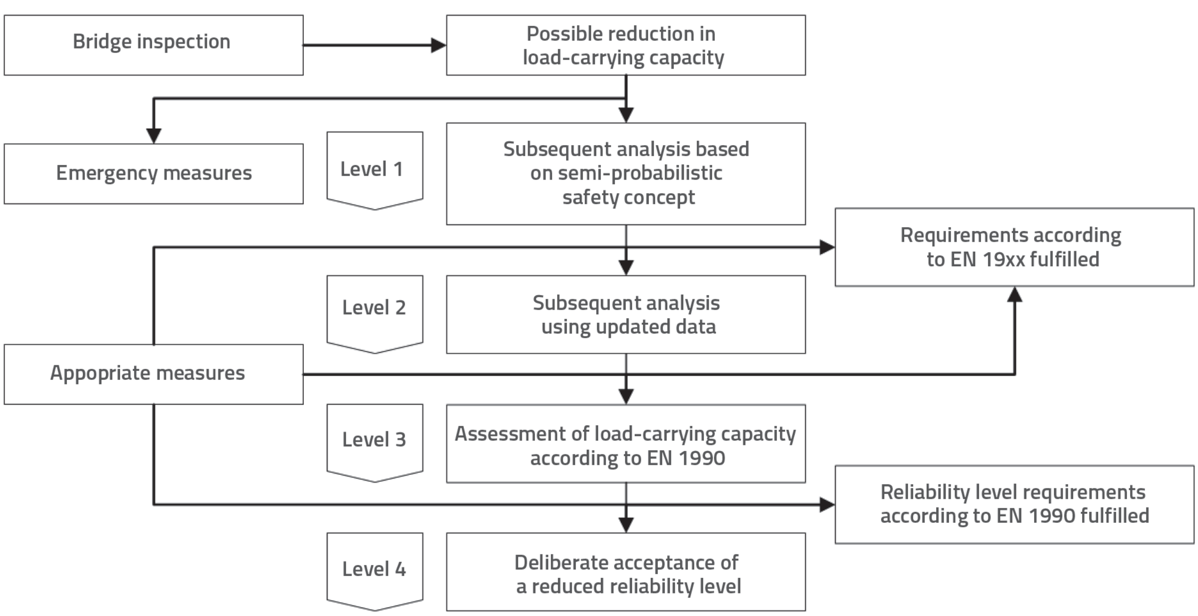
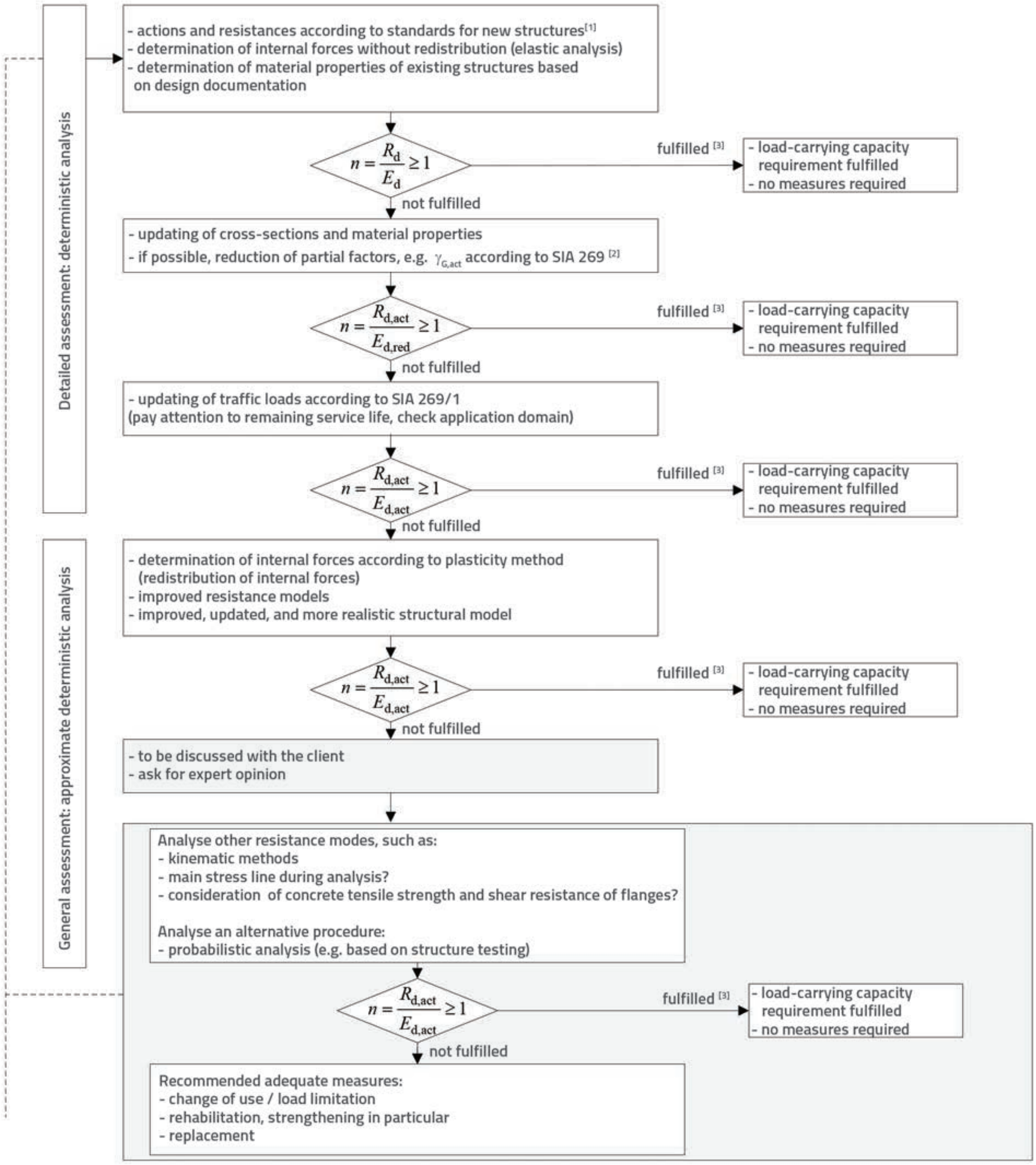


