Partial factors for assessment of existing reinforced concrete bridges

Milan Holicky, Jana Markova and Miroslav Sykora
Klokner Institute, Czech Technical University in Prague, Czech Republic

Abstract: Simplified conservative procedures for the reliability assessment of existing bridges based on the methods applied for design of new structures may lead to expensive repairs. The submitted paper is aimed at the development of methods for the reliability assessment of existing reinforced concrete bridges, taking into account principles of new European standards Eurocodes and international documents ISO. Considering actual conditions of existing bridges, the partial safety factors given in Eurocodes for structural design are modified using probabilistic methods. The outlined procedures are applied in the assessment of a reinforced concrete bridge. It appears that the partial factors may be reduced considering a target reliability level specified for actual conditions of existing bridges.

1 Introduction

Rehabilitation of numerous existing reinforced concrete road bridges is presently an urgent issue of bridge engineers and responsible authorities in the Czech Republic. Decisions concerning existing bridges should be based on the reliability assessment, taking into account deterioration aspects, actual resistances and loadings. It has been recognised that simplified conservative procedures based on the design methods applied for new structures may lead to expensive repairs. It is well-known that contemporary prescriptive requirements on new structures are often more severe than the provisions of original national codes. Nevertheless, existing bridges that might not fulfil these requirements, may still serve their purpose for a specified working life.

In accordance with ISO 13822 [14], probabilistic methods may effectively be applied in the assessment of existing structures. General principles for estimation of failure probability of deteriorating structures are provided in the informative Annex E of this document. The submitted paper is aimed at the development of methods for the reliability assessment of existing reinforced concrete bridges considering principles of the new

European standards (Eurocodes) EN 1990 [3] and EN 1991-2 [6] as well as of the international documents ISO 13822 [14] and ISO 2394 [15]. Target reliability levels for reinforced concrete bridges are modified on the basis of empirical relationships proposed by ALLEN [2] and SCHUEREMANS and VAN GEMERT [21], taking into account economic and societal consequences of failure. The partial safety factors are then derived using probabilistic methods, considering actual conditions of bridges including deterioration due to unfavourable environmental effects and fluctuations of a traffic load.

The outlined procedures are applied in the assessment of an existing reinforced concrete road bridge. Partial factors are estimated on the basis of new information about the bridge conditions and requirements for a remaining working life.

2 Time-variant failure probability

It is herein assumed that:

- Resistance of a bridge can be described by a monotonically decreasing function $\mathrm{R}[R_0, \mathrm{g}(t)]$ where R_0 is the random initial resistance and $\mathrm{g}(t)$ is the degradation function,
- Occurrence of a time-variant (traffic) load $Q(t)$ can be approximated by a rectangular wave renewal process with the mean renewal rate λ,
- Load intensities Q_i are identically distributed independent variables.

The simplified models for the time-variant resistance and traffic load are indicated in Fig. 1.

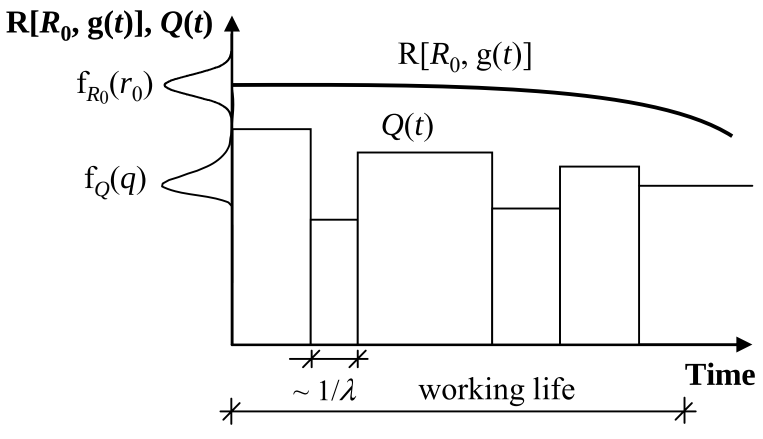

Fig. 1: Decreasing resistance and traffic load within a working life

The instantaneous failure probability can be defined as follows:

$$p_\mathrm{f}(t) = \mathrm{P}\{\mathrm{Z}[\mathbf{X}(t)] < 0\} \approx C_\mathrm{SORM}\Phi[-\beta(t)] \qquad (1)$$

with $Z(\cdot)$ limit state function
$\mathbf{X}(t)$ basic variables
t point in time
C_{SORM} curvature correction factor
$\Phi(\cdot)$ cumulative distribution function of the standardised normal variable
$\beta(t)$ FORM reliability index.

