10.1515/tvsb-2017-0015

Transactions of the VŠB – Technical University of Ostrava

Civil Engineering Series, Vol. 17, No. 1, 2017

paper #15

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ANALYSIS OF PARTIAL FACTORS FOR NEW EN 1990 BASIS OF DESIGN

Abstract

Presently the new generation of Eurocodes is under development. It is expected that the basis of structural design given in current EN 1990 will be considerably revised including the format for load combinations in the Ultimate Limit States and new categorisation of consequence classes for construction works. It is expected that the partial factors for self-weight of structural members could be reduced and factors for variable loads more differentiated. Calibration of partial factors should be performed for actions and materials including also structural glass and FRP polymers.

Keywords

Partial factors, reliability index, target reliability, probabilistic models, model uncertainty.

1 INTRODUCTION

For the verification of structures in the Ultimate limit states (ULS), the new format is proposed in the draft of EN 1990 for basis of structural design in the upcoming second generation of Eurocodes. It is expected that the fundamental load combination, exp. (6.10), for the verification of ULS will be recommended as a basic load combination, and the partial factors for actions should be more differentiated. The partial factor for self-weight of structural members having low variability is proposed to be reduced (1,2) than for other permanent actions where the presently given value (1,35) is expected to remain. The partial factors for imposed loads and climatic actions are intended to be newly calibrated. Presently the unique partial factor (1,5) is recommended in Eurocodes for nearly all variable loads what is in some cases rather conservative, however for some climatic actions it might also lead to decrease of safety level below the recommended target reliability indices.

During implementation of the current generation of Eurocodes some countries decided to select lower target reliability level (β_t = 3,4) to which their partial factors and other reliability elements were calibrated. Presently the target reliability index β_t = 3,8 is recommended for 50 years return period in EN 1990 [1], while for bridges this value is provided for 100 years period.

More detail categorisation of structures is currently given in new EN 1990 proposing five consequence classes CC1 to CC5, and two subcategories in the classes CC1 to CC3. However, the target reliability indices for these subcategories have been not specified yet. Another issue is the relation of the new guidance for structural robustness with the new categorisation of structures, where general provisions are presently given only.

2 FUNDAMENTAL LOAD COMBINATION

Eurocode EN 1990 [1] introduces for the fundamental combination of actions in permanent and transient design situations (ULS of type STR) three alternative procedures, denoted here as load combinations (6.10), (6.10a,b) and (6.10a,$_{mod}$,b). Considering for simplicity one permanent action *G*

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and two variable actions, e.g. an imposed load Q and a wind action W, the combination rule (6.10) is given for the design value of action effect E_d as:

$$E_d = \gamma_G G_k + \gamma_Q Q_k + \gamma_W \psi_W W_k, \tag{1}$$

An alternative procedure (6.10a,b) is expressed as:

$$E_d = \gamma_G G_k + \gamma_Q \psi_Q Q_k + \gamma_W \psi_W W_k \tag{2}$$

$$E_d = \xi \gamma_G G_k + \gamma_Q Q_k + \gamma_W \psi_W W_k \tag{3}$$

where the less favourable action effect from (2) and (3) should be considered ("+" means here in combination). In addition, EN 1990 [1] allows further modification of alternative (6.10a,b), denoted as alternative (6.10a$_{mod}$,b), simplifying equation (2) by considering permanent loads only, thus the load effect is then:

$$E_d = \gamma_G G_k \tag{4}$$

The less favourable action effect resulting from (3) and (4) is then considered. If the leading action is wind W, then in equations (1) and (3) instead of reducing the wind action W by factor ψ_W, the imposed load Q should be reduced by appropriate factor ψ_Q. Factors γ_G, γ_Q and γ_W denote the partial factors of actions G, Q and W.

To investigate resulting load effects under various intensities of variable actions, the characteristic values of actions G_k, Q_k and W_k are related using quantity χ given as the ratio of variable actions Q_k+W_k to total load $G_k+Q_k+W_k$, and the ratio k of accompanying action W_k to the main action Q_k as:

$$\chi = (Q_k+W_k)/(G_k+Q_k+W_k),\ k = W_k/Q_k \tag{5}$$

The parameter χ may be commonly expected in a range from 0 to 0,6 for common buildings, however sometimes approaching to 1, e.g. in case of snow loads on light-weight steel roofs of industrial halls or hypermarkets.