Figure 2. Schematic view of assessment according to ONR 24008 [47]

The remaining service life and operating conditions must be defined during assessment or planning of maintenance measures. The existing bridge condition assessment is of utmost significance for the identification of current and possible deficiencies. The assessment of the remaining service life, based on bridge condition, enables planning of measures aimed at increasing durability, which can be achieved either through repairs or by slowing down the deterioration process. At that, it is significant to make a distinction between structural deficiencies and durability issues because the time frame for repair measures can be extended. The deterioration of bridges is mostly caused by water and chlorides, and so the main focus must be placed on testing the waterproofing and chloride penetration levels. Current material properties are of crucial significance for structural assessments, and so visual inspections must be complemented with appropriate laboratory testing in order to enable an accurate



[1] taking into account valid standards and rule books
[2] according to SIA 269 the partial factor $\gamma_{G,act}$ = 1,20 instead of γ_G = 1,35 provided that permanent actions are updated in accordance with SIA 269/1
[3] lower values should be adopted for accidental actions (impact, earthquake) in accordance with relevant stadards

Figure 3. Bridge assessment procedure according to SIA 269 [48]

assessment of bridge condition, and to properly estimate further deterioration processes.

The structure safety is considered appropriate if the required level of reliability of a structure has been verified by calculations, or if the possibility of structural collapse is held under control through additional or emergency safety measures. Additional safety measures include limiting the use and remaining service life, limiting traffic load, increasing frequency and scope of structural inspections, etc. Emergency safety measures include limiting the use of the structure, supporting of the structure, more stringent supervision, traffic closure, evacuation of people in emergency situations, etc.

The general procedure for assessing reliability of existing bridges is presented in Figure 3 [48]. The index (subscript) "act" refers to updated data. The verification of adequacy of safety measures applied during maintenance is defined by the efficiency of measures,, which is estimated using the EF_M coefficient, defined by the expression:

$$EF_M = \frac{\Delta R_M}{SC_M} \tag{7}$$

where ΔR_M is the reduction of risk due to implementation of maintenance measures, while SC_M are costs required for fulfilment of safety requirements. A safety measure for maintenance is considered adequate if $EF_M \geq 1{,}0$, and it is usually inadequate if $EF_M < 1{,}0$. The consequences of failure are expressed as the ratio ρ between direct costs C_F in case of failure and C_W costs, which are needed to repair the structure after failure $\rho = C_F / C_W$. The target value of the related reliability index β_0 is presented in Table 8 for the 1-year reference period. The values of β_0 are equal to the ones given in Table 3.