In accordance with RACKWITZ [19], an upper bound on the failure probability, related to the reference period $(0,t_d\rangle$, can be obtained for the considered case as follows:

$$p_f(0,t_d) \leq \lambda C_{SORM}\Phi[-\beta(t_d)]\left\{\frac{1-\exp[f'_\beta(t_d)]t_d}{|f'_\beta(t_d)|}\right\} \leq \lambda C_{SORM}\Phi[-\beta(t_d)]C_T \tag{2}$$

with $f_\beta'(t)$ time derivative of the function $f_\beta(t) = \ln\{\Phi[-\beta(t)]\}$
C_T time correction factor.

More details are provided by RACKWITZ [19].

3 Deterioration models

Road bridges are gradually deteriorating due to various adverse factors that may significantly influence their performance and safety. The main factors include chemical attacks (chlorides, atmospheric CO_2, pollutants SO_2 and NO_x), physical effects (frost, scouring), overloaded trucks, natural disasters (floods, extreme winds) and vandalism. It has been recognised e.g. by VU and STEWART [25] that deterioration of reinforced concrete bridges occurs mainly due to chloride contamination. The other factors also contribute, but to a lower extent. Except for coastal areas, aggressive chloride environment is primarily caused by use of de-icing salts. Chlorides may diffuse through a protective concrete cover or corrosion may be initiated by cracking. In this section, deterioration models proposed by VU and STEWART [25] and by ENRIGHT and FRANGOPOL [7] are briefly described.

3.1 Model assumed by VU and STEWART [22]

In this model, penetration of chlorides is described by Fick's second law of diffusion. The chloride content $C(x,t)$ at a distance x from the concrete surface at a point in time t is:

$$C(x,t) = C_0\{1 - \mathrm{erf}[x/(2\sqrt{tD})]\} \tag{3}$$

with C_0 surface chloride content
D diffusion coefficient
erf error function.

The model for the diffusion coefficient developed by PAPADAKIS et al. [18] is applied, taking into account variations of the aggregate-to-cement and water-to-cement ratios and mass densities of cement and aggregates. Model uncertainties are described by the coefficient θ_D.

The chloride concentration must reach a critical threshold chloride concentration C_r at the depth c (concrete cover), to initiate corrosion of reinforcement. Time to initiation of corrosion due to chlorides is then determined from equation (3).

Corrosion may also be initiated by wide cracks caused particularly by bending, that may permit greater migration of moisture, oxygen, and carbon dioxide through the concrete cover. The maximum crack width is approximately considered as:

$$w_{max}(t) = [1 + k(t)]w_{max} \tag{4}$$

with $k(t)$ factor incorporating the time-dependent growth of a crack width due to duration of permanent actions
w_{max} maximum crack width.

A negative exponential relationship is used for the factor $k(t)$. Time to initiation follows from the condition $w_{max}(t) = w_{lim}$ where w_{lim} is the critical crack width.

At time τ since the corrosion initiation (in years), the corrosion rate is:

$$i_{corr}(\tau) = \theta_{icorr} 0.85 i_{corr0} \tau^{-0.29} \tag{5}$$

with θ_{icorr} model uncertainties of the corrosion rate.

The initial corrosion rate i_{corr0} (in $\mu A/cm^2$) is given by:

$$i_{corr0} = [37.8(1- w/c)^{-1.64}] / c \tag{6}$$

with c concrete cover (in cm)
w/c water-to-cement ratio, obtained as $27 / [f_c + 13.5]$ where f_c is the concrete compressive strength in MPa.

Due to corrosion, the diameter of reinforcement bars $d(t)$ is reduced at an arbitrary point in time t as follows:

$$d(t) = \begin{cases} d_0 & \ldots\ t \le T_I \\ \max[d_0 - 2 \times 0.0116 \times 1.2\ i_{corr0}(t - T_I)^{0.71}; 0] & \ldots\ t > T_I \end{cases} \tag{7}$$

with d_0 initial diameter of bars
T_I time to initiation, taken as the minimum of the times to initiation due to chlorides and cracking.

Probabilistic models of the basic variables are indicated in Tab. 1.

It is assumed that highly localised pitting is spatially distributed and few bars only will be affected by the pitting. Therefore, the localised pitting may not significantly influence structural resistance and general corrosion described by relationship (7) is considered.

