RELIABILITY ASSESSMENT OF EXISTING STRUCTURES

1. Introduction

EN Eurocodes, presently being implemented into the system of national standards nearly in the whole Europe, are particularly intended for the design of new structures. Supplementary rules for the verification of existing structures are still missing. General requirements and procedures for the assessment of existing structures based on the theory of structural reliability are provided in ISO 13822 [1]. For the application of this standard in building practice in the Czech Republic, six National annexes were developed. Selected provisions and new terms including material independent problems for the assessment of existing structures are explained in the first Annex NA. Testing of existing structures and materials, and also principles of experimental evaluation are introduced in the National Annex NB. Material oriented Annexes NC to NF provide recommendation for determination of material properties for reinforced concrete, steel, timbre, composites and masonry including tables for conversion among existing and presently valid classes of concrete.

2. Reliability assessment of existing structures

The design of existing structures is as a rule based on different approaches given in the original standards, including the method of allowable stresses or the safety factor method. If the structure is designed by means of the partial factor method, then the procedures for determining the characteristic and design values of basic variables, the rules for load combinations and analytical models commonly differ from provisions given in current standards. For reliability analyses of structures, the partial factor method or probabilistic methods may be applied according to ISO 13822 [1].

2.1. Partial factor method

The partial factor method represents the basic method in new European and international standards. The values of partial factors for actions and material properties are recommended on the basis of calibrations and good practice in construction. The required reliability level of a structure could be considered for determining the values of partial factors. For the 50 years design working life of a structure and ultimate limit state, the target value of reliability index $\beta_t = 3{,}8$ is commonly recommended (for the reliability class RC 2 according to EN 1990 [2]). Where justifiable, other requirements for the target reliability index may be considered for existing structures than for new ones. The partial factors for existing structures could be in those cases modified.

2.2. Probabilistic methods

The probabilistic methods may be in specific cases, e.g. in time dependent problems, applied with advantage for the verification of existing structures or for the assessment of residual lifetime. For the probabilistic analysis of existing structures, the limit state function g($\boldsymbol{X}$) is specified for the vector $\boldsymbol{X}$ of basic variables. It is assumed that the structure is reliable for the condition g($\boldsymbol{X}$) > 0. The failure probability P_f is determined on the basis of integration over region Z($\boldsymbol{X}$) < 0, where the structure is not reliable, given as

$$P_f = \int_{Z(X)<0} \varphi_X(\mathbf{x})\mathrm{d}\mathbf{x} \qquad (1)$$

where $\varphi_X(\boldsymbol{x})$ is the joint probability density for the realisation of vector $\boldsymbol{x}$. Another reliability indicator is the generalized reliability index β defined on the basis of the failure probability P_f as $\beta = -\Phi^{-1}(P_f)$, where Φ is the standardised normal distribution function. Resulting failure probability P_f or reliability index β of the structural member is compared with the target probability $P_{f,t}$ or target reliability index β_t.

3. Partial factors for required reliability level

The concept of design values may be used for determining the partial factors of the basic variables applied for the verification of existing structures. The partial factors γ_i for the basic variables X_i having a favourable effect on structural reliability (resistance) may be determined as

$$\gamma_i = x_{ik} / x_{id} \qquad (2)$$

where the values of partial factors γ_i are commonly greater than one. The procedure for applying the partial factors based on expression (4) for the reliability assessment of existing structures is described as follows. The structural resistance R (e.g. concrete strength, yield strength of steel) may be often described by two-parametric lognormal distribution. The characteristic value of resistance is commonly defined as 5 % lower fractile. The partial factor γ_R is according to expression (2) given as

$$\gamma_R = \exp(-\,1{,}645\ V_R)/\exp(-\,\alpha_R\ \beta\ V_R) \tag{3}$$

where V_R is the coefficient of variation for resistance R and β the required reliability index (e.g. target value), the coefficient 1,645 is the value of 5% fractile of the standardised normal distribution and the sensitivity factor α_R = 0,8. The partial factor for resistance γ_R versus the coefficient of variation V_R for three reliability classes RC1 to RC 3 is illustrated in Fig. 1.