The standard SIA 269/1 regulates the updating of representative values of actions. The factors for traffic load adjustment on road bridges, dependent on the type, span, and cross section, are given in Table 9. These values are used for bridges with bi-directional traffic, 6-9 m in width, and for two-lane bridges on motorways, 9-12 m in width.

Traffic adjustment models for railway bridges presented in SIA 269/1 are taken over from the European standard EN 15529 [31] in accordance with categorisation of European railway lines.

If further calculations, conducted at all levels, do not provide for a sufficient level of reliability, then the so called empirical analysis can be conducted. This analysis can be expected to provide a sufficient level of reliability if all of the following conditions are met:

- detailed inspections do not reveal damage or deficiencies that may reduce the load-carrying capacity,
- structural behaviour has been satisfactory over an extended period of bridge use,
- similar experiences exist regarding behaviour of comparable structures,
- no change of use is anticipated during the remaining service life of the structure, and
- the risk of structural failure, and the consequences of such failure, are acceptable.

If the expected sufficient level of reliability is based on the empirical analysis, then additional safety measures must be conducted.

4.3.3. German guideline

In addition to the German guideline for assessment of existing road bridges [23], the German Federal Institute for Rood Construction (BAST) has issued a number of accompanying documents such as the B 83 [49] that explains the concept of subsequent analysis, B 89 [50] that provides information

Table 8. Target values of reliability index β_0 for one-year reference period [23]

Efficiency of measures EF_M	Consequences of failure		
	Small $\rho < 2$	**Medium (moderate)** $2 < \rho < 5$	**Serious** $5 < \rho < 10$
Small: $EF_M < 0{,}5$	$\beta_0 = 3{,}1$ ($P_f \approx 10^{-3}$)	$\beta_0 = 3{,}3$ ($P_f \approx 5\ 10^{-4}$)	$\beta_0 = 3{,}7$ ($P_f \approx 10^{-4}$)
Medium: $0{,}5 \leq EF_M \leq 2{,}0$	$\beta_0 = 3{,}7$ ($P_f \approx 10^{-4}$)	$\beta_0 = 4{,}2$ ($P_f \approx 10^{-5}$)	$\beta_0 = 4{,}4$ ($P_f \approx 5\ 10^{-5}$)
Large: $EF_M > 2{,}0$	$\beta_0 = 4{,}2$ ($P_f \approx 10^{-5}$)	$\beta_0 = 4{,}4$ ($P_f \approx 5\ 10^{-5}$)	$\beta_0 = 4{,}7$ ($P_f \approx 10^{-6}$)

Table 9. Load adjustment factors (LM1) for road bridges [23]

Type of bridge structure		Span [m]	$\alpha_{Q1,act}$	$\alpha_{Q2,act}$	$\alpha_{qi,act}\alpha_{qr,act}$
Beams	Box	20 – 80	0,70	0,50	0,50
	Two webs	20 – 80			0,40
	More webs	15 – 35			
Slabs	Slabs	10 – 30			
Slab bridges and other bridge types		5,3 – 10	0,60	0,40	0,40
		< 5,3	0,50	0,40	0,40

Table 10. Comparison of reliability indices, reference periods, and partial factors for actions according to the considered standards, for analysis of new and existing road bridges for the ultimate limit state

Standard	Reliability index β		Reference period (years)		Partial safety factor for actions γ			
					Permanent γ_{G1} (γ_{G2})		Traffic γ_Q	
	New	Existing	New	Existing	New	Existing	New	Existing
CAN/CSA-S6-06 [17]	3.75	2.50	100	5	1.20 (1.50)	1.10 (1.25)	1.70	1.35
MBE [22]	3.50	2.50	75	5	1.25 (1.50)	1.25 (1.50)	1.75	1.35
NEN 8700 [18]	4.3[a]	3.3[b]	100	15	1.40	1.25[b]	1.50	1.25[b]
ONR 24008 [16]	4.3[a]	4.3[a]	100	–[c]	1.35	1.20	1.35	1.35
SIA 269 [23]	4.3[a]	4.3[a]	100	–[c]	1.35	1.20	1.50	1.50
Nachrechnungs Richtlinie [20]	4.3[a]	4.3[a]	100	–[c]	1.35	1.20[d]	1.50	1.50

[a] for the consequence class CC3 and the 50-year return period
[b] for bridges that are unfit for use
[c] not presented
[d] for structural concrete only; for structural steel and other actions γ_G = 1.35

about the subsequent analysis of concrete bridges aimed at estimating load-carrying capacity of existing structures, and B 82 [51] that explains traffic models for subsequent analysis of existing road bridges. A detailed explanation of all provisions of the guideline is also presented in [52].