Tab. 1: Probabilistic models for basic variables of the deterioration models.

Model	Symbol	Variable	Unit	Distr.	Mean	CoV
VU and STEWART [25]	C_0	Surface chloride content	kg/m^3	LN0	3.0	0.5
	D	Diffusion coefficient	cm^2/s	N	2e-8	0.45
	θ_D	Model unc. diffusion coeff.	-	N	1.0	0.2
	C_r	Critical threshold chloride conc.	kg/m^3	U	0.6^*	1.2^*
	w_{max}	Maximum crack width	mm	N	0.2^{**}	0.4
	w_{lim}	Critical crack width	mm	U	0.3^*	0.6^*
	θ_{icorr}	Model unc. of corrosion rate	-	N	1.0	0.2
ENRIGHT and FRANGOPOL [7]	T_I	Time to initiation	year	LN0	5	0.25
	k_1	Degradation constant	-	LN0	5e-3	0.1

N – normal distribution, LN0 – lognormal distribution with the lower bound at the origin, U – uniform distribution; * lower/upper bound; ** derived assuming that design satisfies the condition $P(w_{max} < w_{lim} = 0.3\text{ mm}) = 0.9$

3.2 Model assumed by ENRIGHT and FRANGOPOL [7]

In this model, medium and high moment resistance degradation are distinguished. For the medium case considered hereafter, simplified degradation function is written as follows:

$$g(\tau) = \begin{cases} 1 & \ldots\ t \leq T_I \\ 1 - k_1(t - T_I) & \ldots\ t > T_I \end{cases} \tag{8}$$

with k_1 random variable.

Probabilistic models for the time of corrosion initiation T_I and degradation constant k_1 given in Tab. 1 follow from parametric studies of reinforced concrete beams subjected to uniform corrosion initiated by chlorides described by ENRIGHT and FRANGOPOL [8].

3.3 Remarks on the deterioration models

It is important to note that in accordance with ALLAM et al. [1], the mechanical properties of steel and concrete are assumed to be unaffected by the corrosion in the deterioration models. It has been indicated by RAFIQ et al. [20] that significant uncertainties are related to models for reinforced concrete deterioration. Studies available in literature provide scattered data for description of chloride penetration and reinforcement corrosion. For example, in the study by FABER and ROSTAM [9] the models for diffusion coefficient, surface chloride concentration and critical threshold chloride concentration are considerably different from those considered by VU and STEWART [25]. Based on investigation of concrete bridges in Denmark, a different probabilistic model for the critical threshold chloride concentration has been proposed by HENRIKSEN and STOLTZNER

[12]. Apparently, experimental data are needed for development of a realistic deterioration model.

It is also emphasized that the model assumed by ENRIGHT and FRANGOPOL [7] has been developed from limited data and should be used with caution as noted by FABER et al. [10]. It is indicated that the medium deterioration case by ENRIGHT and FRANGOPOL [7] may actually describe severe deterioration.

4 Models for load effects

4.1 Traffic load

In the present study it is assumed that the most unfavourable effect is caused by passage of heavy vehicles on the bridge as explained later in the numerical example. The traffic load effect basically consists of a static and dynamic component. In general, this effect depends on many parameters including a span length, vehicle weight, axle loads, axle configuration, position of a vehicle on the bridge (transverse and longitudinal), number of vehicles on the bridge (multiple presence), and stiffness of structural members.

Traffic data collected in European countries have been statistically analysed within the development of EN 1991-2 [6]. Data including extreme loads corresponding to different return periods are provided by FLINT and JACOB [11]. The extreme values are given for lorries with different numbers of axles. It follows that annual extreme of the static traffic load Q due to a passage of the heavy vehicle may be approximated by the Gumbel distribution with the low coefficient of variation of about 0.03.

The dynamic component of the load is caused mainly by the vibrations of the vehicle induced by the irregularities of the pavement. The dynamic amplification φ with the mean 1.10 and coefficient of variation 0.10 is accepted in the present study.

Recently, an attempt has been made by VU and STEWART [25] to predict a future traffic development. It appears that the prediction of future trends of configuration of axles and vehicle weights includes a considerable degree of uncertainty. Therefore, an auxiliary variable θ_Q is introduced to describe uncertainties in the traffic load effect. In accordance with VON SCHOLTEN et al. [24], the mean and coefficient of variation are taken as 1 and 0.15, respectively. Probabilistic models for the traffic load effect are given in Tab. 2.

