České vysoké učení technické v Praze Kloknerův ústav

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# Hodnocení existujících konstrukcí

# Assessment of existing structures

**Summary**

The effective use of existing structures provides benefits for society and economy of each country, helping to preserve values of historic and cultural character. The methods for assessment of existing structures based on the probabilistic basis of the theory of structural reliability may facilitate further exploitation of a structure or acquisition of valuable information for its rehabilitation.

No European prescriptive document is currently available for the assessment of existing structures. Eurocodes focus particularly on the design of new structures. That is why the international standard ISO 13822 for the assessment of existing structures is an important document, although of a rather general character. In order to facilitate application of the standard in building practice, six National annexes have been developed at the CTU in Prague.

The partial factor method is generally applied for reliability assessment or rehabilitation of existing structures as well as for the design of new structures. At the present time, the probabilistic methods of the theory of structural reliability are gaining more and more significance. The principles of probabilistic methods may be applied for the reliability differentiation of existing structures, where economic, social and sustainability considerations are of particular importance. The concepts of probabilistic methods may be applied for determining partial factors of the basic variables used in the design of new structures, and also in the verification of existing structures. Practical applications have to be based on the required reliability level and on appropriate theoretical models of the basic variables. The actual material and geometric properties, the load history and adverse environmental effects should be known. The quality of construction work during execution and maintenance of the structure has a significant influence.

This lecture provides a basis for the assessment of existing structures including reliability differentiation, modification of partial factors and upgrading of basic variables based on new information. The assessment of existing structures is briefly shown on two selected examples analysed at the Klokner Institute. The first example, of an existing school building, illustrates the need for a complex approach in the assessment of construction works in cases where the structural failures result from a combination of various errors in the project and execution, due to insufficient control or total lack of control. The second example, of a detailed assessment of a structural member, shows the application of probabilistic methods for the determination of structural reliability.

It is shown that the application of new international documents based on the principles of probabilistic methods of the theory of structural reliability make it possible to achieve the required reliability level of a repaired or upgraded existing structure for its intended working life.

**Souhrn**

Efektivní využívání existujících staveb přináší společnosti a ekonomii každého státu celou řadu výhod a přispívá k zachování hodnot historického a kulturního charakteru. Metody hodnocení existujících konstrukcí založené na pravděpodobnostních zásadách teorie spolehlivosti umožňují racionálně rozhodnout o nejvhodnějším způsobu využívání existující stavby nebo o její případné obnově.

Pro hodnocení existujících staveb není dosud k dispozici žádný evropský předpis, neboť Eurokódy jsou převážně určeny pro navrhování nových konstrukcí. Mezinárodní norma ISO 13822 pro hodnocení existujících konstrukcí je proto důležitým, avšak zatím dosti obecným dokumentem. Praktické uplatnění této normy v ČR umožňuje šest národních příloh připravených na ČVUT v Praze.

Pro hodnocení spolehlivosti existujících konstrukcí nebo pro navrhování jejich obnov se stejně jako při navrhování nových konstrukcí používá metoda dílčích součinitelů. Stále větší význam však dnes nabývají pravděpodobnostní metody teorie spolehlivosti. Zásady pravděpodobnostních metod se mohou s výhodou uplatňovat pro diferenciaci spolehlivosti existujících konstrukcí, u kterých je důležité hledisko ekonomické a sociální i aspekty trvalé udržitelnosti. Pravděpodobnostní koncepce lze využít při stanovení dílčích součinitelů základních veličin jak u návrhu nových, tak také pro ověřování spolehlivosti existujících konstrukcí. Praktické aplikace však musí vycházet z požadované úrovně spolehlivosti a vhodných teoretických modelů základních veličin. Důležité jsou při tom znalosti o skutečných vlastnostech existujících materiálů, o historii průběhu zatížení a nepříznivých účincích prostředí. Významný vliv má jakost provádění a údržba stavby během používání.

Přednáška uvádí zásady hodnocení existujících konstrukcí včetně možnosti diferenciace spolehlivosti, úprav dílčích součinitelů a aktualizace základních veličin na základě nových dat. Problematika hodnocení existujících konstrukcí je ilustrována na dvou vybraných příkladech analyzovaných v Kloknerově ústavu. Příklad existující školní budovy ukazuje potřebu komplexního přístupu k hodnocení stavby všude tam, kde je zdrojem poruch kombinace řady chyb v projektu i provádění umožněná nedostatečnou nebo zcela chybějící kontrolou. Příklad podrobného průzkumu existujících balkónů panelových domů ilustruje použití pravděpodobnostních metod pro zjištění úrovně spolehlivosti konstrukce.

Ukazuje se, že uplatnění nových postupů hodnocení existujících konstrukcí založených na zásadách pravděpodobnostních metod přispívá k dosažení požadované úrovně spolehlivosti obnovované existující konstrukce po dobu její plánované životnosti.

**Klíčová slova**

existující konstrukce, hodnocení, obnova, ukazatele spolehlivosti, diferenciace spolehlivosti

**Keywords**

existing structures, assessment, rehabilitation, reliability indices, reliability differentiation


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**Contents** **Page**

# 1 INTRODUCTION

The continued use of existing structures is of a great importance for each country, and has significant economic, social and cultural impacts. Many new factories, buildings and bridges were built in the Czech Republic and in other European countries in the 1960s. Therefore, many construction works, including panel houses, are now reaching the end of their working life, and require assessment and rehabilitation to assure their further continued safe and economical exploitation. Figure 1.1 shows the time period of road bridge completion in selected countries, indicating the need for their rehabilitation [1].

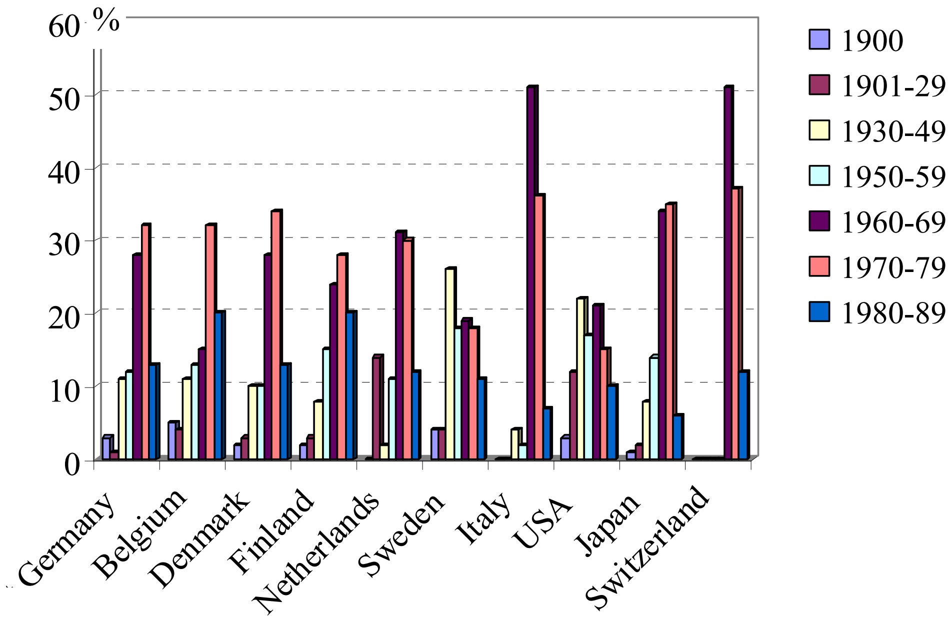



Fig. 1.1 The time period of road bridge completion [1].

The approach to assessment and life extension of existing construction works is a complex process, and a wide range of considerations should be taken into account. The effects of the process of execution and the subsequent working life of the structure, during which it may deteriorated due to environmental influences, the effects of abnormal actions, misuse or other adverse events, have to be taken into account. The level of knowledge achieved in the process of investigation always incorporates some uncertainties concerning the behaviour of the basic variables. For an integrated assessment of existing structures only general recommendations are currently given in prescriptive documents.

The assessment of an existing structure differs in many aspects from procedures taken during the design of a new structure and may require the application of sophisticated methods, in many cases beyond the scope of common standards for structural design. The prescriptive documents cannot be directly applied for the assessment, as the actual state of the structure and its materials must be considered. Moreover, the current standards have often more severe requirements than the standards that were applied at the time when the structure was originally designed. Although some existing structures appear to have a lower reliability level than is presently required for new structures, they may continue to fulfil the required functions for many years.

This begs the question how to specify a suitable reliability level for an existing structure when it is to be repaired or upgraded. For the optimal decision on structural rehabilitation, the principles of the theory of structural reliability, risk assessment and cost-benefit analysis may be applied.

Fig. 1.2 illustrates the percentage of new construction works and rehabilitations between 2001 to 2005 based on information supplied by the Ministry of Industry and Trade of the Czech Republic. Approximately a quarter of all outputs in the Czech construction industry are based on rehabilitations of construction works.