The following partial factors were defined for checking ultimate limit states at the second and third levels: $\gamma_{G,set}$ = 1,0 for actions due to support displacements, γ_Q = 1,35 and ψ_0 = 0,8 for actions due to temperature change and $\gamma_{G,cs}$ = 1,0 for action due to concrete shrinkage. At that, the forces obtained by linear elastic calculation for temperature change and support displacement actions may be reduced to 40 % if a more accurate analysis is not conducted. In case of steel bridges, the partial factor for permanent actions is not reduced (γ_G = 1,35), while in case of composite bridges it amounts to γ_G = 1,35 for self- weight of steel components, and to γ_G = 1,20 for self-weight of the concrete slab.

Measurement results obtained by load testing under service loads are analysed using the subsequent third level analysis. These results are related to structural deformations at critical points and deformation measurements at selected parts of the structure. The measurements cover the real behaviour of the structure at service loads, while also providing instructions for a more realistic description of structural behaviour. However, this calculation can only be used for calibration of calculation models used but, due to complexity of calculation and significant consumption of work, it can only be used in special cases, and in consultation with competent authorities.

The subsequent calculation at the highest fourth level comprises research methods for checking adequate load-carrying capacity of the structure, i.e. special geometrical and materially nonlinear procedures. This verification of adequate load-carrying capacity can be conducted, if needed, by direct determination of the design probability of failure by means of probabilistic methods. It can be combined with the second and third levels, but it can be used only in special cases, and in consultation with competent authorities.

According to subsequent calculation results, bridges are classified into three classes, namely A, B, and C. Class A covers bridges for which it was established by the first level verification that the load-carrying capacity and serviceability requirements are fulfilled without limitations. Class B covers bridges for which for which no service-related limitations were obtained by higher level calculations, and class C covers bridges for which service-related limitations have been obtained by calculations conducted at all levels.

If a bridge belongs to class C, then appropriate traffic limiting measures have to be taken, such as those involving definition of the minimum allowable distance between heavy vehicles in the queue and during traffic jams, overpassing ban for heavy vehicles, weight limitations for heavy vehicles, speed limits and axle-load limitations.

4.4. Comparison of standards for assessment of existing bridges

Although most of the above mentioned standards are related to road bridges only, those specified in [16, 23] also include railway bridges. The analysis mostly concerns bridge superstructure, while only general instructions are given for the analysis of structural bearings, substructure, and foundations. The ultimate load-carrying capacity is relevant for assessing reliability of existing bridges, while the serviceability limit is checked in exceptional cases only. The reliability index β is directly mentioned only in [17, 18, 22]. It is assumed in all standards that the design resistance is equal to that used in the design of new bridges, except in American standard MBE [22] where the design resistance is reduced depending on the condition factor (inspection based assessment) and structural system factor (redundancy), but in total no more than 15%. The comparison of target reliability indices β, reference period, and partial factors for actions in accordance with the considered standards for the analysis of new and existing road bridges for the ultimate limit state, is given in Table 10.

It can be concluded that, although the same reliability methods are used for calibration, the correlation between the target reliability index β, and selected partial factors for actions γ for the analysis of existing bridges actually depends on the standard used in a particular country, because it is based on the standard that is used in that country for the analysis of new bridges. North American and European standards for the analysis of new bridges greatly differ from one another. In European standards [16, 20, 23], only the partial factors for permanent actions are reduced during the second level analyses (only if dimensions are measured on the structure), while only partial factors for traffic load are reduced in the American standard [22]. In addition, in North American standards the existing bridges are assessed only for permanent actions and dominant variable action (traffic load), while in European standards all other variable actions are also considered. The SIA 269 and ONR 24008 are the only available standards for the analysis of existing railway bridges. The rules applied are generally similar to those used for the analysis of road bridges but, evidently, a greater attention is paid to fatigue or, alternatively, to the assessment of the remaining service life. It is interesting to note that special provisions for the analysis of existing bridges were already given in the German standard for railway bridges prepared in 1925 [53].

