Verification of existing reinforced concrete structures using the design value method

M. Sykora & M. Holicky

1 INTRODUCTION

Existing structures are often affected by severe environmental influences that may yield deterioration and gradual loss of their durability and reliability. Hence upgrades of such structures including design of adequate construction interventions are becoming an important issue. Construction interventions may also become necessary in case of a change in use, concern about faulty building materials or construction methods, discovery of a design/construction error, structural damage following extreme events, complaints from users regarding serviceability etc.

Rehabilitation of these structures is a matter of a great economic significance as more than 50 % of all construction activities apply to existing structures, (Diamantidis, Bazzurro 2007). Decisions about various interventions should be always a part of the complex assessment of a structure, considering relevant input data including information on actual material properties.

(Sykora, Holicky 2012) indicated that the assessment of existing structures differs from structural design primarily in the following aspects:

- Increased costs of safety measures,
- Lower periods of the remaining working life,
- Different information on actual structural conditions (inspections, tests, measurements).

These aspects need to be adequately captured in the reliability assessment.

At present existing structures are mostly verified using simplified deterministic procedures based on the partial factor method. Commonly the partial factors recommended for the design of new structures are applied. However, such assessments are often conservative and may lead to expensive repairs.

More realistic verification of actual performance of existing structures can be achieved by the design value method in accordance with (EN 1990 2002) and (ISO 2394 1998). The submitted study intends to clarify applications of this method in verifications of existing reinforced concrete structures. A numerical example supplements general procedures and illustrates how the design values and partial factors can be derived for different target reliability levels and remaining working lives.

The study is based on working materials prepared for the *fib* Special Activity Group 7 "Assessment and Interventions upon Existing Structures", Working Group "Reliability and safety evaluation" and is intended to become a part of the *fib* bulletin. Comments of WG members are acknowledged.

2 GENERAL FORMULATION

Partial factors derived in this study are intended to be applied in conjunction with the load combination rules (6.10), or (6.10a,b) given in (EN 1990 2002). In case of structures without prestressing, the reliability verification format can be written as:

$$R_d \geq E_d = \Sigma_j \gamma_{G,j} G_{k,j} \text{ "+" } \gamma_{Q,1} Q_{k,1} \text{ "+" } \Sigma_i \gamma_{Q,i} \psi_{0,i} Q_{k,i}; \quad j \geq 1, i > 1 \tag{1}$$

or, alternatively, the less favourable of the two following expressions:

$$R_d \geq E_d = \Sigma_j \gamma_{G,j} G_{k,j} \text{ "+" } \Sigma_i \gamma_{Q,i} \psi_{0,i} Q_{k,i}; \quad j \geq 1, i \geq 1$$
$$R_d \geq E_d = \Sigma_j \xi_j \gamma_{G,j} G_{k,j} \text{ "+" } \gamma_{Q,1} Q_{k,1} \text{ "+" } \Sigma_i \gamma_{Q,i} \psi_{0,i} Q_{k,i}; \quad j \geq 1, i > 1 \tag{2}$$

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

The partial factors γ_X shall be derived from the actual distribution of the variable X (based on prior information, or results of tests or the combination of both). The characteristic values X_k are defined in EN 1990 (2002) and shall be based on actual material properties and actions. Their derivation is, however, not treated in this study. Values of the factors ξ and ψ_0 are to be accepted from (EN 1990 2002).

3 MATERIAL FACTOR γ_M

The design value f_d of the material property f can be defined by the relationship:

$$f_d = f_k / \gamma_M \tag{3}$$

The partial factor of a material property can be obtained as a product of:

$$\gamma_M = \gamma_{Rd}\,\gamma_m = \gamma_{Rd1}\,\gamma_{Rd2}\,\gamma_m \tag{4}$$

where γ_{Rd1} = partial factor accounting for model uncertainty; γ_{Rd2} = partial factor accounting for geometrical uncertainties; and γ_m = reliability-based partial factor accounting for variability of the material and statistical uncertainty.

3.1 *Model uncertainty factor* γ_{Rd}

γ_{Rd1} = 1.05 for concrete strength and γ_{Rd1} = 1.025 for reinforcement may be assumed in common cases, (fib SAG 9 2010). However, larger model uncertainty may need to be considered e.g. for punching shear in the case when concrete crushing is governing.

A value of γ_{Rd2} = 1.05 may be assumed for geometrical uncertainties of the concrete section size or reinforcement position, (fib SAG 9 2010). When relevant measurements on an existing structure indicate insignificant variability of geometrical properties, γ_{Rd2} = 1.0 may be considered.

Alternatively, the partial factor γ_{Rd} can be obtained from the following relationship based on a lognormal distribution:

$$\gamma_{Rd} = 1 / [(\mu_{\theta R} / \theta_{Rk}) \exp(-\alpha_R \beta\, \delta_{\theta R})] \tag{5}$$

Table 1. Statistical characteristics of model uncertainties for resistance and load effects (indicative values).