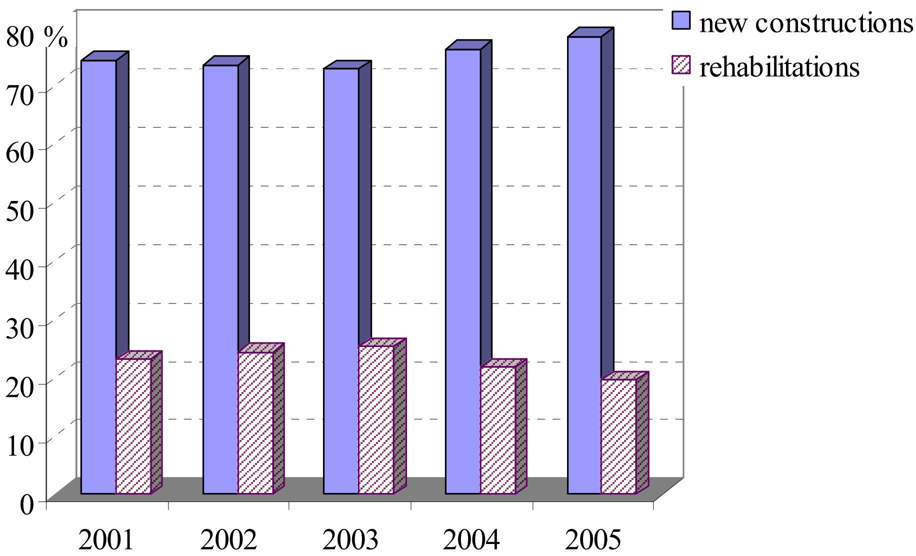



Fig. 1.2 Proportion of new construction works and rehabilitations.

It is expected that within a short time period only EN Eurocodes will be applied for structural design in all Europe. However, although the scope of the key Eurocode EN 1990 [2] for the basis of structural design enables its provisions to be used for structural appraisal of existing construction works, for the design of repairs and alterations or for assessing changes of use, there are no operational rules. A new informative annex to EN 1990 [2] for the assessment of existing structures will now be developed in the Technical committee of CEN/TC 250.

For the assessment of construction works, Czech standard ČSN 73 0038 [3] was developed as one of small number of prescriptive documents for structural appraisal in the world. It may be mentioned here that this standard was used as a background document during the development of international document ISO 13822 [4] for the assessment of existing structures. Although the national standard [3] was a very useful document, the required links to Eurocodes, and also to ISO standards were missing, in particular links to the new ISO 13822 [4]. Therefore, it was decided to implement ISO 13822 [4] into the system of Czech standards and to incorporate selected information from [3] into six National annexes of [4] in co-operation of the Klokner Institute and the Faculty of Civil Engineering CTU in Prague with the Technical and Testing Institute (TAZÚS).

Besides the above mentioned standards, other European or international prescriptive documents may be used for the assessment of structures. Some basic information concerning existing structures is given in ISO 2394 [5] and the principles of statistical evaluation are provided in ISO 2854 [6]. For assessment and reinforcement of structures in seismic situations, EN 1998-3 [7] may be applied.

## 2 BASIC TERMINOLOGY

Several new terms are provided in ISO 13822 [4] (e.g. rehabilitation, safety plan, utilisation plan, residual lifetime), while some useful terms applied in the original Czech standards are no longer used (reconstruction, defect). Selected terminology is explained in the National annex of [4], in order to prevent misunderstanding. The links among terms are illustrated in Fig. 2.1.

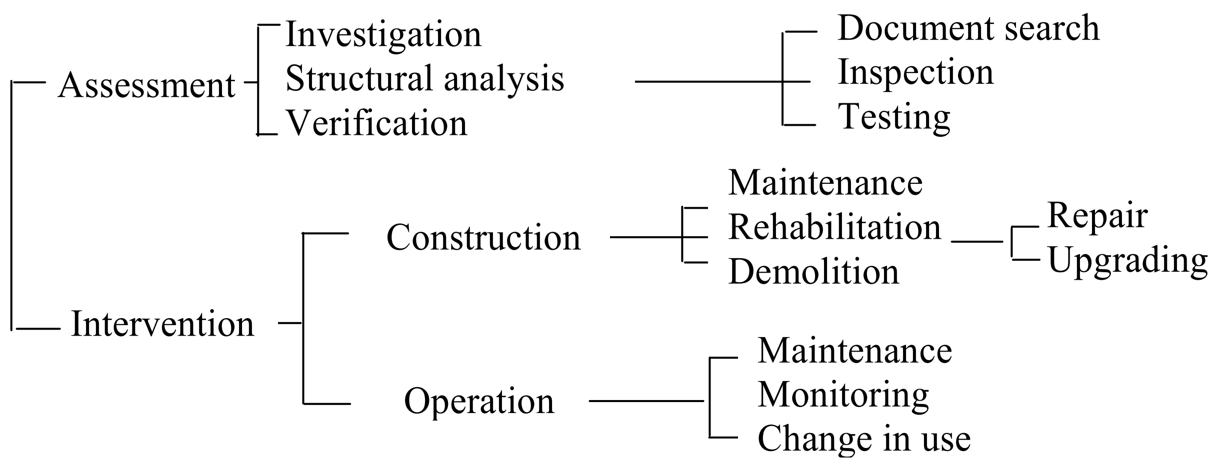



Fig. 2.1 Hierarchy of terms used for structural assessment.

## 3 GENERAL PROCEDURE FOR STRUCTURAL ASSESSMENT

The general procedure for structural assessment may be found in prescriptive documents [3, 4] or in specific technical conditions developed e.g. for industrial devices and power plants [20]. The handbook [8] prepared within the Joint Committee on Structural Safety (JCSS), an international scientific organisation, also provides guidance for structural assessment.

## 3.1 General aspects

The following circumstances may lead to a reliability assessment of an existing structure including

- planned change of exploitation or extension of working life
- reliability verification demanded by owners, insurance companies or responsible authorities
- repair of an existing structure deteriorated e.g. due to time dependent environmental effects or affected by accidental actions
- doubts concerning the actual reliability of the structure (presence of damage, clearly inadequate serviceability).

The reliability assessment of an existing structure focuses on making sure that the structure will perform safely over a specified remaining working life. The assessment is based on verification of the structural resistance, taking into account the actual properties of the structural materials, the geometry and action effects. A visual inspection is an important tool facilitating decisions concerning an existing structure. In some cases, continuous observation of important parameters affecting the overall behaviour of the structure, such as settlement, is also used.

For the assessment of structures, the currently valid codes for verification of structural reliability need to be applied, while the standards valid in the period when the structure was designed are considered as guidance documents only. As a rule, the assessment need not to be performed for those parts of existing structure that would be not affected by structural changes or which are not obviously damaged or not suspected of having an insufficient reliability level.

One important requirement should be kept in mind during the assessment, though it is not explicitly given in ISO 13822 [4]: to preserve the appearance and materials of historical structures.

## 3.2 Basic steps of assessment

The assessment procedure for an existing structure consists of the following steps indicated in the flow-chart given in Annex A:

- specification of the assessment objectives
- scenarios related to structural condition and actions
- preliminary assessment (study of available documentation, preliminary inspection, decision on immediate actions, recommendation for detailed assessment)
- detailed assessment (detailed documentary search, detailed inspection, material testing, determination of actions and structural properties, verification of structural reliability)
- a report, including a proposal for construction intervention
- repetition of the sequence, if needed.

When the preliminary assessment indicates that the structure is reliable for its intended use over the remaining life, a detailed assessment may not be required. However, if the structure seems to be in a dangerous condition, a detailed assessment and immediate intervention may have to be undertaken.

At the end of the assessment, the cost considerations have to be taken into account, including social and historic aspects. If structural repairs are needed, the costs and risks associated with each of the interventions should be estimated.

# 4 RELIABILITY VERIFICATION OF EXISTING STRUCTURES

## 4.1 Methods for structural verification

The design of existing structures is as a rule based on different approaches given in 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 the basic variables, the rules for load combinations and analytical models commonly differ from the provisions given in current standards. For analyses of structures, Eurocodes offer the most advanced partial factor method, based on the principles of the theory of structural reliability. However, it is well known that the requirements of new European standards on actions are often more severe than the provisions of the original national codes.

The partial factor method is generally applied for the reliability assessment or rehabilitation of existing structures, as well as for the design of new structures. The probabilistic methods of the theory of structural reliability are at the present time becoming more and more significant. The principles of probabilistic methods may be applied for the reliability differentiation of existing structures, where particularly economic, social and sustainability considerations are of great importance.

## 4.2 Partial factor method

The partial factor method is the basic method given in new European and international standards. The partial factors for actions and material properties are recommended on the basis of calibrations and good practice in construction. Probabilistic methods, including the FORM method may be applied for determining the values of partial factors for actions and material properties based on the required reliability level of structures [11, 12]. In common cases for the 50-year design working life of structures, the recommended target reliability index is $\beta_t = 3{,}8$ for the ultimate limit states. In case where some other reliability level for existing structures may be required, the partial factors can be appropriately adjusted.