For a given design value of the load effect E_d, the characteristic values of individual actions G_k, Q_k, W_k can be expressed using variables χ and k as follows:

$$G_k = \frac{E_d}{(\xi)\gamma_G + \dfrac{((\psi_Q)\gamma_Q + k(\psi_W)\gamma_W)\chi}{(1+k)(1-\chi)}} \tag{6}$$

$$\chi = (Q_k+W_k)/(G_k+Q_k+W_k),\ k = W_k/Q_k \tag{7}$$

The factors ξ, γ and ψ indicated in the first relationship of (6) in brackets are applied in the same way (either yes or no) as in equations (1), (3) for appropriate combination rules.

Presently available information indicates that one third of 18 CEN available countries has selected the load combination (6.10), exp. (1), and one sixth the combination rule (6.10a,b), twin exps. (2, 3). Both procedures (6.10) and (6.10a,b) are allowed to be used in eight countries, and the procedure (6.10a,$_{mod}$,b), twin exps. (3, 4), in two countries only. In countries where it is allowed to use two alternative combinations, one alternative is commonly preferred and the second might be used under specific conditions, e.g. procedure (6.10a,b) is recommended in the Czech Republic because leads to more balanced reliability of structures, however the combination (6.10) may also be applied. Three countries did not provide any preference for the national selection of load combination.

Most countries accepted recommended values of partial factors for materials given in Eurocodes with exception of one country for concrete and reinforcement, four countries for steel structures and three countries for timber structures. Big differences exist for masonry structures where the partial factors are given in a rather broad range where different categorization of masonry and classes of execution are nationally considered in most CEN countries.

3 PROBABILISTIC ANALYSIS

EN 1990 [1] allows a design directly based on the probabilistic methods. In accordance with the principles of these methods the basic variables are considered as random variables with appropriate distributions, Holicky [5,6,7]. It is verified that a limit state of a member is exceeded with a probability lower than the target value p_t given as:

$$P(g(X) < 0) < p_t \qquad (8)$$

Here g(X) denotes the limit state function, for which the inequality g(X) < 0 indicates that the limit state is exceeded. The condition (8) may be replaced by the inequality $\beta > \beta_t$, where β denotes the reliability index. EN 1990 [1] recommends the target probability $p_t = 7{,}24 \times 10^{-5}$ for ULS of common buildings corresponding to the reliability index $\beta_t = 3{,}8$ for 50 year design working life.

Reliability analysis is based on the limit state function g(X) corresponding to load effect given e.g. by equation (1) and resistance of a generic structural member:

$$g(X) = \theta_R\, R - \theta_E\,(G + Q + W) \qquad (9)$$

where:

X – is the vector of basic variables,

θ_R – is the factor expressing the uncertainty of the resistance model,

θ_E – is the factor expressing the uncertainty of the action effect model.

An important step in any reliability analysis is the specification of probabilistic models for the basic variables in the limit state function (9).

The probabilistic models of actions are related to their characteristic values used for the determination of the design values of actions, see Table 1. The permanent action is described by normal distribution (N), variable actions by Gumbel distribution (GUM) and material strength by lognormal distribution (LN). These models are primarily intended as "conventional models" in time invariant reliability analysis of structural members using Turkstra's combination rule, see PMC [8].

Tab.1: Probabilistic models of basic variables

Basic variable	Distribution	Units	Char. value	μ_X	σ_X
Permanent	N	MN/m^2	G_k	G_k	$0{,}1\mu_X$
Imposed (5 y.)	GUM	MN/m^2	Q_k	$0{,}2Q_k$	$1{,}1\mu_X$
Imposed (50 y.)	GUM	MN/m^2	Q_k	$0{,}6Q_k$	$0{,}35\mu_X$
Wind (1 y.)	GUM	MN/m^2	W_k	$0{,}3Q_k$	$0{,}5\mu_X$
Wind (50 y.)	GUM	MN/m^2	W_k	$0{,}7Q_k$	$0{,}35\mu_X$
Concrete resistance	LN	MPa	R_k	R_k+2σ_X	0,15
Steel resistance	LN	MPa	R_k	R_k+2σ_X	
Load uncertainty	LN	-	θ_E	1,0	0,10
Resistance uncertainty	LN	-	θ_R	1,0	0,05

4 RESULTS OF ANALYSIS

Selected results of reliability analyses of some concrete and steel structural members are shown in Figures 1 to 4. The four sets of partial factors selected in National Annexes of some countries are taken into account and achieved reliability level compared with the target reliability index $\beta_t = 3{,}8$ recommended for common structures in the reliability class RC2.