4.2 Permanent Actions

In addition to the traffic load, road bridges are exposed to permanent actions due to the self weight of structural and non-structural elements permanently connected to the bridge, including waterproofing, surfacing and other coatings. In the present study, the effect of the permanent actions G is approximately described by the normal distribution with the mean equal to the nominal value G_{nom} (considered in design) and coefficient of variation 0.1 as indicated in Tab. 2.

Tab. 2: Probabilistic models for the load effects.

Symbol	Variable	Unit	Distr.	Char. value	Partial factor	Mean	CoV
Q	Static traffic load (annual extreme)	kN	Gum	900*	1.35	715	0.03
φ	Dynamic amplification	-	LN0	-	-	1.15	0.1
θ_Q	Model uncertainty of the traffic load effect	-	LN0	-	-	1.0	0.15
G	Permanent action	-	N	G_{nom}	1.35	G_{nom}	0.1

Gum – Gumbel distribution (maximum values); * including dynamic effects

5 Partial factors

Major advantage of the probabilistic assessment is that a resulting failure probability (or alternatively reliability index, see EN 1990 [3]) can be directly compared with a target reliability level. However, in civil engineering practice the partial factor method or other methods (safety factor method, allowable stresses) are more often used for the assessment of existing bridges. In these methods a reliability level is not directly estimated and, thus, may differ for bridges made of different materials and exposed to different actions.

In the following the partial factor method introduced in the Eurocodes is discussed only. The assessment is based on design values of basic variables derived from characteristic values and partial factors. Partial factors of the action with an unfavourable effect and of resistance with a favourable effect on structural reliability are defined as follows, respectively:

$$\gamma_{Xi} = x_{id} / x_{ik};\ \gamma_{Xi} = x_{ik} / x_{id} \tag{9}$$

The subscript "k" denotes a characteristic value and the subscript "d" refers to a design value. To achieve a target reliability level, the partial factors for the time-variant case may be derived from the design values obtained by:

$$x_{id} = \mathrm{F}_i^{-1}\{\Phi[-\alpha_i(t_d)\beta_{red}]\} \tag{10}$$

with $\mathrm{F}^{-1}(\cdot)$ inverse cumulative distribution function
$\alpha(t_d)$ FORM sensitivity factor
β_{red} reduced reliability index defined below.

It is assumed that the target reliability index β_t, related to a specified working life t_d, is known. In accordance with RACKWITZ [19], the reduced reliability index is derived from the target reliability index as follows:

$$\beta_{red} = -\Phi^{-1}[\Phi(-\beta_t) / (\lambda\, C_{SORM}\, C_T)] \quad (11)$$

6 Numerical example

6.1 Model of the bridge

Reliability of a simply supported reinforced concrete slab bridge is further analysed. The 30-year old road bridge is exposed to a repeated application of de-icing salts. The deck slab is 15.6 m wide and 0.6 m thick (the averaged value), layers of the pavement are 0.1 m thick in total. Span length is 11 m. Probabilistic models for variables describing the initial resistance given in Tab. 3 are chosen taking into account data provided by JCSS [16]. The probabilistic model for the model uncertainty of resistance is based on experimental measurements of concrete beams reported by HOLICKY et al. [13].

Tab. 3: Probabilistic models for resistance variables.

Symbol	Variable	Unit	Distr.	Char. value	Partial factor	Mean	CoV
A_s	Reinforcement area	m^2/m	N	$A_{s,nom}$	-	$A_{s,nom}$	0.03
f_y	Yield strength of reinforcement	MPa	LN0	500	1.15	560	0.054
d	Diameter of reinforcement bars	mm	deterministic	25	-	25	-
h	Slab height	m	N	0.6	-	0.6	0.017
c	Concrete cover	mm	Gamma	50	-	60	0.17
α	factor of long-term load effects on the concrete compressive strength	-	deterministic	0.85	-	0.85	-
f_c	Concrete compressive strength	MPa	LN0	30	1.5	37.5	5
θ_R	Model uncertainty of resistance	-	N	-	-	1.08	0.1

Gum – Gumbel distribution (maximum values)

The bridge of a low clearance passes over traffic lanes. Therefore, the slab soffit is likely to be exposed to de-icing salt chlorides from car spray at similar concentrations as the slab deck, VU and STEWART [25]. All the material properties and all the deterioration parameters are assumed to be constant across the entire bridge and the basic variables are mutually statistically independent.