### 4.3 Probabilistic methods

Probabilistic methods may be in specific cases used with advantage for verification of existing structures or for the assessment of residual lifetime, when appropriate probabilistic models of the basic variables are known. General guidance for the probabilistic verification of structures including models of basic variables, are given in the JCSS Probabilistic Model Code [9].

For the probabilistic assessment of existing structures, the limit state function g(***X***) should be specified for the vector ***X*** of basic variables. The variables entering the limit state function are random variables or random fields. Their characteristics may be described by means of probability theory and mathematical statistics. It is assumed that the structure is reliable for inequality g(***X***) > 0. The reliability indicator, the failure probability $P_\mathrm{f}$ is given as

$$P_\mathrm{f} = \int_{\mathrm{g}(X)<0} \varphi_X(\boldsymbol{x})\mathrm{dx} \tag{4.1}$$

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 $\beta$, defined on the basis of the failure probability $P_\mathrm{f}$, given as

$$\beta = -\Phi^{-1}(P_\mathrm{f}) \tag{4.2}$$

where $\Phi$ is the standardised normal distribution function. The failure probability $P_\mathrm{f}$ and reliability index $\beta$ are equivalent indicators, their relationship is indicated in Table 4.1 [2, 10].

Table 4.1 Relationship between the reliability index $\beta$ and failure probability $P_\mathrm{f}$.

| $P_\mathrm{f}$ | $10^{-1}$ | $10^{-2}$ | $10^{-3}$ | $10^{-4}$ | $10^{-5}$ | $10^{-6}$ | $10^{-7}$ |
|---|---|---|---|---|---|---|---|
| $\beta$ | 1,3 | 2,3 | 3,1 | 3,7 | 4,2 | 4,7 | 5,2 |

The failure probability $P_\mathrm{f}$ or reliability index $\beta$ of the structural member is compared with the target value of probability $P_\mathrm{f,t}$ or reliability index $\beta_\mathrm{t}$. The existing structure is considered to be reliable for the condition

$$P_\mathrm{f} < P_\mathrm{f,t} \text{ or } \beta > \beta_\mathrm{t} \tag{4.3}$$

Several methods may be applied for an analysis of failure probability (e.g. direct integration, analytical or simulation methods, combinations of methods). General procedures for determination of failure probability are given in [11,12,13] and some software products are available e.g. Comrel, Diana [10].

The same target reliability level may be required for existing structures as for new structures. It should be mentioned here that for existing structures not only the safety aspects are important. Social and economic criteria might also influence the decision on the required reliability level.

Sustainability considerations are more important for the assessment of existing structures than for the design of new structures. Selected differences in the evaluation of functional criteria on structural reliability considering new and existing structures are indicated in Table 4.2. The goal of “minimum structural intervention” is as a rule applied for most existing structures when existing materials are used as much as possible in structural rehabilitations. The actual properties of the materials and geometry should be evaluated.

Tab. 4.2 Different viewpoints in the evaluation of functional criteria.

| Viewpoints | Existing structures | New structures |
|---|---|---|
| Economic | commonly significant incremental costs for reliability enhancement | commonly lower incremental costs for reliability enhancement |
| Social | may be significant in the case of restriction or exclusion of exploitation and preservation of cultural heritage | commonly less significant than for existing structures |
| Sustainability | existing building materials would be preferably used (leading to reduction of waste materials) | commonly new materials are used |

The target values of indices $\beta_t$ and associated failure probability $P_{f,t}$ (in brackets) for one-year reference period recommended in [9] are given in Table 4.3.

Table 4.3 The target reliability index $\beta_t$ (and associated failure probability $P_{f,t}$) for a one-year reference period and ultimate limit states.

| Relative costs of safety measures | Minor failure consequences | Moderate failure consequences | Large failure consequences |
|---|---|---|---|
| Large | $\beta_t = 3,1$ ($P_f \approx 10^{-3}$) | $\beta_t = 3,3$ ($P_f \approx 5\times10^{-4}$) | $\beta_t = 3,7$ ($P_f \approx 10^{-4}$) |
| Normal | $\beta_t = 3,7$ ($P_f \approx 10^{-4}$) | $\beta_t = 4,2$ ($P_f \approx 10^{-5}$) | $\beta_t = 4,4$ ($P_f \approx 5\times10^{-6}$) |
| Small | $\beta_t = 4,2$ ($P_f \approx 10^{-5}$) | $\beta_t = 4,4$ ($P_f \approx 5\times10^{-5}$) | $\beta_t = 4,7$ ($P_f \approx 10^{-6}$) |

# 5 PARTIAL FACTORS FOR REQUIRED RELIABILITY LEVEL

## 5.1 General

The concept of design values may be used for determining the partial factors of the basic variables applied in the verification of existing structures. The partial factors $\gamma_i$ for basic variables $X_i$ having an adverse effect on structural reliability (action effects) may be determined as

$$\gamma_i = x_{id} / x_{ik} \tag{5.1}$$

and factors having a favourable effect on structural reliability (resistance) as

$$\gamma_i = x_{ik} / x_{id} \tag{5.2}$$

where the values of partial factors $\gamma_i$ are commonly greater than one.

The procedure for applying partial factors based on expressions (5.1) and (5.2) for the reliability assessment of existing structures is described as follows.

## 5.2 Partial factors for resistance

The structural resistance $R$ (e.g. yield strength of steel) may often be described by two-parametric lognormal distribution. The characteristic value of resistance is commonly defined as 5 % lower fractile. The partial factor $\gamma_R$ is according to expression (5.2) given as

$$\gamma_R = \exp(-\,1{,}645\ V_R)/\exp(-\ \alpha_R\ \beta\ V_R) \tag{5.3}$$

where $V_R$ is the coefficient of variation for resistance and $\beta$ the required reliability index (e.g. target value), the coefficient 1,645 is the value of a 5 % fractile of the standardised normal distribution and the sensitivity factor $\alpha_R = 0{,}8$ [2, 10]. The partial factor for resistance $\gamma_R$ versus the reliability index $\beta$ for three values (0,05; 0,10; 0,15) of the coefficient of variation $V_R$ is shown in Fig. 5.1.

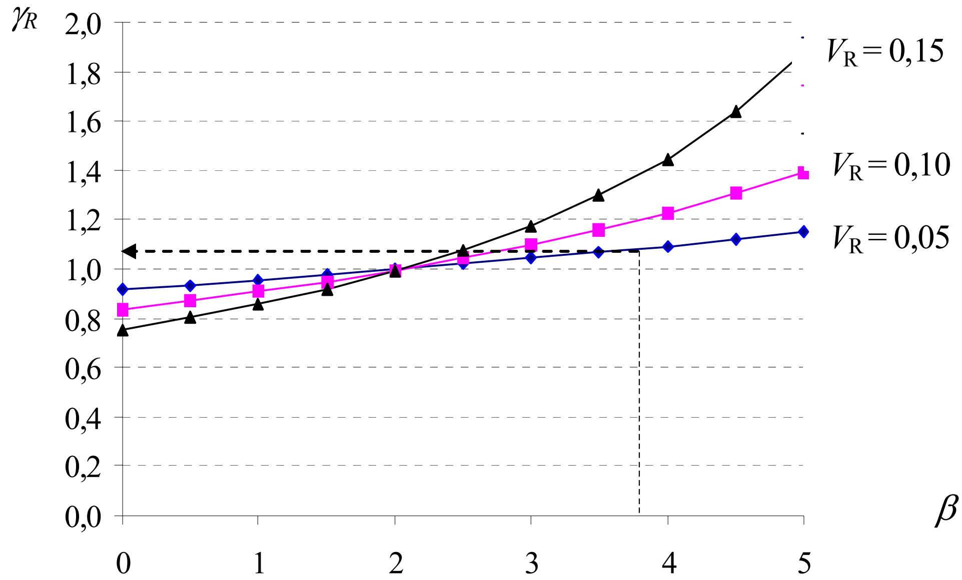



Fig. 5.1 The partial factor for resistance $\gamma_R$ versus reliability index $\beta$.

Fig. 5.1 indicates that for the recommended reliability index $\beta$ = 3,8 and coefficient of variation $V_R$ = 0,05 the partial factor for resistance should be about $\gamma_R$ = 1,1. However, direct application of the results illustrated in Fig. 5.1 may be rather difficult because the characteristic value of the material strength of a construction material like steel corresponds in reality to lower probability than 5 % given in EN 1990 [2].

For reliability assessment of existing structures, the above introduced procedure may be applied particularly when the partial factors are based on test results of the relevant properties of construction materials.