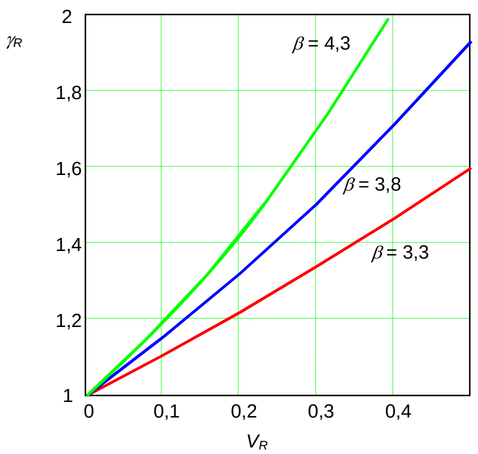

Fig. 1. The partial factor for resistance γ_R versus the coefficient of variation V_R

For the coefficient of variation V_R = 0,1 and structure of a common class CC2, the partial factor of resistance should be about γ_R = 1,15. However, the direct application of results illustrated in Fig. 1 may be rather difficult because the characteristic value of material strength of a building material like steel corresponds in reality to lower probability than 5 % given in EN 1990 [2]. This fact is due to the effort to increase the production quality of materials leading to increase the mean of the strength of real production.

4. An example of reliability assessment according to ISO 13822

The procedure for the reliability assessment of existing structures is shown on a selected example of deteriorated balcony beams of panel houses in the North part of the Czech Republic. The Klokner Institute investigated 230 balconies of panel houses (construction system T0-6B-BTS). Detailed visual inspection proved that the reinforcement of concrete slabs was strongly affected by corrosion due to deteriorated insulation. The carbonation of concrete was visible particularly in front parts of balconies. Results of inspection revealed that the beams were made from concrete of class C 16/20 and reinforcement S 200 with diameter 0,008 m. The beams were cantilevered 0,90 m, their width 3,50 m. The actual position of reinforcement considerably differed from assumptions made in the original design (concrete cover 0,01 m, bar spacing 0,15 m). The distribution of concrete cover c for all 4890 measurements is shown in Figure 2. The actual concrete cover, varying from 0,002 m to 0,065 m, did not provide adequate protection against adverse environmental influences and considerably decreased the load-bearing capacity of balcony components. The quality of construction work was very poor.

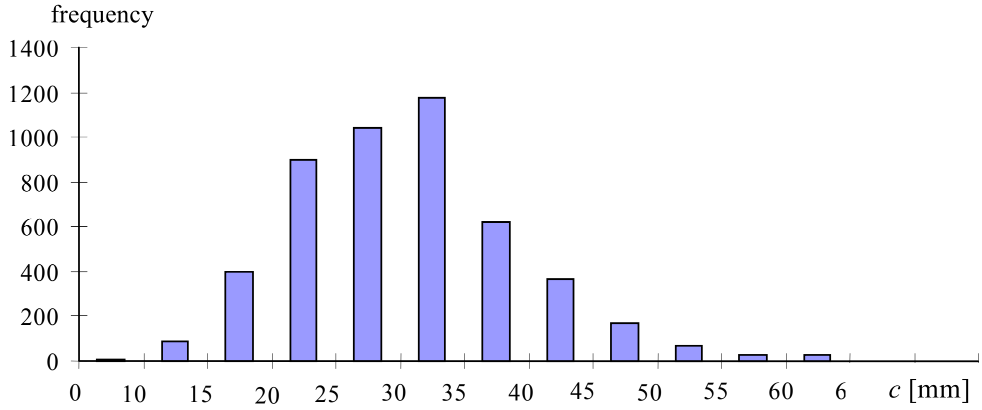

Fig. 2. Distribution of concrete cover c for 230 balconies (4890 measurements, mean μ = 0,026 m, standard deviation σ = 0,009 m, skewness α = 0,58)

4.1. Verification of balconies according to partial factor method

The verification of beams based on the partial factor method according to ČSN 73 1201 [3] reveals that the design resistance of a beam M_{Rd} = 6,0 kNm is greater than the design load effects M_{Ed} = 3,15 kNm on 1 bm of a beam. In case that the reduction of the area of reinforcement due to corrosion and its actual position is considered, the condition $M_{Rd} > M_{Ed}$ may not be fulfilled yet. The results of analysis for three reinforcement areas A_s are shown in Fig. 3 (for design area A_{s1}= 100 % and reduced areas A_{s2} = 90 % and A_{s3} = 75 %). The design action effect M_{Ed} is also illustrated.