Effects of the load models included in EN 1991-2 [6] have been compared by the deterministic finite element analysis. It follows that a special vehicle (the Load Model 3) has a more unfavourable effect than the mixture of lorries and cars described by the Load Model 1. Therefore, the probabilistic model for the load effect due to passage of a heavy vehicle is considered as described above. Axle spacing and distribution of axle loads are assumed in accordance with EN 1991-2 [6].

6.2 Deterministic verification

It is verified by an inspection that bottom reinforcement consists of 8 bars of the diameter 25 mm and the reinforcement area is thus 3.93×10^{-3} m^2/m. Due to corrosion, the area is reduced to 3.73×10^{-3} m^2/m. To satisfy requirements of the Eurocodes, deterministic verification of the maximum bending moment is:

$$A_s \frac{f_{yk}}{\gamma_s}\left[h-\left(c+\frac{d}{2}\right)-\frac{A_s \frac{f_{yk}}{\gamma_s}}{2\alpha \frac{f_{ck}}{\gamma_c}}\right] \geq \gamma_G \mathrm{E}(G_k)+\gamma_Q \mathrm{E}(Q_k) \tag{12}$$

with γ_S partial factor for steel reinforcement
γ_c partial factor for compressive concrete strength
γ_G partial factor for permanent action
γ_Q partial factor for traffic load
$\mathrm{E}(G_k)$ effect of the characteristic (nominal) value of the permanent action
$\mathrm{E}[Q_k)]$ the most unfavourable effect due to passage of the heavy vehicle (characteristic load).

The most unfavourable effect due to passage of the heavy vehicle has been determined by the finite element analysis, considering the dispersal of wheel loads in accordance with EN 1991-2 [6].

The set of partial factors recommended for design of concrete bridges in EN 1992-2 [4], EN 1991-1-1 [5] and EN 1991-2 [6] is further used, $\gamma_S = 1.15$, $\gamma_c = 1.5$, $\gamma_G = 1.35$ and $\gamma_Q = 1.35$. Using the characteristic values of the basic variables given in Tab. 2 and 3 in equation (12), it follows that the minimum reinforcement area is 4.31×10^{-3} m^2/m. It is, therefore, concluded that reliability of the bridge is insufficient.

6.3 Probabilistic reliability analysis

The limit state function may be written as follows:

$$\mathrm{Z}(t)=A_s \mathrm{g}(t) f_y\left[h-\left(c+\frac{d}{2}\right)-\frac{A_s \mathrm{g}(t) f_y}{2\alpha f_c}\right]-\mathrm{E}(G)-\mathrm{E}(Q) \tag{13}$$

Using equation (7), the degradation function g(t) is derived for the model by VU and STEWART [25] as $[1 - d(t) / d_0]^2$. For the model by ENRIGHT and FRANGOPOL [7], the degradation function, given by equation (8), is applied on the whole resistance given in equation (13) by the term "$A_s f_y[\ldots]$".

For the models given in Tab. 3, the coefficient of variation of the initial resistance is about 0.12, which is in a good agreement with the resistance characteristics of reinforced concrete bridges published by NOWAK [17].

Failure probabilities are estimated using relationship (2) for the design in accordance with the Eurocodes (initial reinforcement area of 4.31×10^{-3} m^2/m). For convenience, the resulting reliability levels shown in Fig. 2 are provided in terms of the reliability index, see EN 1990 [3].

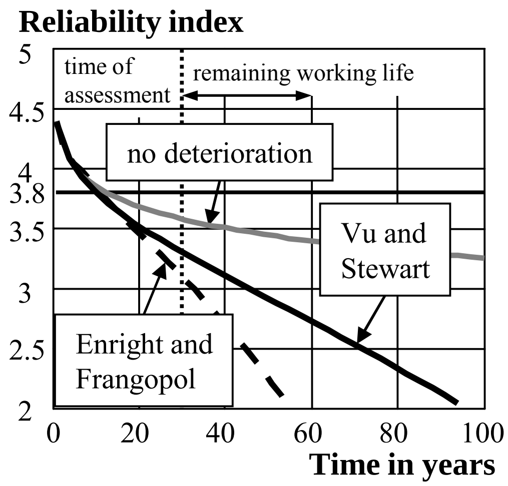

Fig. 2: Time-dependent reliability index for the bridge designed according to the Eurocodes

It follows from Fig. 2 that the design based on Eurocodes approximately yields the reliability index 3.3 for the case of no deterioration and the working life of 100 years. When considering the deterioration, the resulting reliability level is significantly decreased. The obtained reliability index is dependent on the assumed model of deterioration. It appears that influence of deterioration becomes important for time greater than 15 years. Very severe deterioration and low reliability levels are predicted particularly considering the model by ENRIGHT and FRANGOPOL [7].

