BASICS FOR ASSESSMENT OF EXISTING STRUCTURES

Milan Holický, Vladislava Návarová, Roman Gottfried, Michal Kronika
Jana Marková, Miroslav Sýkora, Karel Jung

Lifelong Learning Programme

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Editors:
Milan Holický, Klokner Institute, Czech Technical University in Prague, Czech Republic

Authors:
Milan Holický, Klokner Institute, Czech Technical University in Prague, Czech Republic
Vladislava Návarová, Secondary Technical School of Civil Engineering, Č.Budějovice, Czech Republic
Roman Gottfried, Secondary Technical School of Civil Engineering, Č.Budějovice, Czech Republic
Michal Kronika, Secondary Technical School of Civil Engineering, Č.Budějovice, Czech Republic
Jana Marková, Klokner Institute, Czech Technical University in Prague, Czech Republic
Miroslav Sýkora, Klokner Institute, Czech Technical University in Prague, Czech Republic
Karel Jung, Klokner Institute, Czech Technical University in Prague, Czech Republic

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Edited by:
Milan Holický, Klokner Institute, Czech Technical University in Prague, Czech Republic

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FOREWORD

The Leonardo da Vinci Project CZ/11/LLP-LdV/TOI/134005 *"Vocational Training in Assessment of Existing Structures"* addresses the urgent need to educate students, young engineers and professionals about the assessment of existing structures. The future of the entire construction industry depends on its moving from new constructions towards maintenance, repair and rehabilitation of existing structures. The safety assessment of existing structures therefore plays an important role.

The assessment of existing structures is an urgent issue of great economic significance in most countries around the world, as more than 50 % of all construction activities concerns existing buildings, bridges and other civil engineering works. At present, the Eurocodes which will be used in all CEN Member countries are primarily focused on the design of new structures. Additional operational rules for existing structures are still missing. The international standard ISO 13822 provides only general principles for the assessment of existing structures, and these should be further developed for their effective operational use in practice.

The current project addresses the urgent need for the implementation of principles for the assessment and verification of existing structures to be put into practice in the Czech Republic and other partner countries. The project is supported by the Czech Chamber of Chartered Engineers (ČKAIT). The project consortium, under the leadership of the Klokner Institute of the Czech Technical University in Prague (KI CTU), consists of the Secondary Technical School of Civil Engineering (CZ) and the research institutions and universities from four EU Member States (DE, ES, IT, NL), plus one associated country (TR). All the researchers in the partnership are involved in research projects dealing with the reliability assessment of existing structures. They participate in the national and international standardization activities within the organizations CEN and ISO.

The project outcomes include vocational training materials based on documents from the international research organization Joint Committee on Structural Safety JCSS and international research projects, the selected outcomes of the previous project of the Leonardo da Vinci Programme (developed by 5 partners of the present consortium in 2008-2010) as well as on background documents to the new European and international standards.

The basic project outcomes are 3 handbooks. Handbook 1 "Innovative Methods for the Assessment of Existing Structures" is focused on methodologies to assess and evaluate the condition of existing structures. The methodologies provided are independent of the type of structure and material, and are compatible with the background methodologies used in the Eurocodes. Operational techniques for the assessment of existing structures and associated case studies are presented in Handbook 2 of this project. The present Handbook 3 "Basics for Assessment of Existing Structures" represents a simplified - "2 in 1" - version of Handbooks 1 and 2, adapted for the purposes of secondary technical school students.

The authors believe that the material in Handbook 3 is presented in a comprehensible way, supported with examples, and many references are provided for background material and further study.

České Budějovice and Prague, 2013

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CHAPTER 1 - INTRODUCTION: STANDARDS FOR STRUCTURAL ASSESSMENT

Milan Holický and Jana Marková[1]
[1]Klokner Institute, Czech Technical University in Prague

Summary

There are still missing harmonised European rules for the assessment of existing structures. Recent working meetings of the Technical Committee CEN/TC 250 of the European Committee for Standardization revealed that the preparation of the new Eurocode for the assessment of existing structures is needed. Presently the international standard ISO 13822 may be applied for the harmonisation of rules for the assessment of existing structures with basis and requirements of Eurocodes. It is expected that in the second generation of Eurocodes a new standard will be developed focusing on the assessment and verification of existing structures.

1 CURRENT STATE

Assessment of existing structures is an important topic for experts working in construction in most industrial countries, where rehabilitation including repairs and upgrading of construction works represent about half of all construction activities. It is due to several circumstances including following items.

- Existing structures represent substantial, continually increasing economic contribution
- Users are interested in a new way of exploitation of existing structures
- Many existing structures do not fulfil requirements of currently valid standards.
- An European standard for the assessment and retrofitting of existing structures has not been developed yet.
- Assessment of existing structures often requires knowledge overlapping the framework of standards for the design of new structures.
- Assessment should be focused on minimal construction interventions to existing structures.
- Civil engineers, owners and representatives of governmental authorities need new guidance for the assessment of existing structures.

Presently new European standards are implemented in most CEN Member countries and applied for the design of new structures. Original national standards for structural design are withdrawn or revised in order to harmonize national prescriptive documents in all Member States with respect to requirements of Eurocodes or ISO standards.

However, the Eurocodes serve mainly for the design of new structures. There have not been introduced explicit provisions for the assessment of existing structures and for design of their repairs or upgrading till now. For this purpose the international standard ISO 13822 [1] based on the same principles as Eurocodes is intended which may be supplemented by national provisions based on practice of regional construction industry. Original national standards (ČSN 73 0038 [2] in the Czech Republic) may be included to the National Annexes (the standard ISO 13822 is implemented in the Czech Republic as ČSN ISO 13822 [1]).

2 ISO 13822

ISO 13822 [1] provides general requirements and procedures for the assessment of existing structures (buildings, bridges, industrial structures etc.), based on the principles of structural reliability and taking into account specific problems of existing structures. This is a materially independent prescriptive document applicable to the assessment of any type of existing structure designed and executed according to theoretical basis and original design rules or based on long-term experience and professional procedures.

Translation of ISO 13822 [1] to the Czech language and coordination of the development of six National Annexes was assured by the Klokner Institute CTU in Prague in co-operation with the Faculty of Civil Engineering and the Technical and Test Institute for Construction Prague.

The partial factor method or probabilistic methods may be applied for the reliability analyses of structures according to CSN ISO 13822 [1]. General procedures are recommended for the determination of actions and material properties. ISO 13822 [1] explains why current standards for structural design are not sufficient for the reliability assessment of existing structures, for the design of their repairs or upgrading. Present design codes do not provide procedures for the assessment of the current state of existing structures and resistance of materials. Moreover, they are not dealing with uncertainties due to real use of construction works and history of action effects. Residual working life and purpose of application should be also taken into account. Some existing structures might be sufficiently reliable despite they comply with requirements of current, often more strict requirements of currently valid standards for structural design.

National Annexes represent important parts of ISO standard implemented in the Czech Republic as ČSN ISO 13822 [1].

- Annex NA supplements selected provisions of ISO 13822 and concerns general, material independent matters of structural assessment. Some terms are explained which are still not common in national standards (e.g. assessment, rehabilitation, plan of safety measures) while some national terms are not applied in standard ISO 13822 [1] (e.g. conversion, reconstruction, defect). There are introduced procedures for determination of actions on structures and actual material properties.
- Annex NB deals with testing of existing structures and materials. It provides general principles for experimental verifications and makes references to prescriptive documents for the testing of materials and structures.
- Annex NC gives provisions for specification of properties of concrete, reinforcement and prestressing reinforcement for existing structures.
- Annex ND gives provisions for steel, cast iron and composite steel concrete structures.
- Annex NE gives guidance for specification of properties of timber and composite timber concrete structures.
- Annex NF includes basic provisions for the assessment of properties of existing masonry elements and mortars including procedure for the evaluation of masonry strength.

The requirements for safety and serviceability specified in the international standard ČSN ISO 13822 [1] are in principle the same as those recommended for the design of new structures. There are, however, some fundamental differences between the criteria for design of new structures and assessment of existing structures indicated in Table 1. Generally, it is required to minimize structural intervention to existing structures and to use existing materials. Actual properties of existing materials should be, however, carefully assessed.

Table 1. Different criteria for the assessment of performance requirements for reliability of structures.

Criteria	Existing structures	New structures
Economical	incremental cost for increasing structural safety is commonly high	incremental cost of increasing structural safety is commonly lower
Social	may be significant due to reduction or disruption of serviceability and preservation of heritage values	commonly less significant than for existing structures
Sustainability	in large measure existing materials are used, leading to reduction of waste and recycling	commonly new materials are applied

3 FORESEEN DEVELOPMENT OF STANDARDS FOR EXISTING STRUCTURES

National development

Implementation of ISO 13822 [1] into the system of national prescriptive documents facilitates to develop an operational document for the reliability assessment of existing structures in the CEN Member States according to the principles of Eurocodes. ISO 13822 [1] is an important international standard and both, CEN and some European countries (e.g. UK, Slovakia) are interested in this document.

It is foreseen that some National Annexes will be supplemented with some still missing information, mainly the National Annex NE for the assessment of existing masonry structures. It is also necessary to introduce more detail information concerning procedures for the specification of design values of basic variables, the load-bearing capacity of existing structures and determination of reliability level with respect to the consequences of failures (categorisation of structures) and remaining working life of structures. Complementary provisions for some specific structures are missing (e.g. bridges). Therefore, the amendment of the standard ISO [1] including translation of the Annex I is under preparation supplementing the original standard by provisions for the assessment of heritage structures. It is foreseen to develop a new part of the National Annex with supplementary information concerning heritage structures for national conditions.

International development

Currently, in Europe there are missing common design rules for the assessment and retrofitting of existing structures which should be consistent with Eurocodes. Problems of new European standards for existing structures have been dealt with in the framework of the Technical Committee CEN/TC 250 since 2005 year. Some CEN Member States are willing to develop new rules for existing structures based on the same principles as Eurocodes, other countries are interested to apply their own national standards and national approaches of the assessment. New advisory panel of CEN/TC 250 convenor (prof. M. Holický from the Klokner Institute CTU is also a member) was charged with the development of a study indicating whether a new Eurocode for the assessment of existing structures should be prepared and what would be its scope. The document N 737 [7] is a background for preparation of plans for further evolution of Eurocodes. The plan was prepared within the Technical Committee CEN/TC 250 and submitted for the European Commission (EC). After many negotiations it was prepared a final proposal of Mandate M/515 [13] on the basis of expectation of a certain financial support of EC for evolution of Eurocodes.

Following main contributions of the new Eurocode for the assessment and retrofitting of structures are assumed on the basis of document N 737 [7] and proposal of mandate M/515:

- provide new harmonised European technical rules for existing structures harmonized with basic requirements of Eurocodes (fulfilment of requirements for mechanical resistance, stability and resistance to fire including aspects of durability and economy),
- development of construction works in urban and industrial areas, and also of infrastructure leading to repairs, upgrading and enlargement of existing structures,
- preparation of new provisions for analyses of existing structures facilitating to identify their potential that could be included to new development plans,
- upgrading of existing structures with application of new technologies for retrofitting, improve the quality of energetic effective building envelope,
- application of more precise methods for verification of existing structures facilitating removal of unneeded conservatism while assuring required safety,
- provide a better understanding among owners, users, designers, manufacturers of construction products (facilitating application of new materials and products for existing structures),
- facilitating exchange of services in construction between the Member States,
- effective commercialisation and application of construction precast members,
- more easy use of materials and products properties which are taken into account in analyses,
- preparation of common design tools and software,
- competitiveness growth of European construction companies, producers, users of standards and clients (the volume of cases of retrofitting of existing structures increases in Europe, USA, China, India and other states).

The Technical Committee CEN/TC 250 will co-operate with EOTA, with CEN/TCs for construction product and also with building companies in the European technological platform.

It is expected that the development of standard for the assessment and retrofitting of existing structures will make it possible to effectively exploit existing structures. Safety and robustness of existing structures against adverse actions will be increased. The new developments of product standards are foreseen in the framework of the preparation of this standard.

Some requirements on existing structures and bridges and their economic and social assets are introduced in Table 2.

Table 2. Requirements on existing construction works and assets

Structure	Demand	Growth drivers and needs
Buildings	Sustainability of development	Reusing of existing buildings in towns
	Energy saving (heating)	Reducing of energy loss
	Energy saving (cooling)	Reducing of energy loss
	Fire protection	New evacuation plans, prevent of fire spread, improvement of fire resistance
	Safety	adaption to new occupancies and uses, increase of resistance against accidental and seismic actions
	Serviceability and security	Improvement of stiffness, serviceability, elevators
	Acoustic	Improvement of acoustic properties
Bridges	Sustainable development	Using existing traffic roads
	Security of use	Requirements for dimensions, deflections, clearances
	Safety	Fulfilment of requirements on load bearing capacity, resistance to accidental and seismic actions
	Durability	Reduction of maintenance costs, enhancement of remaining working life

The new standard for the assessment and retrofitting of existing structures should include following principles

- currently valid standards should be applied for the verification of structural reliability while original codes applied in the structural design should have informative character only,
- actual characteristics of construction materials, actions, geometrical data and information concerning structural behaviour should be applied.

New Eurocode for existing structures should include

- methodology of collecting, evaluation and data updating,
- applications of partial factor method including possibility for direct use of probabilistic methods consistent with Eurocodes,
- assessment of target reliability level for existing structures, consideration of remaining working life, consequence of failure and costs on safety measures
- assessment based on previous satisfactory past performance,
- structural interventions and preparation of report with results of assessment.

According to the document N 737 [7] the preparation of a new European standard for the assessment of existing structures and for their retrofitting should start as soon as possible. New European prescriptive documents for the assessment of existing structures have been developed and therefore, a later harmonisation of the standards into one European document would be considerably difficult. Harmonisation of all testing procedures for construction materials and products is also important.

Presently existing construction works form around 60 % of the total construction works for which it is necessary to prepare guidance for their assessment. It is expected that development of new Eurocode will be supported by the European Commission, JRC research centre, National standard bodies, CEN, EOTA and research organisations dealing with prenormative research. Guidance paper L [8] and document N 250 [9] will be applied for the development of new Eurocode used within the preparation of all EN Eurocodes.

Backgrounds for the development of Eurocode for the assessment of existing structures

National and also international prescriptive documents may be applied for the development of new Eurocode for structural assessment. Besides standard ISO 13822 [1] it is foreseen to use the Bulletin fib [10], reports [11,12] and document [13].

National standards are also available for structural assessment in several CEN Member States, e.g. in UK (Highways Agency Requirements), in Germany (DS805, Leitfaden für den Sicherheitsnachweis Vorhandener Straßenbrücken), in Switzerland (SIA 269), in Austria and in the Czech Republic (ČSN 73 0038 [2] where selected provisions were implemented to the National Annexes of ČSN ISO 13822 [1]).

The new European prescriptive document for the FRP (Fibre Reinforced Polymers), currently under development should be also an important document for the retrofitting of existing structures.

4 CONCLUDING REMARKS

Eurocodes serve mainly for the design of new structures. However, harmonised European rules are still missing for the assessment of existing structures. Therefore, a new document has been prepared in the framework of the Technical Committee CEN/TC 250 confirming needs for the preparation of new Eurocode for the assessment of existing structures.

Implementation of ISO 13822 to the system of Czech standards makes it possible to apply the same principles for the assessment of existing structures on which the Eurocodes are based. There are introduced supplementary data and information about traditional procedures used in Czech construction. It is expected that ČSN ISO 13822 [1] will become one of the background documents for the preparation of new Eurocode for the design of existing structures.

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CHAPTER 2: GENERAL FRAMEWORK

Milan Holický[1], Miroslav Sýkora[1]

[1]Klokner Institute, Czech Technical University in Prague, Czech Republic

Summary

Assessment of existing structures is in many aspects different from that taken in designing a new structure. The effects of the construction process and subsequent life of the structure, during which it may have undergone alteration, deterioration, misuse, and other changes to its as-built (as-designed) state, must be taken into account. In general actual variation in the basic variables describing actions, material properties, geometric data and model uncertainties should be taken into account. Taking into account these documents the main principles for assessment of existing structures may be summarized as follows:

- Available scientific knowledge and know-how including currently valid codes should be applied; historical practice and provisions valid when the structure was built (designed), should be used as guidance information only;
- Actual characteristics of structural material, action, geometric data and structural behaviour should be considered; the original documentation including drawing should be used as guidance material only.

The most important step of the whole assessment procedure of existing structures including evaluation of inspection data and updating of prior information concerning strength and structural reliability, described in detail in Handbook 1, are summarised in this Handbook in a condense and operational form.

1 INTRODUCTION

1.1 Background documents

Three International Standards ISO 2394 [1], ISO 13822 [2] and ISO 12491 [3], related to the assessment of existing structures, have been recently developed. Moreover, ISO 13822 [2] contains an annex focused on heritage structures. Additional information may be found in a number of scientific papers and publications, for example in [4], [5] and [6]. Examples of practical procedures and technique are presented in recent papers [7] and [8].

1.2 General principles

Assessment of existing structures is becoming a more and more important and frequent engineering task. Continued use of existing structures is of a great significance due to environmental, economic and socio-political assets, growing larger every year. These aspects are particularly relevant to heritage buildings that always constitute a great historical, social and economic value.

General principles of sustainable development regularly lead to the need for extension of the life of a structure, in majority of practical cases in conjunction with severe economic constraints. That is why assessment of existing structures often requires application of sophisticated methods, as a rule beyond the scope of traditional design codes. Nevertheless, apart from few national codes, three International Standards ISO 2394 [1], ISO/CD 13822 [2] and ISO 12491 [3], related to assessment of existing structures, have been recently developed.

The approach to the assessment of existing structures is in many aspects different from that taken in designing the structure of a newly proposed building. The effects of the construction process and subsequent life of the structure, during which it may have undergone alteration, deterioration, misuse, and other changes to its as-built (as-designed) state, must be taken into account.

However, even though the existing structure may be investigated several times, some uncertainty in the basic variables and structural behaviour shall always remain. Therefore, similarly as in design of new structures, actual variation in the basic variables describing actions, material properties, geometric data and model uncertainties are taken into account by partial factors or other code provisions.

In general, an existing structure may be subjected to the assessment of its actual reliability in case of:

- rehabilitation during which new structural members are added to the existing load-carrying system;
- adequacy checking in order to establish whether the existing structure can resist loads associated with the anticipated change in use of the facility, operational changes or extension of its design working life;
- repair of a building, which has deteriorated due to time dependent environmental effects or which has suffered damage from accidental actions, for example, earthquake;
- doubts concerning actual reliability of the structure.

In some circumstances assessments may also be required by authorities, insurance companies or owners or may be demanded by a maintenance plan.

2 PRINCIPLES OF ASSESSMENT

Two main principles are usually accepted when assessing existing buildings:

- Currently valid codes for verification of structural reliability should be applied, codes valid in the period when the structure was designed should be used only as guidance documents.
- Actual (estimated) characteristics of structural materials, actions, geometric data and structural behaviour should be considered, the original design documentation including drawings should be used as guidance documents only.

The first principle should be applied in order to achieve similar reliability level as in case of newly designed structures, taking only account of economic aspects as indicated below. The second principle should avoid negligence of any structural condition that may affect actual reliability (in favourable or unfavourable way) of a given structure.

Most of the current codes are developed assuming the concept of limit states in conjunction with the partial factor method. In accordance with this method, which is mostly considered here, basic variables are specified by characteristic or representative values. The design values of the basic variables are determined on the basis of the characteristic (representative) values and appropriate partial factors.

It follows from the second principle that a visual inspection of the assessed structure should be made whenever possible. Practical experience shows that inspection of the site is also useful to obtain a good feel for actual situation and state of the structure.

As a rule the assessment need not to be performed for those parts of the structure that will not be affected by structural changes, rehabilitation, repair, change in use or which are not obviously damaged or are not suspected of having insufficient reliability [2].

In general, the assessment procedure consists of the following steps (see the flow chart in [2]):

- specification of the assessment objectives required by the client or authority;
- scenarios related to structural conditions and actions;
- preliminary assessment:
  - study of available documentation;
  - preliminary inspection;
  - preliminary checks;
  - decision on immediate actions;
  - recommendation for detailed assessment;
- detailed assessment:
  - detailed documentary search;
  - detailed inspection;
  - material testing and determination of actions;
  - determination of structural properties;
  - structural analysis;
  - verification of structural reliability;
- report including proposal for construction intervention;
- repeat the sequence if necessary.

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. Conversely if the structure seems to be in dangerous or uncertain condition immediate interventions and detailed assessment may be necessary.

3 INVESTIGATION

Investigation of an existing structure is intended to verify and update the knowledge about the present condition (state) of a structure with respect to a number of aspects. Often, the first impression of the structural condition will be based on visual qualitative investigation. The description of possible damage of the structure may be presented in verbal terms like: 'unknown, none, minor, moderate, severe, destructive'. Very often the decision based on such an observation will be made by experts in a purely intuitive way.

A better judgement of the structural condition can be made on the basis of (subsequent) quantitative inspections. Typically, the assessment is a cyclic process when the first inspection is supplemented by subsequent investigations. The purpose of the subsequent investigations is to obtain a better feel for the actual structural condition (particularly in the case of damage) and to verify information required for determination of the characteristic and representative values of all basic variables. For all inspection techniques, information on the probability of detecting damages if present, and the accuracy of the results should be given.

The statement from the investigation contains, as a rule, the following data describing

- actual state of the structure;
- types of structural materials and soils;
- observed damages;
- actions including environmental effects;
- available design documentation.

Proof loading is a special type of investigation. Based on such tests one may draw conclusions with respect to:

- the bearing capacity of the tested member under the test load condition;
- other members;
- other load conditions;
- the behaviour of the system.

The inference in the first case is relatively easy; the probability density function of the load bearing capacity is simply cut off at the value of the proof load. The inference from the other conclusions is more complex. Note that the number of proof load tests needs not to be restricted to one. Proof testing may concern one element under various loading conditions and/or a sample of structural elements. In order to avoid an unnecessary damage to the structure due to the proof load, it is recommended to increase the load gradually and to measure the deformations. Measurements may also give a better insight into the behaviour of the system. In general proof loads can address long-term or time-dependent effects. These effects should be compensated by calculation.

4 BASIC VARIABLES

In accordance with the above-mentioned general principles, characteristic and representative values of all basic variables shall be determined taking into account the actual situation and state of the structure. Available design documentation is used as a guidance material only. Actual state of the structure should be verified by its inspection to an adequate extent. If appropriate, destructive or non-destructive inspections should be performed and evaluated using statistical methods.

For verification of the structural reliability using the partial factor method, the characteristic and representative values of basic variables shall be considered as follows:

(a) Dimensions of the structural elements shall be determined on the basis of adequate measurements. However, when the original design documentation is available and no significant changes in dimensions have taken place, the nominal dimensions given in the documentation may be used in the analysis.

(b) Load characteristics shall be introduced with the values corresponding with the actual situation verified by destructive or non-destructive inspections. When some loads have been reduced or removed completely, the representative values of these loads (actions) can be reduced or appropriate partial factors can be adjusted. When overloading has been observed in the past it may be appropriate to increase adequately representative values.

(c) Material properties shall be considered according to the actual state of the structure verified by destructive or non-destructive inspections. When the original design documentation is available and no serious deterioration, design errors or construction errors are suspected, the characteristic values given in original design may be used.

(d) Model uncertainties shall be considered in the same way as in design stage unless previous structural behaviour (especially damage) indicates otherwise. In some cases model factors, coefficients and other design assumptions may be established from measurements on the existing structure (e.g. wind pressure coefficient, effective width values, etc.).

Thus the reliability verification should be backed up by inspection of the structure including collection of appropriate data. Evaluation of prior information and its updating using newly obtained measurements is one of the most important steps of the assessment.

5 EVALUATION OF INSPECTION RESULTS

Using results of an investigation (qualitative inspection, calculations, quantitative inspection, proof loading) the properties and reliability estimates of the structure may be updated. Two different procedures can be distinguished:

(a) Updating of the structural failure probability.
(b) Updating of the probability distributions of basic variables.

Direct updating of the structural reliability (procedure (a)) can be formally carried out using the following basic formula of the probability theory:

$$\mathrm{P}(F|I) = \frac{\mathrm{P}(F \cap I)}{\mathrm{P}(I)} \tag{1}$$

where P denotes probability, F local or global failure, I inspection information, and $\cap$ intersection of two events. The inspection information I may consist of the observation that the crack width at the beam B is smaller than at the beam A. An example of probability updating using equation (1) is presented e.g. in [6].

The updating procedure of a univariate or multivariate probability distribution (procedure (b)) is given formally as:

$$\mathrm{f}_X(x|I) = C\ \mathrm{P}(I|x)\ \mathrm{f}_X(x) \tag{2}$$

where $\mathrm{f}_X(x|I)$ denotes the updated probability density function of X, $\mathrm{f}_\mathrm{X}(x)$ denotes the probability density function of X before updating, X a basic variable or statistical parameter, I inspection information, C normalising constant, and $\mathrm{P}(I|x)$ likelihood function.

An illustration of equation (2) is presented in Figure 1. In this example updating leads to a more favourable distribution with a greater design value x_d than the prior design value x_d. In general, however, the updated distribution might be also less favourable than the prior distribution.

The updating procedure can be used to derive updated characteristic and representative values (fractiles of appropriate distributions) of basic variables to be used in the partial factor method or to compare directly action effects with limit values (cracks, displacements). More information on updating may be found in ISO 12491 [3].

Once the updated distributions for the basic variables $f_X(x)$ have been found, the updated failure probability $P(F|I)$ may be determined by performing a probabilistic analysis using common method of structural reliability for new structures. Symbolically it can be written

$$P(F|I) = \int_{g(X)<0} f_X(x \mid I)\mathrm{d}x \tag{3}$$

where $f_X(x|I)$ denotes the updated probability density function and $g(x) < 0$ denotes the failure domain ($g(x)$ being the limit state function). It should be proved that the probability $P(F|I)$, given the design values for its basic variables, does not exceed a specified target value.

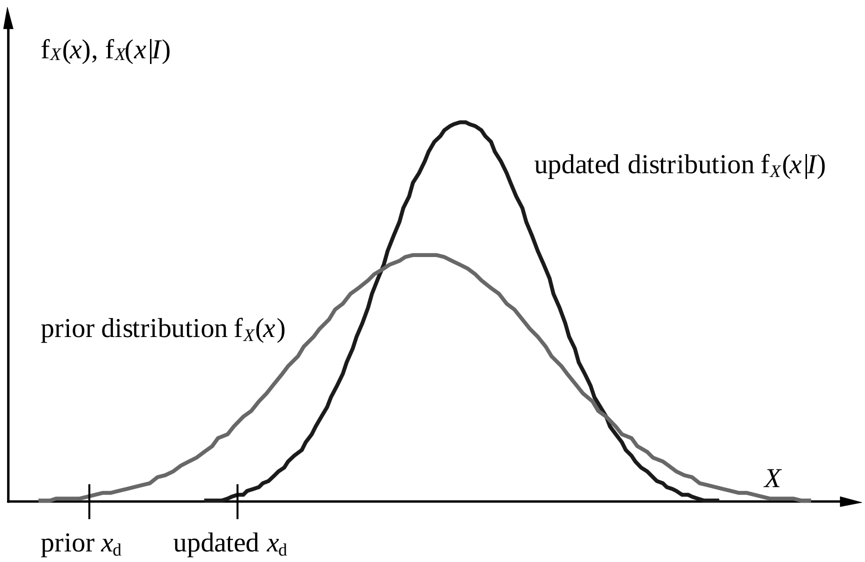

Figure 1: Updating of probability density function for an expected variable *X*.

A more practical procedure is to determine updated design values for each basic variable (procedure (b)) that is discussed in Chapter 6.

6 STRUCTURAL ANALYSIS

Structural behaviour should be analysed using models that describe actual situation and state of an existing structure. Generally the structure should be analysed for ultimate limit states and serviceability limit states using basic variables and taking into account relevant deterioration processes.

All basic variables describing actions, material properties, load and model uncertainties should be considered as mentioned above. The uncertainty associated with the validity and accuracy of the models should be considered during assessment, either by adopting appropriate factors in deterministic verifications or by introducing probabilistic model factors in reliability analysis.

When a structure is analysed, conversion factors reflecting the influence of shape and size effect of specimens, temperature, moisture, duration-of-load effect, etc., should be taken into account. The level of knowledge about the condition of components should be also considered. This can be achieved by adjusting the assumed variability in either the load carrying capacity of the components or the dimensions of their cross sections, depending on the type of structure.