Category	Symbol	Model type	μ_θ	δ_θ
Resistance concrete members	$\theta_{R,M}$	Bending moment	1.1	0.1
	$\theta_{R,Vc}$	Diagonal compression in web	1.4	0.25
	$\theta_{R,Vs}$	Tensile force in web	1.0	0.05
	$\theta_{R,N}$	Axial compression	1.0	0.05
Load effects	$\theta_{E,M}$	Bending moment	1.0	0.1
	$\theta_{E,V}$	Shear forces	1.0	0.1
	$\theta_{E,N}$	Axial forces	1.0	0.05

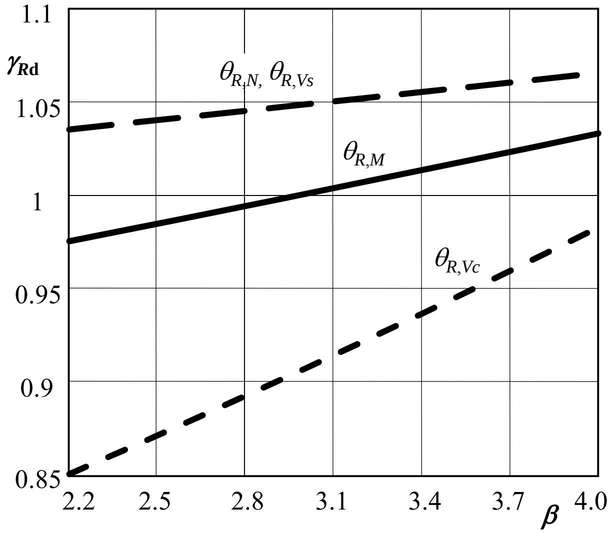

Figure 1. Variation of the partial factor γ_{Rd} with the target reliability β for $\alpha_R = 0.32$.

where $\mu_{\theta R}$ / θ_{Rk} = ratio of the mean to the characteristic value of the model uncertainty in resistance θ_R (bias); α = sensitivity factor in accordance with (EN 1990 2002); β = target reliability index; and δ = coefficient of variation.

Table 1 indicates statistical characteristics of resistance model uncertainties based on the background materials of *fib* SAG7, (JCSS 2001) and (Holický, Retief et al. 2007). The lognormal distribution is assumed for all the model uncertainties. Considering the models given in Table 1, variation of the partial factor γ_{Rd} with the target reliability β for $\alpha_R = 0.32$ is indicated in Figure 1.

The selection of $\alpha_R = 0.32$ deserves additional comments. Annex C of (EN 1990 2002) recommends α_R = 0.8 is for resistance variables. When γ_{Rd} and γ_m are assessed separately considering $\alpha_R = 0.8$, overly conservative designs may be obtained. For instance (Taerwe 1993) thus assumed $\alpha_R = 0.4 \times 0.8 = 0.32$ for "non-dominant" resistance variable.

In principle the factor γ_{Rd} should be applied to the resistance as follows:

$$R_d = R(f_{ck}/\gamma_c, f_{yk}/\gamma_s,\ldots) / \gamma_{Rd} \quad (6)$$

where R = resistance function; f_c = concrete compressive strength; and f_y = yield strength of reinforcement.

The following approximation, consistent with Equation 4, can be applied in common cases:

$$R_d \approx R[f_{ck}/(\gamma_{Rd} \times \gamma_c), f_{yk}/(\gamma_{Rd} \times \gamma_s),\ldots] \quad (7)$$

3.2 *Material factor* γ_m

Assuming a lognormal distribution of the material property and characteristic value defined as a 5% fractile, the partial factor γ_m is obtained as follows:

$$\gamma_m = f_k / f_d = \exp(\alpha_R\beta - 1.645)\delta_m \quad (8)$$

Variation of the partial factor γ_m with the coefficient of variation δ_m is shown in Figure 2 for $\alpha_R = 0.8$ and target reliabilities β = 2.3, 3.1, 3.8 or 4.3 (very low, low, medium and high failure consequences in ULS, respectively, (ISO 13822 2010)).

As an example it is considered that tests of material properties yield coefficients of variation $\delta_c = 0.15$ and $\delta_s = 0.05$. For estimation of the flexural resistance, the following partial factors are obtained using Equation 4 and Figures 1 and 2:

for $\beta = 3.8$:
$\gamma_C = 1.03 \times 1.23 = 1.27$; $\gamma_S = 1.03 \times 1.07 = 1.10$
for $\beta = 3.1$: (9)
$\gamma_S = 1.00 \times 1.13 = 1.13$; $\gamma_S = 1.00 \times 1.04 = 1.04$

Note that the partial factor $\gamma_C = 1.5$ provided in (EN 1992-1-1 2004) has been derived considering γ_{Rd1} = 1.05, $\gamma_{Rd2} = 1.05$, $\delta_c = 0.15$, $\beta = 3.8$ and the additional uncertainty due to the fact that in structural design, the concrete strength is assessed from samples not taken from a structure (expressed by an additional factor 1.1).

The partial factor $\gamma_S = 1.15$ provided in (EN 1992-1-1 2004) has been derived considering $\gamma_{Rd1} = 1.025$, $\gamma_{Rd2} = 1.05$, $\delta_s = 0.06$ and $\beta = 3.8$.

4 PERMANENT ACTION FACTOR γ_G

The design value G_d of the permanent action effect G is defined by the general relationship:

$$G_d = \gamma_G G_k = F_G^{-1}[\Phi(-\alpha_E \beta)] \tag{10}$$

where F^{-1} = inverse cumulative distribution function; and Φ = cumulative distribution function of a standardised normal variable.

The partial factor can be obtained as follows:

$$\gamma_G = \gamma_{Ed,g}\, \gamma_g \tag{11}$$

where $\gamma_{Ed,g}$ = partial factor accounting for the model uncertainty in estimation of the load effect using the load model; and γ_g = reliability-based partial factor accounting for variability of the permanent action, statistical uncertainty and uncertainties related to the model of permanent action.