6.4 Partial factors modified for different target reliability levels

Results of the deterministic verification indicate that rehabilitation of the bridge is necessary to comply with the requirements of the Eurocodes for new structures. An owner of the bridge may then decide to reduce a reliability level or remaining working life. It is thus assumed that the target reliability index is 3.8 for 30 years remaining working life. No rehabilitation within this period is planned.

The partial factors of the basic variables can be obtained from relationships (9) and (10), using the reduced reliability index 4.5 following from equation (11). To derive the set of the four partial factors used in the Eurocodes, the partial factor γ_s is then obtained from the individual partial factors of resistance model uncertainties, reinforcement area, yield strength and degradation model parameters. The partial factor γ_c is derived from the partial factors of height of slab, concrete cover and concrete strength. The partial factor γ_G includes the partial factors of the load effect model uncertainties and permanent action and the partial factor γ_Q is a product of the partial factors of the load effect model uncertainties, dynamic amplification and traffic load effect. The obtained partial factors and corresponding minimum reinforcement area (at the design stage, thus without influence of corrosion) are given in Tab. 4.

Tab. 4: Partial factors for different reliability levels.

Target reliability	Deterioration model	Measurements	γ_s	γ_c	γ_G	γ_Q	A_s in 10^{-3} m^2/m
3.8 (100 years)	No deterioration	no	1.34	1.75	1.11	1.38	4.70
3.8	VU and STEWART	no	1.54	1.64	1.11	1.30	5.15
3.8	ENRIGHT and FRANGOPOL	no	1.76	1.65	1.12	1.31	5.92
3.3	VU and STEWART	no	1.46	1.61	1.10	1.25	4.73
3.3	ENRIGHT and FRANGOPOL	no	1.67	1.59	1.11	1.26	5.44
3.3	VU and STEWART	yes	1.36	1.56	1.01	1.33	4.28
2.7	VU and STEWART	yes	1.29	1.51	1.01	1.27	3.91

It appears that the partial factors γ_s and γ_c, derived for the case of no deterioration, working life of 100 years and the target reliability index 3.8, are rather greater than the recommended values while γ_G is significantly lower. Considering the remaining working life of 30 years and target reliability index 3.8, the partial factor γ_s is considerably greater than the recommend value 1.15. This is consistent with findings of previous studies by VU and STEWART [25] and VAL et al. [23] where importance of yield strength and degradation aspects (included here in γ_s) on reliability has been indicated. The partial factors γ_c and γ_Q are in a partial agreement with the recommended values while the partial factor γ_G is lower than the recommended value. Note that the model uncertainties in resistance can be considered separately by the partial factor γ_{Rd} as indicated in EN 1990 [3]. The partial factor γ_s would then be significantly lower than those in Tab. 4.

Apparently, the requirement on reinforcement area is not satisfied. Therefore, it is decided to reduce the target reliability level. Modified target reliability levels for RC bridges are estimated on the basis of empirical relationships proposed by ALLEN [2] and SCHUEREMANS and VAN GEMERT [21], taking into account economic and societal consequences of failure. In accordance with ALLEN [2], the target reliability index may be reduced by 0.5 for structures with gradual failure with probable warning where likelihood of fatality/injury given the failure is 0.2. Thus, the target reliability index 3.3 is considered in the following.

It follows from Tab. 4 that particularly the partial factor γ_s is decreased. However, reinforcement area of the existing bridge is still insufficient. Therefore, the bridge is inspected to obtain new information on the basic variables. As the theoretical models for the basic variables given in Tabs. 1 to 3 are based on general knowledge, intended to cover "common" reinforced concrete bridges, it is expected that the measurements shall make it possible to reduce variability of variables describing the considered bridge.

It has been recognised in earlier studies by VU and STEWART [25] and VAL et al. [23] that particularly variability of the basic variables describing the deterioration process and concrete cover influence the structural reliability. Therefore, measurements at the time of assessment are focused on these variables. In addition data on the actual permanent action are collected. It is known from previous regular inspections that time to initiation of corrosion is five years. It is observed that reduction of the reinforcement area is about 5 %. It can be shown that the corresponding corrosion rate is about 2.3 $\mu A/cm^2$. To include a measurement error, coefficient of variation of this corrosion rate is considered as 0.05. It is confirmed that the permanent action corresponds to the assumptions made at the design stage. The mean is thus taken as the nominal value and the coefficient of variation is reduced to 0.03. Statistical evaluation of the measurements of concrete cover indicates that mean is 0.05 m and coefficient of variation is 0.06.