When deterioration is observed, the relevant mechanisms shall be identified and a deterioration model predicting the future performance of the structure shall be determined on the basis of theoretical or experimental investigation, inspection, and experience.

7 VERIFICATION

Reliability verification of an existing building shall be made using valid codes of practice, as a rule based on the limit state concept. Attention should be paid to both the ultimate and serviceability limit states. Verification may be carried out using partial safety factor or structural reliability methods with consideration of structural system and ductility of components. The reliability assessment shall be made taking into account the remaining working life of a structure, the reference period, and changes in the environment of a structure associated with an anticipated change in use.

The conclusion from the assessment shall withstand a plausibility check. In particular, discrepancies between the results of structural analysis (e.g. insufficient safety) and the real structural condition (e.g. no sign of distress or failure, satisfactory structural performance) must be explained. It should be kept in mind that many engineering models are conservative and cannot be always used directly to explain an actual situation.

The target reliability level used for verification can be taken as the level of reliability implied by acceptance criteria defined in proved and accepted design codes. The target reliability level shall be stated together with clearly defined limit state functions and specific models of the basic variables.

The target reliability level can also be established taking into account the required performance level for the structure, the reference period and possible failure consequences. In accordance with ISO 2394 [1], the performance requirements for assessment of existing structures are the same as for design of a new structure. Lower reliability targets for existing structures may be used if they can be justified on the basis of economical, social and sustainable consideration (see Annex F to ISO/CD 13822 [2] and numerical example in [8]).

An adequate value of the reliability index β should be in general determined [2] considering appropriate reference period. For serviceability and fatigue the reference period equals the remaining working life, while for the ultimate limit states the reference period is in principle the same as the design working life specified for new structures (50 years for buildings). This general approach should be in specific cases supplemented by detailed consideration of the character of serviceability limit states (reversible, irreversible), fatigue (controllable, incontrollable) and consequences of ultimate limit states (economic consequences, number of endangered people, loss of the cultural heritage value).

8 ASSESSMENT IN THE CASE OF DAMAGE

For an assessment of a damaged structure the following stepwise procedure is recommended:

1) Visual inspection

It is always useful to make an initial visual inspection of the structure to get a feel for its condition. Major defects should be reasonably evident to the experienced eye. In the case of very severe damage, immediate measures (like abandonment of the structure) may be taken.

2) Explanation of observed phenomena

In order to be able to understand the present condition of the structure, one should simulate the damage or the observed behaviour, using a model of the structure and the estimated intensity of various loads or physical/chemical agencies. It is important to have available documentation with respect to design, analysis and construction. If there is a discrepancy between calculations and observations, it might be worthwhile to look for design errors, errors in construction, etc.

3) Reliability assessment

Given the structure in its present state and given the present information, the reliability of the structure is estimated, either by means of a failure probability or by means of partial factors. Note that the model (structural analysis) of the present structure may be different from the original model. If the reliability is sufficient (i.e. better than commonly accepted in design) one might be satisfied and no further action is required.

4) Additional information

If the reliability according to step 3 is insufficient, one may look for additional information from more advanced structural models, additional inspections and measurements or actual load assessment.

5) Final decision

If the degree of reliability is still too low, one might decide to:

- accept the present situation for economical reasons;
- reduce the load on the structure;
- repair the building;
- start demolition of the structure.

The first decision may be motivated by the fact that the cost for additional reliability is much higher for existing structure than for a new structure. This argument is sometimes used by those who claim that a higher reliability should be generally required for a new structure than for an existing one. However, if human safety is involved, economical optimisation has a limited significance.

9 FINAL REPORT AND DECISION

The final report on structural assessment and possible interim reports (if required) should include clear conclusions with regard to the objective of the assessment based on careful reliability assessment and cost of repair or upgrading. The report shall be concise and clear. A recommended report format is indicated in Annex G to ISO/CD 13822 [2].

If the reliability of a structure is sufficient, no action is required. If an assessment shows that the reliability of a structure is insufficient, appropriate interventions should be proposed. Temporary intervention may be recommended and proposed by the engineer if required immediately. The engineer should indicate a preferred solution as a logical follow-up to the whole assessment in every case.

It should be noted that the client in collaboration with the relevant authority should make the final decision on possible interventions, based on engineering assessment and recommendations. The engineer performing the assessment might have, however, the legal duty to inform the relevant authority if the client does not respond in a reasonable time.

In the case of heritage structures minimisation of construction interventions is required in rehabilitation and upgrades, but sufficient reliability should also be guaranteed. When dealing with the preservation of heritage buildings, it may be difficult to propose construction interventions that respect all requirements for preservation of the heritage value. Modern principles of interventions seem to include the following aspects:

- Unobtrusiveness and respect of the original conception,
- Safety of the construction,
- Durability of materials,
- Balance between costs and available financial resources,

and in some cases also:

- Removability,
- Compatibility of materials,
- Indoor environment quality including aspects of comfort, security and accessibility.
-

10 CONCLUDING REMARKS

The main principles for assessment of existing structures are:

- Currently valid codes for verification of structural reliability should be applied, codes valid in the period when the structure was designed, should be used only as guidance documents;
- Actual characteristics of structural material, action, geometric data and structural behaviour should be considered; the original design documentation including drawing should be used as guidance material only.

The most important step of the whole assessment procedure is evaluation of inspection data and updating of prior information concerning strength and structural reliability. It appears that a Bayesian approach can provide an effective tool.

Typically, assessment of the existing structures is a cyclic process in which the first preliminary assessment is often supplemented by subsequent detailed investigations and assessment. A report on structural assessment prepared by an engineer should include a recommendation on possible intervention. However, the client in collaboration with the relevant authority should make the final decision concerning possible interventions.

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ANNEX TERMINOLOGY

Extracted from ISO CD 8930.2 [9]

structure	Organised combination of connected parts designed to provide resistance and rigidity against various actions.
structural elements or structural components	Physically distinguishable parts of a structure, including structural members (such as columns, beams, slabs, shells) and also joints.
structural system	The system formed by the structural elements of a construction works, and the way these elements function together.
maintenance	The routine activities to be performed during the working life of a structure in order to preserve fulfilment of requirements for reliability. Note: to restore the structure after an accidental or seismic event is normally outside the scope of maintenance.
assessment (of the reliability of a structure)	Total set of activities performed in order to find out if the reliability of the structure is acceptable or not.
compliance	Fulfilment of specified requirements
risk	Danger that an undesired event represents for humans, environment or properties. Note: risk can be expressed in terms of possible consequences of the undesired event, and associated probabilities.
failure	Insufficient load-bearing capacity or inadequate serviceability of a structure or structural element
capacity	Ability of a structure (or a part of it) to withstand without failure. For instance: deformation capacity, rotation capacity, load-bearing capacity.
robustness	Ability of a structure to withstand events (like fire, explosion, impact) or consequences of human errors, without being damaged to an extent disproportionate to the original cause.
design criteria	Quantitative formulations describing the conditions to be fulfilled for each limit state.
limit states	States beyond which a structure no longer satisfies the design criteria. These boundaries between desired and undesired performance of the structure are often represented mathematically by “limit state functions”.
limit state function	A function of basic variables, whose attainment of the ‘0’ value characterises a limit state.
ultimate limit states	States associated with collapse, or with similar forms of structural failure. Note: they generally correspond to the loss of load-carrying capacity of a structure or structural element.

serviceability limit states	States corresponding to conditions beyond which specified service requirements for a structure or structural element are no longer met. <u>Note</u>: they are related to user's comfort, risk of deterioration, or intended maintenance.
irreversible serviceability limit states	Serviceability limit states where some consequences of actions exceeding the specified service requirements will remain when the actions are removed.
Reversible serviceability limit states	Serviceability limit states where no consequences of actions exceeding the specified service requirements will remain when the actions are removed.
serviceability criterion	Design criterion for a serviceability limit state.
serviceability constraint	Limit value for a particular serviceability criterion.
reliability or structural reliability	Ability of a structure (or a structural element) to fulfil specified requirements - for safety, serviceability, and durability - over the design working life. It may be evaluated as the probability that the structure will not attain a specified limit state during a specified reference period.
element reliability	Reliability of a structural element which has one single dominating failure mode.
system reliability	The reliability of a structural element which has more than one relevant failure mode, or the reliability of a system of more than one relevant structural element.
probabilistic methods	Calculation methods in which the relevant basic variables are treated as random. <u>Note</u>: this term covers both reliability index methods and fully probabilistic methods.
reliability index	A substitute for the failure probability p_f , defined by $\beta = -F^{-1}(p_f)$, where F^{-1} is the inverse standardised normal distribution.
target reliability level	The level of reliability required ensuring acceptable safety and serviceability.
reliability class	Class (of structures or structural elements) for which a particular specified level of reliability is required
reliability differentiation	The socio-economic optimisation of the resources to be used to build construction works, taking into account all the expected consequences of failures and the cost of the construction.
structural safety	Ability (of a structure or structural element) to resist, with a specified level of reliability, the expected actions (and also specified accidental phenomena) during its construction and anticipated use. <u>Note</u>: the structural safety is related to the ultimate limit states

serviceability	Ability (of a structure or structural element) to show a specified level of reliability during its normal use. Note: the serviceability is related to the serviceability limit states
limit states method	Calculation method in which the intention is to prevent the structure from exceeding specified limit states.
basic variables	A specified set of variables representing physical quantities which characterise actions and environmental influences, geometrical quantities, and material properties (including soil properties).
primary basic variables	A specified set of basic variables, whose variability is of primary importance in design.
model uncertainties	Uncertainties related to the accuracy of a model. For instance: physical uncertainties, statistical uncertainties.
statistical uncertainties	Uncertainties related to the values of statistical parameters, or to the choice of the statistical distributions of the basic variables.
method of partial factors	Calculation method in which allowance is made for the uncertainties and variability assigned to the basic variables by means of representative values, partial factors and, if relevant, additive quantities.
reliability elements	Numerical quantities used in the partial factor format, by which the specified degree of reliability is assumed to be reached. Note: the reliability elements are normally partial factors and additive quantities.
Importance factor	Factor by which the importance of the possible consequences of failure of a given structure is taken into account.
characteristic value	Value (of an action or a material or a geometrical property) chosen - either, on a statistical basis, so that it has a prescribed probability of not being exceeded towards unfavourable values - or, on a non-statistical basis, for instance on acquired experience or on physical constraints (i.e. nominal value)
design value	Value (of a basic variable) used in a design criterion. Note: this value is obtained - either by multiplying or dividing a characteristic value by a partial factor (in case of an action or a material property) - or by applying an additive or subtractive element (to a geometrical data) - or by assessment on the basis of tests.
nominal value	Value fixed on a non-statistical basis, for instance on acquired experience or on physical constraints.
deterministic method	Calculation method in which all basic variables are treated as non-random.
design working life	Duration of the period during which a structure or a structural element, when designed, is assumed to perform for its intended purpose with expected maintenance but without major repair being necessary.
durability	Ability of a structure or a structural element to maintain adequate performance for a given time under expected actions and environmental influences.
life cycle	Total period of time during which the execution and use of a

	construction works takes place.
remaining working life	The period for which an existing structure is intended/expected to operate with planned maintenance.
design situation	Set of conditions under which the design is required to demonstrate that relevant limit states are not exceeded during a specific time interval.
persistent design situation	Design situation that is relevant during a period of time of the same order as the design working life of the structure. Note: generally it refers to conditions of normal use, including possible extreme loading from wind, snow, imposed loads, earthquakes in areas of high seismicity, etc.
transient design situation	Design situation which is relevant during a much shorter period than the design working life of the structure, and which has a high probability of occurrence. Note: it refers to temporary conditions of the structure, of use, or exposure, e.g. during construction or repair.
accidental design situation	Design situation involving possible exceptional conditions for the structure – in use or exposure -, including flooding, fire, explosion, impact or local failure.
seismic design situation	Design situation involving the exceptional conditions when the structure is subjected to a seismic event.
hazard	Exceptionally unusual and severe event, e.g. an abnormal action or environmental influence, insufficient strength or resistance, or excessive deviation from intended dimensions.
environmental influences	Chemical, biological, or physical influences on a structure. They may deteriorate the materials constituting the structure, which in turn may affect its reliability in an unfavourable way.
action	- a set of concentrated or distributed forces acting on a structure (direct action), or - a set of deformations or accelerations imposed on a structure or constrained within it (indirect action).
individual action (or single action)	Action which can be assumed to be statistically independent in time and space of any other action acting on the structure. Note: an individual action may consist of several components, partially correlated together; for example a thermal action may have a uniform component and a gradient component, a traffic load has vertical and horizontal components.
permanent action	Action which is likely to act throughout a given reference period of time, and for which the variation in magnitude with time around its mean value is negligible, or for which the variation is monotonic (i.e. always in the same direction) until the action attains a certain limiting value.
variable action	Action which is likely to act during a given design situation, and for which the variation in magnitude with time is neither negligible nor monotonic.
accidental action	Action which is foreseeable but unlikely to occur with a significant value during the design working life of the structure.

fixed action	Action that has a fixed distribution and position over a structure (or a structural element). This means that the magnitude and direction of each individual force (or deformation or acceleration) are determined unambiguously for the whole structure when determined at one point of it For instance: a static water pressure.
free action	Action that may have any spatial distribution over the structure within given limits. for instance : load of persons on a floor, vehicles on a bridge
load arrangement	Identification of the position, magnitude and direction of a free action.
dynamic action	Action that causes significant acceleration to a structure (or a structural element).
static action	Action that does not cause significant acceleration to a structure (or structural element).
quasi-static action	Static action representing a dynamic action including its dynamic effects.
bounded action	Action which cannot exceed a certain value (exactly or approximately known).
sustained action, transient action	A qualitative distinction, referring to the duration of actions: e.g. the weight of the furniture on a floor is a sustained action, whereas the weight of persons on the floor is a transient action.
self weight	<u>Note</u>: one should avoid the expression "*dead load*" on account of its ambiguity.
prestress	Permanent action resulting from the application of controlled forces to a structure and/or of controlled deformations to it.
geotechnical action	Action transmitted to the structure by the ground, fill or groundwater.
seismic action	Action that arises due to earthquake ground motions.
imposed load	<u>note</u>: one should avoid the expression "*live load*" on account of its ambiguity.
construction load	Load specifically related to execution activities.
reference period	A chosen period of time used as a basis for assessing the design value of variable and/or accidental actions.
representative values of an action	Representative value of an action: a value assigned to the action for a specific purpose, for instance the verification of a limit state.
characteristic value of an action	The principal representative value of an action. It is chosen - either, when a statistical base is available, so that it can be considered to have a prescribed probability of not being exceeded (towards unfavourable values) during a reference period, - or from acquired experience - or on physical constraints. <u>Note</u>: the "reference period" shall take into account the design working life of the structure and the duration of the design situation.

combination value of a variable action	Value chosen for an action in combination with others - in so far as it can be fixed on statistical bases – so that the probability that the effects of the combination will be exceeded is approximately the same as when only the characteristic value of the action is present. This 'combination value' may be expressed as a part of the characteristic value by using a factor $\psi_0 \leq 1$.
frequent value of a variable action	Value determined – in so far as it can be fixed on statistical bases – so that either the total time, within the reference period, during which this value is exceeded is only a small given part of the reference period, or the frequency of this exceedance is limited to a given value. This 'frequent value' may be expressed as a part of the characteristic value by using a factor $\psi_1 \leq 1$.
quasi-permanent value of a variable action	Value determined so that the total period of time for which it will be exceeded is a large fraction of the reference period. The 'quasi-permanent value' may be expressed as a part of the characteristic value by using a factor $\psi_2 \leq 1$.
load case	A set of actions (including load arrangements and imposed deformations) and imperfections, taken into account simultaneously for a particular verification.
combination of action (or load combination)	Set of the design values of different simultaneous actions used for the verification of the structural reliability for a particular limit state.
fundamental combination of actions	Combination of permanent actions and variable actions (the leading action plus the accompanying actions) used for studying an ultimate limit state.
accidental combination of actions	Combination for accidental design situations, involving either an explicit accidental action (e.g. fire or impact) or the situation after an accidental event.
characteristic combination of actions	Combination of permanent and variable actions used for studying a service limit state, where one of the variable actions has its characteristic value.
frequent combination of actions	Combination of permanent and variable actions used for studying a service limit state, where one of the variable actions has its frequent value.
quasi-permanent combination of actions	Combination of permanent and variable actions used for studying a service limit state, where all the variable actions have their quasi-permanent value.
strength	Property of a material indicating its ability to resist mechanical actions. note: it is usually given in units of stress.
characteristic value of a material property	A specified fractile of the statistical distribution of the material property in the supply produced within the scope of the relevant material standard
conversion factor, conversion function	Factor (or function) which converts properties obtained from test specimens to properties corresponding to the assumptions made in calculation models.
design value of a material property	Value obtained - either by dividing the characteristic value by a partial factor γ_M,

	- or by direct determination.
geometrical imperfections	Deviations from the intended geometry of a structure or a structural component
characteristic value of a geometrical quantity	The characteristic value of a geometrical quantity corresponds to - usually the dimension specified in the design - where relevant, a prescribed fractile of the statistical distribution of the quantity.
design value of a geometrical quantity	The design value of a geometrical quantity corresponds to - usually a nominal value - where relevant, a prescribed fractile of the statistical distribution of the quantity. Note: the design value of a geometrical property is generally equal to the characteristic value. However, it may differ in cases where the limit state under consideration is very sensitive to the value of the geometrical property, for example when considering the effect of geometrical imperfections on buckling. In such cases, the design value will normally be established as a value specified directly, for example in an appropriate European Standard or Pre-standard. Alternatively, it can be established on a statistical basis, with a value corresponding to a more extreme fractile (i.e. a rarer value) than applies to the characteristic value.
resistance	Capacity of a structural element or a cross-section of a structural member to withstand actions without mechanical failure. For instance: tension resistance, bending resistance, buckling resistance.
design resistance	Value of a resistance incorporating partial factors
effects of actions (or action effects)	The effects of actions (or action effects) on structural elements (e.g. internal force, moment, stress, strain) or on the whole structure (e.g. deflection, rotation).
structural analysis	Determination of the effects of actions in a structure or part of it. A distinction is generally made between global analysis (considering the whole structure), member analysis (e.g. about buckling), and local analysis (e.g. a cross-section, a connection, a weld).
structural model	An idealisation of the structure, used for the purposes of analysis, design and verification.
calculation model	A simplified description of a physical reality, suitable for calculation. For instance: model for actions, structural analysis model, behaviour model.
damage	Unfavourable change in the condition of a structure that may affect structural performance
deterioration	A process that adversely affects the structural performance including reliability over time due to: - naturally occurring chemical, physical or biological actions - normal or severe environmental actions - repeated actions such as those causing fatigue - wear due to use - improper operation and maintenance of the structure
deterioration model	A mathematical model that describes structural performance as a function of time taking deterioration into account

inspection	On-site non-destructive examination to establish the present condition of a structure.
investigation	Collection and evaluation of information through inspection, document search, load testing and other testing.
load testing	Test of the structure (or part of it) by loading to evaluate its behaviour or properties, or to predict its load bearing capacity
material properties	Mechanical, physical or chemical properties of structural materials
monitoring	Frequent or continuous, normally long-term, observation or measurement of structural conditions or actions.
repair (of a structure)	Improvement of the condition of a structure by restoring or replacing existing components that have been damaged.
safety plan	Plan specifying the performance objectives, the scenarios to be considered for the structure, and all present and future measures (design, construction, or operation, - e.g. monitoring) to ensure the safety of the structure.
structural performance	A qualitative or quantitative representation of the behaviour of a structure (e.g. load bearing capacity, stiffness, etc.) in terms of its safety and serviceability.
upgrading	Modifications to an existing structure to improve its structural performance.

CHAPTER 3
PROCEDURES FOR THE ASSESSMENT OF EXISTING STRUCTURES

Vladislava Návarová[2]

[2]SPŠS, České Budějovice, Czech Republic

Summary

This chapter deals with the general requirements and assessment procedures for existing structures, based on the principles of reliability of structures and the consequences of faults and failures.

1 INTRODUCTION

An assessment of existing structures should be based on EN standards. These standards, however, aren't always sufficiently explicite and operational.

That is the reason why ISO 13822 has been supplemented by several aannexes, which provide particular steps of assessment of existing structures. In general this standard recommends that the load-bearing capacity of particular supporting members be specified, taking into account actual loads and material properties including the influence of structural degradation.

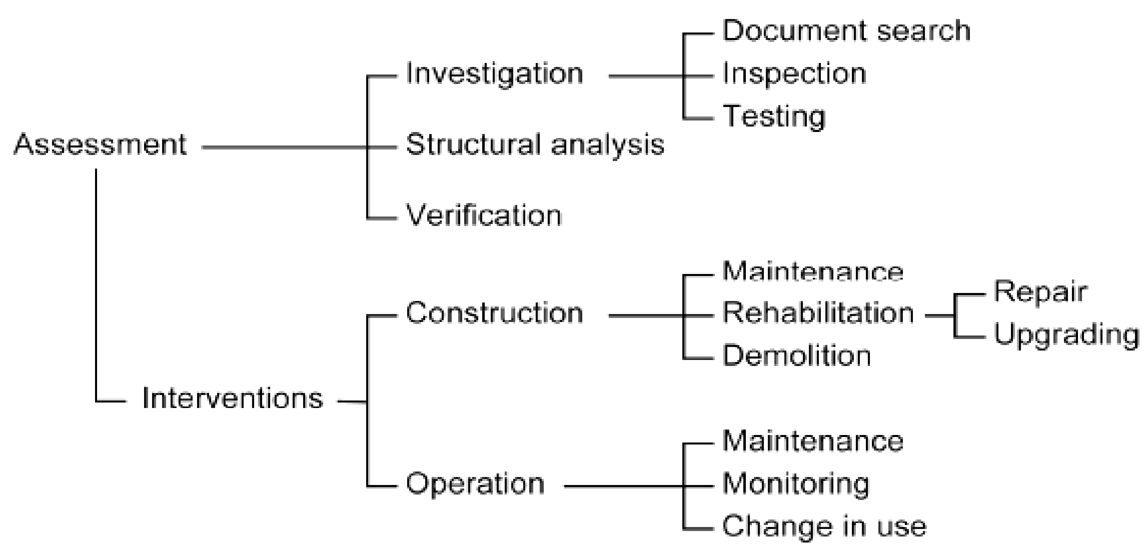
The following circumstances can lead to starting of assessment:

- Degradation of a structure – faults and defects have appeared in the object
- Change in use
- Extension of the working life of existing structures
- Changes of an object, leading to a change in the load
- Required check of working life
- Extraordinary load of existing structures

2 HIERACHY OF TERMS

Annex A
(informative)

Hierarchy of terms



assessment
Set of activities performed in order to verify the reliability of an existing structure for future
investigation
Collection and evaluation of information through inspection, document search, load testing and other testing
inspection
On-site non-destructive examination to establish the present condition of the structure
testing
Tests of material qualities or load testing
analysis
Determining the effects of actions on a structure, determining the causes of observed damage or irregular behaviour
verification
The establishment of a target level of reliability – the level for securing acceptable safety and reliability
measure
Changes proposed to secure a desired level of safety and reliability of a structure
maintenance
Routine intervention to preserve appropriate structural performance
rehabilitation
Work required to repair, and possibly upgrade, an existing structure

repair (of a structure)
Improve the condition of a structure by restoring or replacing existing components that have been
damaged
upgrading
Modifications to an existing structure to improve its structural performance
demolition
Work needed to remove an existing structure
monitoring
Frequent or continuous, normally long-term, observation or measurement of structural conditions or actions
change in method of use
Requirements for the change in the method of use of an existing structure which will secure a desired level of safety and reliability of the structure

3 ASSESSMENT PROCEDURE ACCORDING TO ČSN ISO 13822

3.1 ENTRY DATA

Before the starts assessment of existing structures we have to obtain entry data from a client. From this data we specify the objective of the assessment of an existing structure. The objective of the assessment of an existing structure can be as follows:

- to assess the possibility of a change in use of premises (for example change of attic into a flat)
- to assess the reliability of an existing structure in case of a change in the static model of structures (for example floor additions)
- to assess the reliability of an existing structure with respect to its degradation (the defects and faults of a structure)
- to assess the reliability of a structure with respect to its damage by extraordinary loading

On the basis of preliminary architectural and structural design, we establish the scope of change of structural conditions or scope change load. We assess the possible scenarios of functioniong of existing structures with the inclusion of the influence of change load change, static model of existing structures and changes in the rigidity of an existing structure.

3.2 PRELIMINARY ASSESSMENT

3.2.1 INVESTIGATION

a) DOCUMENTS SEARCH AND STUDY

We perform an examination of the documents and assess its completeness, concerning both extent and accuracy. Next we try to secure all available data on existing structures:

- the history of structural interventions to the existing object and existing structures, especially the weakening of existing structures and the decrease in rigidty of a property

- the history of structural interventions to neighbouring objekts, provided they have an influence on the investigated objekt
- significant effects of the environment on existing structures, such as the extraordinary load as ea fire, flood etc.

b) PRELIMINARY INSPECTION

For an verifikation the real structural state of existing structures and for documenting the faults and defects we perform a preliminary inspection of the object [1]. The inspection is performed with using simple testing and measuring methods.

Figura1. - Photograph of structural changes

At an inspection we collect information about the real state of existing structures:

1. the actual dimensions of particular existing structures and the actual dimensions of follow-up structural compositions. Measurement of the actual dimensions is a prerequisite for:
   - the determination of the real data of characteristic permanent loading actions (for the procedure for determining the degree of permanent loading actions, see Chapter 4)
   - the determination of the real dimensions of the load-bearing members of the existing structure

   the technical solution of details of existing structures
2. the actual load area of existing structures
3. the static model of particular existing structures
4. the fulfillment of structural principles valid for existing structures
5. the conformity of the original project documentation with the actual state of existing structures
6. the materials used for existing structures

7. the surface characteristics of existing structures by means of visual observation and with the help of preliminary surface testing

During the inspection we document data on the failures of existing structures, such as

1. surface charakteristics
2. visible deformations
3. stability loss
4. cracks on existing structures – widths and patterns
5. corrosion and spalling
6. biological actions
7. changes in the surface characteristics of existing structures

While performed the preliminary inspection we photo-document the state of existing structures. During the preliminary inspection we can apply plaster strips and other simply tools for monitoring failures.

The result of a preliminary inspection is

1. a description of the actual conditions of an existing structure including the dimensions, surface characteristics and statik model
2. a quality classification of an existing structure according to the condition and the degree of damage

c) PRELIMINARY INSPECTION OF MASONRY STRUCTURES

By preliminari inspection we search surface change, materials degradation and the cracks. An occurence of defects and cracks in masonry structures indicates a deterioration in structural capacity. We record and document any visible defects during the preliminary inspection. With masonry, we distinguish between structural and non-structural defects [2].

Structural defects are caused by statik load that eventually have a dynamic element. Most often they become evident as deformations, cracks, crushing and local damage.

Non-structural defects are caused by environmental actions, such as increased humidity, temperature, chemical or biological actions.

Cracks in masonry structures can be classified according to the following criteria:

- causes of crack occurence
- scope of deformations to masonry structure
- location of cracks in relation to the masonry elements
- position and shape of cracks - straight, cranked, vertical, horizontal or diagonal
- length and width of cracks
- type of masonry and structure

The inspection has to conclude from the shape, location and direction of the cracks whether the cracks are tensile, pressurized or skidding. Masonry damage often occurs as a result of temperature fluctuations and the consequent occurence of volume changes or as a result of humidity in the environment.

By inspection it is appropriatte to record actual crack pattern and deformations and to find the dependence between the cracks and changes on floor levels.

Figura 2 - Photograph of failures in the masonry

During a preliminary inspection we deduce the physical and mechanical properties of the masonry from surface properties, a visual inspection or with the help of simple tools. The strenght of the masonry depends on to:

- the strenght of the bricks pressure and tension, dimensions of the bricks
- the strenght of the mortar pressure and tension, width of mortar
- technology of implementation masonry, the influence of the mortar consistency and the absorptivity of the bricks
- applied bricklaying
- a faults resulting from design documents
- a faults resulting from applied bricklaying technology
- a faults resulting from the use of the object

d) PRELIMINARY INSPECTION OF TIMBER STRUCTURES

Timber bearing structures can have, under normal circumstances, a shorter working life than other structures. During an inspection of timber structures we observe material charakteristics, structural model and actual dimensions of elements and the possible actions of wood-decaying fungi or wood-destroying insects [3].