## 5.3 Partial factors for actions

### 5.3.1 Permanent load

The normal distribution is commonly applied for permanent loads. In many cases, the characteristic value $G_k$ may be considered by the mean $\mu_G$. The design value of permanent load $G_d$ may be determined as

$$G_d = \mu_G - \alpha_G \beta \sigma_G = \mu_G (1 + 0{,}7\beta V_G) \tag{5.4}$$

where $\mu_G$ denotes the mean, $\sigma_G$ the standard deviation, $V_G$ the coefficient of variation and $\alpha_G$ the sensitivity factor [10, 12]. The partial factor $\gamma_G$ of the permanent action $G$ is given as

$$\gamma_G = G_d / G_k = \mu_G (1 - \alpha_G \beta V_G) / \mu_G = 1 - \alpha_G \beta V_G \tag{5.5}$$

where the sensitivity factor $\alpha_G = -0{,}7$, the target reliability index $\beta_t = 3{,}8$ and the coefficient of variation $V_G = 0{,}1$ are assumed [10]. The factor $\gamma_G$ may be determined for the fifty-year design working life as

$$\gamma_G = 1 + 0{,}7\,\beta V_G = 1 + 0{,}7 \times 3{,}8 \times 0{,}1 = 1{,}27 \tag{5.6}$$

The value of partial factor $\gamma_G = 1{,}27$ is a little lower than recommended in EN 1990 [2] where also the material uncertainties are included. The influence of the reliability index $\beta$ on the value of partial factor $\gamma_G$ is shown in Fig. 5.2, considering 5 % and 10 % values of the coefficient of variation $V_G$.

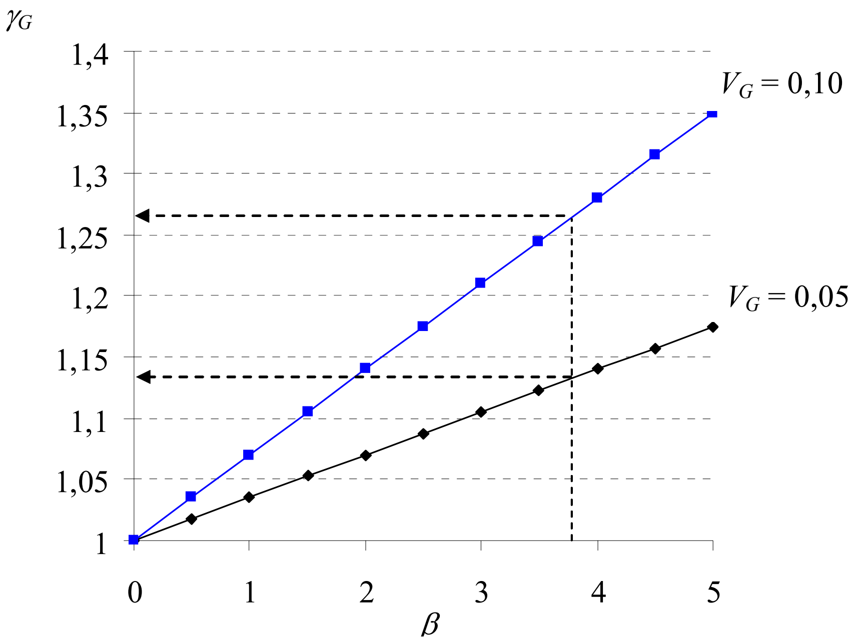



Fig. 5.2 The partial factor $\gamma_G$ of permanent action versus the reliability index $\beta$ for 5 % and 10 % coefficient of variation $V_G$.

### 5.3.2 Variable actions

To determine the design values for variable actions, the characteristic values $Q_k$ of actions are commonly used as a basis, and partial factors $\gamma_Q$ are applied according to the recommendations of the standards [4, 14, 19]. For climatic actions, more detailed information may be obtained for the site location from the Czech Hydrometeorological Institute, which may determine the characteristic value of climatic action for a required return period. Direct assessment of the design value $Q_d$ of variable action or partial factor $\gamma_Q$ may be influenced by statistical uncertainties concerning the type of probabilistic distribution and its relevant statistical parameters. Therefore, this procedure is not commonly used. The value of partial factor $\gamma_Q$ for variable action $Q$ can be determined on the basis of a selected probabilistic model for the considered action. For example, for the same mean of variable action and the coefficient of variation $V_Q$ = 0,4, the partial factor $\gamma_Q$ is given under the assumption of the type of probabilistic distribution in a relatively large interval $1{,}27 \leq \gamma_Q \leq 1{,}46$ (for common reliability class RC2). Thus, when other information is not available, the recommended partial factor $\gamma_Q$ = 1,5 for variable action should rather be applied in the structural assessment. In case that the reliability differentiation of structures is taken into account, the partial factor $\gamma_Q$ may be selected in a range from 1,35 to 1,65. It should be noted that model uncertainties are not considered here.

The variable action $Q$ may often be described by the Gumbel distribution, e.g. climatic actions or the sustainable component of the imposed load. The characteristic value of action is defined in Eurocodes as the upper fractile of the probabilistic distribution for the basic time period corresponding to the probability of 2 %. The partial factor $\gamma_Q$ of a variable action may be determined according to [10] as

$$\gamma_Q = \frac{1 - V_Q\{0{,}45 - 0{,}78\ln N + 0{,}78\ln[-\ln(\Phi(-\alpha_Q\,\beta))]\}}{1 - V_Q[0{,}45 + 0{,}78\ln(-(\ln 0{,}98))]} \qquad (5.7)$$

where $V_Q$ denotes the coefficient of variation for the basic time interval and $N$ is the number of expected changes of load intensity during the assumed design working life of structure. For imposed loads, the sustained and intermittent components are distinguished regarding time variation. The probabilistic models for different categories of imposed loads are described in the Probabilistic Model Code [9].

In the following study of partial factors, three categories of use are considered: residential areas (category A), office areas (category B) and school areas (category C). The occurrence rate $\lambda$ of sustained load changes is recommended in [9] for assumed categories of use as $\lambda_A = 10$, $\lambda_B = 7$ and $\lambda_C = 5$. For imposed load the number $N_i$ of changes in the fifty-year design working life is specified as $N_A = 5$ (residential areas), $N_B = 7$ (office areas) and $N_C = 10$ (school areas).

The partial factor $\gamma_Q$ versus the coefficient of variation $V_Q$ of an imposed load of category B, taking into account three reliability classes of structures RC1 to RC3, is illustrated in Fig. 5.3.

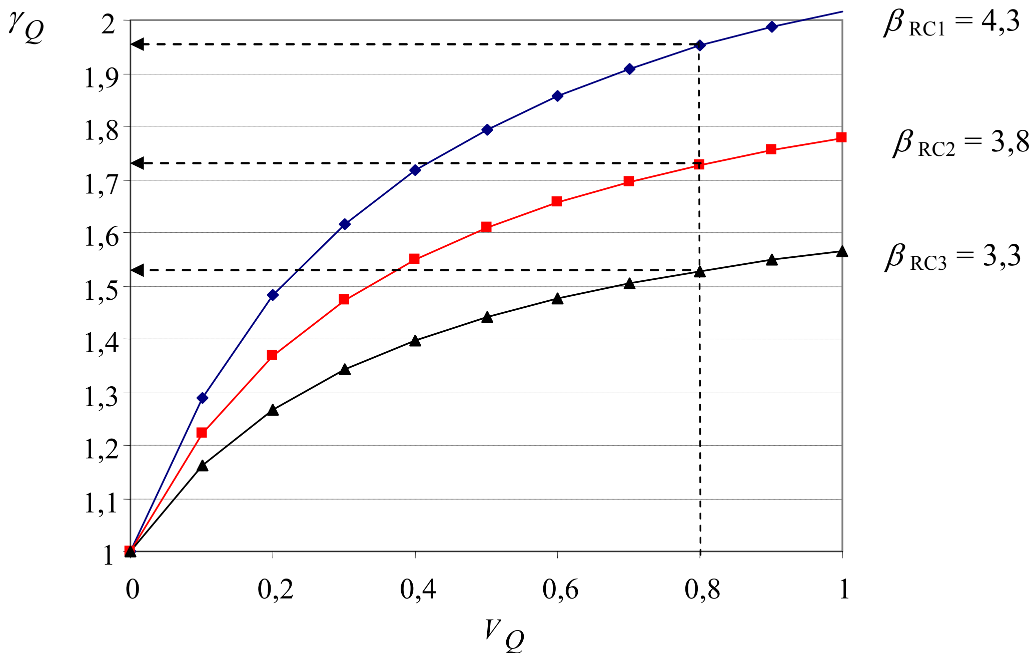



Fig. 5.3 The partial factor $\gamma_Q$ of variable action versus the coefficient of variation $V_Q$ for three reliability classes RC1 to RC3 (in office areas).

The theoretical values of partial factor $\gamma_Q$ versus the coefficient of variation $V_Q$ for expected changes of load $N$ in considered three categories of use A to C are illustrated in Fig. 5.4.