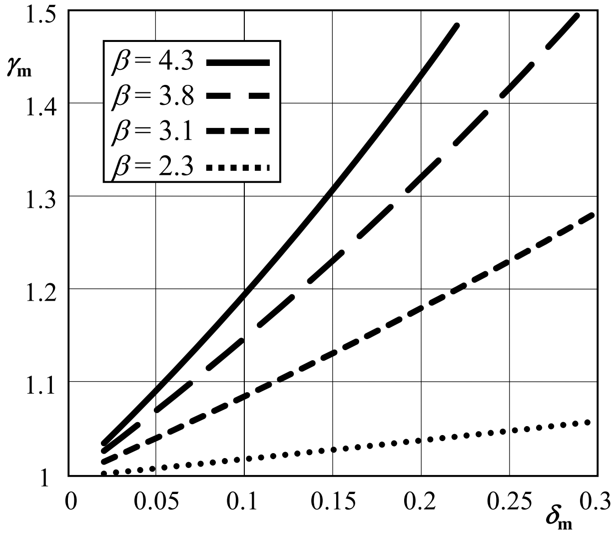

Figure 2. Variation of the partial factor γ_m with the coefficient of variation δ_m for $\alpha_R = 0.8$ and $\beta = 2.3$, 3.1, 3.8 or 4.3.

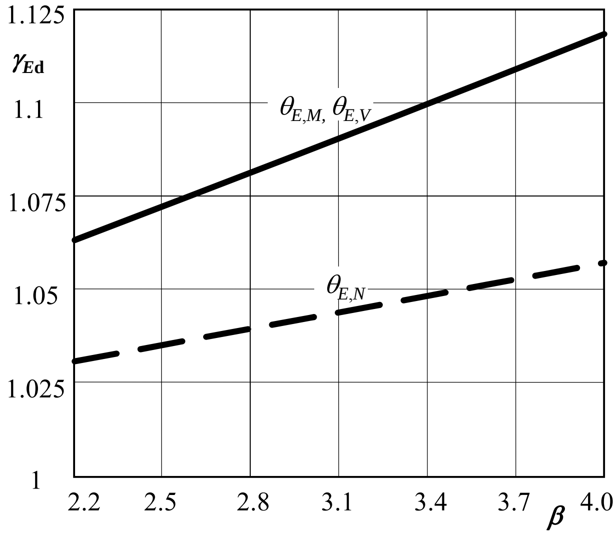

Figure 3. Variation of the partial factor γ_{Ed} with the target reliability β for $\alpha_E = -0.28$ (model uncertainty of an unfavourable action).

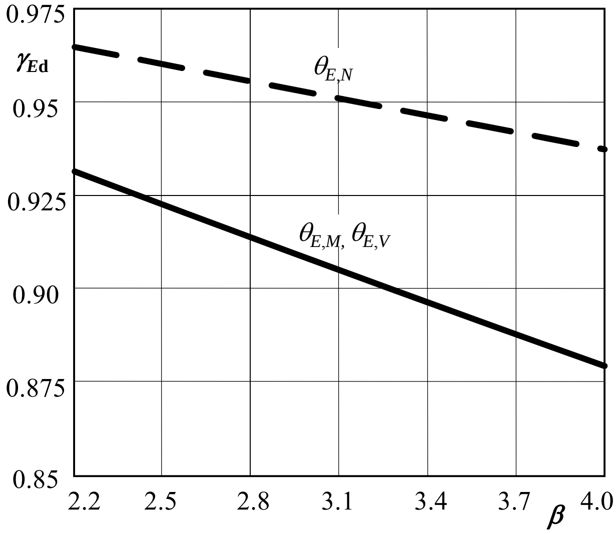

Figure 4. Variation of the partial factor γ_{Ed} with the target reliability β for $\alpha_{E,fav} = 0.32$ (model uncertainty of a favourable permanent action).

4.1 *Model uncertainty factor $\gamma_{Ed,g}$*

$\gamma_{Ed,g} = 1.05$ is normally assumed in structural design for an unfavourable action and $\gamma_{Ed,g} = 1.0$ for a favourable action, (fib SAG 9 2010).

Alternatively, the partial factor γ_{Ed} (in principle the same relationship holds for $\gamma_{Ed,g}$ and $\gamma_{Ed,q}$ denoting the partial factor accounting for model uncertainty in estimation of the variable load effect) can be obtained from the following relationship based on a lognormal distribution:

$$\gamma_{Ed} = (\mu_{\theta E} \,/\, \theta_{Ek}) \exp(-\alpha_E \beta \,\delta_{\theta E}) \tag{12}$$

where $\mu_{\theta E} / \theta_{Ek}$ = ratio of the mean to the characteristic value of the model uncertainty in load effect θ_E (bias) and $\delta_{\theta E}$ the coefficient of variation of θ_E.

Assuming probabilistic models given in Table 1, variation of the partial factor γ_{Ed} obtained from Equation 12 with the target reliability β is indicated in Figure 3 for $\alpha_E = -0.28$ ("non-dominant" variable of an unfavourable action) and in Figure 4 for $\alpha_{E,fav} = 0.32$ ("non-dominant" variable of a favourable permanent action). Note that favourable effects of variable actions are not considered in reliability verifications based on the partial factor method and thus $\gamma_{Ed,q}$ for favourable variable actions is not provided.