The most common causes of defects of timber structures are as follows:

- rainwater running into the structure and consequent wood decay
- increased humidity in the environment
- over limit loads
- damage of timber mass caused by wood-decaying fungus or wood-destroying agents

The most common defects of timber structures are as follows:

- over limit deformation of elements
- cracks in the wooden element
- damage due to pressure on the wooden element
- damage of timber structure caused by fungi, insects or rot

During an inspection of a timber structure, we pay a special attention to the surface structure of the timber, its structure under the surface, the dimensions of the timber element, the depth of the timber damage and its extent and the size of the deformation of elements.

Figura 3 - Photograph of an example of wood-destroying insect action

The condition of the timber structures in the light of any damage is assessed by a visual inspection, by tapping on the timber element and by spading into the timber. Precise confirmation of the presence of wood-decaying fungi or insects is carried out by a specialized mycologist during a detailed inspection.

During a preliminary inspection, the physical and mechanical properties of the timber are deduced from surface properties through a visual inspection or with the help of simple tools.

e) PRELIMINARY INSPECTION OF STEEL STRUCTURES

During a preliminary inspection of a steel structure we observe deformations and transformations, structural patterns, the actual dimensions of elements and the occurence of corrosion. Next we observe the design and the state of the links within steel structures, including binders and the condition of the bearing welds of a steel structure.

The most common causes of defects of timber structures are as follows:

- excessive humidity
- aggressive environment
- over limit loads
- faults in design documents – insufficient dimensions of elements

The most common defects of steel structures are:

- over limit deformation of elements
- loss of stability by buckling and tilting
- weakening of steel by corrosion
- failure of joits and welds

During an inspection of a steel structure, special attention is paid to the surface structure of the steel, the extent of the corrosion, the dimensions of the steel element, the state of the joits and the condition of the bearing welds and jag bolts and the level of deformation of the steel elements.

The condition of the steel elements is judged first by a visual inspection. If there is no reason for doubt, the level of strength is presumed from information contained in the original documentation, from historical records classifying the material used, and from corresponding data, visual inspection and information derived from an examination of surface rigidity.

The defect situation of the steel structure is then documented.

f) PRELIMINARY INSPECTION OF REINFORCED CONCRETE STRUCTURES

Reinforced concrete structures are noted for their long working life. Working life can be reduced due to design document faults, e.g. [4]:

- insufficient tensile bar
- insufficient web reinforcement
- insufficient surface layer of reinforcement
- insufficient length of bearing members

Working life can also be reduced due to technology mistakes, e.g.

- implementation at low or minus temperatures
- incorrect positioning of reinforced
- failures cover
- unsufficient concrete processing
- improper care after placement

The working life of reinforced concrete structures can also be reduced by

- excessive humidity
- aggressive environment
- extreme temperatures
- loads that exceed the maximum acceptable limits

Figura 4 - Photograph of degradation of a reinforced concrete armature.

The most common defects of reinforced concrete structures are:

- armature corrosion and subsequent unreliability of a structure
- excessive deformation and transformation of elements
- tension cracks
- skidding cracks
- pressure cracks

During an inspection of a reinforced concrete structure, special attention is paid to the surface structure of the concrete, the extent of corrosion, the dimensions of the reinforced concrete element, the location and size of cracks and the extent of transformation of particular elements.

The condition of reinforced concrete elements is judged first by a visual inspection. If there is no reason for doubt, the level of rigidity is presumed from information contained in the original documentation, from historical records classifying the material used, and from corresponding data, visual inspection and information derived from an examination of surface rigidity.

The defect situation of the reinforced concrete structure is then documented.

If there is no reason for doubt, the level of rigidity is presumed from information contained in the original documentation, from historical records classifying the material used, and from corresponding data, visual inspection and information derived from an examination of surface rigidity.

The defect situation of the reinforced concrete structure is then documented.

3.2.2 ANALYSIS

As the standard recommends, the load capacity of the bearing members and structures should be specified with respect to the actual load, including the influence of real degradation of an existing structure.

The first stage is to carry out a classification of an existing structure in relation to the status and extent of the damage.

We classify as serious all the faults and defects that fundamentally affect the reliability of an existing structure as a whole. Mainly they are active (i.e. still developing) faults and defects which gradually evolve and spread.

We classify as less serious all the faults and defects which are found locally and do not fundamentally affect the reliability of existing structures as a whole.

When classifying faults, various criteria can be used. With regard to their seriousness they can be classified as:

a. Minor faults and defects – lesser faults and defects which do not affect the reliability of an existing structure, those faults and defects which are structurally unimportant, those which are of an aesthetic only, and whose repair would have no effect on the reliability of the structure.

b. Major faults and defects – faults and defects which have a high probability of leading to the collapse of an existing structure, or increasing the reliability of an existing structure, faults and defects which are static important and which require timely intervention.

c. Critical faults and defects –called emergency faults and defects – those which can be hazardous to people inside or near the structure and which require immediate intervention.

Next, the structural models of particular parts of a construction are specified, and we describe the causes of observed faults and defects, or the reasons for abnormal behaviour. When repair work is not sufficient, the cause for the fault must be found and measures proposed which will eliminate the cause [5].

Ways of detecting the causes of faults:

- checking structural members and elements, i.e. comparing existing structures and their real state with original specifications.
- visual inspection of faults
- inspection of faults with simple tools
- partial removal of surface layers
- deep boring of an examined element
- load testing
- local or band probes
- observation of changes in time

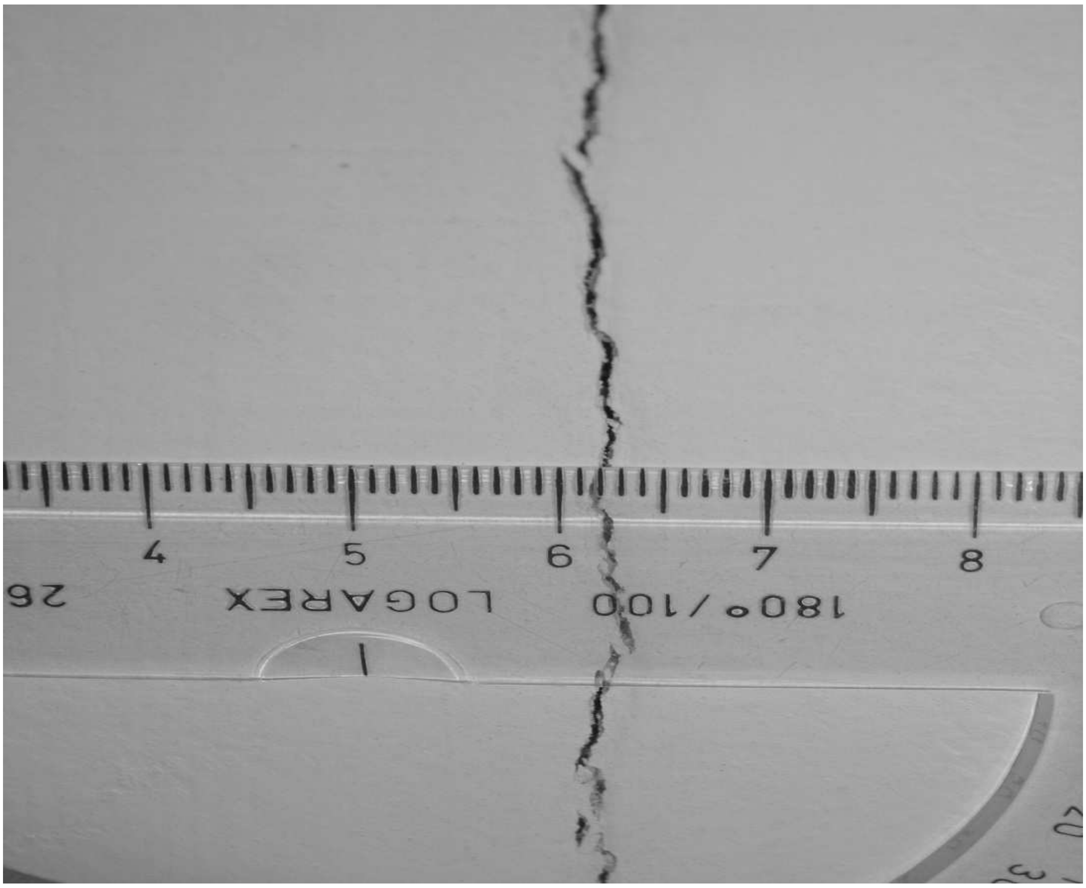


Figura 5 - Photograph of documenting faults

Ways of observing changes over time:

- with the help of plaster indicators – strips of 8-10mm width, on which the start date is recorded and later the dates of follow-up checks
- with the help of a numeric gauge – two ground pins are positioned at the edges of a crack and their respective drift is measured
- with the help of a deformemeter – a structure is rigged up with discs, whereby any motion is recorded
- with the help of surveying tools

3.2.3 PRELIMINARY VERIFICATION OF A STRUCTURE

The preliminary assessment is an assessment of the reliability and level of public safety of an existing structure with respect to recorded data.

According to the ISO standard, an earlier level of acceptable behaviour pertaining to an existing structure under examination can be used as a starting point. Structures designed and executed according to earlier valid standards can be regarded as reliable for all kinds of load, except when there is an extraordinary case of load in the following circumstances:

- no critical or major faults and defects are documented during the inspection
- acceptable conduct of the structure over a sufficiently long period is recorded
- no changes in load of the existing structure under inspection are predicted
- basic transfer of load and tension is secured

Based on a preliminary inspection, the following is stated:

LOAD:

a) actual weight, bulk density, permanent load

- The characteristic value of the actual weight of an existing structure can be determined by data taken from the preliminary inspection using statistical methods.
- The bulk density of particular layers can be determined from the median value of bulk density. In other cases the standard for determining load is followed.
- The characteristic value of the permanent load capability is determined by an estimate of the average, and by an estimate of the divergence from the average. The ISO standard recommends taking at least five samples.

b) utility load

- For the characteristic values of the utility load of existing structures, see the chart for standards of load in structural engineering, according to category A-K.
- The characteristic value of load by relocatable partition walls can be generally seen as an addition to utility load, on condition that the load distribution is secured and the actual weight of the partition is less than 3,0kN/m (the partition weight itself is less than 1,0kN/m – addition 0,5kN/m^2; the partition weight itself is 1,0 – 2,0kN/m – addition 0,8kN/m^2; the partition weight itself is 2,0 – 3,0kN/m – addition 1,2kN/m^2).

c) Snow load

- When assessing existing structures, certain problems can occur as a result of the more restrictive standard for snow load.
- Provided an existing structure does not comply with the tightened requirements for snow load, the structures can be strenghtened, or the reliability of the existing structure can be conditioned, by removing snow when a certain height of snow is reached.

d) Wind load

- The wind load of existing structures is determined by valid EN standards.

MATERIAL PROPERTIES:

- It is essential to take into account the actual material properties which are determined or checked by the inspection of existing structures. When determining material properties the influence of the degradation must be taken into account.

DIMENSIONS:

- The dimensions of an existing structure are given in the designs and provided by inspection and measuring.

Provided that the particular elements of load an existing structure are given, as well as its material properties and dimensions, an assessment of the reliability of a structure for an ultimate limit states.We perform for these limit states combinations of load according to partial factors in the valid EN standards.

3.2.4 DECISION ON IMMEDIATE MEASURE

Provided a preliminary inspection or check indicates that a structure is in a dangerous or emergency condition, an immediate decision on prompt and appropriate action must be taken in order to reduce the danger to public safety. This decision must be reported to the client. He/she is then obliged to intervene without delay.

3.2.5 RECOMMENDATIONS FOR FURTHER PROCEDURE

After the preliminary check, the need for any further possible action is assessed:

A) A detailed assessment is unnecessary

The preliminary check is sufficient to assess the reliability of an existing structure. The preliminary inspection or preliminary check indicates that the structure is reliable for its intended use over its remaining working life. In this case, a detailed assessment is unnecessary and it is possible to move on to reporting the results. Based on the preliminary inspection, a report on the results of the assessment is produced, including an appraisal and a decision taken on whether an existing structure is sufficiently reliable or not.

B) A detailed assessment is necessary

The critical parts of an existing structure are specified, these being necessary for any further assessment based on a detailed inspection of the existing structure.

3.3 DETAILED ASSESSMENT

3.3.1. DETAILED INSPECTION

A detailed inspection is carried out with the help of both destructive and non-destructive methods. Load-bearing tests of existing structures are also used. The extent and nature of a detailed inspection of an existing structure depends on the particular structure and requirements resulting from the detailed inspection.

During a detailed inspection, the following steps can be taken:

e) time-dependent inspection of structural defects and faults, followed by a determination of their causes
f) measuring transformations of existing structures during a service load
g) measuring transformations of existing structures during load tests
h) sampling in order to determine material properties, such as the rigidity of the material

Before the testing part of the preliminary inspection, it is necessary to agree on the plan and procedure of the testing with the client and the certified testing organization which will carry out the tests. The extent of the tests and the number of samples used must be specified in a testing plan. During sampling, it is necessary to proceed in such a way that the safety of the existing structure is not jeopardized.

During the detailed inspection, we record the progress of the inspection, including the date and time. The documenting of all material samples, and the recording of the results of all tests and measurements, is an essential part of the detailed inspection.

An observation is made of the effects of the environment and any surrounding traffic e.g. the infuence of vibrations on the structure. When appropriate, the impact on the structure of environmental temperature and humidity is noted.

3.3.2. DETAILED ANALYSIS

The structural analysis carried out as part of the preliminary inspection of the structure is accompanied by an analysis of the samples, an assessment of the time dependence of any structural defects (such as cracks) and eventually a report concerning the results of load testing, provided these tests have been carried out:

a) the assessment of samples – determination of the material properties of the structure
b) the assessment of the time dependence of the defects
c) load testing results

It is advisable to compare the test results to the anticipated values based on the available documentation, and on the results of the preliminary checks. If there is a large discrepancy from the anticipated result, this discrepancy should be reviewed, and eventually additional tests should be carried out.

3.3.3. DETAILED VERIFICATION

A detailed assessment is an evaluation of the reliability of an existing structure with respect to the documented data derived from a detailed inspection of existing structures.

The assessment must result from the concept of limit states, and can be carried out using the method of partial factors or the methods of reliability theory.

An economic and social aspect admits bigger differences between the reliabilities of existing and newly designed structures. These differences are implemented in the assessment with the help of the target reliability level. For existing structures lower target reliability levels can be used, provided they are justified on the basis of socio-economic aspects.

The partial factors, which are listed in current standards, can be, in the case of existing structures, modified according to the results of the inspection and the tests (see annex).

Examples of target reliability levels are given in the following chart. A particular procedure for determining the target reliability level and thereby determining the partial factors is given in the annex of this manual.

Table F.1 – Illustrations of target reliability level (ISO13822)

Limit states	Target reliability index	Reference period
serviceability		
reversible	0.0	intented remaining working life
irreversible	1.5	intented remaining working life
fatigue		
can be inspected	2.3	intented remaining working life
cannot be inspected	3,1	intented remaining working life
Ultimate		
very low consequence of failure	2.3	L_S in years a)
low consequence of failure	3.1	L_S in years a)
medium consequence of failure	3.8	L_S in years a)
high consequence of failure	4.3	L_S in years a)

3.4. ASSESSMENT RESULTS

The results of the assessment of an existing structure will be clearly described in a report on the results of the assessment of the existing structure. Detailed contents of the report on the results of the assessment of existing structures are given in the annex.

The outcomes of the assessment should clearly describe the state of the existing structure in view of reliability and safety. In the conclusion, those elements which are satisfactory and those are not should be clearly described. Next it is necessary to determine the conditions for the use of the structure and to sum up the proposed structural and operational measures. The conclusion should also clearly define the eventual discrepancy between the results of the structural assessment and the actual condition of the structure, e.g. the fact that although the assessment indicates it is not safe, the structure actually does not reveal any defects.

The safety and serviceability of the structure must be evaluated at the end of the assessment.

a) Safety assessment (according to ISO 13822)

If the structures were designed and executed according to standards that were valid earlier, and even if the standards derived from time-proven structural experience were not used, it is assumed that they are safe for all kinds of load-bearing, except for extraordinary actions (including seismic) on the condition that:

- thorough inspection does not find any signs of significant damage, overloading or degradation;
- the structural system is assessed, including critical details and their assessment in view of tension transfer;
- the structure shows satisfactory behaviour during a sufficiently long period of time, in the course of which unfavourable actions occured due to usage and environmental effects;
- the estimate of the degradation, when the current state and planned maintenance are considered, secures a sufficient durability;
- after a sufficiently long period of time, no changes which could significantly increase the load occur, and neither are any such changes expected.

b) Serviceability assessment (according to ISO 13822)

If the structures were designed and executed according to standards that were valid earlier, and even if the standards derived from time-proven structural experience were not used, it is assumed that they are safe for all kinds of load-bearing, except for extraordinary actions (including seismic) on the condition that:

- thorough inspection does not find any signs of significant damage, overloading or degradation;
- the structure shows satisfactory behaviour during a sufficiently long period of time, in view of damage, overloading, degradation, transformation or vibration;
- no changes occur in the structure or its usage which could significantly change its load-bearing capacity, including environmental actions on the structure or any part thereof; and an expected course of degradation, determined with respect to the current condition and planned maintenance, does not jeopardize the working life of the structure.

A proposal of structural measures to be taken, or a utilization plan, is part of the report on the results of the assessment of the existing structure.

Structural measures can be proposed, such as rehabilitation, repair, upgrading and demolition. A detailed description of the proposed structural measures on existing structures, having been drawn up on the basis of the foregoing assessment, is part of the report. The design documentation of the proposed structural measures is not part of this report, it is part of the next level of documentation.

A utilization plan can be proposed as a subsequent monitoring of an existing structure, or as requirements for a change (reduction) of operational, eventually climatic, actions – i.e. a change in usage of the structure.

The report on the results of the assessment of the existing structure serves as a basis for a decision on the part of the client regarding further actions related to the assessed existing structure. In the case of a client who does take an appropriate course of action in order to secure the safety of the general public – for example at an emergency situation of the existing structure – the author of the report can (in specified cases must) inform the authorities.

4. THE STRUCTURE OF THE REPORT

For a model report see the annex C. Basic structure of a report according to ISO 13822

1 Titler page

The following items should be included: title, date, client and author (full name and address of the civil engineer or the company).

2 Name of the engineer and/or firm

The names of the persons who carried out the assessment along with the names of the client representatives and other participants.

3 Summary

The problem is summed up clearly and briefly in one or two pages, important parts of the inspection are stated along with the main conclusions and recommendations, including all important objections and/or rejections.

4 Table of contents

The following items should be included:

a) scope of the assessment;
b) description of the structure;
c) investigation;
– reviewed documents,
– inspection items,
– procedures of sampling and testing,
– test results;
d) analysis;
e) verification;
f) data analysis;
g) review of intervention options;
h) conclusions and recommendations;
i) reference documents and literature;
j) annexes.

CONCLUDING REMAKS

When carrying out the evaluation of existing structures, despite all the care taken in the preliminary investigation avoided uncertainties. These uncertainties in the assessment must then specify the detailed investigation.

Details of these procedures are clearly described and illustrated in the standard ISO 13,822. This standard ISO 13822 is not in conflict with CSN 730038, standard ISO 13822 complements the CSN 730038 with other criteria and information.

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CHAPTER 4: METHOD FOR ASSESSMENT OF BUILDINGS – PARTIAL FACTOR METHOD

Roman Gottfried[1]

[1]Secondary Technical School of Civil Engineering, Resslova 2, České Budějovice

Summary

Existing structures have often been designed using the method of allowable stresses or safety factors. If they were designed by the preceding partial factor method, then the procedures for the determination of characteristic and design values of basic variables, load combinations and commonly used computing models might differ from current European and international standards. The basic method of European EN Eurocodes regulations as well as international ISO regulations is the partial factor method.

1 INTRODUCTION

The partial factor method for the assessment of existing structures, or for designing their recovery, has its advantages, and also disadvantages. It is not always obvious, if it is necessary to apply the same values of load partial factors and material properties as when designing new structures. The requirements of Eurocodes for the design of new structures are usually more conservative than those given in previous ČSN standards.

The procedures for reliability verification of structures using the partial factor method are described in detail in ČSN EN 1990 1990 [1] and ISO 2394 [2]. These standards specify how to determine the characteristic and design values of basic variables (load, materials, geometrical data). They also provide rules for combinations and procedures for determining load effects and structural resistance. The recommended values of partial factors for load effects and material properties were determined based on calibrations, by comparing them with previous standards and also by using probabilistic procedures. A certain reliability level for structures has been assumed (in common cases, the life span of a structure is 50 years and the ultimate limit state is the standard value of the reliability index β=3,8 – the method of determining it is not covered in this chapter). If the reliability index for an existing structure under consideration is different from that accepted for structural design, then it is possible to adjust the partial factors for verification of the existing structure. In common cases it is recommended that the values of partial factors be applied according to current standards and the characteristic values of material and geometric properties be determined taking into account the actual state of the existing structure.

2 PRINCIPLES OF DESIGNING BASED ON LIMIT STATES USING THE PARTIAL FACTOR METHOD

Characteristic elements confidence partial factors combination

When using the partial factor method, it must be verified in all real design situations that none of the limit states has been exceeded (ultimate and serviceability limit states). In reliability verification the following values must be used in design calculation:

- design values of actions (design values of **load effects**)
- design values of **material properties,** dimensions etc.

2.1. Load

Design values of load F_d are determined by multiplying the representative values of load F_{rep} and the partial factor of load γ_F. The representative values of load are determined by multiplying the characteristic values of load F_k and the (combination) coefficient ψ.

The above mentioned design values of load Fd enable the load combinations to be set for different design situations for the ultimate limit state. The design value of load effects, E_d, in these combinations mustn´t exceed the design value of the relevant resistance R_d ($E_d \leq R_d$).

The characteristic values of load F_k and the representative values of load F_{rep}, mentioned above, enable to set load combinations for the serviceability limit state. Using the load combinations determined for the serviceability limit state, the design value of load effects (for example deformations, cracks etc.) is calculated. The design value of load effects in these combinations must not exceed the design value of relevant serviceability criterion ($E_d \leq C_d$).

2.1.1 Charakteristik load values

The numerical values of characteristic load F_k are generally specified:

- in the corresponding technical regulation (for example ČSN EN 1990) by an average, upper or lower limit, or possibly by a nominal value (no relation to any known statistic distribution).
- in the project or by the relevant responsible authority (for example ČHMÚ) on condition that all general provisions of the relevant regulation are observed (for example ČSN EN 1990)

The principles for the determination of numeric values of characteristic loads F_k differ for specific types of time dependent loads. From the point of view of variability in time we classify these basic types of loading:

- permanent – G – e. g. the weight of structures, permanent equipment of structures etc.
- variable – Q – e. g. imposed loads on ceilings, snow, wind etc.
- accidental– A – e. g. explosions, vehicle impact etc.

2.1.1.1 Values of permanent characteristic loads

The numerical values of permanent characteristic loads are presented:

- by a single value G_k – the average of values gained by measurement, if the variability of the measured G is small and does not significantly change during the working life(the variation coefficient is not bigger than 0,1 – for determining the characteristic value of the self-weight, which is a substantial part of total load, the value of variation coefficient is not bigger than 0,05 – determination of the variation coefficient V_x is not covered in this chapter)
- by a single value G_k – an average density specified in ČSN EN 1991-1-1 (Actions on structures - General Load - Densities, Self-Weight and Imposed Loads on Structures) multiplied by nominal dimensions.

- by two values of $G_{k.inf}$ (lower) and $G_{k.sup}$ (upper) – the value of $G_{k.inf}$ is the value in the place of quantile 0,05 and the value of $G_{k.sup}$ is in the place of quantile 0,95 in the normal (Gauss) statistic distribution G, if the variability of measured G is not small (i.e. the variation coefficient is bigger than 0,1, or it concerns a structure with a big sensibility to variability of G).

Probability density $\varphi(u)$

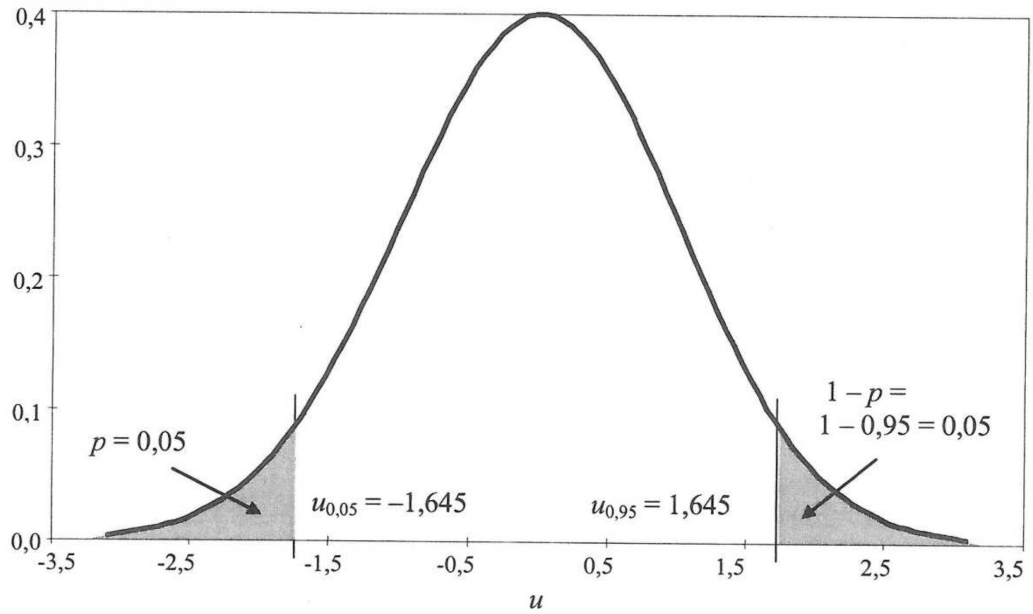

Figure 1: The upper and lower quantile of standardized random quantity U (in this case G) with normal distribution.

In most cases we assume that the variability G is small and it will be determined as a single value by an average from measurements or as a single value determined in ČSN EN 1991-1-1.

For determination of numeric characteristic values of permanent load in existing structures it is recommended to take into account the actual state of a given structure determined for example by testing.

EXAMPLE 1: see “Handbook for the Assessment of Existing Structures“[4]. When determining the characteristic value of concrete density we assume that:

- the density has normal distribution,
- the average value m_G determined by measurement is 16,8 kN/m³,
- the standard deviation a) σ_G = 0,5 kN/m³, b) σ_G = 1,8 kN/m³ (standard deviation is calculated from the dispersion σ_G^2 as $\sigma_G = \sqrt{\sigma_G^2}$, the dispersion is a sum of products of deviations squared in individual measurements with the frequency of their occurrence in [%])

Variation coefficient $V_G = \sigma_G/m_G$. For a) V_G = 0,03, b) V_G = 0,10.

In the case of a) the variability is low (V_G = 0,03) and if the structural self-weight has no significant influence on the structural reliability, it is sufficient to determine a single characteristic value G_k as an average G_k = 16,8 kN/m³.

In the case of b) the variability is high (V_G = 0,10) and it is necessary to distinguish the characteristic value in cases, when the structural self-weight has an unfavourable influence ($G_{k.sup}$) and when a favourable one ($G_{k.inf}$). In the image 2.1. we can read the value of quantile $u_{0,05}$ = - 1,645 and the value of quantile $u_{0,95}$ = +1,645

$G_{k.inf} = m_G(1+ u_{0,05} \times V_G)$ = 16,8(1 – 1,645 x 0,1) = 14,0 kN/m³

$G_{k,sup} = m_G(1+ u_{0,95} \times V_G) = 16{,}8(1 + 1{,}645 \times 0{,}1) = 19{,}6$ kN/m³

It shows that for higher values of variation coefficients the upper and lower value G_k can differ significantly and it is necessary to consider them separately. This procedure, the case of a) and b), is applicable when a sufficient number of tests has been performed.

The characteristic values of permanent loads determined by testing can also be acquired based on the procedure stated in the national appendix NA.2.5 to the article 4.6.3 of ČSN ISO 13822 [5] standard. Bases of the design of structures – Assessment of Existing Structures.