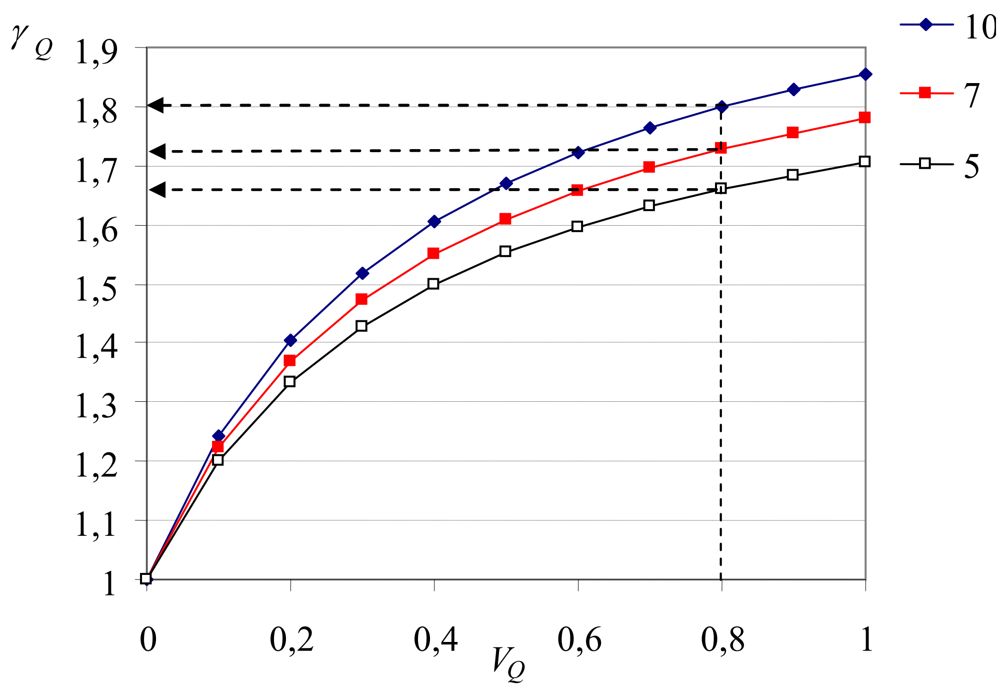



Fig. 5.4 The partial factor $\gamma_Q$ of imposed load $Q$ versus the coefficient of variation $V_Q$ for assumed load changes $N$ (categories of use A to C).

It follows from Fig. 5.4 that for the considered reliability index $\beta = 3{,}8$ and the coefficient of variation in about $V_Q \approx 0{,}8$, the partial factor $\gamma_Q$ varies from 1,67 to 1,8 for the three assumed categories of use A to C. It may be noted that they represent the theoretical values only under the assumption of a Gumbel distribution and, in some cases, more appropriate probabilistic models may have to be used. Moreover, the characteristic values given in standards are recommended greater than those determined for the 0,98 fractile. For example, EN 1991-1-1 [24] recommends for office areas a common range of characteristics values in the interval from 2 to 3 kN/m$^2$ while the theoretical 0,98 fractile is lower, approximately from 2,1 to 2,3 kN/m$^2$. This traditional approach may lead to the reduction of theoretical values of partial factors for imposed loads to $\gamma_Q = 1{,}5$.

# 6 UPDATING OF RELIABILITY ESTIMATES

## 6.1 Procedures for updating

The properties of basic variables and reliability estimates of the structure may be updated on the basis of an investigation. The following procedures may be used:

(a) direct updating of the structural failure probability
(b) updating of the probability distribution of the basic variables.

(a) Direct updating of failure probability is based on the basic formula of probability theory [10]

$$\mathrm{P}(F|I) = \frac{\mathrm{P}(F \cap I)}{\mathrm{P}(I)} \tag{6.1}$$

where P denotes probability, $F$ local or global failure, $I$ inspection information, $\cap$ intersection of two events, and | means conditional upon. The inspection information $I$ may consist e.g. of the result of measurements of existing structural member deflection or crack width.

(b) The updating procedure of a probability distribution is given as:

$$\mathrm{f}_X(x|I) = C\,\mathrm{P}(I|x)\,\mathrm{f}_X(x) \tag{6.2}$$

where $\mathrm{f}_X(x|I)$ denotes the probability density function of a basic variable or statistical parameter $X$ after updating by inspection information $I$, $\mathrm{f}_X(x)$ the probability density function of $X$ before updating, $C$ normalising constant, and $\mathrm{P}(I|x)$ likelihood function of finding information $I$ for the given value $x$ of $X$. An illustration of prior $\mathrm{f}_X(x)$ and updated $\mathrm{f}_X(x|I)$ probability density based on formula (6.2) is shown in Fig. 6.1.

In the example illustrated in Fig. 6.1 the updating leads to a more favourable distribution with a greater updated design value $x_d$ than the prior value. It should be mentioned here that the design value of the updated distribution may also be lower than the design value of the prior distribution.

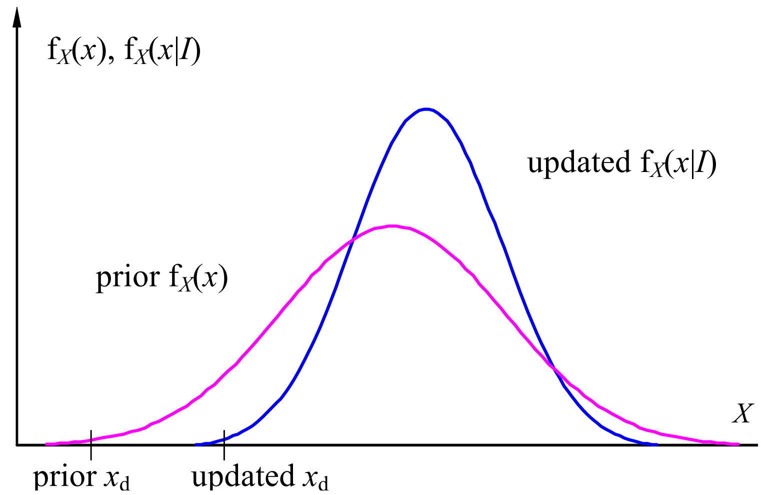



Fig. 6.1. Prior and updated probability density for variable $X$.

## 6.2 Updating of failure probability

For the updated distributions of basic variables $f_X(x)$, the updated failure probability $P(F|I)$ in procedure (a) may be determined by performing a probabilistic analysis using a common procedure given as

$$P(F|I) = \int_{g(x<0)} f_X(x\,|\,I)\mathrm{d}x \tag{6.3}$$

where $f_X(x|I)$ denotes the updated probability density function, $g(x)$ the limit state function and condition $g(x) < 0$ the failure domain.

The existing structure will be considered as reliable if the probability $P(F|I)$, given the design values for its basic variables, should not exceed a specified target value.

## 6.3 Updating of characteristic and design values

The updating procedure for probability distribution (b) can be applied for determining the updated fractiles of the basic variables (characteristic or design values). For determining the updated fractiles, the Bayesian methods may be applied [10].

The design values $x_d$ for each basic variable $X$ may also be updated using formulae specified in [8] for the appropriate type of probability distribution. For normal and lognormal distribution of $X$, the design value of variable is given as

$$x_d = \mu(1 - \alpha\beta V) \tag{6.4}$$

$$x_d = \mu\exp\left(-\alpha\beta\sigma - 0{,}5\sigma^2\right) \tag{6.5}$$

where $\alpha$ is the sensitivity factor, $\beta$ target reliability index, $V$ updated coefficient of variation and variance $\sigma^2 = \ln(1+V^2)$. The characteristic value $x_k$ is given as

$$x_k = \mu(1 - kV) \tag{6.6}$$

$$x_k = \mu \exp\left(-k\sigma - 0{,}5\sigma^2\right) \tag{6.7}$$

where $k$ = 1,64 is usually used as the 5 % fractile of the standardised normal distribution. If the updated characteristic values are determined only and the partial factors recommended in Eurocodes are applied, this procedure may lead to somewhat conservative results.

## 6.4 Estimation of structural condition based on new information

This example deals with the probability estimation that existing balconies (presented in Chapter 7) may have a reinforcement concrete cover less than 0,01 m and could be endangered by carbonation and subsequent corrosion. The task is to specify updated (posterior) probabilities $p_i'' = \mathrm{P}(B_i|A)$ of the event $B_i$ (hypothesis) updated with respect to the result of a new inspection (event A). The updated probability is based on the relationship given as

$$p_i'' = \frac{p_i' l_i}{\sum_j p_j' l_j} \tag{6.8}$$

where $l_i$ is the likelihood of an adverse event $i$. It is known from previous inspection of selected balconies that the prior probability of event $B_1$ is $p_1' = \mathrm{P}(B_1) = 0{,}05$ given that the actual reinforcement concrete cover is less than 0,01 m, and the updated probability of event $p_2' = \mathrm{P}(B_2) = 0{,}95$ given the actual concrete cover is greater than 0,01 m. During subsequent inspection of balconies, new experimental tests were carried out, using more precise instruments. The new tests revealed that the likelihood of event $B_1$ is $l_1 \propto \mathrm{P}(A|B_1) = 0{,}15$ and the likelihood of event $B_2$ is $l_2 \propto \mathrm{P}(A|B_2) = 0{,}85$. The updated (posterior) probabilities are determined from relationship (6.8) as follow

$$p_1'' = \frac{p_1' l_1}{\sum_{j=1}^{2} p_j' l_j} = \frac{0{,}05 \times 0{,}15}{0{,}05 \times 0{,}15 + 0{,}95 \times 0{,}85} = 0{,}009 \tag{6.9}$$

$$p_2'' = \frac{p_2' l_2}{\sum_{j=1}^{2} p_j' l_j} = \frac{0{,}95 \times 0{,}85}{0{,}05 \times 0{,}15 + 0{,}95 \times 0{,}85} = 0{,}991 \tag{6.10}$$

The updated probability distribution $p_i''$ is therefore more favourable than the prior probability distribution $p_i'$ indicating that most balconies may have greater durability than it was firstly assumed.