In principle the factor γ_{Ed} obtained from Equation 12 should be applied to the load effect as follows:

$$E_d = \gamma_{Ed}\, E(\gamma_g\, G_k,\ \gamma_q\, Q_k, \ldots) \tag{13}$$

However, the following approximation consistent with Equation 11 can be used in most cases:

$$E_d \approx E[(\gamma_{Ed} \times \gamma_g)G_k,\ (\gamma_{Ed} \times \gamma_q)Q_k, \ldots] \tag{14}$$

4.2 *Factor for the permanent action γ_g*

Assuming a normal distribution of the permanent action and characteristic value defined as the mean value, the partial factor γ_g is obtained as follows:

$$\gamma_g = G_d \,/\, G_k = 1 - \alpha_E \beta\, \delta_g \tag{15}$$

Variation of the partial factor γ_g with the coefficient of variation δ_g is shown in Figure 5 for $\alpha_E = -0.7$ (unfavourable action) and in Figure 6 for $\alpha_{E,fav} = 0.32$ (favourable action) and the target reliabilities $\beta = 2.3$, 3.1, 3.8 or 4.3. Note that $\alpha_{E,fav} = 0.8$ should be accepted when a favourable permanent action significantly contributes to a structural resistance, e.g. in case of stabilizing forces.

As an example it is considered that in-situ measurements yield $\delta_{g0} = 0.05$ for self-weight and $\delta_{g1} = 0.1$ for other permanent actions. For estimation of bending moments and shear forces, the following partial factors for unfavourable permanent actions are obtained using Equation 11 and Figures 3 and 5:

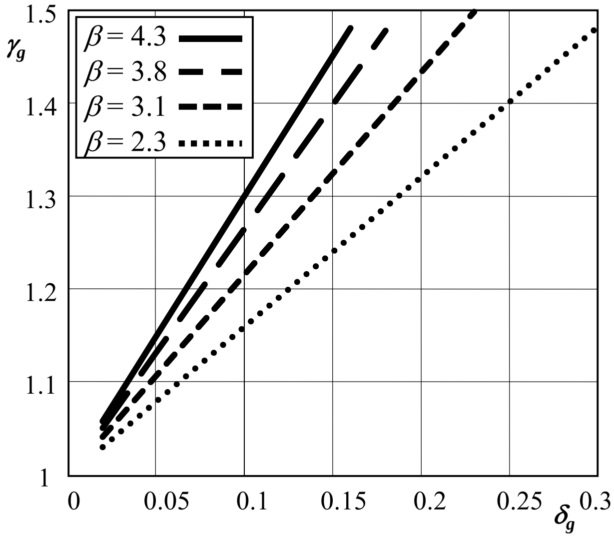

Figure 5. Variation of the partial factor γ_g with the coefficient of variation δ_g for β= 2.3, 3.1, 3.8 or 4.3, α_E = -0.7 (unfavourable action).

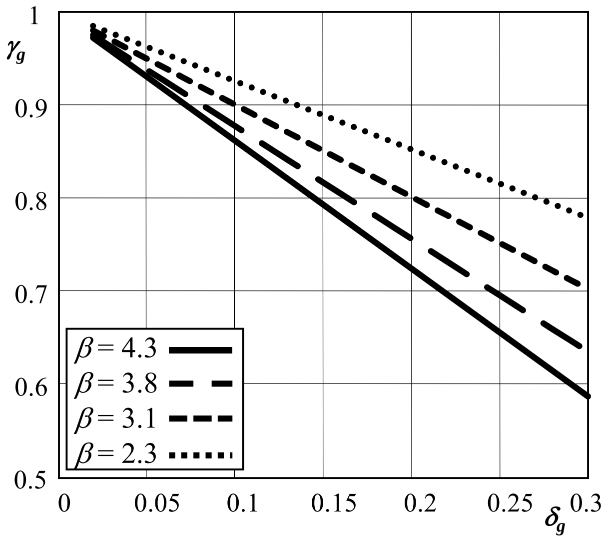

Figure 6. Variation of the partial factor γ_g with the coefficient of variation δ_g for β= 2.3, 3.1, 3.8 or 4.3, $\alpha_{E,\text{fav}}$ = 0.32 (favourable action).

for β= 3.8:
γ_{G0} = 1.11 × 1.13 = 1.25; γ_{G1} = 1.11 × 1.27 = 1.41
for β= 3.1: (16)
γ_{G0} = 1.09 × 1.11 = 1.21; γ_{G1} = 1.09 × 1.22 = 1.33

Note that the partial factor γ_G = 1.35 given in (EN 1990 2002) has been derived considering γ_{Ed} = 1.05, δ_g = 0.1 and β= 3.8.

5 VARIABLE ACTION FACTORS γ_Q

In many cases no additional information on variable loads, except that provided by valid codes for structural design, is available in the assessments of existing structures. The characteristic values and partial factors for variable loads should then be based on recommendations of such codes. However, if applicable partial factors should be adjusted to a specific situation of the existing structure with respect to assumed reliability levels and remaining working lives.

When site- or structure-specific data on variable loads can be gathered and a detailed assessment needs to be made, partial factors for variable loads may be derived using the procedure described in this section. The design value Q_d of the variable action effect Q is defined by the general relationship:

$$Q_d = \gamma_Q Q_k = F_{Qtref}^{-1}[\Phi(-\alpha_E \beta)] \tag{17}$$

where Q_{tref} = maxima of the load effect of a variable action related to a reference period for the reliability verification t_{ref}.

The partial factor γ_Q can be obtained as follows:

$$\gamma_Q = \gamma_{\mathrm{Ed},q}\ \gamma_q \quad (18)$$

where $\gamma_{\mathrm{Ed},q}$ = partial factor accounting for model uncertainty in estimation of the load effect from the load model; and γ_q = reliability-based partial factor accounting for variability of the variable action, statistical uncertainty and uncertainties related to the model of variable action.

5.1 *Model uncertainty factor* $\gamma_{\mathrm{Ed},q}$

$\gamma_{\mathrm{Ed},q} \approx 1.1$ is commonly assumed in structural design for an unfavourable variable action. Alternatively, the partial factor $\gamma_{\mathrm{Ed},q}$ can be obtained using the procedure described in section 4.1.