For the model by VU and STEWART [25], the required reinforcement area decreases to 4.28×10^{-3} m^2/m as indicated in Tab. 4. In this case, the design according to Eurocodes yields a sufficient reliability. However, the required reliability level is still not achieved for the existing bridge. Therefore, it is decided to modify the target reliability level using the model proposed by SCHUEREMANS and VAN GEMERT [21]:

$$\beta_t = -\Phi^{-1}(SC\ t_D\ AC\ CF\ /\ (np\ W) \times 10^{-4}) \approx 2.7 \tag{14}$$

with SC social criterion factor (recommended value for bridges 0.5)
AC activity factor (recommended value for bridges 3)
CF economical factor (1 for serious consequences of failure)
np number of endangered persons (here taken as 15)
W warning factor (0.1 for gradual failure with likely warning).

Considering the target reliability index 2.7, the required reinforcement area is 3.91×10^{-3} m^2/m. In this case the existing bridge complies with requirements on the target reliability level. The derived partial factors for the actions are reduced as compared with the recommended values while the partial factor γ_c is equal to and γ_s is rather greater than the recommended values.

It is noted that the bridge should be re-assessed each about five years as after this period traffic conditions and resistance of a deteriorating structure are likely to have changed beyond accuracy of the applied models, VU and STEWART [25]. In particular it is difficult to predict with any degree of confidence long-term deterioration processes. Hence, the predicted reliability level may only be relatively accurate for shorter time periods since previous assessment or updating.

It should be noted that influence of inspections and possible upgrades of the bridge are not included in the presented model. It is foreseen that the partial factors could be lower when regular inspections and upgrades are guaranteed. The predicted reliability levels and derived partial factors are rather conservative as they are based on an upper bound on the failure probability given by equation (2). In addition, the fact that the bridge has survived 30 years could be taken into account.

7 Conclusions and outline of the future research

The following conclusions may be drawn from the reliability analysis of the deteriorating reinforced concrete road bridge:

- The obtained reliability indices are considerably decreased when the deterioration is taken into account.
- The resulting reliability level is dependent on the assumed model for deterioration due to chloride contamination.
- For existing reinforced concrete road bridges, target reliability levels may be different than those for new bridges.
- The target reliability levels may be modified taking into account economic and societal consequences of failure as well as costs of upgrades.
- Partial factors for the assessment of existing bridges may be reduced considering a modified target reliability level and updated models of basic variables based on results of inspections.
- The partial factor for steel reinforcement, covering resistance uncertainties, yield strength variability and degradation effects, varies within the range from 1.3 to 1.7.
- The derived partial factors for concrete strength and traffic load correspond to the recommended values 1.5 and 1.35.
- The partial factor for permanent actions may be considered by the reduced values 1.0 – 1.1 when information on an actual permanent action on a bridge is available.

It is emphasized that the obtained numerical results are significantly dependent on the assumed probabilistic models and should be considered as informative only. To provide unambiguous recommendations for specification of the partial factors for existing reinforced concrete road bridges, the following aspects will further be taken into account:

- Shear failure criterion, improved deterioration models (bond cracking, delamination, etc.) and refined load models will be included,
- Reliability analyses will be conducted for different ratios of the permanent actions and traffic loads to consider different lengths of span,
- The target reliability levels will be estimated by cost optimisation, considering economic and societal consequences of failure.

Acknowledgements

This paper has been conducted in the Klokner Institute, CTU in Prague, in the framework of the research project 1F82C/072/910 Advanced methods for reliability assessment of existing road bridges, supported by the Ministry of Transport of the Czech Republic.