An effect of alternative load combination rules for a reinforced concrete beam is illustrated in Figure 1 considering design according to EN 1992-1-1 [2]. Obviously the combination (6.10a,b) given by twin exps. (2,3) leads to a better balanced reliability than the combination rule (6.10), exp. 1). In combination (6.10a$_{mod}$,b), exps. (3,4), the recommended values of partial factors are applied according to EN 1990 [1]. Combination (6.10a$_{mod}$,b)$_{NA}$ is applied with NDPs selected in some CEN countries where the values of partial factor for permanent load are applied γ_G = 1,2 in exp. (4) and γ_G = 1,0 in exp. (3), for adverse variable loads γ_Q = 1,5, for concrete γ_c = 1,45 and for reinforcement γ_s = 1,2, leading in this case also to satisfactory reliability of a structural member.

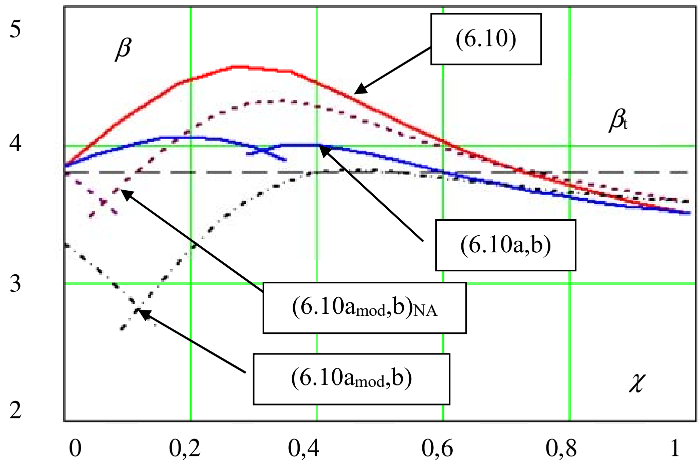

Fig.1: Variation of the reliability index β of the reinforced concrete beam with the load ratio χ for the alternative load combinations.

Figure 2 shows the significance of national decision concerning the value of reduction factor ξ for permanent actions in alternative (6.10a,b) on the reliability of the concrete beam which is then approaching the alternative (6.10). It should be noted that three countries have selected the value of reduction factor ξ = 0,89 (instead of CEN recommended value 0,85), and the value ξ = 0,925 one country.

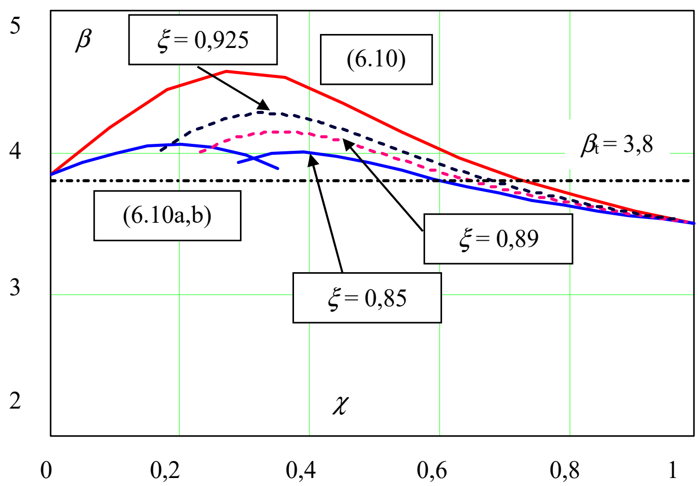

Fig.2: Variation of the reliability index β of the concrete beam with the load ratio χ for the combination (6.10), and for combination (6.10a,b) considering ξ = 0,85, 0,89, 0,925.

The reliability analysis of a short axially loaded concrete column designed according to Eurocodes, considering the CEN recommended set of partial factors, indicates that the reliability index β is greater than the target value β_t (3,8) for the combinations (6.10) and (6.10a,b), see Figure 3. However, for the combination (6.10a_{mod},b) the reliability of the column is not meeting requirements for prevailing permanent loads.

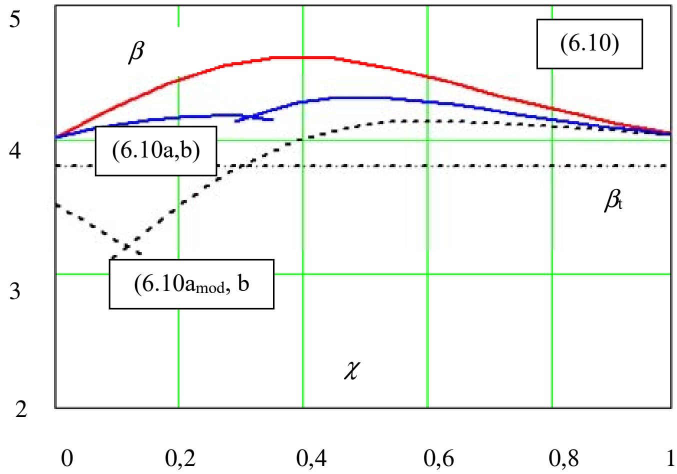

Fig. 3: Variation of the reliability index β of the short column with the load ratio χ.