5.2 *Factors for the variable loads* γ_q

The partial factor for the variable load q related to the target reliability level β and reference period t_{ref} may be obtained as follows:

$$\gamma_q = \mathrm{F}_{q\mathrm{tref}}^{-1}[\Phi(-\alpha_E\beta),\ t_{\mathrm{ref}}]\ /\ q_{\mathrm{k}} \quad (19)$$

where q_{tref} = maxima of the variable load during the reference period t_{ref}. The distribution of the load maxima should be based on the same period t_{ref} as used for the reliability index β, (EN 1990 2002).

In general the remaining working life t_{r} may differ from the reference period t_{ref}. Nevertheless, for existing structures exposed to deterioration, it is mostly required to consider t_{ref} equal to t_{r}.

In common cases (EN 1990 2002) allows to approximate the sensitivity factor α_E by the value -0.7 for the leading variable action and by -0.28 for an accompanying variable action. However, available measurements may often lead to reduction of uncertainties related to resistance and permanent action effect when assessing existing structures. Then, the sensitivity factors for the resistance and permanent actions decrease and the absolute values of the sensitivity factor for the variable actions increase. However, this case-specific effect can only be treated by a full-probabilistic approach and thus is not further considered in this study.

In general the variable load depends on the time-variant component $q_0(t)$ and on the time-invariant component C_0 (including uncertainties related to an accepted load model). In most cases the maxima of the variable load related to t_{ref} can be obtained as a product of both components:

$$q_{\mathrm{tref}} = C_0 \times \max_{\mathrm{tref}}[q_0(t)] = C_0 \times q_{0,\mathrm{tref}} \quad (20)$$

Indicative probabilistic models for the time-invariant and time-variant components of selected variable loads are given in Table 2. The models are based on information provided in (JCSS 2001) and (Holický, Sýkora 2011). More detailed data (e.g. for imposed loads in common types of buildings) are provided by (JCSS 2001). The models in Table 2 should be considered as informative and should always be carefully revised taking into account actual loading, structural conditions, and experimental data relevant for a particular structure.

Assuming the Gumbel distribution of the time-variant component, the mean of $q_{0,\mathrm{tref}}$ is obtained as:

$$\mu_{q0,\mathrm{tref}} = \mu_{q0} + 0.78\sigma_{q0}\ \ln\,(t_{\mathrm{ref}}\ /\ t_0) \quad (21)$$

where t_0 = basic reference period for $q_0(t)$ (e.g. 1 year for climatic loads, 5 years for the sustained part of imposed loads in office buildings, see (CEN/TC250 1996)). The standard deviation remains the same, $\sigma_{q0} = \sigma_{q0,\mathrm{tref}}$. Note that other theoretical models such as a three-parameter lognormal distribution may be more suitable than the Gumbel distribution for some variable loads.

In many cases it can be considered – as an approximation – that q_{tref} has a Gumbel distribution with the following parameters:

$$\mu_{q,\mathrm{tref}} \approx \mu_{C0}\ \mu_{q0,\mathrm{tref}};$$
$$\delta_{q,\mathrm{tref}} \approx \sqrt{(\delta_{C0}^2 + \delta_{q0,\mathrm{tref}}^2 + \delta_{C0}^2\ \delta_{q0,\mathrm{tref}}^2)} \quad (22)$$

where $\delta_{q0,\mathrm{tref}} = \sigma_{q0}\ /\ \mu_{q0,\mathrm{tref}}$. Consequently, the partial factor is assessed as:

$$\gamma_q \approx$$
$$\mu_{q,\mathrm{tref}}/q_{\mathrm{k}} \times [1-\delta_{q,\mathrm{tref}}(0.45+0.78\ln(-\ln\Phi(-\alpha_E\beta_{\mathrm{t}})))] \quad (23)$$

where q_{k} = characteristic value applied in the assessment.

As an example the partial factor of the snow load γ_q, obtained from Equation 19, is shown in Figures 7 and 8 for various coefficients of variation δ_S and different t_{ref}, respectively. The different β-values and α_E = -0.7 are considered.

It is emphasised that the partial factors are dependent on the assumed probabilistic models. In the reliability verification of a particular structure, probabilistic models should be carefully specified taking into account actual loading, structural conditions, and relevant experimental data. More detailed analysis should be based on a full-probabilistic approach.

Table 2. Indicative probabilistic models of selected variable loads.

X	Variable	Distr.	μ_X / X_k	δ_X
C_{0W}	Time-invariant component of the wind action	LN	0.65	0.3
v_b	Annual maxima of the basic wind velocity	LN*	~ 1 / {1 - δ_{vb}[0.45 + 0.78ln(-ln 0.98)]}	**
W	Annual maxima of the basic wind pressure	Gum	~ (1 + δ_{vb}^2) / {1 - δ_{vb}[0.45 + 0.78ln(-ln 0.98)]}^{2}	***
C_{0S}	Time-invariant component of the snow load	LN	1	0.15
S	Annual maxima of snow load on the ground	Gum	~ 1 / {1 - δ_S[0.45 + 0.78ln(-ln 0.98)]}	**
C_{0Q}	Uncertainty of the imposed load model	LN	1	0.1
Q	5-year maxima of imposed load (offices)****	Gum	0.2	1.1

*Three-parameter lognormal distribution. **Should be based on meteorological data. Indicative values of δ_S: 0.6-0.7 (lowlands in the Czech Republic), 0.4-0.6 (mountains in the Czech Republic). Value of δ_{vb} should always be based on local meteorological data since it is dependent on terrain roughness, orography and altitude. ***$\delta_W \approx \delta_{vb}$ (4 - δ_{vb}^2 + 6$\delta_{vb}\omega_{vb}$)$^{0.5}$ / (1 + δ_{vb}^2) where ω denotes a sample skewness (in the case of insufficient data, Gumbel distribution may be assumed and ω_{vb} = 1.14 may be considered). ****Assuming a sustained part of the imposed load is dominating over an intermittent part.