# 7 EXAMPLES OF RELIABILITY ASSESSMENT

The Klokner Institute has long-term experience in reliability assessment of various types of existing structures including power plants, cooling towers and buildings damaged in various accidental situations, including the floods in

Bohemia in 2002. The basic requirements for resistance, serviceability and durability of structures have to be fulfilled for existing structures. The structures should have also adequate robustness to sustain adverse events such as explosions or accidental impacts [2, 23].

An example of a building having inadequate structural robustness is shown in Fig. 7.1, where missing ring ties and low spatial stiffness caused progressive collapse of a considerable part of a structure affected by a gas explosion. Fig. 7.2 illustrates the localised damage of the building due to the gas explosion where basic structural measures were provided.

Fig. 7.1 Progressive collapse.

Fig. 7.2 Localised failure.

Two selected examples of reliability assessment of existing structures, described in detail in [15, 24], will be characterised briefly. Firstly, the example of a Prague primary school, where various gross human errors and inadequate quality control in design and also in the process of execution caused serious financial losses and restrictions in use of the school building. Secondly, reliability assessments of the balconies in panel houses, based on detailed inspection.

### 7.1 Reliability assessment of a primary school

A new public school built in the 1990s in Prague 6 to replace a three-year old school building that had burn out while the roof was being repaired provides an example of a reliability assessment. The new school consists of three separate parts: the main four-storey building, a sports hall and dining hall [24].

### 7.1.1 Observed defects

The reliability assessment of the primary school was a demanding task, as many different faults were detected during investigation:

- inadequate design, e.g. inadequate simplifications of models during structural analysis, ill-considered serviceability requirements, insufficient design documentation without appropriate detailing
- errors during execution, e.g. incorrect changes of floor layers, various changes of construction materials, heavier composition of terrace layers, poor quality of workmanship due to lack of supervision
- insufficient control during execution, and consequently a tendency of the state owner to overlook the defects.

Many problems were caused by heating malfunctions, and by faults in the insulation and plumbing. The composition of the floor layers of the sports hall was changed during execution and the steam-proof insulation was omitted, although the school kitchen was located directly underneath. Therefore, the parquet floor was soon significantly distorted due to the humidity of the school kitchen.

Visible deflections and cracks were observed in the reinforced concrete slabs of the dining hall due to the inadequate area of reinforcement specified in the design and also due to the heavier composition of the terrace layers over the slabs. A detailed structural analysis proved that both the ultimate and serviceability limit states were not fulfilled. Additional supporting columns therefore, had to be supplemented to reinforce the structure of the dining hall.

Another defect involved the gradual damage to the protective layers against atmospheric corrosion of the steel load-bearing structure of the separate sports hall building, which started to flake off in a short time. The Klokner Institute proposed protective measures to enhance the durability of the structure.

Further damage was observed one year after the school had been completed [24]. Cracking in the floors, partition walls and cladding components of the main building were detected, accompanied by malfunctioning doors and flaking plaster, as shown in Figures 7.3 and 7.4.

### 7.1.2 Assessment of the structure

Verification of the main school building showed that the load-bearing structure fulfilled the requirements for the ultimate limit states, but that the reinforced concrete slabs were insufficiently rigid. The cracking of the partition walls and facing was due to excessive deflections of the reinforced concrete slabs, which did not comply with the serviceability requirements.

It was obvious that most of the defects would not have appeared if adequate care had been given to appropriate detailing in the project, to serviceability and durability requirements, as well as to quality assurance during execution.

Fig. 7.3 Damaged plasters in the classroom.

Fig. 7.4 Cracks in partition walls

## 7.2 Reliability assessment of existing balconies

The Klokner Institute investigated more than two hundred balconies of panel buildings assembled from the T0-6B-BTS construction system [15]. The poor quality of the concrete as well as inappropriate detailing enhanced the deterioration effects and contributed to the dangerous state of the balconies.

### 7.2.1 Load-bearing balcony components

The cross-section of a balcony is shown in Fig. 7.5, while Fig. 7.6 illustrates deterioration effects on a selected balcony above a building doorway.

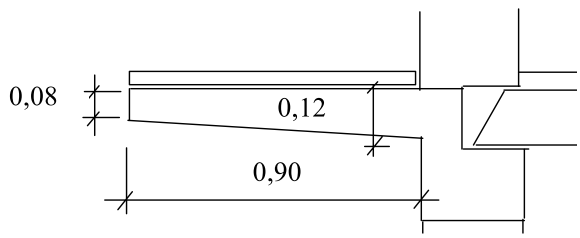



Fig. 7.5 Balcony cross-section in m.


Fig. 7.6 Deterioration of balcony.

### 7.2.2 Inspection and material testing

A detailed inspection, including verification of geometrical properties and material tests, was carried out to assess the actual state of existing balconies. The investigation included non-destructive testing of concrete compressive strength, supplemented by tests of selected specimens, measurements of reinforcement position and depth of concrete carbonation. The actual spatial location of the reinforcement in the precast components did not comply with the original design.

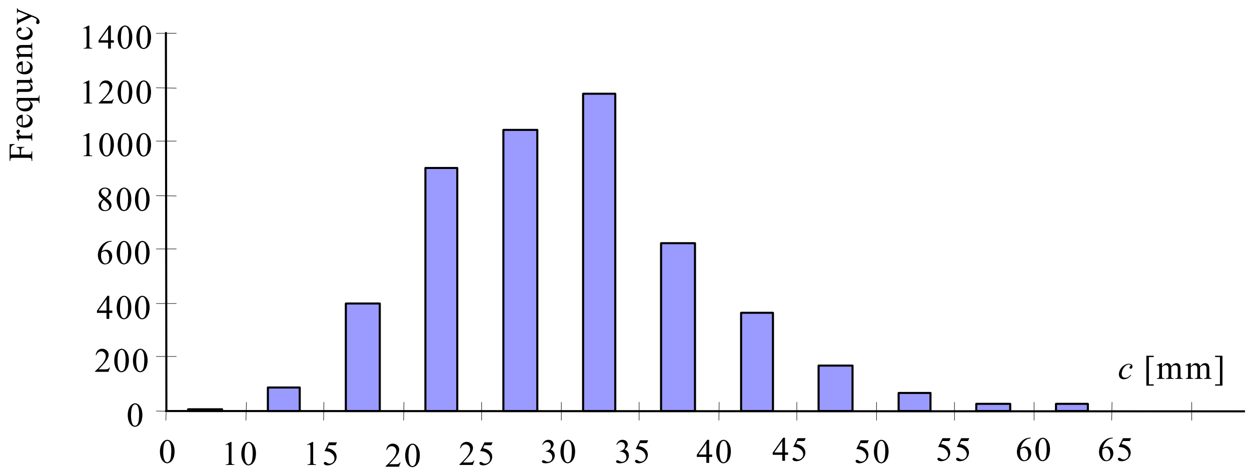



Fig. 7.7 Distribution of concrete cover $c$ for 230 balconies (4890 measurements, mean $\mu$ = 0,026 m, standard deviation $\sigma$ = 0,009 m, skewness $\alpha$ = 0,58).

The measurements were statistically evaluated for each balcony, for each panel building and also for the total number of balconies. The distribution of reinforcement concrete cover $c$ for all 4890 measurements is shown in Fig. 7.7. The concrete cover varied from 0,002 to 0,065 m, on many balconies not providing adequate protection against adverse environmental influences.

The design distance of the bars is 0,15 m. However, the actual distance of the reinforcement, based on experimental measurements, varied from 0,05 m to 0,20 m, and the total number of bars per one balcony varied from 20 to 26 bars.

### 7.2.3 Reliability verification of balcony components

The verification of the existing balcony components is based on the standard [22]. An analysis of the balconies revealed that for the geometric and material properties considered in the original design, the basic inequality $M_{Rd} > M_{Ed}$ between the maximum design bending moment $M_{Ed}$ due to external forces and the design resistance $M_{Rd}$ was satisfied.