From the research results of n samples $g_1, g_2, \dots, g_n$ the characteristic value of permanent load is determined using an average m_G and the standard deviation s_G based on the relations:

$$G_k = m_G \pm k_n \times s_G,\ \text{kde}\ m_G = \Sigma g_i/n\ \text{a}\ s_G^2 = \Sigma(g_i - m_G)^2/(n-1)$$

Coefficient k_n depends on the number of extracted samples and it is mentioned below in the chart NA.1. of standard [5].

Tab.1 - Coefficient values k_n for the determination of the permanent load characteristic value based on the number of extracted samples.

number of samples n	factor k_n	number of samples n	factor k_n
5	0,69	15	0,35
6	0,6	20	0,3
7	0,54	25	0,26
8	0,5	30	0,24
9	0,47	40	0,21
12	0,39	>50	0,18
For intermediate values of the factor k_n samples determined by linear interpolation The factor k_n is determined by assuming a normal distribution permanent load			

EXAMPLE 2: See handbook "Specification of actions by the Assessment of Existing Structures"[6]. When determining the characteristic values of concrete density we assume that:

- the number of testing measurements is small (in this case 6)
- an average m_G determined based on measurement is 16,8 kN/m³
- standard deviation s_G = 1,8 kN/m³ (standard deviation is calculated from dispersion s_G^2 as $s_G = \sqrt{s_G^2}$)
- $k_n = 0{,}6$ (from the chart NA.1 of standard [5])

$G_k = 16{,}8+ 0{,}6 \times 1{,}8 = 17{,}88$ kN/m³ in the case that the self-weight of a structure has an unfavourable influence.

$G_k = 16{,}8- 0{,}6 \times 1{,}8 = 15{,}72$ kN/m³ in the case that the self-weight of a structure has a favourable influence.

This procedure is applicable when there are a small number of tests.

2.1.1.2 Values of variable characteristic loads

The numerical values of variable characteristic loads Q_k are presented:

- by an upper or lower value with determined probabilities that it will not be exceeded during the specific reference period,
- by a nominal value that may be determined, if a relevant statistical distribution is unknown.

The numerical values of variable loads Qk are provided in the relevant charts and parts of Eurocode 1.

For characteristic values of climatic loads, we usually consider the distribution of extreme values during a specific reference period with the probability of 0,02 being exceeded during one year. This equals an average return period of 50 years for the time dependent part of the load.

2.1.1.3 Values of accidental loads

The numerical values of accidental loads are determined directly in the design values Ad for the specific project.

2.1.2 Representative values of actions

The representative values of actions Frep are determined by the characteristic value F_k multiplied by the combination coefficient ψ. Generally, we assume $F_{rep} = \psi\ F_k$. The coefficient ψ acquires values of 1,0 or ψ_0, ψ_1, or ψ_2. These coefficients express a decrease in the probability of exceeding the design values of actions for several variable actions at the same time.

For permanent actions the coefficient ψ is considered with the value of 1,0. It is possible to assume:

$$G_k = G_{rep} \tag{4.1}$$

For variable loads we consider the coefficient ψ with a value of 1,0 or ψ_0, ψ_1, or ψ_2. Individual coefficients (ψ = 1,0, ψ_0, ψ_1, or ψ_2) are given for individual load combinations (see below – load combinations for the ultimate limit state and the serviceability limit state).
Generally, it is possible to assume:

$$Q_{rep} = \psi\ (\psi_0,\ \psi_1, \text{ or } \psi_2) \times Q_k. \tag{4.2}$$

Values for ψ_0, ψ_1, or ψ_2 are provided in ČSN EN 1990 – see Table 2

Table 2 - Recommended values of ψ factors for buildings

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-			
Category A : domestic, residential areas	0,7	0,5	0,3
Category B : office areas	0,7	0,5	0,3
Category C : congregation areas	0,7	0,7	0,6
Category D : shopping areas	0,7	0,7	0,6
Category E : storage areas	1,0	0,9	0,8
Category F : traffic area, vehicle weight ≤ 30kN	0,7	0,7	0,6
Category G : traffic area, 30kN < vehicle weight ≤ 160kN	0,7	0,5	0,3
Category H : roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)*			
Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
NOTE The ψ values may be set by the National annex. * For countries not mentioned below, see relevant local conditions.			

2.1.3 Design values of actions

Design values of actions F_d are determined by the representative values of actions F_{rep} multiplied by the partial coefficient of actions γ_F.

Generally, it is possible to state $F_d = F_{rep}\,\gamma_F$. (4.3)

Design values of actions are determined by the relations:

$G_d = \gamma_F\ G_k$ for permanent loads (γ_F can be expressed in this equation as γ_G) (4.4)

$Q_d = \gamma_F\ \psi\ Q_k$ for variable loads (γ_F can be expressed in this equation as γ_Q) (4.5)

A_d = always determined by a value for a specific project

2.1.4 Design values of load effects

Design values of load effects Ed are usually provided from a simplified relation:

$E_d = E\{\gamma_{Fi}\ F_{repi};\ a_d\}$, i ≥1 (4.6)

a_d is the design value of geometric data

F_{repi} is the representative value of action (see above)

γ_{Fi} is the value of the partial factor (see below)

2.1.5. Partial factors of actions γ_F

The partial load factor γ_F takes into account:

- unfavourable deviations of action
- inaccuracies of the actions model
- the uncertainties of load effects determination (generally, load effects also depend on material properties – e. g. statically indefinite structures).

The partial load factor γ_F is determined as the product of the model uncertainty factor γ_{Ed} and the partial load factor γ_f.

$\gamma_F = \gamma_{Ed}\ \gamma_f$ (4.7)

γ_{Ed} is the model uncertainty factor, which takes into consideration model uncertainties of load effects and in some cases uncertainties of load models.

γ_f is the partial load factor, which takes into consideration possible unfavourable deviations of load values from the representative values.

The values of partial load factors γ_F in limit states of load-bearing capacity, regarding material damage, are consider based on recommendation in ČSN EN 1990 as:

- a permanent load with a favourable effect (γ_{Ginf}): γ_f = 0,875, γ_{Ed} = 1,20, $\gamma_F \approx$ 1,00
- a permanent load with an unfavourable effect (γ_{Gsup}): γ_f = 1,125, γ_{Ed} = 1,20, γ_F = 1,35
- a variable load (γ_{Qinf}): γ_f = 1,350, γ_{Ed} = 1,10, $\gamma_F \approx$ 1,50

2.1.6 Load combinations for load-bearing capacity limit states

2.1.6.1. combinations for permanent and temporary design situations

(**EQU**) The limit state is used for an assessment of the static balance of a structure as a whole. We consider the possibility of e. g. tilting, sinking, emergence of a structure etc. The following condition must be verified for this limit state:

$E_{d,dst} \leq E_{d,std}$ (4.8)

$E_{d,dst}$ is the design value of a destabilizing load effect

$E_{d,std}$ is the design value of a stabilizing load effect

The strength of structural materials or the foundation soil are not usually decisive.

Load effects *E*d in a combination for the limit state **EQU** can be expressed:

$$\sum_{j\geq 1}\gamma_{G,j}G_{\mathrm{k,j}}"+"\gamma_P P"+"\gamma_{Q,1}Q_{\mathrm{k},1}"+"\sum_{\mathrm{i}>1}\gamma_{Q,i}\psi_{0,i}Q_{\mathrm{k},i} \tag{4.9}$$

$\gamma_{G,j}$= 1,1 if the permanent load has an unfavourable effect (destabilizing), $\gamma_{G,j}$= 0,9 if the permanent load has a favourable effect (stabilizing)
$\gamma_{Q,1}$ ($\gamma_{Q,i}$) = 1,5 if the variable load has an unfavourable effect (destabilizing) , $\gamma_{Q,1}$ ($\gamma_{Q,i}$) = 0 if the variable load has a favourable effect (stabilizing)
P (γ_p) denotes the load prestress

(**STR**) The limit state is used for verifying the mechanical resistance of load-bearing structures and elements, when the geotechnical load is not taken into account. It is usually the limit state associated with achieving the structural material strength (the concrete strength, the slip limit of reinforcement, the timber strength etc.). It monitors the inner failure of a structure, or of load-bearing elements.

(**GEO**) The limit state is used for designing load-bearing elements, which involves a geotechnical load (bases, posts, underground walls etc.) It considers a possible failure of the foundation soil in sites where the soil firmness or the rock foundation are important for the load capacity.
The following condition must be verified for these limit states:

$$E_d \leq R_d \tag{4.10}$$

E_d is the design value of load effect (inner strength, momentum etc.)
*R*d is the design value of relevant load capacity

Load effects E_d in a combination for the limit state **STR** and/or **GEO** can be expressed:

$$\sum_{j\geq 1}\gamma_{G,j}G_{\mathrm{k,j}}"+"\gamma_P P"+"\gamma_{Q,1}Q_{\mathrm{k},1}"+"\sum_{\mathrm{i}>1}\gamma_{Q,i}\psi_{0,i}Q_{\mathrm{k},i} \tag{4.11}$$

Or alternatively, as a less favourable combination from the following two expressions:

$$\sum_{j\geq 1}\gamma_{G,j}G_{\mathrm{k,j}}"+"\gamma_P P"+"\gamma_{Q,1}\psi_{0,1}Q_{\mathrm{k},1}"+"\sum_{\mathrm{i}>1}\gamma_{Q,i}\psi_{0,i}Q_{\mathrm{k},i} \tag{4.11a}$$

$$\sum_{j\geq 1}\xi\gamma_{G,j}G_{\mathrm{k,j}}"+"\gamma_P P"+"\gamma_{Q,1}Q_{\mathrm{k},1}"+"\sum_{\mathrm{i}>1}\gamma_{Q,i}\psi_{0,i}Q_{\mathrm{k},i} \tag{4.11b}$$

$\gamma_{G,j}$= 1,35 if the permanent load has an unfavourable effect, $\gamma_{G,j}$= 1,0 if the permanent load has a favourable effect
$\gamma_{Q,1}$ ($\gamma_{Q,i}$)= 1,5 if the variable load has an unfavourable effect, $\gamma_{Q,1}$ ($\gamma_{Q,i}$) = 0 if the variable load has a favourable effect
ξ = 0,85 (reduction coefficient for unfavourable permanent loads)
P (γ_p) denotes the load prestress

When assessing existing structures it is convenient to consistently use the alternative expressions (4.11a) and (4.11b) of combinations for **STR** and **GEO**. The use of the reduction coefficient for unfavourable permanent loads ξ or the use of the combination coefficients $\psi_{0,1}$ a $\psi_{0,i}$ often approximates the load values considered in standard frames ČSN EN to original loads.

EXAMPLE 3: Determination of partial factors of load γ_F for individual loads in combinations for the limit states EQU and STR on the beam with an overhanging end.

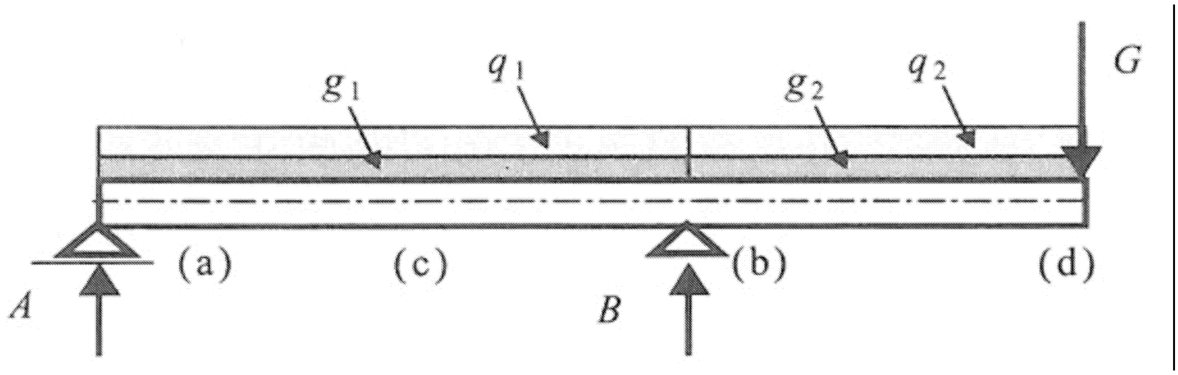

Figure 2: Beam with overhangs - assuming three independent permanent loads g_1,g_2,G and two independent variables loads q_1, q_2.

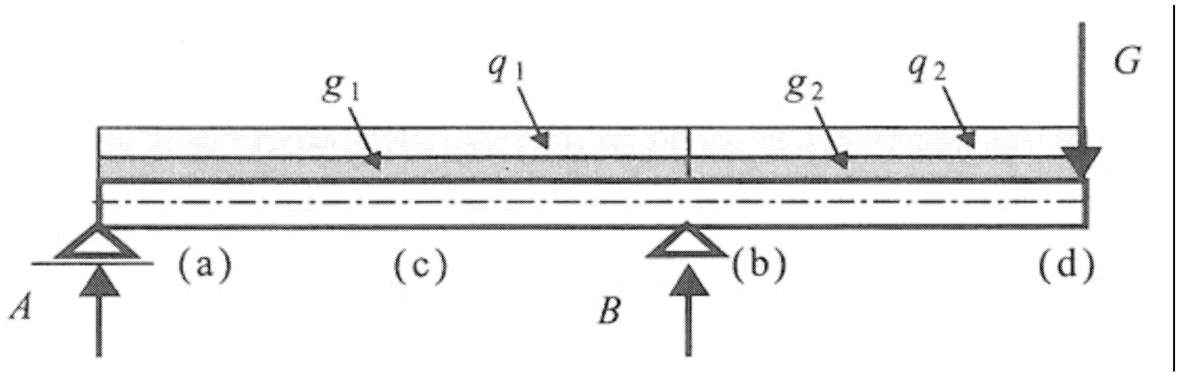

Figure 3: Beam with overhangs - a decisive combination for determining the maximum reaction B and extreme bending momentum at point b (EQU, STR). $\gamma_{g1} = 1{,}35$, $\gamma_{g2} = 1{,}35$, $\gamma_G = 1{,}35$, $\gamma_{q1} = 1{,}5$, $\gamma_{q2} = 1{,}5$

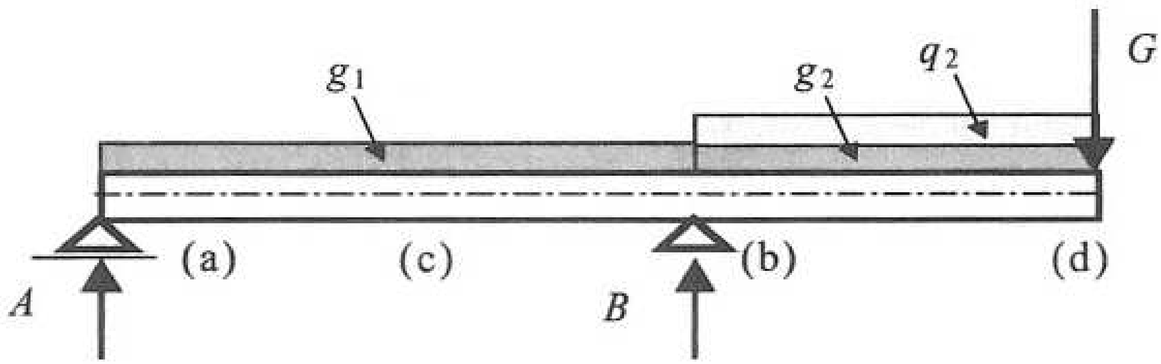

Figure 4: Beam with overhangs - a decisive combination for determining the static balance (reaction A) (EQU). $\gamma_{g1}=0{,}9$, $\gamma_{g2}=1{,}1$, $\gamma_G = 1{,}1$, $\gamma_{q1} = 0{,}0$, $\gamma_{q2} = 1{,}5$

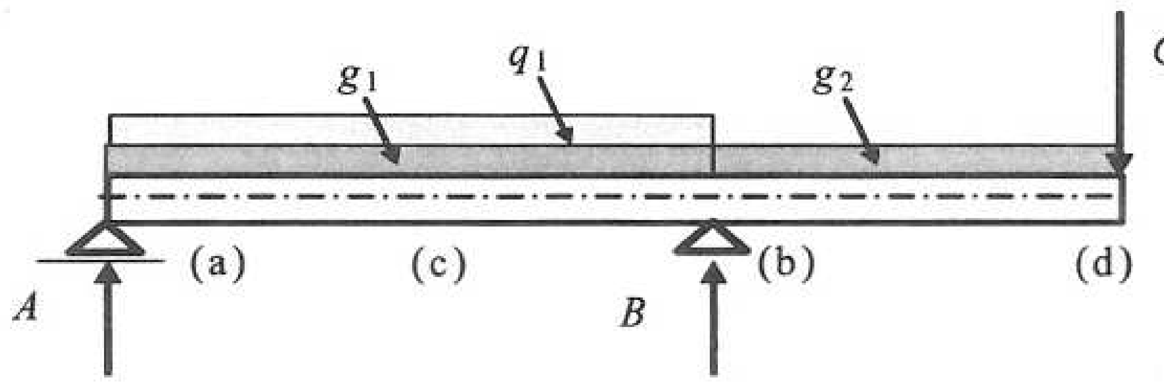

Figure 5: Beam with overhangs - a decisive combination for determining the extreme bending momentum in the field (point c) (STR). γ_{g1}=1,35, γ_{g2}=1,0, γ_G = 1,0, γ_{q1} = 1,5, γ_{q2} = 0,0

2.1.6.2. Combinations for exceptional, seismic and fatigue design situations

Expressions for exceptional, seismic and fatigue design situations are described in ČSN EN 1990

2.1.7. Load combinations for limit states of serviceability

The following condition must be verified in these combinations:

$$E_d \leq \boldsymbol{C}_d \tag{4.12}$$

E_d is the design value of load effect stated in the serviceability criterion and defined by the relevant combination.

C_d is the design value of the relevant serviceability criterion.

Load combinations for the limit states of serviceability are to be considered in the corresponding design situations.

A characteristic combination is usually used for irreversible serviceability limit states and can be expressed:

$$\sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } Q_{k,1} \text{ "+" } \sum_{i>1} \psi_{0,i}\, Q_{k,i} \tag{4.13}$$

A frequent combination is usually used for reversible serviceability limit states and can be expressed:

$$\sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } \psi_{1,1} Q_{k,1} \text{ "+" } \sum_{i>1} \psi_{2,i} Q_{k,i} \tag{4.14}$$

A quasi-permanent combination is usually used for long-term effects and the appearance of a structure and can be expressed:

$$\sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } \sum_{i\geq 1} \psi_{2,i} Q_{k,i} \tag{4.15}$$

Note: In the limit states of serviceability, we consider the load without partial factors of load γ_F (γ_G, γ_Q, γ_p).

2.2 Material properties

Design values of material properties X_d are determined by characteristic values of material properties X_k from the relation

$X_d = X_k/\gamma_M$ (4.16)

γ_M is the partial coefficient of material reliability

Based on the design values of material properties we can express the design resistance of materials R_d using the following simplified relation

$R_d = \{X_{k,i}/\gamma_{M,i}\,;\, a_d\}\quad i \geq 1$ (4.17)

a_d is the design value of geometric data

2.2.1. Characteristic values of material properties

The characteristic values of material properties are derived from tests. For their derivation we must consider:

- the test data dispersion
- the statistic uncertainty based on the number of tests
- the a priori statistic knowledge

The numerical characteristic values of material properties (e. g. strength) can be acquired from the following relation based on the number of measurements (tests) and the normal statistical distribution

$X_k = m_x(1\text{-}k_n\, V_x)$ (4.18)

k_n is the value determined based on the number of measurements (tests) for „V_x unknown " or „V_x known ".

Tab.3 **-** The numerical value k_n for the 5% characteristic value is shown in the chart.

N	1	2	3	4	5	6	8	10	20	30	∞
V_x known	2,31	2,01	1,89	1,83	1,8	1,77	1,74	1,72	1,68	1,67	1,64
V_x unknown			3,37	2,63	2,33	2,18	2	1,92	1,76	1,73	1,64

V_x is the variation coefficient that can be marked in most cases as „V_x unknown". This means that the value of this coefficient is not known in advance from previous tests carried out in comparable situations. The coefficient value is to be calculated from the available measurements using the following expression

$V_x = s_x/m_x$ (4.19)

m_x is an average from the available measurements

s_x is the standard deviation expressed $s_x = \sqrt{s_x^2}$ (4.20)

s_x^2 is the dispersion of a given measurement file determined by $s_x^2 = \frac{1}{n-1}\Sigma(x_i - m_x)^2$ (4.21)

x_i is the value of a single measurement

EXAMPLE 4: See handbook – Material Properties Determination for Existing Structures Assessment **[7].**

The determination of the characteristic value f_{ck} of concrete strength under pressure based on measurement results. The characteristic value of strength is defined as the 5% lower quantile $f_{ck} = f_{c0,05}$. The number of measurements n = 24 (34.0, 30.2, 23.2, 25.9, 29.5, 33.3, 34.0, 26.5, 29.8, 29.4, 45.8, 30.3, 32.7, 32.8, 24.1, 32.6, 29.6, 21.7, 33.5, 36.4, 35.3, 32.7, 33.8, 22.3 MPa). An average based on these measurements m_{fc} = 30,80 MPa. A standard deviation of a given measurement file s_{fc} = 5,281 MPa. The variation coefficient $V_{fc} = s_{fc}/m_{fc}$ = 5,281/30,80 = 0,1714. The coefficient value k_n = 1,749 is determined using interpolation from chart 3 for unknown V_x (the variation coefficient is not known from many previous measurements – we know only the variation coefficient from our measurement file).

$f_{ck} = m_{fc}(1\text{-}k_n\;V_{fc})$ = 30,80(1-1,749 x 0,1714) = 21,6 MPa

2.2.2. Design values of material properties

The design values of material properties X_d are determined by the characteristic values of material properties X_k based on the relation

$$X_d = X_k/\gamma_M \tag{4.22}$$

2.2.3. Design values of material resistance

Using the design values of material properties we can express the design resistance of materials R_d based on the following simplified relation

$$R_d = \{X_{k,i}/\gamma_{M,i}\,;\, a_d\} \quad i \geq 1 \tag{4.23}$$

ad is the design value of geometric data (the element dimensions used for calculating the cross-section characteristics – the surface, the cross-section module, the inertia momentum etc. or for determining the load effects). These values can be expressed by nominal values.

2.2.3. Partial factors of material

γ_M is the partial factor of material reliability. In a simplified way we can express it as:

$$\gamma_{M,j} = \gamma_{Rd}\,\gamma_{mj} \tag{4.24}$$

γ_{Rd} is the partial factor that covers the uncertainties of a resistance model including geometric deviations.

γ_{mj} is the partial factor of material properties that takes into consideration:

- possible unfavourable deviations of material properties from the characteristic value
- the random part of the conversion coefficient η (the conversion coefficient expresses the influence – of volume and dimensions, humidity and temperature or other parameters to be considered)

The standard framework ČSN EN recommends using the following partial factor values of material reliability γ_M:

For concrete γ_C = 1,5

For concrete reinforcement γ_s = 1,15

For construction steel γ_s = 1,15 (calculation examples for some cases 1.30, 1.45, 1.50)

For solid wood γ_M = 1,3

For glued laminated wood γ_M = 1,25

For masonry γ_M = 1,15 až 3,0 (the factor value is determined based on masonry material and the category performance using the chart in the national appendix to ČSN EN 1996-1-1 standard [8])

3 CONCLUDING REMARKS

In common cases of existing structure reliability verification, or designing its reconstruction, the partial factor method is applied because it is possible to proceed based on common procedures for designing new structures. However, a problem can occur with existing structures when modelling the time dependent material properties, the load properties and the environmental impact. It is not always obvious, if it is necessary to apply the same values of partial factors of load and material properties as when designing new structures. The requirements of the Eurocodes for load are usually stricter than they were for the previous national standards.

The methodology of determining the partial factors presented in ČSN EN 1990 [1] is systematically based on probabilistic methods of reliability theory. A detailed way of determining the partial factor values is covered in “annexe B” of this handbook or in ISO 13 822 [5]. In the case that a direct procedure for verifying structures using the partial factor method fails, it is possible to verify the partial factor values using these probabilistic methods.

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CHAPTER 5 – EXAMPLES

Vladislava Návarová[2]

[2]SPŠS, České Budějovice, Czech Republic

EXAMPLE 1 - INVESTIGATION AND EVALUATION OF PART A CEILING STRUCTURES

Summary

Due to a fault on the existing ceiling construction above the first floor was requested a stastic evaluation of construction state of part ceiling above the first floor under the the waiting room.

The existing object was built in the years 1972 - 1975 in DIY co-operative way . The original purpose of the object was an administrative building . In 2009 the reconstruction of the object was carried out to establish a dental clinic that is working there until today .

During an inspection of the construction site were found faults of the top layer of the floor - ceramic tiles. It is locally sunken, in the middle of the waiting room it is lengthwise cracked . The soffit of ceiling above the first floor was removed. That consists of mineral squares suspended about 150m below the bottom edge of the bearing construction. After the removal of the soffit were discovered faults of the bottom edge of the ceiling construction.

The archive project documentation of the specified construction work in 2009 wasn't traced, and the archive documents from the period of construction of the building do not exist.

The following report was drawn from the part of the existing ceiling.

1 REPORT

Title page: title, date, client and author

1.1 Introduction

Due to a fault on the existing ceiling construction above the first floor was requested a stastic evaluation of construction state of part ceiling above the first floor under the the waiting room of dental clinic.

1.2 Synopsis

The existing ceiling structure located above the first floor in the part below the waiting room has a faults. In July 2013 was carried out a preliminary inspection of the examined ceiling construction. During the preliminary examination and also through inspection of photos taken during the 2009 reconstruction, as inappropriate technical solution of the ceiling structure is determined as the cause of the defect. A solution is suggested and a detailed inspection of the ceiling structure with the help of a probe is recommended.

1.3 Contents

a) scope of assesment

The purpose of the evaluation is to assess the construction condition of the existing ceiling above the first floor in the area of a waiting room on the second floor where there are repeated failures of the floor top layer - ceramic tiles . The evaluation is required only in the waiting room on the 2nd floor. The main requirement of the evaluation is to ensure safe movement of people in a building with public access.

b) description of the structures

The building was built in 1970s. In 2009, some building modifications were carried out, and was performed completely reconstructed.

The object is a detached building and the ground plan is a rectangle 8.9 m x 18.50 m The object is divided into a two-storey part and a three-storey part. The supporting system of the construction is combined wall . Load bearing outer walls and inner walls are made up of the original brickwork .

The shape of the roof structure is composed of two gabled roofs at a high-level above the 2nd and 3rd floor. The supporting system of the roof consists of purlinsd with full ties.

The whole building is currently used as a dental clinic. The attic space is not used.

c) documents

Inspection of the building on the site on ………….
Photographic documentation of construction work carried out in 2009
The corresponding CSN EN, ISO 13822

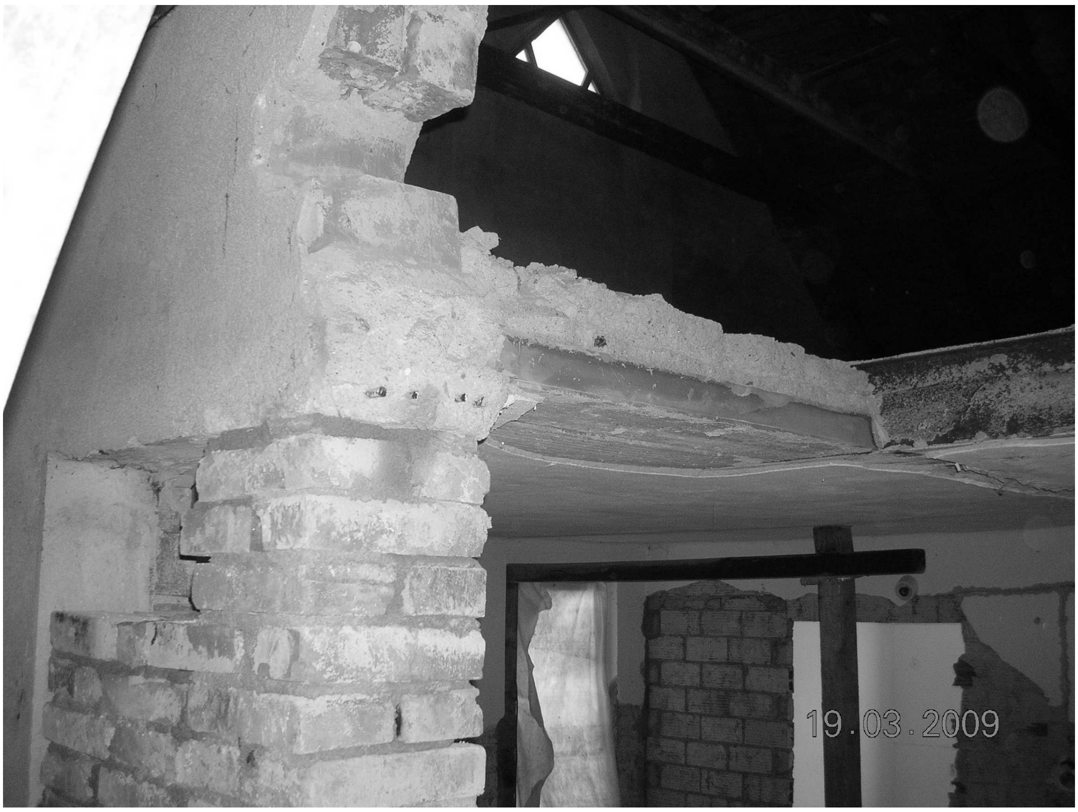
d) preliminary inspection

There are faults in the waiting area on the 2nd floor. The top layer of the floor above the 1st floor consists of ceramic tiles. The ceramic tile are cracked lengthwise and sunken by about 10 mm around the middle of the waiting room. This crack and the drop floor occurs repeatedly even if it is repaired, said the owner. Near the perimeter walls the tiles are sunken by about 20 mm. The ceramic strip on the perimeter wall is torn .