Table 3. Probabilistic models for basic variables considered in the example.

X	Variable	Source of information	Distr.	μ_X / X_k	δ_X	γ_X (β = 3.1)	Other factors
G	Permanent action	In-situ measurements	N	1	0.05	γ_g = 1.11, Figure 5	ξ = 0.85*
C_{0S}	Time-inv. comp. snow load	Table 2	LN	1	0.15	-	-
S	Annual max. snow on ground	Meteorological data	Gum	0.25	0.65	γ_q = 1.12 t_{ref} = 15 y., Eq. 23	ψ_0 = 0.5*
f_c	Concrete comp. strength	Tests (sample size n = 20)	LN	29.4/40	0.15	γ_c = 1.13, Figure 2	-
f_y	Yield strength of reinforc.	Tests (sample size n = 5)	LN	490/560	0.054	γ_s = 1.05, Figure 2	-
h	Height of the beam (0.5 m)	In-situ measurements	N	1	0.02	-	-
b	Width of the beam (0.3 m)	In-situ measurements	det	1	-	-	-
b_{col}	Width and height - column	Study parameter	N	1	0.01m/x_{nom}	-	-
a	Distance reinf. to surface	In-situ measurements	Gamma	1	0.17	-	-
$\theta_{R,M}$	Resist. unc. (bending mom.)	Table 1	LN	1.1	0.1	$\gamma_{Rd,M}$ = 1.00, Figure 1	-
$\theta_{R,N}$	Resist. unc. (axial compr.)	Table 1	LN	1	0.05	$\gamma_{Rd,N}$ = 1.05, Figure 1	-
$\theta_{E,M}$	Load eff. unc. (bend. mom.)	Table 1	LN	1	0.1	$\gamma_{Ed,M}$ = 1.09, Figure 3	-
$\theta_{E,N}$	Load eff. unc. (axial forces)	Table 1	LN	1	0.05	$\gamma_{Ed,N}$ = 1.04, Figure 3	-

*(EN 1990 2002)

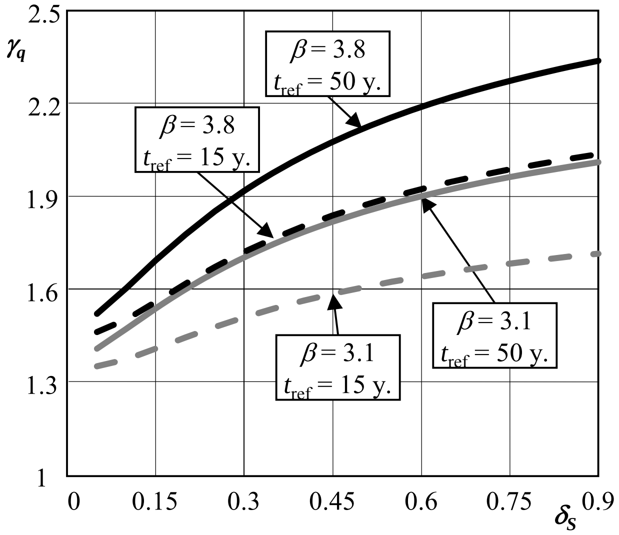

Figure 7. Variation of the partial factor γ_q for the snow load with the coefficient of variation of annual maxima of the snow on the ground (t_{ref} = 15 or 50 years).

6 EXAMPLE OF THE BEAM AND COLUMN EXPOSED TO SNOW LOAD

The structural reliability of an existing reinforced concrete beam and short column exposed to a per-

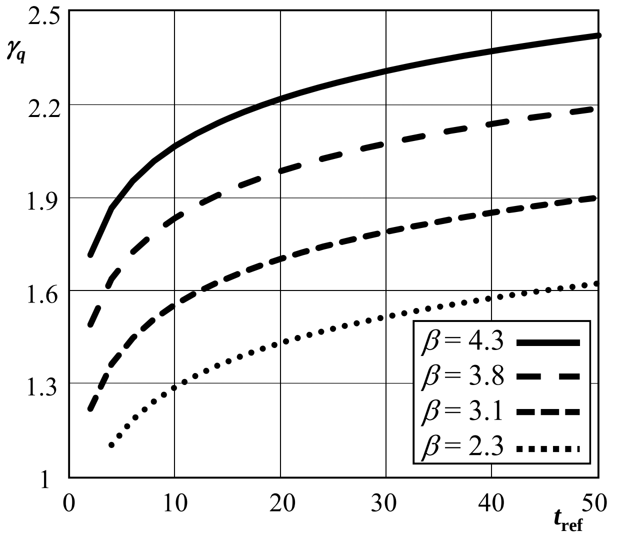

Figure 8. Variation of the partial factor γ_q for the snow load with the reference period (δ_S = 0.6).

manent and snow load is analysed by a full-probabilistic method. Partial factors are obtained by the previously described procedures for the target reliability index β_t = 3.1. The reference period t_{ref} (equal to a remaining working life) is a study parameter. Measurements of material and geometry properties are available and statistical characteristics of the snow load on the ground are provided by a meteorological institute.