Table 11. Partial factors for actions for the second-level analysis of road bridges

Partial factors for road bridges			Notes
Permanent actions γ_G	Self-weight of the structure	1.20[a]	Load should be determined based on real dimensions measured on the structure. If the analysis is not based on real dimensions measured on the structure, then partial factors should be used in accordance with HRN EN 1990 [1].
	Weight of equipment (wearing surface, kerbs, sidewalks, etc.)	1.20[a]	Load should be determined based on real dimensions measured on the structure.
Variable actions	Traffic load excepting the specified heavy cargo transport with a detailed axle-load distribution, one action $\gamma_{Q,1}$	1.35	
	Specified heavy cargo transport with a detailed axle-load distribution	1.20	
	Several simultaneous actions $\gamma_{Q,i}$	–[b]	

[a] for favourable effect: 1,0
[b] analysis according to HRN EN 1990 [1], taking into account the factor ψ.

Table 12. Partial factors for actions as used during the second-level analysis for railway bridges

Partial factors for railway bridges			Notes
Permanent actions γ_G	They act permanently	1.20[a]	Load should be determined based on real dimensions measured on the structure.
	They do not act permanently, parts that initially do not belong to the structure (e.g. ballast)	1.30[a]	The analysis is based on the real measured ballast depth and provisions must be made not to increase this depth.
Traffic actions due to traffic load by rail vehicles $\gamma_{Qi} = \gamma_{Q1}$	Load models SW, real locomotives and wagons (weighed)	1.20	
	Load models for classification of railways (real trains), other vehicles and wagons	1.45	
Variable additional actions γ_{Qi}	Actions due to traction, braking, wind action, temperature action, side impact, and other variable actions	1.10	
Variable additional action as dominant action γ_{Q1}		1.30	

[a] for favourable effect: 1,0

5. Proposal of guidelines for preparation of Croatian standard for assessing reliability of existing bridges

Croatia currently does not have a standard for assessing reliability of existing bridges and so, in practical situations, engineers most often use current standards for new bridges for assessment of existing bridges. This leads to unnecessary and expensive repair and rehabilitation works, and so a new standard is proposed to improve the situation.

All of the above presented standards for assessing reliability of existing bridges rely on assessment procedure consisting of several levels, and so a similar multi-level procedure is also proposed for Croatia. The proposed guidelines for the creation of a new Croatian standard are mostly based on the corresponding Austrian standard [16]. This is the only standard that covers the existing road and railway bridges, while the analysis for new bridges is conducted in accordance with Eurocodes.

The initial first level analysis should be conducted using standard procedures based on the standards that are currently applied for the analysis of new bridges.

The second level analysis allows the use of reduced partial factors for permanent actions as based on detailed analysis of real dimensions (Tables 11 and 12). Traffic load should be adopted in accordance with the applicable standard HRN EN 1991-2 [54] as there are no reliable data about the real traffic. Resistance should be based on updated measured properties of materials, and all possible damage and other structural deficiencies should be taken into consideration. Advanced methods in accordance with *fib* 2010 [55] may be used in the analysis of shear strength, which is often relevant in case of existing bridges.

The use of the first-order reliability method, compliant with HRN EN 1990, Appendix C [1], is proposed for the third-level analysis. It is considered that the verification requirements have been met, if the design reliability index β for the structure under consideration is equal to or greater than the required smallest value of β in accordance with HRN EN 1990. The relevant limit state is determined in the previous second-level analysis. This is followed by modelling of this limit state taking into account variable (dispersed) actions and resistance values, and the model uncertainty. Statistical distribution parameters (mean value, standard deviation and/or coefficient of variation) should generally be calculated according to JCSS Model Code [5]. In order to check adequacy of results, it is necessary to perform, in addition to comparison with second-level results, a subsequent assessment that comprises the analysis of sensitivity and parametric studies. The analysis of sensitivity shows which random variables have the greatest influence on reliability. The parametric studies show the effect of the change in mean value of a random variable on the reliability of the structure, i.e. reduction of mean strength of concrete due to ageing of material.