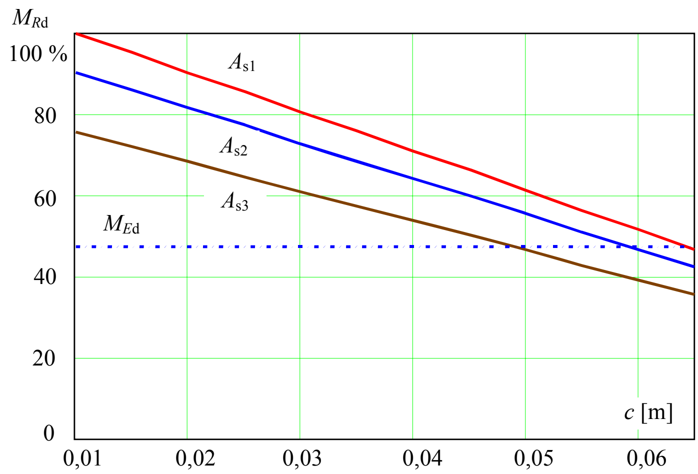

Fig. 3. The variation of resistance moment M_{Rd} with increasing concrete cover c for 3 study cases of reinforcement A_s (for design area A_{s1}= 100 % and reduced A_{s2} = 90 %, A_{s3} = 75 %).

4.2. Probabilistic reliability analysis

For the reliability analysis of balcony, the probabilistic approach according to ISO 13822 [1] is applied. The limit state function for the verification of balconies is given as

$$g = \theta_R\, n\, (\pi\phi^2/4)\, f_y\, [h - c - \phi/2 - 0{,}5\, n(\pi\phi^2/4)\, f_y / f_c] - \theta_E\, (g + p)\, L^2/2 \qquad (4)$$

where all applied basic variables are listed in Table 1 including relevant probability distributions. The probabilistic models of basic variables are based on the provisions of Probabilistic Model Code [4] of the research organisation JCSS and evaluations of test results.

The impact of reduced reinforcement area ΔA_s on the reliability index β for 4 cases of concrete cover c (according to evaluated measurements of individual balconies) is illustrated in Fig. 4. Significant basic variable influencing the reliability of balcony beams is the concrete cover of reinforcement. In case the reduction of the area of reinforcement ΔA_s is not taken into account, then the reliability index β is decreasing from 5,2 to 3,7 for increasing concrete cover from 0,01 to 0,03 m.

Table 1 Probabilistic models of basic variables

Basic variable	Sym-bol	Distribution	Dimension	Mean μ	Standard deviation
Compr. concrete strength	f_c	LN	MPa	24	4
Yield strength	f_y	LN	MPa	240	15
Length of the balcony	L	DET	m	0,90	-
Diameter of a bar	ϕ	DET	m	0,008	-
Number of bars per balcony	n	DET	-	20	-
Balcony depth in embedment	h	LN	m	0,12	0,01
Concrete cover	c	BET	m	0,026	0,009
Uncertainty of resistance	θ_R	LN	-	1,1	0,05 μ
Uncertainty of load effect	θ_E	LN	-	1	0,05
Density of concrete	ρ	N	MN/m^3	nom.	0,06
Imposed load	p	GAM	MN/m^2	0,0008	0,00048