6.1 *Actions*

Model uncertainty factors $\gamma_{Ed,M}$ and $\gamma_{Ed,N}$ for estimation of bending moments (beam) and axial forces (column) are obtained from Figure 3.

A characteristic value of the permanent action $G_k = \mu_G$ and its coefficient of variation δ_G are estimated from measurements on the structure (Table 3). G_k = 50 kNm (bending moment) and G_k = 3.5 MN (axial compressive force) apply for the beam and column, respectively. The partial factor γ_g is obtained from Figure 5.

To cover a wide range of load combinations, the load ratio χ (study parameter) is introduced:

$$\chi = Q_k / (G_k + Q_k) \tag{24}$$

In general the load ratio may vary within the interval from nearly 0 (underground structures, foundations) up to nearly 1 (local effects on bridges, crane girders). For reinforced concrete beams in buildings χ is expected to vary within the range from 0.4 up to 0.7; for columns within the range from 0.1 to 0.6. Given G_k and χ, Q_k can be obtained from Equation 24.

Statistical characteristics of annual maxima of the snow load on the ground are available (Prague airport, $\mu_{S,1} = 0.25$ kN/m^2 and $\delta_{S,1} = 0.65$). The mean of maxima of the snow load on the ground related to the reference period t_{ref} (in years) is derived in case of a Gumbel distribution as follows:

$$\mu_{S,tref} = \mu_{S,1} + 0.78\sigma_{S,1} \ln(t_{ref}) \quad (25)$$

where $\sigma_{S,1} = \sigma_{S,tref}$ = standard deviations of maxima of the snow load on the ground related to 1 year and reference period, respectively.

The characteristic value $s_k = 1$ kN/m^2 is obtained from the Czech snow map. The load effect Q_k obtained from Equation 24 corresponds to the characteristic value s_k multiplied by characteristic values of relevant factors (shape, exposure and thermal coefficients, loading width, span, etc.) $C_k(\chi)$:

$$C_k(\chi) = Q_k(\chi) / s_k \quad (26)$$

The design load effect is obtained as follows:

$$E_d = \max[\gamma_{Ed}\, \gamma_g\, G_k + \gamma_{Ed}\, \gamma_q(t_{ref})\, \psi_0\, C_k(\chi) s_k; \\ \xi\, \gamma_{Ed}\, \gamma_g\, G_k + \gamma_{Ed}\, \gamma_q(t_{ref})\, C_k(\chi) s_k] \quad (27)$$

Partial factors and statistical characteristics of the snow load are provided in Table 3.

6.2 *Resistance*

As a prerequisite, design resistances of the investigated beam and column are assumed equal to the design load effects given by Equation 27, $R_d = E_d$. Flexural resistance of an existing concrete beam is:

$$R(\rho) = \theta_{R,M}\, \rho\, b(h - a)\, f_y\, [h - a - 0.5\rho(h - a) f_y / f_c] \quad (28)$$

The probabilistic models are given in Table 3. To satisfy the condition $R_d = E_d$, the limiting value of the reinforcement ratio is:

$$\rho(\chi, t_{ref}) = \frac{\dfrac{f_{ck}}{\gamma_{Rd,M}\gamma_c}}{\dfrac{f_{yk}}{\gamma_{Rd,M}\gamma_s}} \left[1 - \sqrt{1 - \frac{2E_d(\chi, t_{ref})}{b(h-a)^2 \dfrac{f_{ck}}{\gamma_{Rd,M}\gamma_c}}}\right] \quad (29)$$

Note that for the expected range of $\chi \in \langle 0.4, 0.7 \rangle$ and $t_{ref} \in \langle 1$ y.,50 y.$\rangle$, the reinforcement ratio varies between 0.25-0.75 % and reinforcement properties are governing the failure.

The resistance of a centrically loaded, short square column is given as:

$$R(b_{col}) = \theta_{R,N}\, (f_c + \rho f_y)\, b_{col}^2 \quad (30)$$

The probabilistic models are given in Table 3. In this case, the constant reinforcement ratio $\rho = 0.5$ % is considered and the column width/height b_{col} is derived to achieve $R_d = E_d$:

$$b_{col}(\chi, t_{ref}) = \sqrt{\frac{E_d(\chi, t_{ref})}{\dfrac{f_{ck}}{\gamma_{Rd,N}\gamma_c} + \rho \dfrac{f_{yk}}{\gamma_{Rd,N}\gamma_s}}} \quad (31)$$

For the expected range of $\chi \in \langle 0.1, 0.6 \rangle$ and $t_{ref} \in \langle 1$ y.,50 y.$\rangle$, b_{col} varies between 0.37-0.60 m. For the column concrete failure mode is dominant.

6.3 *Reliability analysis*

The structural reliability of the beam and column is analysed by FORM. The limit state function reads:

$$Z(\mathbf{X}) = \\ \theta_{R,(M \text{ or } N)}\, R(\chi, t_{ref}) - \theta_{E,(M \text{ or } N)}\, [G + C_k(\chi) C_{0S} S_{tref}] \quad (32)$$

Figure 9 shows the variation of the reliability index β with χ for $t_{ref} = 15$ y. and $\beta_t = 3.1$ based on partial factors obtained by the design value method. To illustrate benefits of the method, the reliability index for the par-

tial factors recommended for new structures ($\gamma_C = 1.5$, $\gamma_S = 1.15$, $\gamma_G = 1.35$, $\gamma_Q = 1.5$), independent of χ and t_{ref}, is plotted for the beam.
Figure 10 indicates variation of the reliability index β with the reference period t_{ref} for $\chi = 0.55$ in case of the beam and for $\chi = 0.35$ in case of the column (the middle values of the expected ranges of χ).