The ceiling construction is completed by a soffit of mineral squares that is hung under the ceiling construction by about 15 cm. Part of the mineral soffit was removed during the inspection and the bottom edge of the ceiling construction in the area under the waiting room was inspected. The supporting members of ceilung consists of „I“ steel traverse axially spaced by 1.2 m. Steel traverses are stored on the load-bearing walls with the free-span of 3,8 m [1]. The width of bottom flange was measured as 100 mm, which corresponds to a rolled steel „I“ traverse 220 with the bottom flange of width of 98 mm. Rolled steel traverses do not show over limit deformation .

In the steel traverses there are Hurdis ceiling blocks with straight heads . All visible seams are filled with concrete or cement plaster from the bottom edge. According to the photos, the Hurdis blocks are apparently added with concrete up to the upper edge of the steel traverse [fig.2]. The Hurdis blocks are partially completed with plaster. Below the drop tiles near the outer wall the Hurdis blocks are sunken in the middle and lean against the brick wall on the 1st floor.

Figure 1 – photos of 05/2009, the bottom edge of the ceiling above the 1st floor



Obrázek 2 – fotodokumentace z 03/2009 – skladba stropu nad 1.NP

Where the plaster is there are visible cracks in the plaster perpendicular to the „I“ traverses. These cracks correspond to the position of the interface between individual Hurdis blocks .

The photos taken at the time of the building renovation in 2009 were checked. According to available documentation the composition of the bearing part of the ceiling is as follows:

Plaster 15 mm - only locally on about ½ of the area
Steel traverses – „I“ 220
Hurdis ceiling blocks - 80 mm
concrete to the upper edge of traverse - 140 mm

On this layer, according to the information from the owner, cement screed was made and ceramic tiles laid on an adhesive .

e) preliminary verification

e1) verification of steel traverses I 220

The value of permanent actions is determined according to the above composition. Given that the individual layers were not checked by a probe into the ceiling construction, partial coefficient γ_F for permanent actions is used of value 1.35.

LOAD - CEILING CONSTRUCTION above 1.NP

1. PERNAMENT

		densities KN/m^3	**gk** KN/m^2
ceramic tiles	*10* mm	*23*	*0,230*
cement screed	*40* mm	*24*	*0,960*
concrete screed	*140* mm	*24*	*3,360*
blocks Hurdis	*80* mm		*0,680*
plaster on 1/2 area	*7,5* mm	*20*	*0,150*
podhled minerální	*10* mm	*0,5*	*0,005*
altogether			***5,39***

2. VARIABLE

$q_{k1}=$ *4* KN/m^2 waiting room C2

3.COMBINATION 1 DESIGN VALUES – LIMIT STATE STR - GROUP B

		Pernament load g_d	Variable load q_{d1} KN/m^2	q_{d2} KN/m^2	Total f_d KN/m^2
term 6.10a	*1*	*7,27*	*4,20*		*11,47*
term 6.10b	*2*	*6,18*	*6,00*		*12,18*
term 6.10	*3*	*7,27*	*6,00*		*13,27*

pozn.1 pernament		variable	
for unfavorable	*1,35*	for unfavorable	*1,5*
for favorable	*1*	for unfavorable	*0*
ξ	*0,85*		
ψ	*0,7*		

For the determination total loads is used combinatorial formula according to CSN EN. To verify the ceiling is due to possible variation using a combination of 6.10 - $\boldsymbol{f_d = 13.27}$ **kN/m^2** is used.

The material charakteristics of „I“ traverses was classifieded, according to the date of building, as steel of 37 series. The statik model of steel traverses is simple stored beam with load width of 1.2 m. The beam is secured against tilting .

STEEL BEAM N1

According to ČSN EN

Pernament load :	g_k = *5,39* KN/m^2	coefficient =	*1,35*
Variable load :	q_k = *4,00* KN/m^2	coefficient =	*1,5*
Line load :	p_k = *0,00* KN/bm	coefficient =	*1,35*

1. SECTION AND SPAN

I 220

l_s=	*3,8 m*	E=	*2,1E+11 kPa*
l=	*3,99 m*	I_y=	*0,0000305* m^4
		W_y=	*0,000278* m^3
		m=	*31* kgm^{-1}

2. LOAD

With own weight — Load width : *1,2* m

g_k = *6,78* KN/m	coefficient =	*1,35*
v_k = *4,8* KN/m	coefficient =	*1,5*
p_k = *0,00* KN/bm	coefficient =	*1,35*

3. ASSESSMENT 1st LS

STEEL S 235		f_{yk} =	*235* MPa
$\boldsymbol{M_{gd}}$=	*18,21* KNm	f_{yd} =	*235* MPa

M_{vd}=	*14,33* KNm	Q_d=	*32* KN
M_{ed} =	*32,54* KNm		
σ_{celk}=	***117,0* MPa**	< **f_d=**	***235* MPa**

SATISFIES

4. ASSESSMENT 2nd LS

f_{lim}= *1/* *300* L

= *13,3* mm

f_{gn} =	*3,49* mm
f_{vn} =	*2,47* mm
f_s =	***5,97* mm** < **f_{lim} =** ***13,3* mm**

SATISFIES

Steel ceiling traverse I 220 is satisfactory and the failures do not arise due over limit deformation of the steel ceiling traverse.

e2) verification Hurdis blocks

Ceiling blocks Hurdis with straight heads are along the length of the upper edge and all the gaps are concreted . The Hurdis blocks are directly loaded with a 140mm thick layer of concrete which is a characteristic load value of 3.36 KN/m^2, ie 336 kg/m^2 . Due to the faults of the ceramic tile it can be assumed that the base layer of the ceramic tiles ie the cement screed or concrete screed does not transmit the variable load in the waiting room to the steel beams . This variable load is transmitted through the concrete screed to the Hurdis blocks and only then to the I 220 steel beams. The most significant faults of the ceramic tiles are in the place frequented by people in the waiting room - in the centre of the room and also at the seating place.

All visible seams are filled with concrete or cement plaster from the bottom edge. According to the photos, the Hurdis blocks are apparently added with concrete up to the upper edge of the steel traverse [fig.2]. The Hurdis blocks are partially completed with plaster. Below the drop tiles near the outer wall the Hurdis blocks are sunken in the middle and lean against the brick wall on the 1st floor.

f) data analysis

The data available from the preliminary inspection are sufficient to determine the cause of failures of the ceiling above the first floor. Due to the use of the waiting room a detailed inspection of the compostion of the ceiling above the first floor will be performed at the start of work on the proposed measures.

g) review of intervention options

One option is to remove existing layers of the floor from above, pulling the concrete above the Hurdis blocks and performing a new layers. After the removal of these layers an on-site inspection will decide whether or not and on what conditions and measures it will be possible to keep the Hurdis blocks in the ceiling construction.

h) conclusions and recommendations

The available data indicate that the faults of the ceiling construction above the first floor under the waiting area are caused by improper technical solution of the composition of the ceiling construction, namely by:

- full setting in concrete of the Hurdis blocks over the entire area of the upper surface
- missing layer for variable load transfer to the steel beams

The existing state of ceiling construction under the waiting room shows a faults and it is in a dangerous condition. It can lead to a collapse of the bottom part of the Hurdis blocks including plaster. The following measures need to be taken immediately.

The existing floor layers of the ceiling above the 1st floor in the waiting room area will be removed to expose the existing Hurdis blocks . After that an examination of the state of the Hurdis blocks will have to be carried out and each block will be evaluated individually for the possibility of their further use.

While removing the concrete layer may be the Hurdis blocks damaged. Damaged blocks will be replaced by new ones.

Polystyrene will be placed on the blocks up to the upper edge of the steel beam. Above the steel beams will be performed a 60 mm reinforced concrete slab of C16/20 concrete, as well as reinforcement of Kari grid at the bottom with cover 10mm, profile 8/8 mm, wire spacing 100mm .

During the proces any movement of persons under the worked on ceiling construction will be prevented.

I need to point out to the fact that the ceiling construction in other parts of the building might have been handled in the same improper technical solution and it may be in a dangerous condition too. I recommend carrying out a detailed survey of the ceiling above the 1st floor.

i) references

STANDARD ČSN EN 1990, 1991, 1993
ČSN ISO 13822

2 CONCLUDING REMARKS

In the assessment the ceiling construction was used methodology evaluation by the ČSN ISO 13822. It is important to determine the purpose for which the assessment is performed . In this case, the purpose of the assessment is to assess the faults of the existing ceiling construction under the waiting room, and to eliminate potential threat to people in an area with public access.

That example shows that sometimes without the detailed inspection despite all the care taken in the preliminary inspection, uncertainties might not be avoided. We deduce these uncertainties in the assessment from local circumstances and relayed data.

Therefore, in this evaluation, a detailed investigation is recommended. Since it is a building with public access, the relevant local planning authority was notified of the state of the ceiling construction.

Since it is costly for the property owner to close down operations for the time necessary to perform a detailed surve investigation and remedial action, it was agreed to propose a support construction for the ceiling. Also, the soffit will be removed in all areas of the 1st floor and a careful inspection of the failures of the bottom edge of ceiling was carried out.

EXAMPLE 2 - INVESTIGATION AND EVALUATION OF THE MUSIC PAVILION

Summary

Due to a failure of the existing structure of the music pavilion a structural assessment of the structural condition of the building was requested. The purpose of the evaluation is to make a decision on further action to secure the structure or to perform a new substructure.

The substructure of the existing structure was built in 1923. In 1940s a wooden roofing construction and walls were built above the substructure.

Currently, the main sewer is being reconstructed, using a tunneling shield. There has been a drop of the front of the building by about 130 mm. The staircase on both the right and left hand side has been cracked in the area where it is placed on a reinforced concrete frame. The structure was temporarily secured by a facility with „I“ steel traverses inserted below the lower edge of the reinforced concrete beam.

The archive project documentation object was not found.

The following report on the assessment of the existing structure was drawn.

1 STRUCTURE ASSESSMENT REPORT

1.1 Introduction

The assessed Music Pavilion is located in the middle of the town park on land No. XXX k.ú. XXXXX. Currently, near the Music Pavilion there is going on a reconstruction of the main sewer, and a tunneling shield is being used.

1.2 Summary

After the start of reconstruction of the main sewer the existing structure was damaged. A preliminary examination of the existing structure was carried out. On the front of the Music Pavilion the upper edge of the concrete slab tilted by about 130 mm. Its reinforced concrete frame including a monolithic plate is tilted toward the centre of the park.

A preliminary examination of the existing structures produced conclusions and recommendations. Therefore, neither a detailed examination was carried out, nor sampling was proposed.

Based on the preliminary examination a temporary relocation of the wooden pavilion structure and demolition of the existing reinforced concrete structure including foundation was proposed. After the reconstruction of the main sewer has been finished, a new reinforced concrete structure will be made, designed to comply with ČSN EN Standards. The design of this structure will be part of a separate project documentation. After the proposed reinforced concrete structure is built, the original wooden structure fitted and fixed to this structure.

1.3 Contents

a) scope of assesment

The purpose of the evaluation is to assess the structural condition of the building – to evaluate the faults connected with driving the underground sewers close to the property and assess the overall structural condition of the building and to design measures to secure the building. The assessment of the entire building is required.

b) description of the structure

The existing pavilion [fig. 1, 2, 3] is structurally composed of two parts, which were built in two stages.

I. In 1923 the lower part of the building - a 7.32 x 8.85 m stage was built.

The foundation is laid on spread footings of plain concrete – foundation blocks. The supporting structure consists of reinforced concrete frame. The 250 x 250 mm reinforced concrete columns are complemented by two-way reinforced concrete beams. On the reinforced concrete beams a monolithic reinforced concrete slab on three levels is placed. In the middle of the front of the stage (when viewed from the park) a concrete balcony is built on the substructure. On both sides of the front there are quadrant staircases. A reinforced concrete railing was built along the perimeter of the concrete slab.

The groundfloor is only compressed soil. The 1st floor consists of monolithic reinforced concrete slab. The height of the stage structure is 1.44 m, 1.62 m and 1.80 m above the groundfloor.

The area under the load-bearing reinforced concrete structure is encircled by 150mm thick facework along its perimeter.

Figure 1 – Music Pavilion, front view

Figure 2 – Music Pavilion, side view

Figure 3 – Music Pavilion, rear view

II. In 1940s a wooden roof construction and walls were built on the substructure.

The roofing corresponds with the shape of the reinforced concrete structure, at the back it continues to form a semicircle. The supporting structure consists of arched trusses anchored by columns in the reinforced concrete structure. The front side is an open wooden roof structure, the arched truss is completed with a steel pull rod. From other sides is the wooden structure is covered with wooden flooring. The covering is made of copper.

c) podklady

Site inspection on XX.XX.XXXX
Building project documentation of the current state of the structure
Photographic documentation of the structure before starting work on stamping the main sewer
Photographic documentation of the current state
Reference ČSN EN Standards

d) preliminary inspection

The upper edge of the reinforced concrete slab on the front of the Music Hall dropped by about 130 mm. The reinforced concrete frame including monolithic plate is tilted toward the centre of the park. The staircase on both the right and left hand side got cracked in the area where it is placed on a reinforced concrete frame. Its right hand side around the middle of the rail is damaged by vertical cracks.

A temporary stability facility with steel girders and below the lower edge of the reinforced concrete beam was built. At the front the beams exceed the structure by about 2.5 m. An „I“ steel beam is at its loose end supported and deposited on an existing terrain.

On the rear side, horizontal cracks in the reinforced concrete columns can be seen at the lower edge of the reinforced concrete beam. These cracks were repaired in the past. On the left hand side there are vertical cracks in the reinforced concrete railing, which have also been repaired before.

At the bottom of the reinforced concrete slab the concrete cover layer of the bearing reinforcement plate has fallen off. The reinforcement is totally corroded , it can be easily peeled off. The reinforcement is no longer capable of transmitting load effects in reinforced concrete slab (tensile strength). The bottom side of the plate is continuously damaged by cracks [fig.5].

The the lower carrier reinforcement of the reinforced concrete medium girder is completely exposed, concrete has fallen off in the thicknesses of up to a few centimeters, and in the middle third of the beam the lower part of the concrete is completely missing. The reinforcement beam is completely corroded and can no longer perform the function of supporting the reinforced concrete section - transmitting tensile forces [fig.4].

The reinforced concrete beams are damaged by horizontal and diagonal cracks in the area where they are placed.

The central support columns are damaged by horizontal cracks at the lower edge of the girders.

The reinforcement concrete slab cover layer is only about 5 mm. The concrete in the reinforced concrete structure is mixed with fillers with large fraction.

The upper edge of the concrete slabs there are visible cracks in the lines of the supporting girders. On the balcony interface there is a crack along the entire length.

The preliminary inspection of the wooden structure was carried out visually and using simple tools. By drilling a discoloration of wood, wood hardness and structure were found. Next, acoustic echo was evaluated when tapped. The wooden bearing structure of the roof and walls does not show disturbance. The decrease of the supporting columns at the front did not cause any loss of shape stability of the structure.

Figure 4 - damage to the reinforced concrete beam and slab

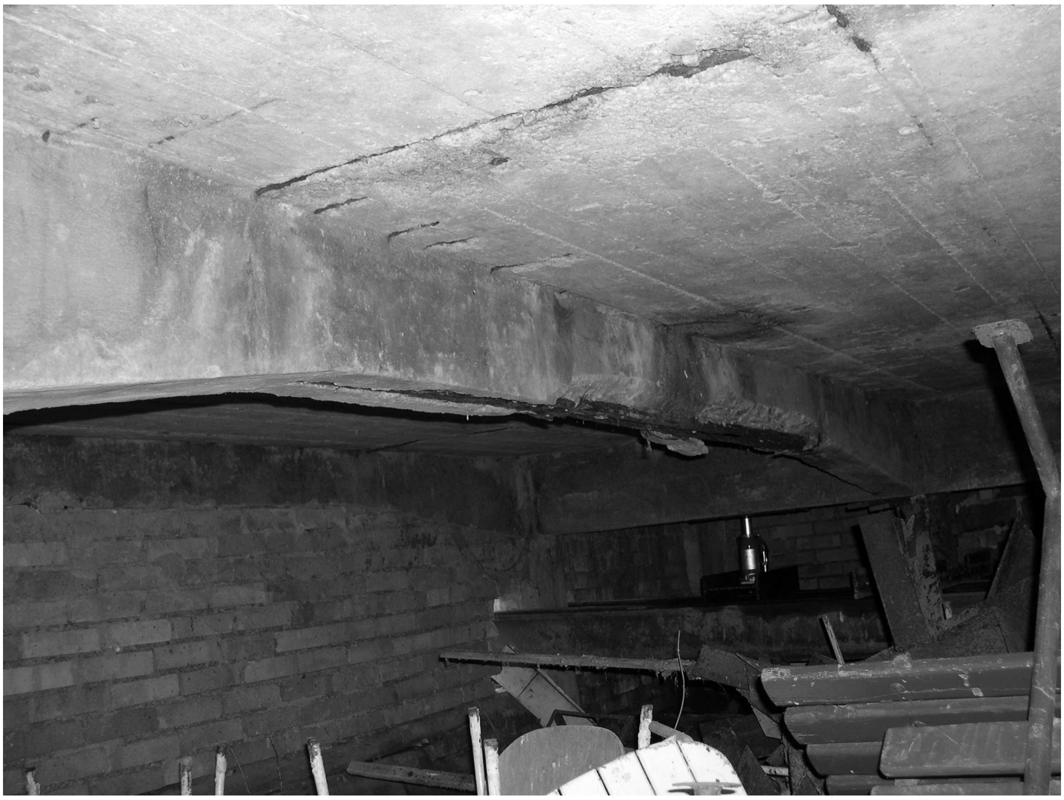


Figure 5 - damage to the reinforced concrete beam and slab

e) preliminary assessment

The drop of the front footings occurred in connection with the reconstruction of the PALACKÉHO SADY Park main sewer using the shield as a result of subsoil decline.

Other damage to the supporting structures is not related to the reconstruction of the main sewer.The reinforced concrete structure of the Music Pavilion is in disrepair. This emergency situation was there even before the reconstruction of the main sewer of the Palackého Sady Park.

Due to corrosion the supporting reinforcement in the reinforced concrete structures has failed to fulfill its supporting function - to transfer tensile forces in the reinforced concrete section. The concrete cover is inadequate.

Judging the age of the cracks and also according to the photographic documentation of the building from before the sewer reconstruction, it is evident that the damage to the supporting parts of the concrete structures happened partially prior to the subsoil decrease caused by the reconstruction of the main sewer in the Palackého Sady Park. The cracks at the bottom and top of the concrete slabs, the cracks in the area where the concrete beams are placed, the horizontal cracks in the outer and inner reinforced concrete columns, and some cracks of the reinforced concrete railing arose before the commencement of the reconstruction of the sewer due to insufficient load bearing of the reinforced concrete structures.

The load failures of the reinforced concrete structures probably occured due to moisture from the enclosure under the concrete slab and inadequate concrete cover of the structure. The nature of the defects does not exclude the possibility of adjusting the existing supporting reinforced concrete structures.

f) data analysis

The data available from the preliminary examination are sufficient to determine the causes of the structure failures.

g) review of intervention options

One option of the measures, according to the investor's requirements, is a temporary relocation of the existing wooden pavilion structure, pulling the reinforced concrete and replacing that with a new reinforced concrete structure.

h) conclusions and recommendations

It can be said that the reinforced concrete monolithic part of the structure of the Music Pavilion is in disrepair. The failures of the existing reinforced concrete structure are not related to the reconstruction of the main sewer, they were caused by the environment. The instability of the building, however, occurred due to the reconstruction of the main sewer .

I propose to remove this reinforced concrete structure, including the existing foundation of the building and replace it with a new reinforced concrete structure.

I suggest a temporary relocation of the wooden pavilion structure and subsequent removal of the existing reinforced concrete structure, including the foundation. Next I recommend to draw a new design of the pavilion substructure. After making the new foundation and the new bearing structure of the stage it will be possible to mount the wooden roof structure and walls again to the new bearing structure.

Since the music pavilion is currently in a state of disrepair, the use of the music pavilion is impossible, and it must be closed to the public. The proposed structures will be made after the completion of the reconstruction of the main sewer.

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i) annex

- Photos of the structure failures before the start of the reconstruction of the main sewer
- Photos of the structure failures of 27/07/20XX

j) references

NORMY ČSN EN 1990, 1991, 1992, 1995
ČSN ISO 13822
ČSN 730038

2 CONCLUDING REMARKS

For the assessment of the ceiling structure the original ČSN 730038 standard was used. The report was adjusted using the methodology of structure assessment according to ČSN ISO 13822. In this case, the purpose of the evaluation is to assess the failures of the existing structure and to recommend further steps for corrective action with regard to the economic aspect of the solution.

The above example shows that the ČSN ISO 13822 standard is not in conflict with ČSN 730038, the ISO ČSN 13822 standard complements the ČSN 730038 with other criteria and information.

During the preliminary structure assessment sufficient evidence for the decision was collected and carrying out a detailed survey is not recommended.

Figure 6 – photo of the current state

The proposed measures have been carried out, and the photo shows the music pavillion currently on a new reinforced concrete structure [fig.6].

EXAMPLE 3 - INVESTIGATION AND EVALUATION OF THE OBJECT VILLA

Vladislava Návarová[2]

[2]SPŠS, České Budějovice, Czech Republic

Summary

Due to proposed construction work and requirement to extend the working life of a object is performed an assessment of an existing structure - a villa located in Půlkruhová Street, Prague.

The existing structure was built in 1920 – 1923, the original purpose of the object being a residential building. This purpose has not changed. Currently a change in the use of the attic has been proposed – as a residential area.

The object is a detached building, with a combined wall support system. The roof is hip with dormers and the roof support structure is made up of a classic binding rafter assemblage.

The object itself consists of basement, ground floor, 1st floor and an attic space. The exterior and interior load bearing walls are masonry, which in its lower part changes to stonework on original mortar. The other walls are masonry.

The north-east corner of the object shows faults. There are cracks of horizontal-diagonal character.

The wall between the staircase and a room is damaged by cracks of diagonal character.

The roof support structure is made up of binding rafter, central ring beam crown and top rafter.

The following report on assessment of part of the existing roof structure has been compiled.

1 REPORT

Title page: title, date, client and author

1.1 Introduction

Účelem hodnocení je posoudit konstrukční stav objektu a provést případný návrh opatření. V rozsahu celého objektu jsou navrženy stavební úpravy, v prostoru 3.NP je navržena změna využití na obytný prostor. Hodnocení je požadováno v rozsahu celého objektu.

1.2 Synopsis

1.3 Content

a) scope of assesment

The purpose of the evaluation is to assess the structural condition of the objekt, to analyse the failures and carry out possible proposed measures. A change in use is proposed for the attic located - as a residential area. The assessment is required for the entire object.

b) description of the object

The object is a detached building of a rectangular floor plan of 9.75 m x 10.08 m. Total height above the ground is 11.16 m. The structure's supporting system is longitudinal wall. The roof is hip with dormers and the roof support structure is made up of a classic binding rafter assemblage.

The object itself consists of basement, ground floor, 1st floor and an attic space. The basement is located only below about a half of the structure (fig. 1). The ground floor and the 1st floor are currently used as residential. The attic is only used as storage. The exterior and interior load bearing walls are of brick masonry, which in its lower part changes to stonework on original mortar. The other walls are of brick masonry.

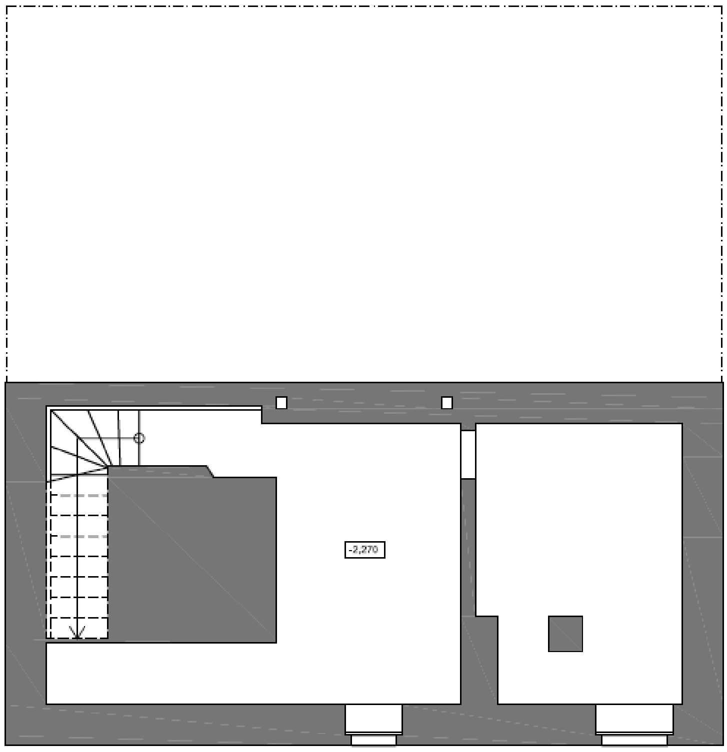

Figure 1 – existing basement

c) documents

1. On-site inspection 27.8.2008
2. Project documents for planning permission
3. Photo evidence
4. Relevant ČSN EN standards

d) preliminary inspection

Preliminary on-site inspection has been carried out. The following structures have been documented:

d1) roof truss (fig.2)

The roof support structure is made up of binding rafter, central ring beam crown and top rafter. The rafters' profile is 100/120 mm, at a distance of about 1.0 m. Middle rafters are wooden beams of 160/200 mm profile. They are supported in the corners by column of 160/160 mm profile and at the free – unsupported corner they are supported by the existing masenry 150 mm thick, across which rafters run as cantilevers. The truss columns are placed on the ceiling construction above the ground floor. Between the staircase and a room the purlin is probably placed on a 150 mm thick wall.

The top rafter is supported by a column which is placed on the ceiling beams above the 1st floor with the help of an additional wooden beam. The roof construction stiffness is secured by planking.