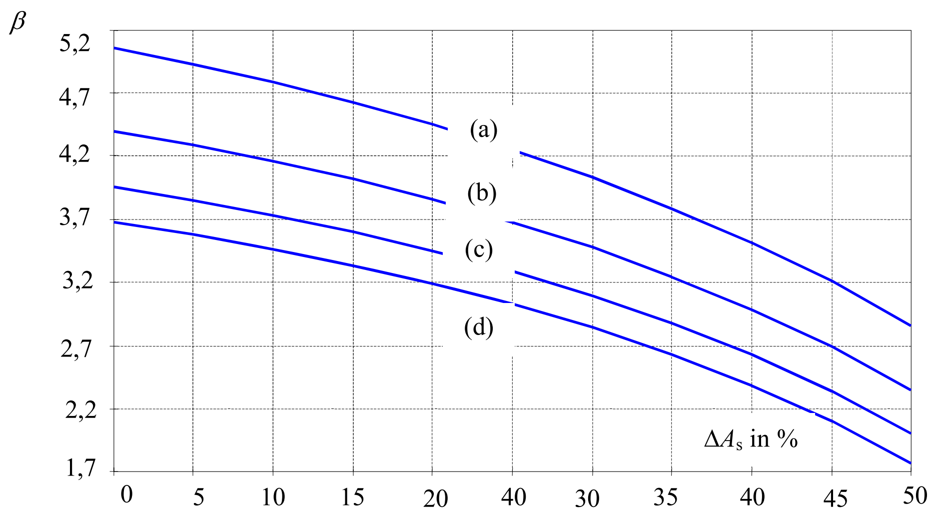

Fig. 4. Reliability index β versus the reduction of reinforcement ΔA_s (in %) for four cases of cover c: (a) mean μ = 0,01 m, (b) 0,02 m, (c) 0,026 m, (d) 0,03 m (coefficient of variation V = 0,35, lower bound a = 0, upper bound b = 3 μ, Beta distribution).

The original design of balconies was consistent with requirements of standards valid in that time. Low-quality production of prefabricated balcony beams including poor quality of construction work on site was leading to significant reduction of the area of reinforcement considered in the original design. The significant basic variable influencing the reliability indices of balconies are the area of reinforcement and concrete cover. The reliability indices considerably decrease with decreasing area of reinforcement due to the corrosion (the index β is reduced to one-half with 50 % reduction of the reinforcement area).

5. Influence of partial factors

The design of existing balconies was influenced by values of partial factors and detailing given in the original Czech standards for structural design (recommended partial factors for permanent and variable actions γ_G = 1,1, γ_Q = 1,3, concrete cover c = 0,01 m). Eurocodes recommend for the verification of the ultimate limit states (type STR) in most cases greater values of partial factors for actions (γ_G = 1,35, γ_Q = 1,5) than ČSN. Fig. 4 indicates the variation of the initial reliability index β (without any reduction of the reinforcement area due to rusting or negligence) with partial factors γ_G and γ_Q for a balcony component, assuming that these were applied in the design of a balcony. The reliability level β_t = 3,8 recommended in Eurocodes is shown in Fig. 5 by a white area.

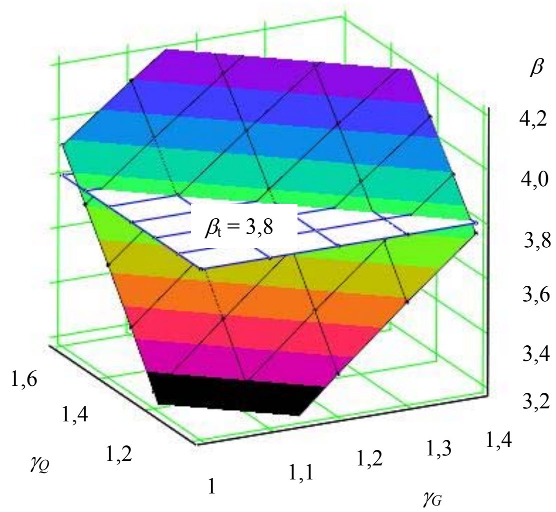

Fig. 5. Variation of the reliability index β with partial factors γ_G and γ_Q

6. Conclusions

The international standard ISO 13822 provides procedures for the reliability assessment of existing structures including the application of probabilistic methods. Probabilistic approach according to this standard facilitates to decide about new exploitation of existing structures. Application of probabilistic methods for the reliability assessment of existing structures is shown on the example of deteriorated balcony beams. Despite the design of beams fulfilled the requirements of originally applied standards, the low quality of production and adverse effects of environment caused significant reduction of reliability level of balconies and evoked the need for their repair. Detailed investigation revealed that the initial reliability index β was in a range from 5,2 to 3,7 for individual beams. When reduced area of reinforcement due to corrosion was considered, the reliability index β was further significantly decreasing.

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References

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Doc. J. Markova is involved in the reliability analyses of structures, actions on structures, concrete structures, probabilistic methods of the theory of structural reliability, national and international standardisation activities.