Literature

[1] Allam, I.M.; Maslehuddin, M.; Saricimen, H.; Al-Mana, A.I.: Influence of Atmospheric Corrosion on the Mechanical Properties of Reinforcing Steel. *Construction & Building Materials* V (1994), p. 35-41

[2] Allen, D.E.: Safety Criteria for the Evaluation of Existing Structures. In: *Proceedings IABSE Colloquium on Remaining Structural Capacity, Copenhagen, Denmark,* 1993

[3] CEN (European Committee for Standardization): *EN 1990 2002. Eurocode - Basis of structural design.* Brussels: CEN, 2002

[4] CEN: *EN 1992-2:2005 Eurocode 2 - Design of concrete structures - Part 2: Concrete bridges - Design and detailing rules.* Brussels: CEN, 2005

[5] CEN: *EN 1991-1-1. Eurocode 1: Actions on structures - Part 1-1: General actions; Densities, self-weight, imposed loads for buildings.* Brussels: CEN, 2003

[6] CEN: *EN 1991-2:2003 Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges.* Brussels: CEN, 2003

[7] Enright, M.P.; Frangopol, D.M.: Reliability-based condition assessment of deteriorating concrete bridges considering load redistribution. *Structural Safety* 21 (1999), p. 159-195

[8] Enright, M.P.; Frangopol, D.M.: Probabilistic analysis of resistance degradation of reinforced concrete bridge beams under corrosion. *Engrg Struct* 20 (1998), p. 960-971

[9] Faber, M.H.; Rostam, S.: Durability and Service life of Concrete Structures – the Owners Perspective. In: *Proceedings to the International IABSE Conference on Application of Risk Analysis in Civil Engineering, Malta,* 2001

[10] Faber, M.H.; Val, D.V.; Stewart, M.G.: Proof load testing for bridge assessment and upgrading. *Engineering Structures* 22 (2000), p. 1677-1689

[11] Flint, A.R.; Jacob, B.: Extreme traffic loads on road bridges and target values of their effects for code calibration. In: *IABSE Colloquium on Basis of Design and Actions on Structures, Delft, 27-29 March 1996.* The Netherlands, 1996

[12] Henriksen, C.F.; Stoltzner, E.: Kloridbetinget Korrosion. *Dansk Beton* (1992)

[13] Holicky, M.; Retief, J.V.; Dunaiski, P.E.: The reliability basis of design for structural resistance. In: *3rd Int Conf SEMC 2007,* Millpress, 2007, p. 1735-1740

[14] ISO (International Organization for Standardization): *ISO 13822. Bases for design of structures - Assessment of existing structures.* Geneve: ISO TC98/SC2, 2003

[15] ISO: *ISO 2394:1998, General principles on reliability for structures.* Geneve, 1998

[16] Joint Committee on Structural Safety: *JCSS Probabilistic Model Code.* 2001 <http://www.jcss.ethz.ch/>

[17] Nowak, A.S.: *Calibration of LRFD Bridge Design Code, NCHRP Report 368.* Transportation Research Board, 1999

[18] Papadakis, V.G.; Roumeliotis, A.P.; Fardis, M.N.; Vagenas, C.G.: Mathematical modelling of chloride effect on concrete durability and protection measures. In: Dhir, R. K.; Jones, M.R. (eds.): *Concrete repair, rehabilitation and protection,* London: E&FN Spon, 1996, p. 165-174

[19] Rackwitz, R.: A Concept for Deriving Partial Safety Factors for Time-variant Reliability. In: Guedes Soares, C. (ed.): *Proceedings of ESREL' 97.* Pergamon, 1997

[20] Rafiq, M.I.; Chryssanthopoulos, M.K.; Onoufriou, T.: Performance updating of concrete bridges using proactive health monitoring methods. *Reliability Engineering & System Safety* 86 (2004), p. 247-256

[21] Schueremans, L.; Van Gemert, D.: Assessing the safety of existing structures: reliability based assessment framework, examples and application. *Journal of Civil Engineering and Management* X (2004), p. 131-141

[22] Stewart, M.G.; Rosowsky, D.V.: Time-dependent reliability of deteriorating reinforced concrete bridge decks. *Structural Safety* 20 (1998), p. 91-109

[23] Val, D.; Bljuger, F.; Yankelevsky, D.: Reliability evaluation in nonlinear analysis of reinforced concrete structures. *Structural Safety* 19 (1997), p. 203-217

[24] von Scholten, C.; Enevoldsen, I.; Arnbjerg-Nielsen, T.; Randrup-Thomsen, S.; Sloth, M.; Engelund, S., et al.: *Reliability-Based Classification of the Load Carrying Capacity of Existing Bridges, Report 291.* Road Directorate, Ministry of Transport, Denmark, 2004, <htpp://www.vejdirektoratet.dk>

[25] Vu, K.A.T.; Stewart, M.G.: Structural reliability of concrete bridges including improved chloride-induced corrosion models. *Structural Safety* 22 (2000), p. 313-333