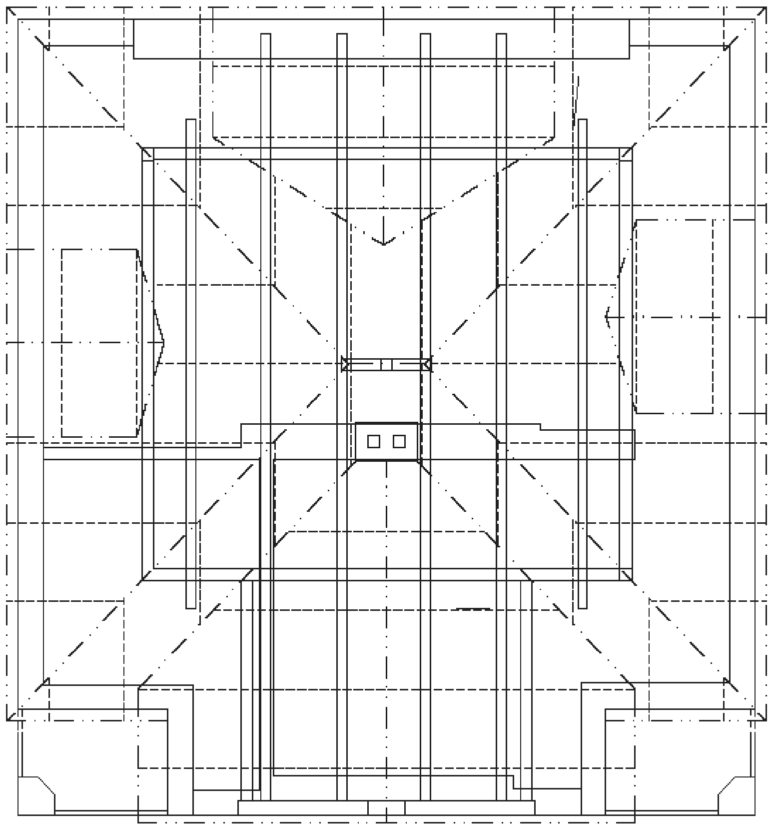

Figure 2 – roof section plan

A visual inspection of the wooden truss elements has been carried out, followed by testing surface qualities of the beams by scratching. No evident signs of faults of the truss elements have been discovered during the preliminary inspection, nor presence of wood-decaying insects or fungi. The scratches showed healthy wood mass only.

No excessive truss deformation has been found.

d2) ceiling construction above the 1st floor (fig. 3)

The ceiling construction is made up of wooden ceiling beams at the level below the purlin crown of 130/150 mm profile at the maximum distance of 1.0 m. These ceiling beams are anchored by nails each to a pair of rafters and they make a collets of the truss. These wooden beams are supported by the truss construction and the middle bearing wall.

A visual inspection of the beams and testing surface qualities of the beams by scratching have been carried out. No evident surface signs of faults of the beams have been founded during the preliminary inspection, nor presence of wood-decaying insects or fungi. No excessive truss deformation has been found.

d3) vertical constructions of the 1st floor

The exterior vertical supporting walls of the 1st floor are made of brick 330 mm thick including plaster. Interior supporting walls are 380mm – 450mm thick. The mortar strength was determined by a non-destructive Schmidt Hammer Test of the value of 1MPa.

The exterior vertical supporting walls show faults in the north-west corner of the structure. The faults – cracks follow the faults in the brickwork on the ground floor.

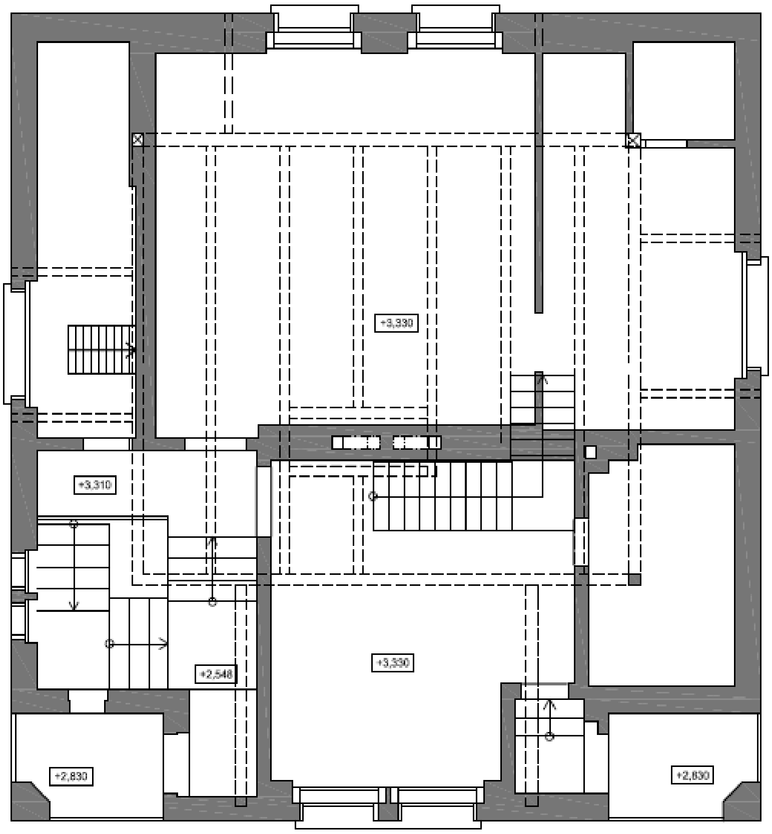

Figure 3 – 1st ground floor plan

d4) ceiling construction above the ground floor (fig. 4)

The ceiling construction above the ground floor is made of wooden beams about 0.95 m away from each other. The beams are of 160/230 mm profile for the light span of 4.78m and 160/215 mm profile for the light span of 3.98 m. On the beams a 24 mm planking was made and 24 mm planking was made under beems with plaster. A visual inspection of the beams and testing surface qualities of the beams by scratching have been carried out. No evident surface signs of faults of the beams have been founded, nor presence of wood-decaying insects or fungi. The scratches showed healthy wood mass only, even close to the beam headers. The ceiling beams show visible deformation.

d5) vertical structures of the ground floor (fig.4)

The faults documented on the ground floor in the figure 4 are marked red.

The existing 150 mm thick brickwork between the staircase and the room is damaged by horizontal-diagonal cracks of the width of up to 15 mm. The inspection shows that similar faults exist also in the masonry between the staircase and the kitchen.

The north-west corner of the structure shows faults. There are horizontal-diagonal cracks up to 20 mm wide and about 2 m long running to both directions from the corner. The cracks continue from the external masonry of the ground floor to the foundation stonework.

Near the interface between the basement and not basement floor of the object there are cracks in the foundation stonework.

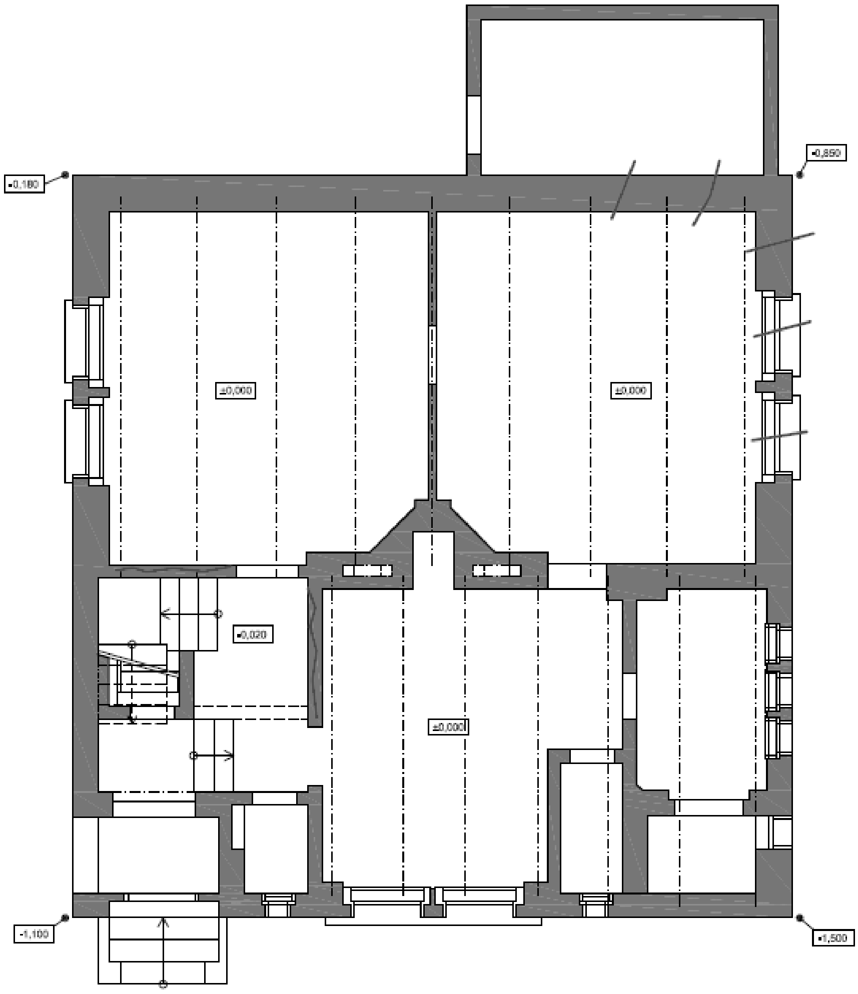

Figure 4 – ground floor plan, the faults

e) data analysis

The preliminary inspection data are sufficient to determine necessary measures due the proposed building work and changes.

A detailed inspection of the foundation situation has been proposed to determine exact causes of the faults in the north-west corner of the structure.

f) preliminary verification

f1) the supporting structure of the roof and the ceiling above the 1st floor (fig.5)

According to the project design the existing layers of the roof structure including the roofing and the planking will be removed. The verification is executed for the new proposed composition of the roof construction. The snow load is done for Prague – snow area 1.

The existing composition of ceiling above the 1st floor will also be removed, and a new composition is proposed in the project design, which expects the use of the attic space as residential. It is considered variable load of the ceiling above the 2st floor for the A place.

The existing rafters comply with the proposed load.

The purlins do not meet the load requirements. The proposed pernament load is increased by the composition of the floor above the 1st floor and the following variable load is proposed on value 1.5 KN/m^2 of the collet. Reinforcement of the existing middle purlins of 160/200mm profile is proposed by a steel liner of U220 and U160 profile (fig.6).

Next, reinforcement of the existing ceiling beams above the 1st floor, the collets is proposed - according to the documentation the existing collet of 130/150 mm profile will be completed with profile 80/160mm wood liner. Where the top rafter supporting post is proposed is new ceiling beam betweem existing ceiling beams.

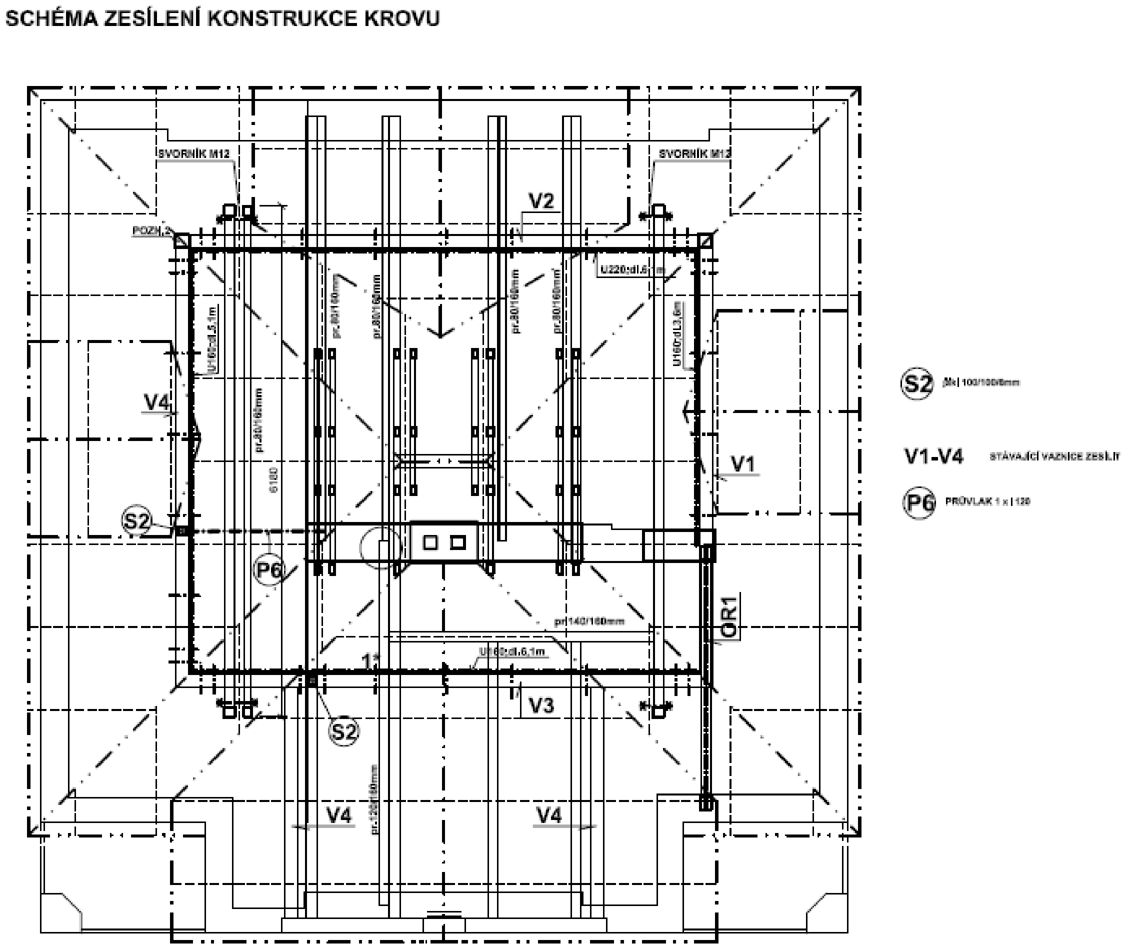

Figure 5 – proposed changes to the truss and ceiling above the 1st floor

The truss structure posts of 160/160mm profile comply with the proposed load, but placing of the posts on the ceiling structure above the 1st floor does not. At the place laying of the two posts in the ceiling structure above the 1st floor is proposed reinforced – the existing ceiling beam will be reinforced by a two-sided steel liner. Concurrence will be secured by bolts. Next, an OR1 steel frame is proposed to replace of one post under the middle purlin. The frame OR1 will be made of 2 x U 160 steel profiles welded into a box.

The existing solution of place laying the purlin on the 150 mm thick wall between the staircase and the room is insufficient. The proposed solution is to supply the post with a supporting purlin. The post will be placed on the supporting wall of the ground floor.

All wooden element joints will have to be secured by bolts.

The roof stiffening is secure by the roof planking.

f2) the supporting structure of the ceiling above the ground floor (fig. 6)

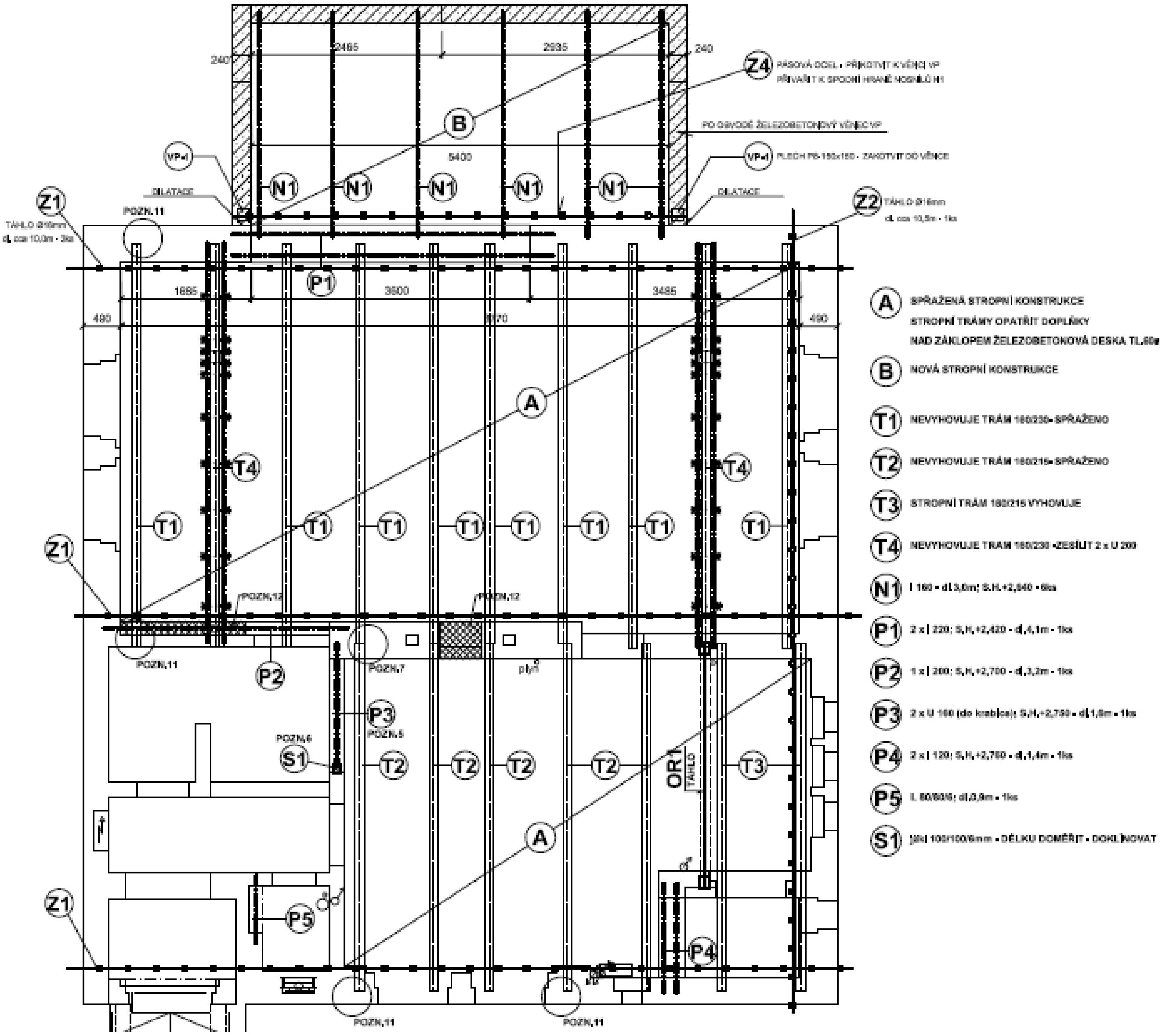

Figure 6 – proposed changes of the ceiling above the ground floor and on the ground floor

The ceiling beams doesn´t comply with the proposed load of the floor. Reinforcement is proposed – an interconnection with an reinforced concret slab. Above the planks a 50 mm thick concrete slab will be concreted reinforced with Kari welded wire fabric. The seams between the beam and the slab will be secured against skidding by a pins (fig. 7, 8).

Reinforcement of the existing beams for placing truss posts is proposed. The existing beam will be reinforced by a two-sided steel liner of U200 profile. Concurrence will be secured by bolts (fig. 9, 10).

Reinforcement of the object at the level below the ceiling structure above the ground floor is proposed using a two-way steel pull rod of 12 mm diameter, which will be anchored outside the object by a bearing plate.

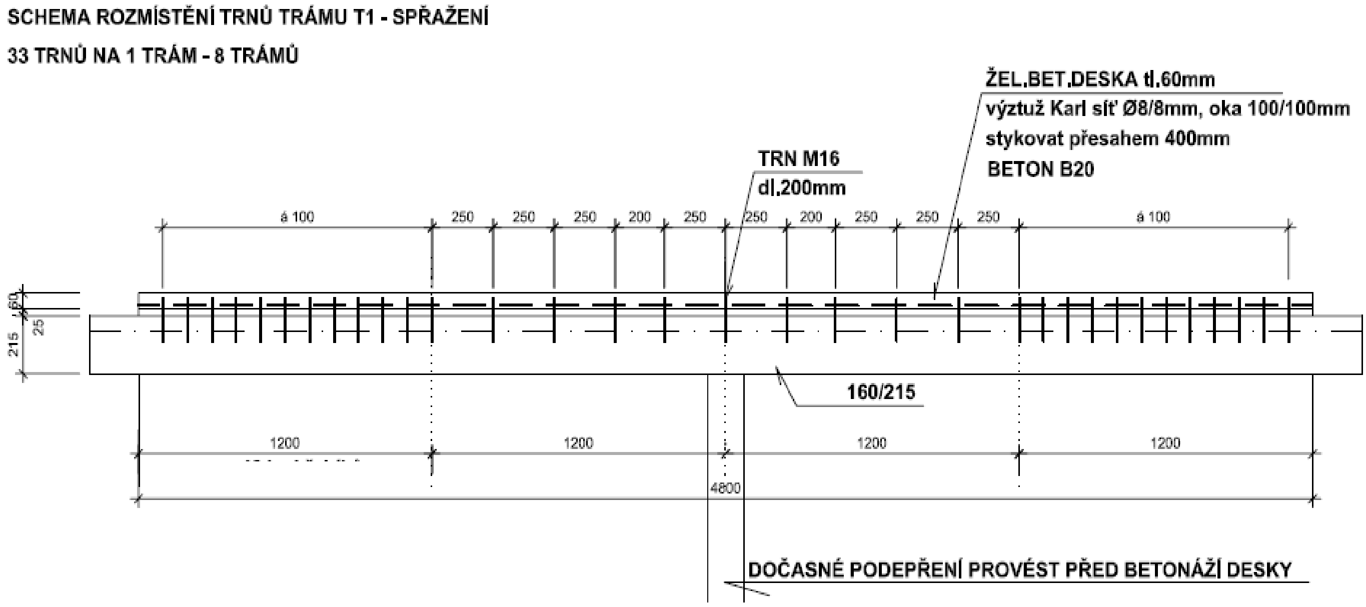

Figure 7 – beam- slab interconnection

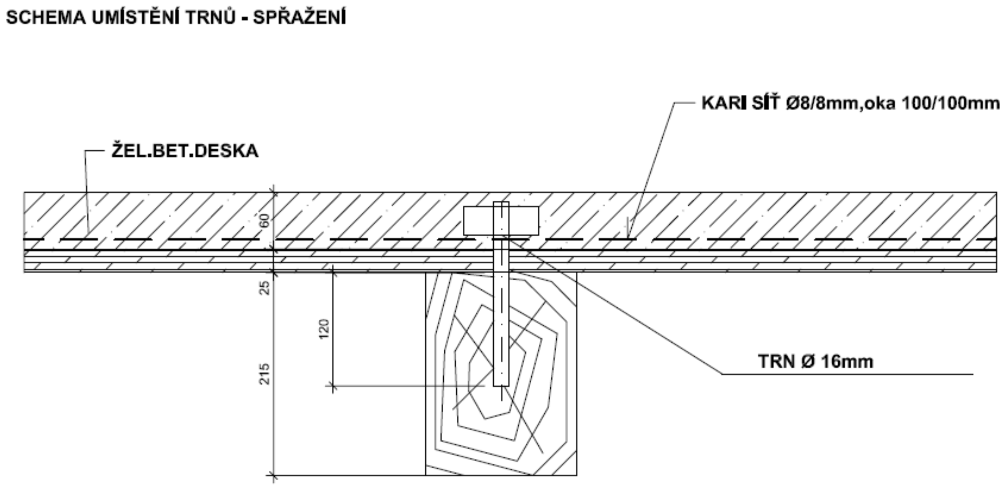

Figure 8 – beam- slab interconnection - section

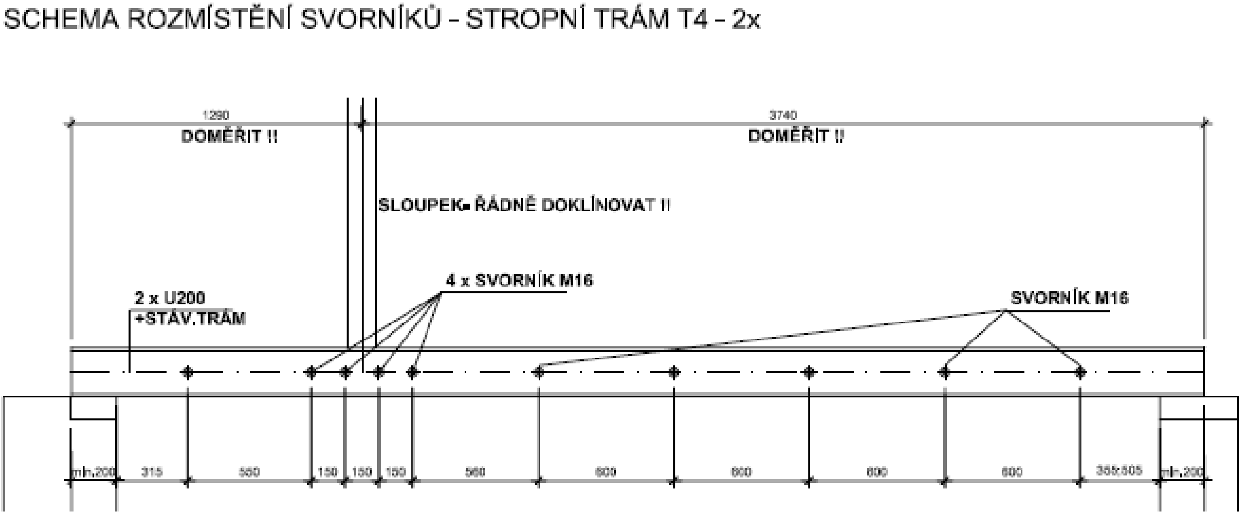

Figure 9 – beam steel liners

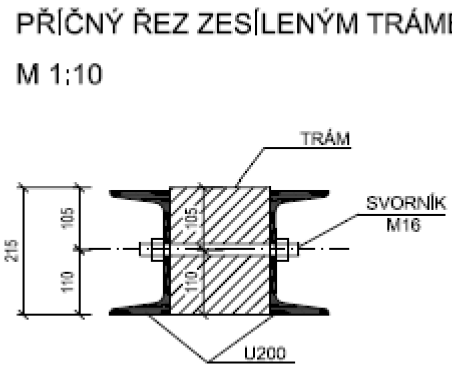

Figure 10 – beam steel liners - section

f3) vertical bearing structure

It is proposed reinforced existing masonry wall 150 mm thick on the ground floor between the staircase and the room – to make a new 175 mm thick masonry - using brick of strenght P10 and 5MPa mortar. The existing masonry must be interconnected with the new masonry with steel pins inserted into the horizontal joints (fig. 6).

Next is proposed an insertion of above mentioned steel column – square tube 100/100/6 mm on the ground floor and 1st floor. The steel column will be placed on both ends with P10 steel anchor plates. The columns will have to be placed so that they would link to the purlin and make up its support in this way (fig.5, 6).

The damaged masonry at the north-west corner will be secured by external horizontal U120 steel rull rods, anchored in the masonry with chemical anchor.

f4) stiffness of the object

The existing structure is nit reinforced by concrete crowns. Reinforcement in both directions is proposed – at the level below the ceiling structure above the ground floor and in part at the level of the floor of the ground floor. 14 mm steel pull rods are proposed, which will be anchored outside the object with the help of a P10 bearing plate

f5) foundation structure

I propose to carry out detailed inspection of the foundation situation. Close to the faults will be carried out probe near the foundation structure to the bottom of foundation. After the probes will be proposed measure to ensure the foundations.

g) review of intervention options

The first option is to carry out the above mentioned reinforcement of the existing structures and elements. Another proposed reinforcement option is to replace the existing elements by new sufficient elements.

g) conclusions and recommendations

The available data show that the existing structures are not sufficient for the proposed change in use and for expanding the work life of the structure.

I recommend that reinforcement of the existing structures is carried out.

A detailed inspection of the basement structures is necessary.

g) references

ČSN EN Standards 1990, 1991, 1992, 1993, 1995, 1996

ČSN ISO 13822

2 CONCLUDING REMARKS

For the assessment of the existing structure the original ČSN 730038 Standard was used. The report was adjusted to comply with the ČSN ISO 13822 methodology of assessment of existing structures. In this case the purpose of the assessment is to evaluate the structure of existing object for the proposed changes in use and for the extension of the work life of the object. Next, to recommend further steps for remedies regarding the economic aspect of the solution.

During the preliminary inspection enough evidence was collected for the assessment of the structures except the foundation structures. A detailed inspection of them is recommended. This detailed inspection can begin after the building work has started.

The above mentioned example shows that despite all the care taken in the preliminary investigation avoided uncertainties. Detailed investigation it is necessary, to exclude uncertainties.

It was agreed, that the detailed inspection of the foundation structures will be carried out after the building work on the object has started.

ANNEX A: EVALUATION OF RESULTS

Milan Holický[2]
[2]Klokner Institute, Czech Technical University in Prague, Czech Republic

A.1 General

The evaluation of statistical data representing a random sample taken from a particular population is frequently the first step in assessment of existing structures. The concept of a general population and the random samples taken from it is introduced and supplemented by the definition of commonly used sample characteristics. Emphasis is put on the moment characteristics, that usually provide the initial background information for the specification of a theoretical model of population. Sample characteristics regularly used in engineering and science describe the location, dispersion, asymmetry and kurtosis of statistical data. The general rules and computational techniques used for determining sample characteristics of a single random sample, and also for the combination of two random samples, are illustrated by examples.