The probabilistic methods and procedures of [8, 9] were used to investigate the reliability level of the deteriorated balcony components. The analysis was based on the limit state function given as

$$g(\theta_E \boldsymbol{E}, \theta_R \boldsymbol{R}) = \theta_R \boldsymbol{R} - \theta_E \boldsymbol{E} \tag{7.2}$$

where $\boldsymbol{E}$ and $\boldsymbol{R}$ are the vectors of the random variables for the action effects and resistance of a component, $\theta_R$ and $\theta_E$ model uncertainties of the resistance and action effects.

The limit state function $g$ for verification of balconies at the ultimate limit states is given as

$$g = \theta_R\, n\, (\pi\phi^2/4)\, f_y\, [h - c - \phi/2 - 0{,}5\, n(\pi\phi^2/4)\, f_y / f_c] \;-\; \theta_E\, (g + p)\, L^2/2 \quad (7.3)$$

where all applied basic variables are listed in Table 7.1 [15].

Table 7.1 Models of basic variables.

| Name of basic variable | Symbol | Distr. type | Units | Mean | St. deviation |
|---|---|---|---|---|---|
| Compressive 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 | $\phi$ | DET | m | 0,008 | - |
| No. of bars per balcony * | $n$ | DET | - | 20 | - |
| Balcony depth | $h$ | LN | m | 0,12 | 0,01 |
| Cover of reinforcement ** | $c$ | BET | m | 0,026 | 0,009 |
| Uncertainty of resistance | $\theta_R$ | LN | - | 1,1 | 0,05 $\mu$ |
| Uncertainty of load effect | $\theta_E$ | LN | - | 1 | 0,05 $\mu$ |
| Density of concrete | $\rho$ | N | MN/m$^3$ | nom. | 0,06 |
| Imposed load | $p$ | GAM | MN/m$^2$ | 0,0008 | 0,00048 |

* The minimum $n$ = 20 obtained experimentally is considered in this analysis.
** The mean and standard deviation are based on measurements of all balconies.

The probabilistic assessment of reliability based on a detailed balcony investigation indicates that the initial reliability level of particular balconies may vary significantly with the characteristics of the concrete cover of reinforcement (reliability index $\beta$ varies from 5,2 to 3,7) which comply with the target value $\beta_t$ = 3,8 [4]. However, the reliability index $\beta$ gradually decreases with diminishing reinforcement area due to reinforcement corrosion ($\beta$ drops to about one half with the reduction of the reinforcement to 50 %) as indicated in Figure 7.9.

The low level of quality control of the manufacturer of the precast load-bearing balcony components had a considerable influence on the statistical characteristics of the basic variables and enhanced the overall uncertainties concerning the actual reliability level of the balconies. Consequently, the theoretical models of the basic variables had to be specified individually, based on the results of detailed inspection. A durability assessment was applied for each balcony (quality of assembly, waterproofing, environmental effects). On the basis of the detailed assessment, it was decided which balconies were in the dangerous state and needed to be strengthened. New waterproofing insulation was provided to meet durability requirements.

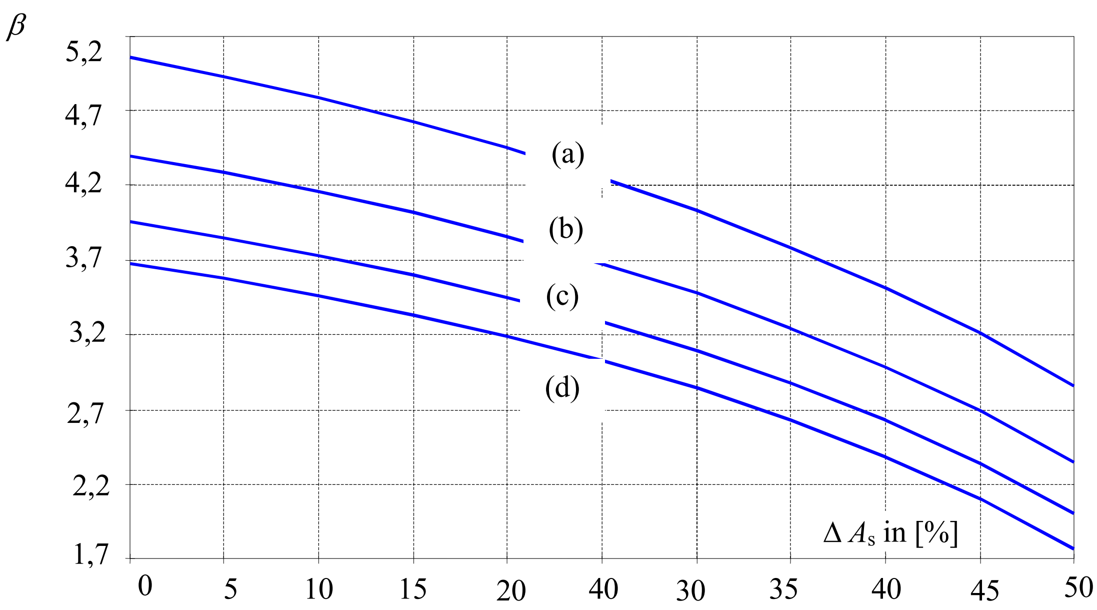



Fig. 7.8 Reliability index $\beta$ of balconies versus reinforcement area $\Delta A_s$ in %, for four cases of cover $c$: mean $\mu = 0{,}01$ m (a), 0,02 m (b), 0,026 m (c), 0,03 m (d).

### 7.2.4 Effects of design parameters

The design of existing balcony components was influenced by the reliability elements and detailing provided in the original Czech standards for structural design (recommended partial factors for permanent and variable actions $\gamma_G$ = 1,1, $\gamma_Q$ = 1,3, concrete cover $c$ = 0,01 m only). Afterwards, the national provision for concrete cover was raised taking into account the new durability requirements of European standards.

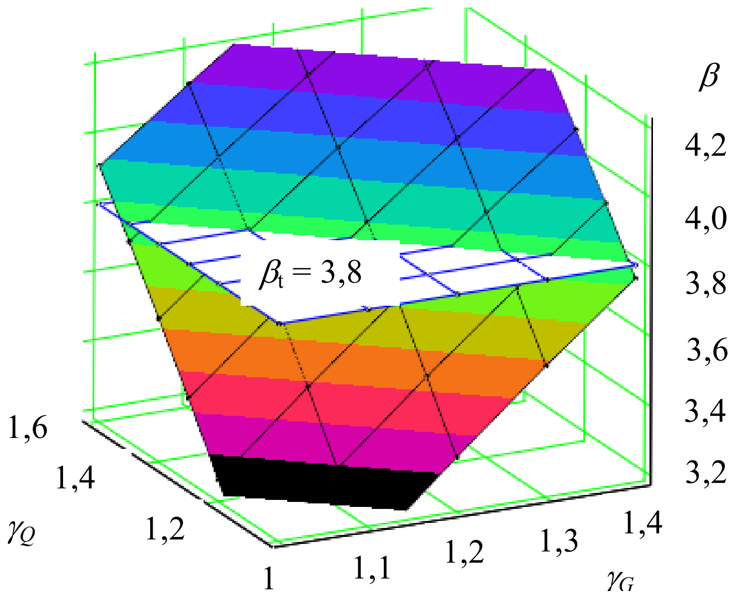



Fig. 7.9. Variation of the reliability index $\beta$ with partial factors $\gamma_G$ and $\gamma_Q$.

Eurocode EN 1990 [2] recommends greater values of partial factors for actions ($\gamma_G$ = 1,35, $\gamma_Q$ = 1,5) for verifications of the ultimate limit states (type STR).

Fig. 7.9 indicates the variation of the initial reliability index $\beta$ (without any reduction of the reinforcement area due to rusting or negligence) with partial factors $\gamma_G$ and $\gamma_Q$ for a balcony component, assuming that these were applied in the design of a balcony component. The reliability level $\beta_t = 3,8$ recommended in Eurocodes is shown in Fig. 7.9 by a white area.

## 8 CONCLUSIONS

The continued use of existing structures is of great importance for social, economic and cultural reasons. Methods for assessment based on the probabilistic principles of the theory of structural reliability enable rational decisions to be made on further exploitation of a structure and the need to be rehabilitated.

The new international standard ISO 13822 for the assessment of existing structures is based on the principles of probabilistic methods. Six National annexes developed at the Czech Technical University in Prague facilitate application of the standard in national practice. It is expected that relevant findings will be used for the development of new informative annex to EN 1990.