It follows from Figures 9 and 10 that:

– The design value approach captures well random properties of the basic variables. For the expected ranges of χ, the obtained β values are reasonably close to the target level for different reference periods t_{ref}.

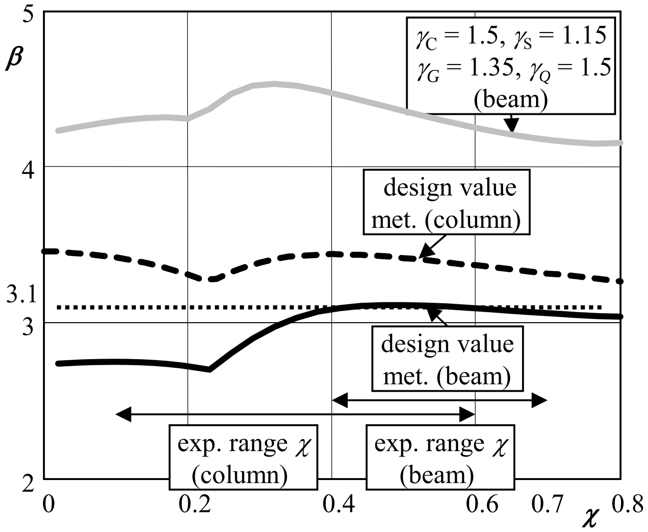

Figure 9. Variation of β with χ for $t_{ref} = 15$ y.

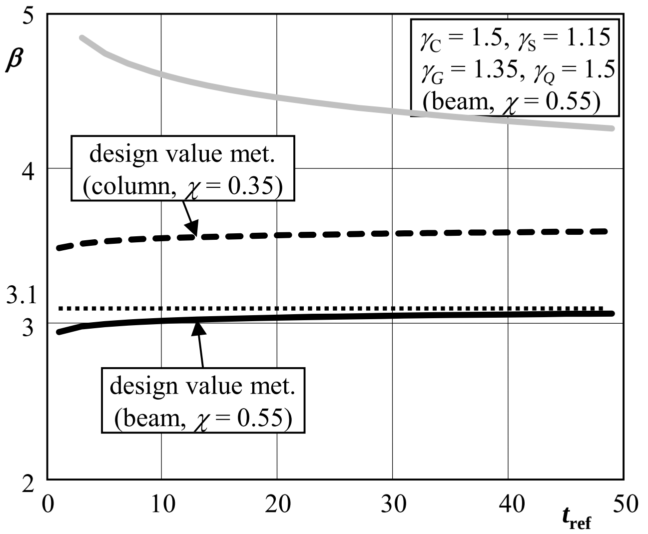

Figure 10. Variation of β with $t_{ref} = 15$ years.

– In case of the beam rather lower reliability levels for small values of χ can be attributed to the underestimated factors $\gamma_{Rd,M}$ (considered value $\alpha_R = 0.32$ is rather low compared to the actual sensitivity factors obtained by FORM).
– Influences of the target reliability ($\beta_t = 3.1$ here) and of information obtained from the measurements are not adequately covered when the partial factors recommended for new structures are considered. In this example application of the recommended values yields overly conservative structural verifications.

7 CONCLUDING REMARKS

Reliability of existing reinforced concrete structures can be efficiently verified using partial factors obtained by the design value method. Numerical example reveals that reliability of members verified by the design value method is close to a specified target reliability for any reference periods and ratios of permanent to variable actions. Results of measurements can be readily included.

REFERENCES

CEN/TC250, 1996. *Background Document EC1:Part1: Basis of Design.* 2nd draft. ECCS.

DIAMANTIDIS, D. and BAZZURRO, P., 2007. Safety acceptance criteria for existing structures, *Special Workshop on Risk Acceptance and Risk Communication*, March 26-27, 2007 2007.

EN 1990, 2002. *Eurocode - Basis of structural design.* Brussels: CEN.

EN 1992-1-1, 2004. *Design of concrete structures - Part 1-1: General rules and rules for buildings.* Brussels: CEN.

FIB SAG 9, 2010. *Revision of partial safety factors (report).* December 2010.

HOLICKÝ, M., RETIEF, J.V. and DUNAISKI, P.E., 2007. The reliability basis of design for structural resistance, A. ZINGONI, ed. In: *Proc. SEMC 2007* 2007, Millpress, pp. 1735-1740.

HOLICKÝ, M. and SÝKORA, M., 2011. Conventional probabilistic models for calibration of codes, M.H. FABER, J. KÖHLER and K. NISHIJIMA, eds. In: *Proc. ICASP11*, 1-4 August 2011 2011, CRC Press/Balkema, pp. 969-976.

ISO 13822, 2010. *Bases for design of structures - Assessment of existing structures.* Geneve, Switzerland: ISO TC98/SC2.

ISO 2394, 1998. *General principles on reliability for structures.* 2nd edn. Geneve, Switzerland: ISO.

JCSS, 2001. *JCSS Probabilistic Model Code.* Zurich: Joint Committee on Structural Safety.

SYKORA, M. and HOLICKY, M., 2012. Target reliability levels for the assessment of existing structures - case study, A. STRAUSS, K. BERGMEISTER and D.M. FRANGOPOL, eds. In: *Proc. IALCCE 2012*, 3 - 6 October 2012 2012, CRC Press/Balkema, pp. 813-820.

TAERWE, L., 1993. *Towards a consistent treatment of model uncertainties in reliability formats for concrete structures.* CEB Bulletin d'Information n° 219 'Safety and Performance Concepts'. Lausanne: CEB.