If the results of the first-, second-, and third-level analyses are not satisfactory, the reliability level required in HRN EN 1990 [1] may be reduced in some exceptional cases through the fourth-level probabilistic analysis. Such special cases include bridge structures that have been proven beyond doubt to possess a considerable reserve in load-carrying capacity, and also the structures that have demonstrated an impeccable behaviour, and where a failure with warning can reasonably be expected. For this analysis, a detailed explanation must be provided, and other appropriate measures, such as permanent supervision or frequent bridge inspections, must be conducted.

6. Conclusion

Research efforts relating to reliability of bridges have recently been increasingly oriented toward the existing bridges. Assessment of an existing bridge has to be conducted because of determined damage, for retrofitting or strengthening of the bridge load-carrying structure, to enable transport by oversize vehicles, to meet demands for introducing higher railway categories (railway bridge categorisation), because of determined structural deficiencies, after emergency events, or after new information is obtained about the load-carrying capacity.

The assessment procedure for these bridges is much more complex than the analysis for new bridges as the existing load-carrying capacity reserves relating to actions, resistance, and safety concept, have to be activated for economic reasons, so that it can be determined by calculation whether such bridges are still fit for use. Reserves relating to actions comprise analysis of real vehicles and application of an appropriate dynamic factor instead of standardised vehicles, and compensation measures that are applied if it is determined through calculations that the use of the bridge must be restricted. Such limitations include definition of driving lanes (traffic only along the middle of the bridge), introducing weight restrictions for heavy vehicles, limiting vehicle speed and axle loads, and specifying the smallest allowable distance between heavy vehicles in case of vehicle queues and traffic jams. The reserves relating to resistance include determination of real properties of materials used in the bridge structure, and the use of advanced more realistic (more accurate) elasto-plastic computation models instead of the usual elastic computation models. The paper also provides a review of latest research in which advanced procedures are presented for inclusion of characteristic tests for materials and loads, for introduction of realistic traffic loads, and for the structural system behaviour, with the possibility of redistribution in the probabilistic frame for the assessment of existing bridges, based on probability and reduction of target reliability levels. The assessment procedure is conducted at several levels. If the bridge fulfils requirements for the initial assessment level, then further verifications are deemed unnecessary.

An overview of some significant characteristics of representative foreign standards for assessing reliability of existing bridges is also given. The presented concepts are the most consistently implemented in the Canadian standard [17] that specifies a smaller reliability index β, which is based on the reduced remaining service life, actual behaviour of structural elements and structural system, level of inspections, and risk category. In the Dutch standard [18], modified partial factors of safety are defined for subsequent analysis of existing bridges using the same procedure for stochastic values on the side of actions and on the side of resistance, based on the deduced adjusted reliability index β. In the standards used in countries we traditionally lean onto [16, 20, 23], the assessment procedure is conducted according to the basic principle of keeping the reliability level equal to that used in the analysis of new bridges, by removing uncertainties in partial factors through detailed measurements and analysis of properties of materials used in the bridge, and through determination of real traffic operated on the bridge. The initial first level assessment is conducted using standard semi-probabilistic procedures according to standards applied for new bridges. The basic information for higher assessment levels includes upgrading of cross section and properties of materials incorporated into the bridge structure, based on detailed inspections and adjustment of traffic load based on appropriate traffic measurements. Thus, the bridges that do not meet initial verification requirements are checked according to a semi-probabilistic second-level procedure, involving the use of reduced partial factors based on such data. Load-carrying capacity requirements are most often satisfied through implementation of this procedure. Higher level assessments are probabilistic and the results greatly depend on the quality of statistical data for action distribution and resistance, and also on the complexity of the limit state under considerationstudy. That is why a detailed sensitivity analysis and comparison with previous results has to be made. The use of such advanced procedures has revealed that in many cases a bridge that does not meet usual safety verifications can in fact safely carry applied loads, without the need for its strengthening or replacement.

As such a standard does not exist in Croatia, guidelines are proposed in the paper for further consideration of this issue, highly significant from both economic and social standpoints. The proposed standard is mostly based on the relevant Austrian standard [16], and involves assessment of existing bridges at several levels while retaining the reliability level equal to that used during analysis of new bridges. This reliability level can be reduced only exceptionally at the highest fourth level, when appropriate alternative measures must be applied.

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