In the present generation of Eurocodes, the material factor γ_M = 1 is recommended for steel structures which was nationally modified in the National Annexes of some Member countries. Following four cases (denoted here by I to IV, see Table 2) characterized by different sets of partial factors for actions and material properties are analysed. The first set (I) of partial factors indicated in Table 2 is based on the recommendations of EN 1990 [1] and EN 1993-1-1 [3], the second set (II) is based on the ENV version of Eurocodes, the remaining sets (III-IV) were selected in some countries.

Tab.2: Probabilistic models of basic variables

No.	γ_M	γ_G	γ_Q	No.	γ_M	γ_G	γ_Q
I	1,0	1,35	1,5	III	1,2	1,2	1,4
II	1,1	1,35	1,5	IV	1,2	1,1	1,4

Figure 4 illustrates the reliability index β as a function of the load ratio χ assuming the four sets of partial factors and considering one variable action acting only. The set I of partial factors provides considerably lower reliability level of a steel member than the set II, however still acceptable for the load ratio χ within the common range from 0,1 to 0,6 for combination (6.10). Comparing reliability indices obtained for the first two cases I and II, it appears that the decrease of the material factor γ_M from 1,1 to 1 leads to the considerable decrease in the reliability index β.

Figure 4 clearly indicates that the partial factor γ_M = 1 for steel should be applied only in the load combinations based on expression (6.10) and not in the load combination (6.10a,b). It appears that the set of partial factors III might be used in the load combination (6.10), the set IV in the combination (6.10a,b).

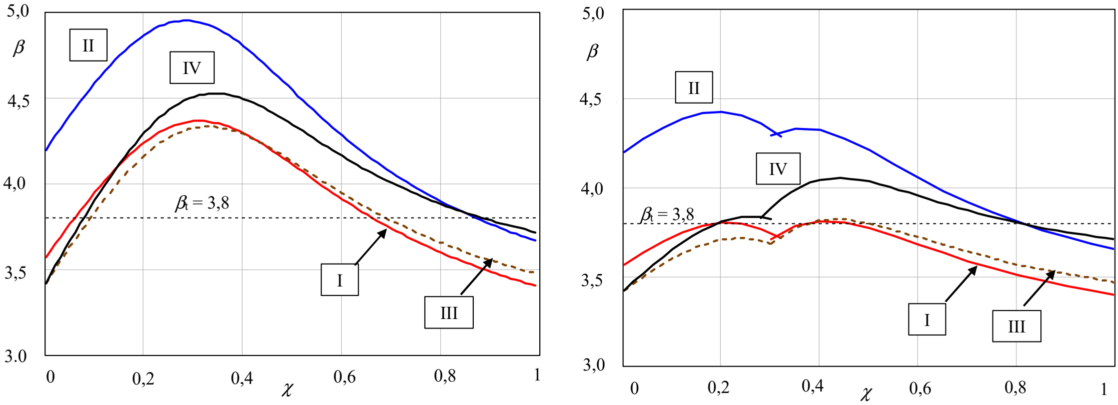

Fig. 4: Variation of the index β for a steel member with the load ratio χ, the load combinations (6.10) (on left side of Fig. 4) and (6.10a,b) (on right side), considering sets I to IV of partial factors.

4 CONCLUDING REMARKS

The reliability of structures, designed according to the alternative combination rules provided in EN 1990 might considerably vary. Combination (6.10) leads to the most reliable but most likely uneconomical solution. Combination (6.10a,b) yields a lower but a more uniform reliability level. It is shown that the reliability of the steel members is significantly affected by the material partial factor γ_M. Obviously, any decrease in the factor γ_M leads to a decrease in the reliability level of the member. It appears that for the partial factors γ_G and γ_Q for actions recommended in EN 1990, the partial factor for steel γ_M = 1 provided in EN 1993-1-1 leads to a considerably lower reliability level than previously used value γ_M = 1,1 in ENV Eurocodes. It appears that γ_M = 1 should not be used in combination (6.10a,b) or its modification, the alternative (6.10a,$_{mod}$,b).

In case that the combination (6.10) should be nationally selected as it is presently recommended as an unique combination in the working draft of new EN 1990, the partial factors for actions and materials, and other safety elements should be recalibrated to optimize the deviation of the structural reliability level from the recommended target reliability.

ACKNOWLEDGMENT

This is the partial outcome of the project TE01020068 Centre of research and experimental development of reliable energetics supported by the Technological agency of the Czech Republic.

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