The concepts of population and random sample are extremely important for the appropriate interpretation of statistical data and their analysis. Population, or "the universe", is the totality of items under consideration. A population may be finite (N sampling units) or infinite. Rather than examining the entire group of N units a small part of the population, that is a sample of n units, may be examined instead. A precise definition regarding a population is often difficult to come by, but must be provided in order to interpret outcomes of statistical investigation correctly [1,2]. An excellent description of the basic technique is given in [3,4] and a short review is provided in [5]. The correct terminology and procedures are available in International Standards [6,7,8].

A sample is one or more units taken from a population and is intended to provide information on that population. It may serve as a basis for decision-making about the population, or about the process which produced it. The term "random sample" refers to the samples that are taken from a population in such a way that all possible units have the same probability of being taken. The number of sampling units, called sample size n, may be considerably different. Commonly, samples are considered to be very small ($n < 10$), small ($n < 30$), large ($n > 30$) or very large ($n > 100$). Obviously, with increasing size the samples become more representative. However, the sampling procedure is equally important.

If a sample is representative of a population, important conclusions about it can often be inferred from an analysis of the sample. This phase of statistics is called inductive statistics, or statistical inference, and is covered in subsequent chapters. The phase of statistics that seeks only to describe and analyse a given sample is called descriptive, or deductive, statistics to which is devoted this Chapter.

Example 1

A structure consists of 70 members of the same type. A random sample of 10 members can be taken from the population of 70 units using a table, or a generator of random numbers within a range of 1 to 70. A sample can then be created by taking the units whose serial numbers are equal to ten generated random numbers.

A.2 Characteristics of Location

The basic characteristic of sample location (or its main tendency) is the sample mean m_X given as

$$m_X = \frac{1}{n}\sum_1^n x_i \qquad \text{(A.1)}$$

Here x_i denotes sample units. If the sample units are ordered from the smallest to greatest unit then the subscripts i are generally changed to (i), and the units are then denoted $x_{(i)}$.

Another characteristic of location is median $\widetilde{m}_X$ defined the point separating ordered sequence of data into two parts such that half of the data is less than the median and half of the data greater than the median.

Example 2

A random sample of measurements of concrete strength contains ten measurements x_i = {27; 30; 33; 29; 30; 31; 26; 38; 35; 32}in MPa. The measured data, in order of scale, is $x_{(i)}$ = {26; 27; 29; 30; 30; 31; 32; 33; 35; 38}in MPa:

The sample mean m_X and the median $\widetilde{m}_X$ are given as

$$m_X = \frac{1}{10}\left(\sum x_i\right) = 31{,}1 \text{ MPa}, \ \widetilde{m}_X = \frac{1}{2}\left(x_{(5)} + x_{(6)}\right) = 30{,}5 \text{ MPa}$$

A.3 Characteristics of Dispersion

The basic characteristic of dispersion is called the variance

$$s_X^2 = \frac{1}{n}\sum_1^n (x_i - m_X)^2 \qquad \text{(A.2)}$$

In practical applications the standard deviation s_X is commonly used instead of “variance”.

Another measure of dispersion that is frequently applied in engineering and science is called the coefficient of variation

$$v_X = \frac{s_X}{m_X} \qquad \text{(A.3)}$$

This is, in fact, a measure of relative dispersion normalised by the sample mean m_X. It is frequently used in engineering when the sample mean m_X is not very small. If the sample mean m_X is relatively small then the standard deviation should be used instead.

In the case of very small samples ($n \leq 10$) additional measure of dispersion, called sample range, is sometimes used; it is defined simply as the difference between of the greatest and smallest sample unit, $x_{(n)} - x_{(1)}$.

In same specific cases also the mean deviation MD, or average deviation, defined as the mean of differences $|x_i - m_X|$ is also used

$$MD_X = \frac{1}{n}\sum_{i=1}^n | x_i - m_X \qquad \text{(A.4)}$$

Example 3

The variance of the sample sample given in Example 3.1 x_i = {27; 30; 33; 29; 30; 31; 26; 38; 35; 32}in MPa is given as

$$s_X^2 = \frac{1}{n}\sum_1^n (x_i - m_X)^2 = 11{,}69 \text{ (MPa)}^2$$

The standard deviation is thus

$$s_X = \sqrt{s_X^2} = \sqrt{11{,}69} = 3{,}42 \text{ MPa}$$

Example 4

The coefficient of variation of the data in the random sample given in Example 3.2 x_i = {27; 30; 33; 29; 30; 31; 26; 38; 35; 32} in MPa, is given as

$$v_X = \frac{3,42}{31,1} = 0,11 = 11\%$$

Example 3.5

Considering ordered measurements from example 3.2 $x_{(i)}$ = {26; 27; 29; 30; 30; 31; 32; 33; 35; 38}in MPa, the variation range and the mean deviations are:

$x_{(n)} - x_{(1)}$ = 38 – 26 = 12 MPa

$$MD_X = \frac{1}{n}\sum_{i=1}^{n}|\ x_i - m_X| \ = 2,72 \text{ MI}$$

A.4 Characteristics of Asymmetry and Kurtosis

The characteristics of asymmetry and peakedness (kurtosis) are used less frequently than the characteristics of location (the mean m_X) and the characteristic of dispersion (the variance s_X^2). However, the characteristics of asymmetry and peakedness provide valuable information about the nature of the sample, in particular the distribution of observation to the left and right of the mean and the concentration of observation about the mean. This information may be extremely useful for determining the appropriate theoretical model (probability distribution) of population.

The following moment characteristics are most often used. The coefficient of asymmetry is defined on the basis of the central moment of the third order as

$$a_X = \frac{1}{ns_X^3}\sum_{i=1}^{n}(x_i - m_X)^3 \tag{A.5}$$

Similarly the coefficient of kurtosis is related to the central moment of the fourth order as

$$e_X = \frac{1}{ns_X^4}\sum_{i=1}^{n}(x_i - m_X)^4 - 3 \tag{A.6}$$

Note that the above defined coefficients of asymmetry and kurtosis should be close to zero for samples taken from population having normal distribution.

The coefficient of asymmetry is positive when more sample data is on the left of the mean, positive when more data is on the right of the mean. The coefficient of kurtosis is positive when the sample data is located mostly in the vicinity of the mean, negative when the data is distributed more uniformly. Both these characteristics (skewness a_X and kurtosis e_X) are strongly dependent on abnormal deviations of some sample units (outliers), or errors, particularly in the case of small samples (n < 30). Then their evaluation may be highly uncertain (and may suffer from so-called statistical uncertainty due to limited data).

Example 6

Considering again data from example 3.2 given as x_i = {27; 30; 33; 29; 30; 31; 26; 38; 35; 32} in MPa, the coefficients of asymmetry and kurtosis are:

$$a_X = \frac{1}{ns_X^3}\sum_{i=1}^{n}(x_i - m_X)^3 = 0{,}46$$

$$e_X = \frac{1}{ns_X^4}\sum_{i=1}^{n}(x_i - m_X)^4 - 3 = -\ 0{,}44$$

The positive coefficient of asymmetry indicates that more observations are on the left of the mean (in fact 6 of 10 values are on the left of the mean). A slightly negative coefficient of kurtosis indicates low peakedness (observed values seem to be distributed slightly more uniformly than those of normal distribution). Note that the investigated sample is very small (10 values only), and the coefficients obtained, a_X and e_X may be inaccurate.

It is interesting to note that there is an empirical relationship between the skewness a_X the mean m_X, the median $\tilde{m}_X$ and the standard deviation s_X (called sometimes as Pearson coefficient of skewness) in the form

$$a_X \approx 3(m_X - \tilde{m}_X)/\ s_X{}^{\circ}$$

Considering the results of previous examples 3.2 and 3.3 $m_X = 31{,}1\ \mathrm{MPa}$, $\tilde{m}_X = 30{,}5\ \mathrm{MPa}$ and $s_X = 3{,}42\ \mathrm{MPa}$ it follows that

$$a_X \approx \frac{3(31{,}1-30{,}5)}{3{,}42} = 0{,}53$$

This seems to be a good approximation of the above obtained moment skewness a_X = 0,46. It also demonstrates the intuitively expected result that if the median $\tilde{m}_X$ is less than the mean m_X, then the skewness a_X is positive. Consequently more data is located left of the mean than right of the mean.

A.5 General and Central Moments

Most of the samples characteristics described above belong to so called moment characteristics that are based on general or central moments of the data. The general moment (about the origin) of the order *l* (*l* = 1, 2, 3, …) is defined as the arithmetic mean of the sum of *l*-powers

$$m_l^* = \frac{1}{n}\sum_{i=1}^{n} x_i^l \tag{A.7}$$

The central moment (about the mean) of the order *l* is similarly given as

$$m_l = \frac{1}{n}\sum_{i=1}^{n}(x_i - m_X)^l \tag{A.8}$$

The moment characteristics can be then defined as follows.

$$m_X = m_1^* \tag{A.9}$$

$$s_X = \sqrt{m_2} \tag{A.10}$$

$$a_X = \frac{m_3}{m_2^{3/2}} \tag{A.11}$$

$$e_X = \frac{m_4}{m_2^2} - 3 \tag{A.12}$$

In numerical calculation it is sometime useful to apply the following relations between the general and central moments

$$m_2 = m_2^* - m_X^2 \tag{A.13}$$

$$m_3 = m_3^* - 3m_X m_2^* + 2m_X^3 \qquad \text{(A.14)}$$

$$m_4 = m_4^* - 4m_X m_3^* + 4m_X^2 m_2^* - 3m_X^4 \qquad \text{(A.15)}$$

When computers are used to evaluate statistical samples equations (A.13) to (A.15) are not directly used.

A.6 Combination of Two Random Samples

Sometimes it is necessary to combine two random samples taken from one population, assuming that the characteristics of both the samples are known, but the original observations x_i are not available. It must be emphasised that only homogeneous samples of the same origin (taken from one population under the same conditions) should be combined. Violation of this important assumption could lead to incorrect results.

Assume that a first sample of the size n_1 has the characteristics m_1, s_1, a_1, while a second sample of the size n_2 has the characteristics m_2, s_2, a_2. Only three basic characteristics are considered here (the coefficients of kurtosis are rarely available for combined samples).The resulting characteristics of a combined sample of the size n can be determined from the following expressions:

$$n = n_1 + n_2 \qquad \text{(A.16)}$$

$$m = \frac{n_1 m_1 + n_2 m_2}{n} \qquad \text{(A.17)}$$

$$s^2 = \frac{n_1 s_1^2 + n_2 s_2^2}{n} + \frac{n_1 n_2}{n^2}(m_1 - m_2)^2 \qquad \text{(A.18)}$$

$$a = \frac{1}{s^3}\left[\frac{n_1 s_1^3 a_1 + n_2 s_2^3 a_2}{n} + \frac{3n_1 n_2 (m_1 - m_2)(s_1^2 - s_2^2)}{n^2} - \frac{n_1 n_2 (n_1 - n_2)(m_1 - m_2)^3}{n^2}\right] \qquad \text{(A.19)}$$

It is interesting to note that the standard deviation s is dependent not only on the standard deviations of two initial samples s_1 and s_2, but also on the means of both the samples. Similarly, the skewness a also depends on the characteristics of the lower order (means and standard deviations). The relationship for the kurtosis is not included as it is not commonly used.

It should be noted that if the original data is available then it can be analysed as one sample; relationships (A.16) to (A.19) can then be used for checking newly obtained results. The most important thing is the verification of the hypothesis that both samples are taken from one population.

Example 7

An example of the practical application of equations (A.16) to (A.19) is shown underneath.

Samples	*n*	*m*	*s*	*a*	*v*
Sample 1	10	30.1	4,4	0,5	0,15
Sample 2	15	29,2	4,1	0,5	0,14
Combined	25	29,56	4,25	0,53	0,14

Note that a different number of sample units may affect the characteristics of the resulting combined sample. An EXCEL sheet has been developed for calculation if this is the case.

Sometimes it may occur that the size of one sample, say n_1, is not known, and only the first two characteristics m_1, s_1 are available. This is a typical situation when updating previous data with the characteristics m_1, s_1, using newly observed data of the size n_2 with the characteristics m_2, s_2. Then the Bayesian approach may be used for assessing the unknown value n_1 and a corresponding degree of freedom ν_1. The following text is presented here as a guide on how to proceed in that case, just for information and without the appropriate mathematical clarification.

In accordance with the Bayesian concept [1, 3], the unknown value n_1 and a corresponding degree of freedom ν_1 may be assessed using the relations for the coefficients of variation of the mean and standard deviation $V(\mu)$ and $V(\sigma)$, (the parameters μ and σ are considered as random variables in Bayes' concept) for which it holds

$$n_1 = [s_1 / (m_1 V(\mu))]^2, \ \nu_1 = 1 / (2 V(\sigma)^2) \tag{A.20}$$

Both unknown variables n_1 and ν_1 may be assessed independently (generally $\nu_1 \neq n_1 - 1$), depending on previous experience with a degree of uncertainty of the estimator of the mean μ and the standard deviation σ of the population. Note that for a new sample it holds that $\nu_2 = n_2 - 1$.

When the sample size n_1 and the degree of freedom ν_1 are estimated, the degree of freedom ν is given as [3, 11]

$$\nu = \nu_1 + \nu_2 - 1 \text{ if } n_1 \geq 1, \ \nu = \nu_1 + \nu_2 \text{ if } n_1 = 0 \tag{A.21}$$

Then the resulting size of the combined sample n and the mean m is given by equations (3.59) and (3.60); the standard deviation s is determined from a modified equation (3.61) as

$$s^2 = \left[\nu_1 s_1^2 + \nu_2 s_2^2 + \frac{n_1 n_2}{n}(m_1 - m_2)^2 \right] / \nu \tag{A.22}$$

The above relationship may be easily applied using the EXCEL sheet or other software tools.

Example 8

Suppose that from the prior production of a given type of concrete the following information is available regarding its strength

$$m_1 = 30{,}1 \text{ MPa}, V(\mu) = 0{,}50, s_1 = 4{,}4 \text{ MPa}, V(\sigma) = 0{,}28.$$

For the unknown characteristics n_1 and ν_1 it follows from equation (3.20) that

$$n_1 = \left(\frac{4{,}4}{30{,}1} \frac{1}{0{,}50} \right)^2 \approx 0, \nu_1 = \frac{1}{2 \times 0{,}28^2} \approx 6$$

Thus, the following characteristics are subsequently considered: $n_1 = 0$ and $\nu_1 = 6$.

To verify the quality of the concrete, new measurements have been carried out using specimens from the same type of concrete. The following strength characteristics have been obtained:

$n_2 = 5$, $\nu_2 = n_2 - 1 = 4$, $m_2 = 29{,}2$ MPa, $s_2 = 4{,}6$ MPa.

Using equations (3.16), (3.17), (3.18) and (3.19), the updated characteristics are as follows:

$$n = 0 + 5 = 5$$

$$\nu = 6 + 4 = 10$$

$$m = \frac{0\times 30{,}1 + 5\times 29{,}2}{5} = 29{,}2 \text{ MPa}$$

$$s^2 = \left[6\times 4{,}4^2 + 4\times 5{,}6^2 + \frac{0\times 5}{5}(30{,}1 - 29{,}2)^2\right]/10 = 4{,}5^2 \text{ MPa}^2$$

Thus, using the previous information, the standard deviation of the new measurements could be decreased from s = 5,6 MPa to s = 4,5 MPa.

However, it should be noted that the combination of the previous information with the current measurements might not always lead to favourable results. For example, if the coefficients of variation are $w(\mu)$=0,2 and $w(\sigma)$=0,6, then the unknown characteristics n_1 and ν_1 follow from equation (3.20) as

$$n_1 = \left(\frac{4{,}4}{30{,}1}\frac{1}{0{,}2}\right)^2 \approx 1;\ \ \nu_1 = \frac{1}{2\times 0{,}6^2} \approx 1$$

In this case

$$n = 1 + 5 = 6$$

$$\nu = 1 + 4 - 1 = 4$$

$$m = \frac{1\times 30{,}1 + 5\times 29{,}2}{6} = 29{,}35 \text{ MPa}$$

$$s^2 = \left[1\times 4{,}4^2 + 4\times 5{,}6^2 + \frac{1\times 5}{6}(30{,}1 - 29{,}2)^2\right]/4 = 6{,}03^2 \text{ MPa}^2$$

In this case, the mean increased slightly from 29,2 to 29,35, while the standard deviation increased considerably, from 5,6 to 6.03. However, this is an extreme case, caused by unfavourable estimates of n_1, ν_1 and ν following on from equations (3.20) and (3.21). In practical applications these equations should be applied with caution, particularly in extreme cases similar to the above example. In connection with this warning, an important assumption mentioned at the beginning of this section should be stressed. Only those samples that are evidently taken from the same population can be used for combining or updating statistical data; otherwise the results of the combination of two random samples may lead to incorrect results.

A.7 Note on Terminology and Software Products

It should be mentioned that documents such as ISO 3534 [7] , [8] and software products EXCEL, MATHCAD and STATISTICA provide slightly modified terminology and definitions for basic moment characteristics.

In general two modifications are commonly used for the characteristic of dispersion.

- The characteristic called here “the sample standard deviation” is also denoted as “the standard deviation of a sample”, or as “the population standard deviation” (when n is the population size), and is given as

$$s_X = \sqrt{\frac{1}{n}\sum_1^n (x_i - m_X)^2} \qquad \text{(A.23)}$$

- The sample estimate of the population standard deviation called here a point estimate of the population standard deviation and denoted by the symbol $\hat{s}_X$ is sometimes called the sample standard deviation

$$\hat{s}_X = \sqrt{\frac{1}{n-1}\sum_1^n (x_i - m_X)^2} \tag{A.24}$$

Expression (A.23) corresponds to equation (A.2) for the sample standard deviation. Expression (A.24) represents a point estimate of standard deviation that is derived from the mean of the distribution describing the sample variance (based on the χ^2 random variable and discussed in [1], [2], [3] and [4]).

Similar modifications of sample characteristics are also available for the skewness and kurtosis. The "sample skewness" *a* defined here by equation (A.5) can be written in simplified form as

$$a_X = \frac{m_3}{m_2^{3/2}} = \frac{1}{n s_X^3}\sum_{i=1}^n (x_i - m_X)^3 \tag{A.25}$$

STATISTICA, EXCEL, MATHCAD and some other software products provide a point estimate of the population skewness $\hat{a}_X$ (see Chapter 8) as

$$\hat{a}_X = \frac{n^2}{(n-1)(n-2)}\frac{1}{n\hat{s}_X^3}\sum_{i=1}^n (x_i - m_X)^3 = \frac{\sqrt{n(n-1)}}{(n-2)}\, a_X \tag{A.26}$$

Note that the population estimate $\hat{s}_X$ is used in equation (3.26). If the sample standard deviation is used then the estimate of the population skewness would be

$$\hat{a}_X = \frac{n}{(n-1)(n-2)}\frac{1}{s_X^3}\sum_{i=1}^n (x_i - m_X)^3 = \frac{n^2}{(n-1)(n-2)}\, a_X \tag{A.27}$$

The factor enhancing the sample skewness a_X in equation (A.27) (the fraction containing the sample size *n*) is slightly greater than the similar factor in equation (A.26) (for $n > 30$ by less than 5 %); the difference diminish with increasing sample size *n*

Similar modifications of sample characteristics may be found for kurtosis based on the central moment of the fourth order (see equation (A.6)). The relevant formulae can be found in the help component of the relevant software products. However, kurtosis is evaluated in practical applications very rarely and only for very large samples ($n > 100$).

A.8 Grouped Data, Histogram

When analyzing large size of statistical data *n*, it is often useful to group them into a limited number of classes *k* (usually $7 \le k \le 20$) and to determine the number of units belonging to each class n_i ($i = 1,2,\dots k$), called class frequency ($\Sigma n_i = n$). Each class is represented by class mark x_i^* which is the midpoint of the class interval limited by its lower and upper class limit.

Commonly, the grouped data are presented graphically in the form of a histogram, which is a column diagram showing frequency n_i or relative frequency n_i/n for each class. Histograms are very useful graphical tools providing valuable information about the overall character of the sample. Visual investigation of the histogram is always recommended. It may provide an initial understanding of the sample nature.

The mean m_X is given by the general moment of the first order (A.7), which for grouped data is written as

$$m_X = m_1^* = \frac{1}{n}\sum_{i=1}^k n_i\, x_i^* \tag{A.28}$$

The central moments (about the mean) of the order *l* are for grouped data given as

$$m_l = \frac{1}{n}\sum_{i=1}^{k} n_i (x_i^* - m_X)^l \qquad \text{(A.29)}$$

The moment characteristics of grouped data can be determined using the general formulae (A.10) to (A.12). Also the relationships between the general and central moments provided by equation (A.13) to (A.15) can be used in the numerical evaluation of grouped data.

Example 9

Results of n = 90 tests of concrete strength are grouped into k = 9 classes as indicated in the table below and in the histogram in Fig. A.1. Visual investigation of the histogram indicates that the sample is well-ordered (without outliers), symmetric (the skewness is expected to be close to zero) and slightly less spiky (more flat) than commonly used normal distribution (a bit of negative kurtosis is expected).

Class i	Class interval in MPa	Class mark x_i^* in MPa	Frequency n_i	Product $n_i\, x_i^*$	Product $n_i\, (x_i^* - m_X)^2$
1	16 to 18	17	1	17	71,309
2	18 to 20	19	3	57	124,593
3	20 to 22	21	12	252	237,037
4	22 to 24	23	15	345	89,630
5	24 to 26	25	20	500	3,951
6	26 to 28	27	18	486	43,556
7	28 to 30	29	11	319	139,062
8	30 to 32	31	8	248	246,914
9	32 to 34	33	2	66	114,173
Sum	-	-	90	2290	1070,222

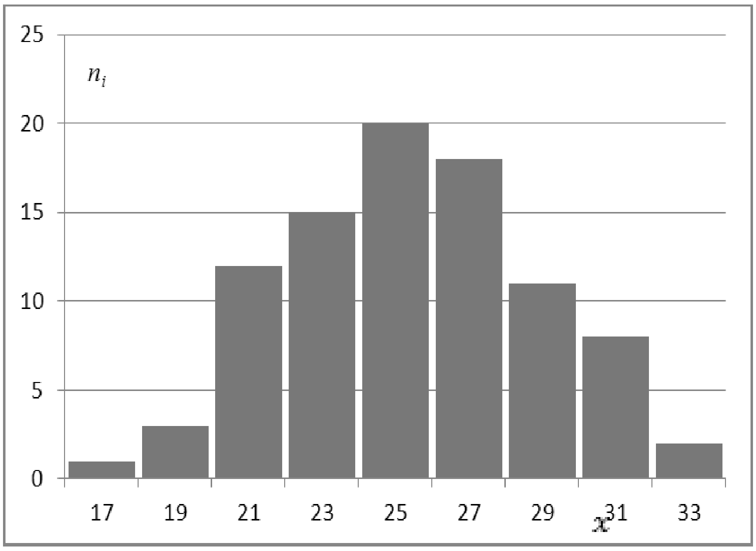

Fig. A.1 Histogram of the grouped data form Example 3.9 (90 observations of concrete strength)

The table shows the class intervals, class marks x_i^* (in MPa), frequency n_i and products $n_i\, x_i^*$ and $n_i\, (x_i^* - m_X)^2$ used to calculate the general moments of the first order, and the central moment of the second order. The moments of the order three and four would be necessary for calculation of the skewness a_X and kurtosis e_X.

It follows from equations (A.7), (A.10) and the numerical results shown in the last row of the above table that the sample mean and standard deviation are

$$m_X = m_1^* = 2290/90 = 25{,}44 \text{ MPa and } s_X = \sqrt{m_2} = (1070{,}222/90)^{0{,}5} = 3{,}45 \text{ MPa}$$

The coefficient of variation v_X = 3,45/25,44 ≈ 0,14 is relatively high and indicates a somewhat low quality of material. The other moment characteristics can be similarly found using the central moments of higher order and general equations (A.11) and (A.12). This way it can be found that the sample skewness is almost zero, a = 0,03, and the kurtosis e = –0,53. So the sample is really symmetrical and slightly more uniform than the normal distribution.

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ANNEX B: PARTIAL FACTORS

Milan Holický[1]
[1]Klokner Institute, Czech Technical University in Prague, Czech Republic

B.1 INTRODUCTION

Important developments in the application of the theory of structural reliability to codified design have intensified since the last revision of ISO 2394 [1] in 1998. In the field of limit states design, the implementation of principles of structural reliability for establishing the basis for structural design for Eurocode in EN 1990:2002 [2] is a significant application and further development of ISO 2394:1998 [1]. Other newly published International Standards include ISO 13822:2001 [3] and ISO 22111:2007 [4]. The scientific basis of structural design was developed extensively in the JCSS Model Code (2001) [5], and has been updated continuously. The context of structural reliability is developed furthermore through standards for risk assessment, such as the International Standard ISO 13824:2009 [6] and the JCSS Guideline on Risk Assessment (2008) [7].

The paper considers the process for the revision of a number of national codes in accordance to principles for standards development (ISO 2394:1998 [1] and EN 1990:2002 [2]), including the reference, supporting background (Gulvanessian et al., 2002), [8], (Holický, 2009) [9] and guiding information (Retief and Wium, 2010) [10]. An important question of the target reliability levels and reliability differentiation for newly designed and existing structures is thoroughly discussed. Further, the reliability bases formed by the First Order Reliability Methods (FORM) [9] are critically reviewed and methods of probabilistic code calibration are presented. Finally, suggestions for possible revisions and updating of the present operational design methods of partial factors are proposed.

B.2 TARGET RELIABILITY LEVEL

The target reliability level required in design of new or assessment of existing structures is the first inevitable step to relate science and practice. Recommended target reliability levels, expressed commonly by reliability indexes $\beta = -\Phi^{-1}(p)$, where $\Phi()$ denotes the distribution function of the standardized normal distribution and p the failure probability, are given in several documents [1,2,5]. In EN 1990 [2] the target reliability index β is given for two reference periods (1 year and 50 years) (see Tab. B.1). No explicit link between the target reliability level and the design working life is provided.

Table B.1. Reliability classification in accordance with EN 1990 [2]

Reliability classes	Consequences of structural failure	Reliability index β for reference period		Examples of buildings and civil engineering works
		1 year	50 years	
RC3 – high	High	5,2	4,3	Bridges, public buildings
RC2 – normal	Medium	4,7	3,8	Residences and offices
RC1 – low	Low	4,2	3,3	Agricultural buildings

It should be underlined that a couple of β values (for 1 year and 50 years) given in Tab. 1 for each reliability class corresponds to the same reliability level. Practical application of these values, however, depends on the time period considered in the verification, which may be linked to available probabilistic information concerning time variant basic variables (imposed load, wind, earthquake, etc.).

For example, considering a structure of reliability class 2 and the design working life 50 years, the reliability index $\beta = 3{,}8$ should be used provided that probabilistic models of basic variables are available for this period. The same reliability level is achieved when the reference period 1 year and $\beta = 4{,}7$ are applied using the theoretical models for one year.

It should be mentioned that for existing structures the target reliability level recommended in EN 1990 [2] given in Table 1 may be modified. In some cases it is allowed (if not necessary) to reduce the reliability index β (as indicated in the Dutch standard [11]). These cases should be discussed with all responsible partners.

B.3 DESIGN VALUE METHOD

The design value method is a very important step from probabilistic design methods toward operational partial factors method. The design value method is directly linked to the basic principle of EN 1990 [2], according to which it should be verified that no limit state is exceeded when the design values of all basic variables are used in the models of structural resistance R and action effect E. Thus, if the design values E_d and R_d of E and R are determined considering the design values of all basic variables, then a structure is considered as reliable, when the following expression holds

$$E_d < R_d \tag{B.1}$$

where the design values E_d and R_d are symbolically expressed as

$$E_d = E\{F_{d1}, F_{d2}, \ldots a_{d1}, a_{d2}, \ldots \theta_{d1}, \theta_{d2}, \ldots\} \tag{B.2}$$

$$R_d = R\{X_{d1}, X_{d2}, \ldots a_{d1}, a_{d2}, \ldots \theta_{d1}, \theta_{d2}, \ldots\} \tag{B.3}$$

Here, E denotes a function describing the action effect, R denotes a function describing the structural resistance, F is a general symbol for actions, X for material properties, a for geometrical properties, and θ for model uncertainties. Subscript 'd' refers to the design values.