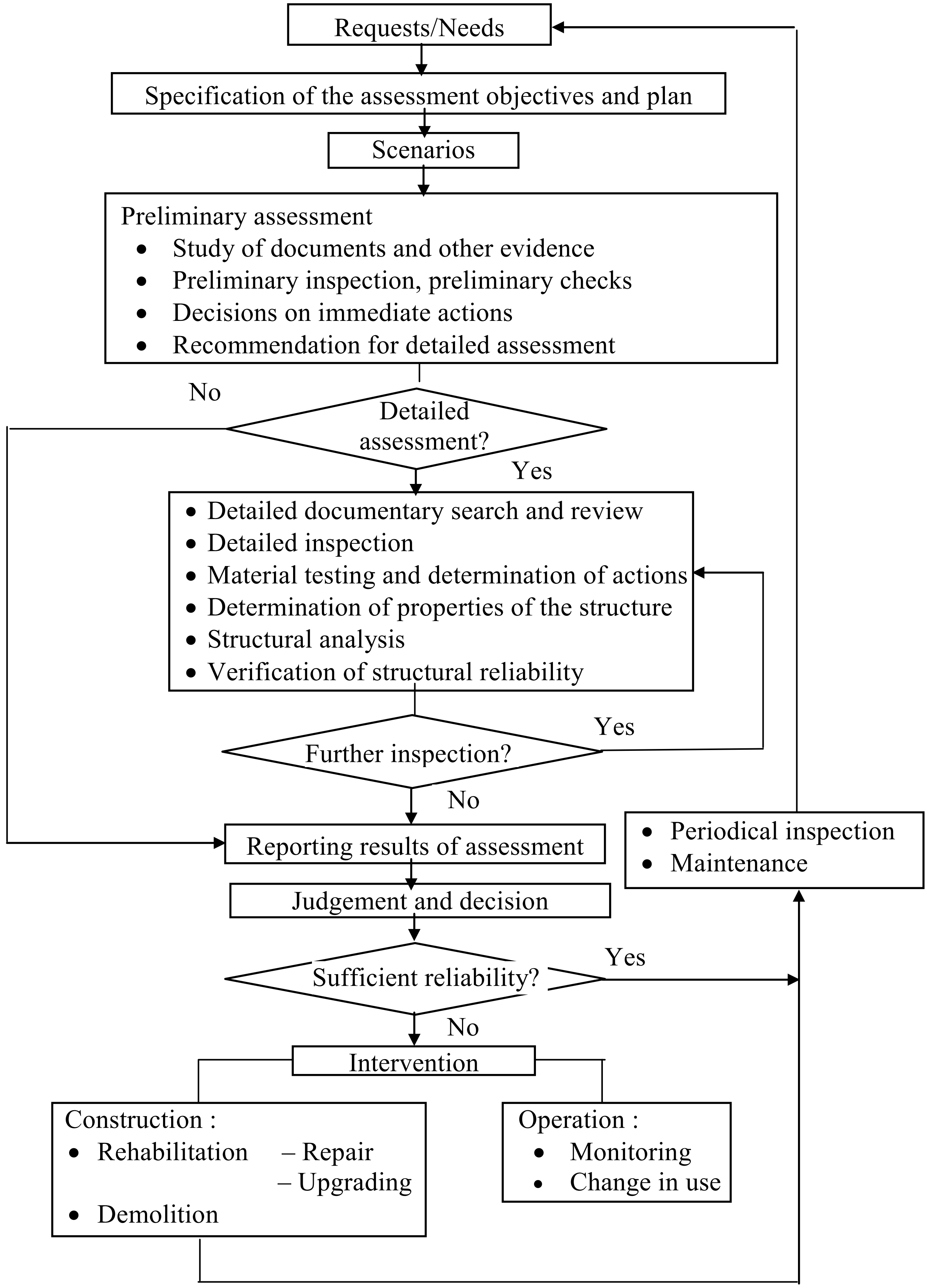
The principles of probabilistic methods can be applied for reliability differentiation of existing structures. The concept of design values may be used to determine partial factors for basic variables applied in the verification of existing structures. Applications of probabilistic methods in practice should be based on the required reliability level of a structure and on appropriate models of basic variables. An important role is played by appropriate quality management on site and adequate maintenance during the design working life. The procedure for assessment of existing structures is illustrated by two examples analysed at the Klokner Institute.

The example of an existing school building illustrates the need for complex assessment when the structural damages result from a combination of various errors in the design and in the process of execution, due to inadequate or non-existent control. The analysis of the main building proves that the structure satisfying the ultimate limit states does not comply with the serviceability limit states. Deflections of the floor slabs caused failures of non-structural parts, such as partitions and finishing.

The second example illustrates detailed structural assessment of prefabricated balconies based on the application of probabilistic methods to verify the structural reliability.

It appears that the application of new methods for structural assessment based on the probabilistic principles of the theory of structural reliability will contribute to the achievement of the required reliability level of existing structures and their durability.

## Annex A General flowchart for the assessment of existing structures according to ISO 13822



**REFERENCES**


[1] State of the art report on assessment and life extension of existing structures and industrial plants, WP6, Saferelnet, Lisbon, 2006

[2] EN 1990 Basis of structural design, CEN, 2002 (implemented as ČSN EN 1990 Zásady navrhování konstrukcí, 2004)

[3] ČSN 73 0038 Navrhování a posuzování stavebních konstrukcí při přestavbách, 1986 (withdrawn, superseded by ČSN ISO 13822 in 2005)

[4] ISO 13822 Bases for design of structures - Assessment of existing structures, ISO, 2003 (implemented as ČSN EN ISO 13822 Zásady navrhování konstrukcí - Hodnocení existujících konstrukcí, 2005)

[5] ISO 2394 General principles on reliability for structures, ISO, 1996 (implemented as ČSN ISO 2394 Obecné zásady spolehlivosti konstrukcí, 2004)

[6] ISO 2854 Statistical interpretation of data - Techniques of estimation and tests relating to means and variance (implemented as Statistická interpretace údajů. Odhady a testy středních hodnot a rozptylů), 1994

[7] EN 1998 -3 Eurocode 8: Design of structures for earthquake resistance Part 3: Assessment and retroffiting of buildings, CEN, 2005 (implementation in the Czech Republic expected in 2007)

[8] Probabilistic Assessment of Existing Structures, *Publication of JCSS*, 2001

[9] Probabilistic Model Code, JCSS, http://www.jcss.ethz.ch, 2001

[10] Holický M., Marková J.: Základy teorie spolehlivosti a hodnocení rizik, skripta, ČVUT, 2005

[11] Ditlevsen O., Madsen H.O.: Structural Reliability Methods, UK, 1996

[12] Haldar A., Mahadevan S.: Probability, Reliability and Statistical Methods in Engineering Design, New York, 2000

[13] Faber M.H.: Risk and Reliability in Civil Engineering, *Textbook*, Malta, 2001

[14] Marková J.: Pravděpodobnostní modely zatížení, Sborník k semináři *Uplatnění pravděpodobnostních metod*, ČVUT KÚ, 2003

[15] Holický M., Marková J.: Durability Assessment of Prefabricated Balconies, In: *Forensic Engineering*, UK, 2005

[16] Melchers R.E.: Structural Reliability, Analysis and Prediction, John Wiley and Sons, 1999

[17] Pume D. a kol.: Průzkumy a opravy stavebních konstrukcí, Praha, 1993

[18] Structural Concrete, Textbook on Behaviour, Design and Performance Updated Knowledge of the CEP/FIP Model Code 1990, 1999

[19] Holický M., Marková J.: Navrhování konstrukcí podle EN 1990, Management spolehlivosti a spolehlivostní metody, *Informační zpravodaj ČKAIT*, 4/2005

[20] Podmínky pro přípravu a kontroly oprav železobetonových konstrukcí ve výrobnách ČEZ, Kloknerův ústav a FAST VUT Brno, 1997

[21] EN 1991-1-1 Eurocode 1 Actions on structures - Self weigt, imposed load, actions on buildings, CEN, 2002 (implemented as ČSN EN 1991-1-1 Objemové tíhy, vlastní tíha, užitná zatížení pozemních staveb, 2004)

[22] ČSN 73 1201 Navrhování betonových konstrukcí, ČNI, 1988

[23] EN 1991-1-7 Eurocode 1 Actions on structures - Accidental actions, 2006 (implementation expected in 2007)

[24] Holický M., Marková J., Reliability Assessment of Primary School, *Failures of Concrete Structures II*, Bratislava, Expertcentrum, 2001

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### Education

Ing. (1979) - Czech Technical University in Prague, Faculty of Civil Engineering, branch Structures and Transport

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### Profession

1979 - 1996 - designer of buildings and bridges in the design or construction companies SSŽ, PÚDIS, Conbuilt, Ltd., C.R. Almeida, Ltd, M-Projekt, Ltd.

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- Research project MSM CEZ J04/98:210000029: Risk Analysis and Reliability of Technical Systems (1999-2004)
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- development of National annexes, translations and reliability analysis of standards EN 1990, EN 1991-1-1, EN 1991-1-5, EN 1991-1-6, EN 1991-1-7 and EN 1998-3
- co-operation on the development of National annexes to ISO 13822
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- member of Technical Committee for Construction Products at the Czech Office for Standards, Metrology and Testing
- member of the Concrete and Masonry organisation
- member of CIB, Structural Serviceability
- member of JCSS (Joint Committee on Structural Safety), WG1 for Reliability of Structures
- member of ENC Group (European National Correspondent Group), Commission Services.