If only two variables E and R are considered, then the design values E_d and R_d may be determined using the following formulae

$$P(E > E_d) = \Phi(+\alpha_E \beta) \tag{B.4}$$

$$P(R \le R_d) = \Phi(-\alpha_R \beta) \tag{B.5}$$

where β is the target reliability index, α_E and α_R, with $|\alpha| \le 1$, are the values of the FORM sensitivity factors [2, 9]. The sensitivity factor α_E is negative for unfavourable actions and action effects (in EN 1990 [2] $\alpha_E = -0{,}7$), the resistance sensitivity factor α_R is positive (in EN 1990 [2], $\alpha_R = 0{,}8$).

B.4 PARTIAL FACTOR METHOD

In accordance with the partial factor methods accepted in EN 1990 [2] the design values of the basic variables, X_d and F_d, are usually not introduced directly into the design expressions. They are commonly expressed in terms of their representative values X_{rep} and F_{rep}, which may be:

- the characteristic values X_k and F_k, i.e. values with a prescribed or intended probability of being exceeded, for example for actions, material properties and geometrical properties;

– the nominal values X_{nom} and F_{nom}, which may be treated as characteristic values for material properties and design values for geometrical properties.

The representative values X_{rep} and F_{rep} should be divided and/or multiplied, respectively, by the appropriate partial factors to obtain the design values X_d and F_d. Considering the representative values X_{rep} and F_{rep} by their characteristic values X_k and F_k, the design values X_d and F_d can be expressed as

$$X_d = X_k / \gamma_M \tag{B.6}$$

$$F_d = \gamma_F F_k \tag{B.7}$$

where γ_M denotes the partial factor of materials properties, and γ_F the partial factor of action. Both partial factors γ_M and γ_F are in most cases greater than 1.

As described in the following sections, both partial factors γ_M and γ_F should include model uncertainties, which may significantly affect the reliability of a structure. As stated in EN 1990, design values for model uncertainties may be incorporated into the design expressions through the partial factors γ_{Ed} and γ_{Rd} applied as follows:

$$E_d = \gamma_{Ed} E\{\gamma_{gj} G_{kj}; \gamma_P P; \gamma_{q1} Q_{k1}; \gamma_{qi} \psi_{0i} Q_{ki}; a_d ...\} \tag{B.8}$$

$$R_d = R\{\eta X_k / \gamma_m; a_d ...\} / \gamma_{Rd} \tag{B.9}$$

Here η denotes a conversion factor appropriate to the material property. The coefficient ψ, which takes account of reductions in the design values of variable actions, is applied as ψ_0 , ψ_1 or ψ_2 to simultaneously occurring accompanying variable actions. The following simplifications may be made to Eqn. (B.8) and (B.9).

a) On the loading side (for a single action or where linearity of action effects exists):

$$E_d = E\ \{\gamma_{F,i} F_{rep,i},\ a_d\} \tag{B.10}$$

b) On the resistance side the general format is given in Eqn. (B.9), and further simplifications may be modified in the relevant material-oriented documents.

The relation between individual partial factors in Eurocodes is schematically indicated in Fig. B.1. In accordance with Fig. B.1 the partial factor γ_F may be fragmented into the load intensity uncertainty factor γ_f and model uncertainty factor γ_{Ed}. Similarly, the partial factor γ_M may be split into the material property factor γ_m and resistance model uncertainty factor γ_{Rd}. Generally, it holds that

$$\gamma_F = \gamma_f\ \gamma_{Ed} \tag{B.11}$$

$$\gamma_M = \gamma_m\ \gamma_{Rd} \tag{B.12}$$

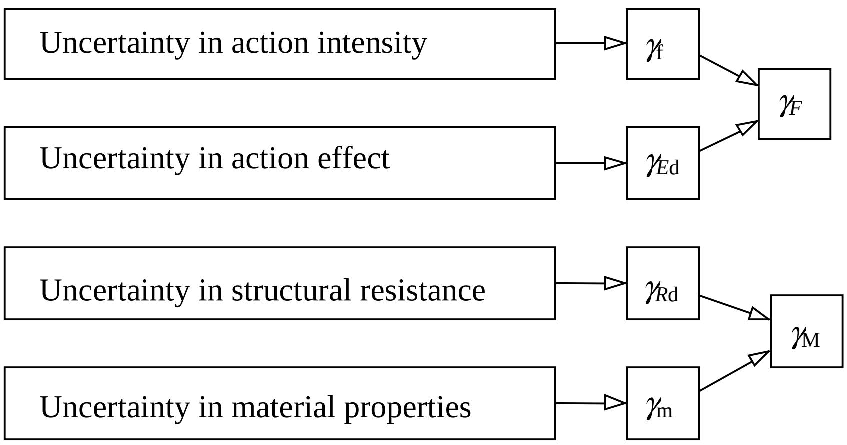

Fig. B.1. Partial factors in Eurocodes (EN 1990:2002 [2])

Numerical values of both factors of model uncertainty depend on particular conditions and should be derived from previous experience and available experimental data. The load effect factor γ_{Ed} may be expected within the interval from 1,05 to 1,15. The resistance factor γ_{Rd} depends on the construction materials and behaviour of the structural member. For example, uncertainty of the bending capacity of a steel beam will be lower (about 1,05) than uncertainty of a welded connection capacity (about 1,15).

B.5 PARTIAL FACTORS FOR MATERIAL

Partial factor for resistance γ_m is defined in Eqn. (B.13) by fractiles X_k and X_d. Taking into account general expression for fractiles of the random variable X the factor γ_m may be written as

$$\gamma_m = \frac{X_k}{X_d} = \frac{\mu_X + u_{0.05}\,\sigma_X}{\mu_X + u_p \sigma_X} = \frac{1 + u_{0.05}\,V_X}{1 + u_p V_X} \qquad \text{(B.13)}$$
$$p = \Phi(-0.8\beta)$$

where V_X denotes coefficients of variation of X, $u_{0.05}$ or u_p denotes 5%- or p-fractile of the standardised random variable having the same probability distribution as the resistance X.

Fig. B.2 and B.3 show the variation of the partial factor γ_R of the material property X with the reliability index β for selected values of the coefficient of variation w_R given for a normal distribution by Eqn. (B.13) (Fig. B.2), and a log-normal distribution by Eqn. (B.14) (Fig. B.3).

Assuming a log-normal distribution of X, then the fractiles u_p in Eqn. (B.13) must be taken from the standardised log-normal distribution. In the case of a log-normal distribution having the lower bound at zero, Eqn. (B.13) may be written as

$$\gamma_m = \frac{X_k}{X_d} = \frac{\frac{1}{\sqrt{1+V_X^2}}\exp\left(u_{0.05}\sqrt{\ln(1+V_X^2)}\right)}{\frac{1}{\sqrt{1+V_X^2}}\exp\left(u_p\sqrt{\ln(1+V_X^2)}\right)} \cong \qquad \text{(B.14)}$$
$$\cong \frac{\exp\left(u_{0.05} \times V_X\right)}{\exp\left(u_p \times V_X\right)}, \quad p = \Phi(-0.8\beta)$$

where u denotes now the normal standardised variable, for which detail tables are commonly available. Note that the approximation indicated in the last expression in Eqn. (14) is fully acceptable for small coefficients of variation V_X (< 0.2).

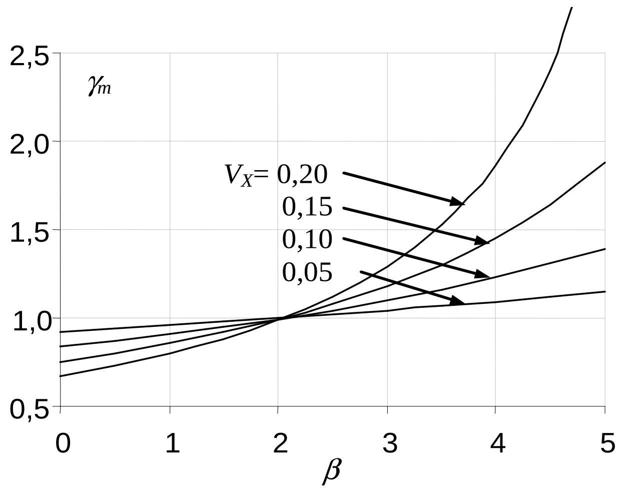

Fig. B.2. Variation of γ_m with β for selected coefficients of variation V_X and normal distribution of X

Fig. B.4 shows the dependence of the partial factor γ_m on the coefficient of variation w_X for three types of distribution functions: a normal N, a log-normal LN with the lower bound at zero (x_0 = 0) and a log-normal distribution LN with the skewness α = 0,5 assuming β = 3,8.

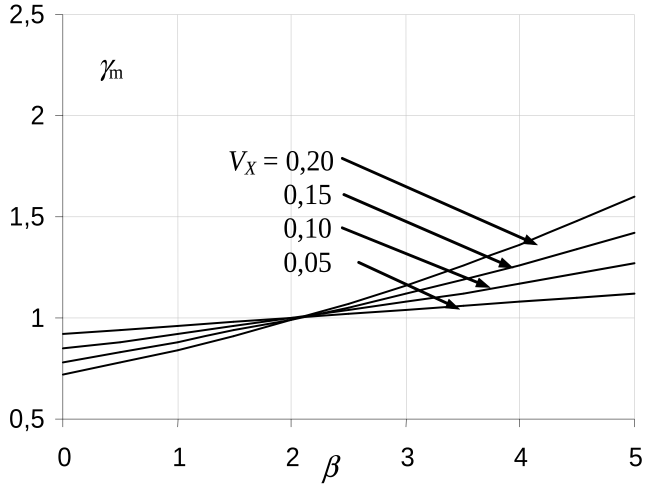

Fig. B.3. Variation of γ_m with β for selected coefficients of variation V_X and log-normal distribution of X.

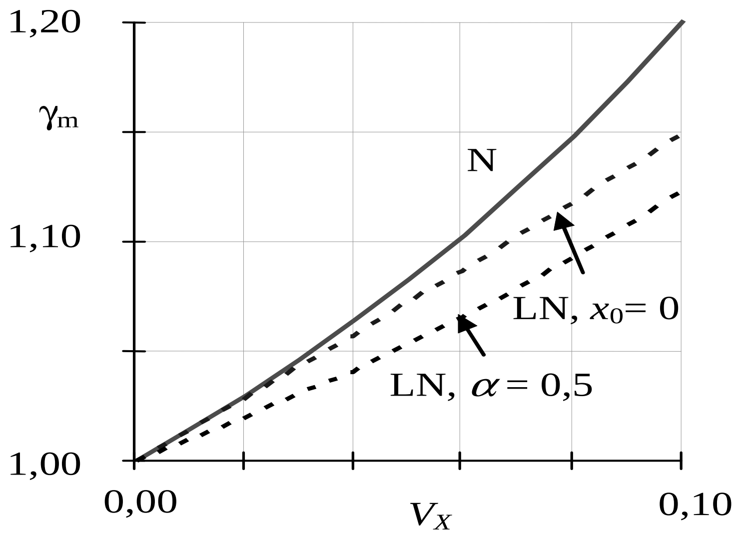

Fig. B.4. Variation of γ_m with β for selected coefficients of variation V_X, normal and log-normal distribution of X

B.6 PARTIAL FACTOR FOR PERMANENT LOAD

Consider a permanent load G (self-weight) having a normal distribution. It is assumed that the characteristic value G_k of G is defined as the mean μ_G:

$$G_k = \mu_G \tag{B.15}$$

Thus the design value G_d is given as

$$G_d = \mu_G - \alpha_G \times \beta \times \sigma_G = \mu_G (1 + 0{,}7 \times \beta \times w_G) \tag{B.16}$$

In Eqn. (B.16) μ_G denotes the mean, σ_G the standard deviation, V_G the coefficient of variation and $\alpha_G = -\,0{,}7$ the sensitivity factor of G.

The partial factor γ_G of G is given as

$$\gamma_g = G_d / G_k \tag{B.17}$$

Taking into account Eqn. (B.15) and (B.16) it follows from Eqn. (B.17) that

$$\gamma_g = (1 + 0{,}7 \times \beta \times V_G) \tag{B.18}$$

Fig. B.5 shows the variation of the partial factor γ_G with the reliability index β for selected values of the coefficient of variation V_G.

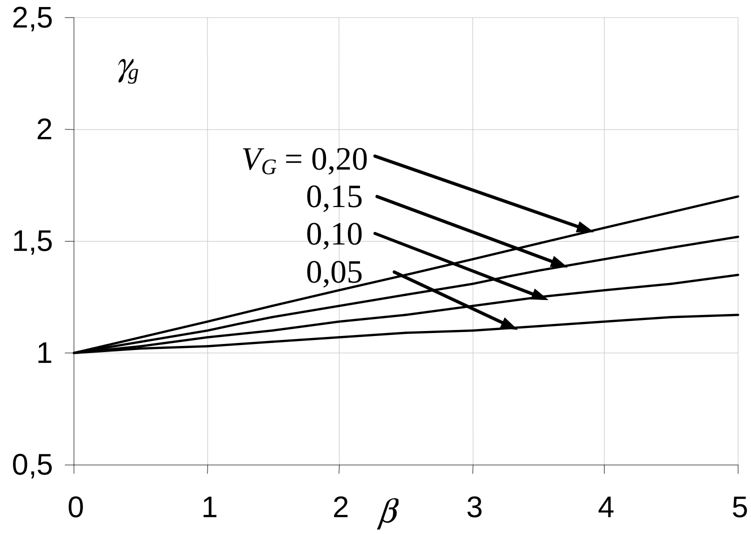

Fig. B.5. Variation of the partial factor γ_G with the reliability index β for selected values of the coefficient of variation V_G

B.7. PARTIAL FACTOR FOR VARIABLE LOADS

A similar procedure as in the case of the permanent load G can be used for estimation of the partial factors γ_Q for variable loads Q. Assuming the Gumbel distribution the characteristic value is usually defined as 0,98 fractile of annual extremes (or extremes related to a certain basic reference period) and is given as

$$Q_k = \mu_Q\,(1 - V_Q\,(0{,}45 + 0{,}78\,\ln(-\ln(0{,}98)))) \tag{B.19}$$

The design value Q_d related to the working life described by period ratio N is given as [9]

$$Q_d = \mu_Q\,(1 - V_Q\,(0{,}45 - 0{,}78\alpha_T\,\ln(N) + 0{,}78\ln(-\ln(\Phi^{-1}(-\alpha_E\beta)))) \tag{B.20}$$

In Eqn. (B.19) and (B.20) μ_Q denotes the mean, w_Q the coefficient of variation of extreme values of Q determined for the basic reference periods (for 1 or 5 years), N denotes the ratio of the working design life, for example 50 years, and the basic reference period. As an example, the period ratio N = 10 (= 50/5) is considered below. Finally, α_E = – 0,7 is the sensitivity factor of Q and α_T is the time-sensitivity factor given by the ratio V'_Q / V_Q, where w'_Q denotes the coefficient of variation of the time-dependent component of Q and w_Q denotes the coefficient of variation of the total Q. When Q depends on time-dependent components only, $V'_Q = V_Q$ and $\alpha_T = 1$. Note that the reliability index β in Eqn. (20) is related to the design working life (for example to 50 years) and not to the basic reference period (for example to 1 or 5 years). The partial factor γ_Q of Q is given as

$$\gamma_Q = Q_d / Q_k \tag{B.21}$$

The partial factor γ_Q of a variable action Q defined by Eqn. (B.21) depends on five parameters. In addition to V_Q, α_E, β (used also in the case of time-invariant basic variables), the partial factor of variable actions γ_Q depends also on the period ratio N and on the time-sensitivity factor α_T. Fig. 6 shows the variation of γ_Q with the coefficients of variation w_Q for selected values of β assuming a Gumbel distribution of Q, and the period ratio N = 10 (the design working life 10 times greater than the basic reference period) and the time-sensitivity factor α_T = 1 (no time-independent components).

It should be noted that the time-variant component may have a considerably lower variability than the total action Q, and, therefore, a reduced coefficient of variation should be considered in Eqn. (B.20) for estimating time-variant effects (α_T < 1). Consequently, the predicted design value Q_d and the partial factor γ_Q would decrease. Without going into details, it appears that the value γ_Q = 1,5, which is recommended in EN 1990 [2], is a reasonable approximation corresponding to the reliability index β = 3,8, the coefficient of variation w_Q = 0,3 (that may be considered as a reduced coefficient of variation of the extremes of Q) and to the period ratio N = 10 (the design working life being 10 times of the basic reference period).

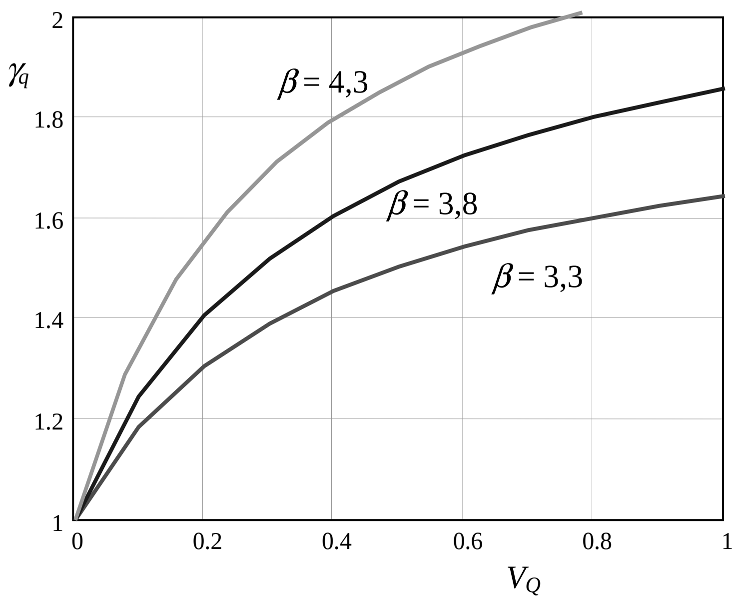

Fig. B.6. Variation of γ_Q with the coefficients of variation w_Q for selected values of β assuming a Gumbel distribution of Q, period ratio N =10 and α_T = 1

Fig. B.7 shows the variation of γ_Q with the reliability index β for selected coefficients of variation V_Q assuming again a Gumbel distribution of Q, and the period ratio N = 10.

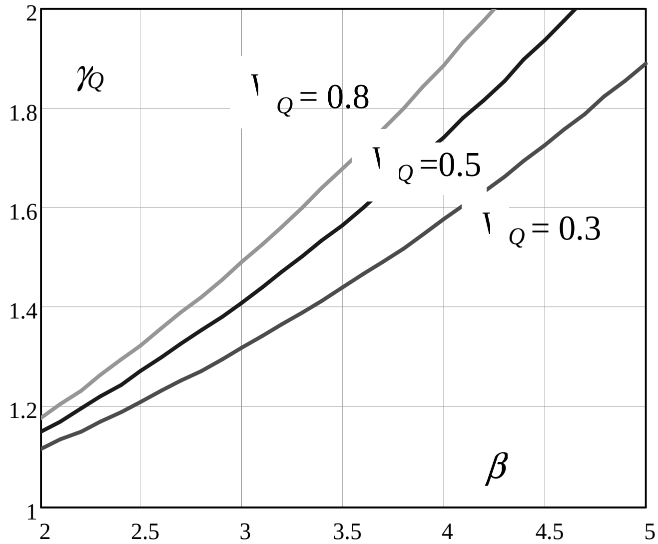

Fig. B.7. Variation of γ_Q with the reliability index β for selected values of the coefficients of variation V_Q assuming a Gumbel distribution of Q, period ratio N = 10 and α_T = 1

B.8 CONCLUDING REMARKS

Scientific methods of structural reliability based on the First Order Reliability Method (FORM) can be effectively used to specify the reliability elements of newly developing structural codes in a general case of several basic variables describing performance of the structure or structural system. Alternatively these methods can be used for the direct reliability analysis of new or existing structures. In both cases the specification of an appropriate target reliability level and its differentiation is of uttermost importance.

However, up to now the assessment of various reliability elements in the new structural codes is partly based on historical and past experience. Such an experience may depend on local conditions including climatic actions and traditionally used construction materials, and, consequently, might be considerably diverse in different countries. That is why a number of reliability elements and parameters in the present suite of European standards including target reliability level and reliability differentiation are open for national choice.

The reliability elements recommended in EN 1990 [2] for new structures seem to be, in general, acceptable. However, the theory of structural reliability indicates that the partial factors for permanent loads may be slightly high (in particular for own weight), the partial factors for some variable loads slightly low (in particular for snow and wind) and the combination factors rather conservative. Nevertheless, the available theoretical methods based on the theory of structural reliability can be also effectively applied for additional calibration and refinement of structural codes when applied to verification of new or existing structures under specific conditions.

The theory of structural reliability is further extremely useful for the specification of the optimum target reliability level and reliability differentiation making allowance for the cost of structures, maintenance and consequences of possible failure of new and existing structures.

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ANNEX C: REPORT ON ASSESSMENT

Michal Kronika[1]
[1]Secondary Technical School of Civil Engineering, Resslova 2, České Budějovice

Summary

This Annex C – Report on Assessment presents an option how to process a report when assessing existing structures. In the first part, a "Report Format" is described following Annex G (informative) of ČSN ISO 13822 *Principles of Construction Designing – Existing Structures Assessment* from July 2005. After that one of many practical examples of such a report can be processed is shown.

1 INTRODUCTION

Assessment of existing structures is in common cases based on a method that includes several working stages. At the end of the assessment some form of report is usually required. The following provisions primarily refer to the final report that is to be issued after the whole assessment process is completed.[1]

In the next subchapter the "report format" is described following annexe G (informative) of ČSN ISO 13822 *Principles of Construction Designing–Existing Structures Assessment* from July 2005.

In subchapter 3 a practical example of the final report is shown.

2 REPORT FORMAT BASED ON ISO 13822
Annex G (informative) ČSN ISO 13822

G.1 Title page

The following items are given: title, date, contract owner and contractor (full name and address of the engineer and/or company).

G.2 Name of engineer and/or firm

The people that have carried out the assessment together with the contract owner´s representatives and other participants are introduced herein.

G.3 Synopsis

The issue is briefly and precisely summarized on one or two pages, significant investigation points are provided, including the main conclusions, recommendations and all important objections and/or rejections.

G.4 Table of contents

The following items are included:

a) scope of assessment;
b) description of structure;
c) investigation;
  - examined documents,
  - inspected objects,
  - sampling and testing procedures, test results;
d) analysis;

[1] Klokner Institute, ČVUT Praha, Prof. Ing. Milan Holický, DrSc., Ing. Jana Marková, PhD. *ČSN ISO 13822, Bases fordesign of structures – Assessment of existing structures.* June 2005, p. 16

e) verification;
f) data analysis;
g) evaluation of possible interventions;
h) conclusions and recommendations;
i) reference documents and literature;
j) annexes.

G.5 Scope of assessment

Reasons for assessment and the task scopes as arranged between the contract owner and the engineer are introduced herein. The assessment procedure is described (see annexe B) and all assessment activities are recorded. The utilization plan and safety precaution plan are determined.

G.6 Description of structure

The following information is briefly and precisely introduced: name, location (address), the load-bearing system including all drawings. Also, the history of the original load-bearing construction, successive changes, previous and present usage purpose are specified herein.

G.7 Investigation

G.7.1 Documents examined

Documents that are available to the engineer including their origin (e.g. letters from the contract owner or their representative, drawings and/or reports from other parties sent by the contract owner) are listed herein.

G.7.2 Inspection items

It is important to be able to verify that the authorised and qualified people have carried out the corresponding number of inspections. Possible restrictions on the inspection´s effectiveness and factors out of the engineer´s range are to be recorded.

G.7.3 Sampling and testing procedure

The origin, number, date and location of the test sample collection are introduced herein. We also introduce the laboratory name and contract measures of procedures for test sample collection and testing. It is important to present the purpose and nature of tests/analyses followed by the summary of results. Further, it is advised to attach copies of the laboratory test reports. In the case of proof-load a test plan and other documents are introduced in the annexe.

G.8 Analysis

It is necessary to introduce the method used for the calculation and also the criteria used for its consideration. The analysis results are to be briefly summarized. Detailed calculations can be presented in the annexe.

G.9 Verification

Verification of the construction´s safety and usability are covered in chapter 7.

G.10 Discussion of evidence

As the title suggests the importance of all results are discussed in this chapter, as described in G.11 and G.12 and especially their importance for the purpose of the evaluation.

Any uncertainties that have remained after the investigation are introduced here, as well as any possible further verification necessity.

G.11 Review intervention options

It is necessary to consider possible variations of measures. For every variation the costs are to be estimated.

G.12 Conclusions and recommendations

G.12.1 Conclusions

Conclusions must be strictly logical expert´s opinions that follow from careful evaluation of the information obtained. It is advisable to briefly describe the accuracy and limits of methods used and the actual importance of the results. Every conclusion should be based on the factual aspects mentioned in previous chapters of the report.

G.12.2 Recommendations

The course of activities is described briefly and in logical succession so that it is practicable for the contract owner and it follows on from the conclusions. For individual pieces of work a rough estimation of costs is provided. Also, the remaining lifespan, the inspection and maintenance plan, and the date of the next assessment are determined.

G.13 Annexes

In annexes the following items should be introduced: drawings, photographs, laboratory documents of tests, calculations etc.

3 AN EXAMPLE OF THE FINAL REPORT

3.1 Title page

Title: STRUCTURAL ASSESSMENT - influence of building modifications on structural stability of the City Hall building, Normální Street 1, Popelín, postcode 00 000
Date: 30/6/2012
Contract owner: Statutory city of xxxxxx, municipal building authority, Nová Street 28.
Contractor: Structural stability office XYZ, Kosmonautů 1825, Novákovice, postcode 00 000

3.2 Name of engineers

Authors of the assessment:	Ing. A. Novák – certified structural engineer, expert witness Ing. P. Nová – certified structural engineer
Contract owner representative:	Ing. F. Starý – certified engineer, technical supervisor of the investor
Other:	Ing. B. Malý – expert on construction foundations

3.3 Synopsis

The assessment purpose is to find the causes of bearing construction failures that have been discovered during the work on the storage space modifications and also to propose possible interventions for their elimination.

On the supporting transverse wall adjoining the passageway and on the spot, where the modifications are being carried out, the following faults have been discovered:

- To the right of the cellar space entry opening (see Fig. 8) there is a horizontal crack in the plaster. The plaster on the spot has been removed and it has been found out that the crack does not continue into the brickwork.

- To the left of the cellar space entry opening there is a horizontal crack on the pillar at the base height of the arched girder above the passageway. Similar crack has been found on the opposite side of the passageway - on the arched girder.

Also, there is a crack on the vertical boundary line between the mentioned pillar´s arched girder above the passageway and the wall adjoining the passageway.
In both cases there are horizontal micro-cracks and cracks (see Fig 8)

A hydrological survey has been carried out on the site of the building before the proposal of safety building modifications. This survey shows the foundation conditions and the load-bearing capacity of the foundation joint, as well as sample collection for determination of quality and bearing capacity of existing brickwork and foundations.

A structural analysis has been carried out considering the load bearing capacity of existing foundation taking into account the on-going building modifications. Also, it considers the proposal for a new lintel including the weakened brick pillar.

The documented failures of the supporting passageway wall are not serious and do not endanger the structural stability of the building in its present state – the above mentioned building modifications have not affected the structural stability.

There is no reasonable connection between the supporting wall failures and the building modifications (carried out according to the project documentation) that would be suggested by the entry data.

It cannot be ruled out that the emergence of the cracks developed during the work on the building modifications. Apparently, this might happen with all buildings where building modifications are being executed. In this case it would involve definitive failures that do not endanger the structural stability of the building.

As it is not possible to rule out other reasons of the failure occurrence than the above mentioned building modifications, it is recommended to monitor the cracks, e. g. by using the plaster bands. In case that the cracks and consequently the building failures would not be definitive, i. e. that there would appear cracks on the plaster bands and the failures would continue, it is advisable to appoint an expert to inspect the whole building.

3.4 Table of contents

3.4.1 Scope of assessment

The assessment purpose is to determine the failure causes of bearing constructions and a proposal of measures for their elimination.

The supporting transverse wall adjoining the passageway and the space where the building modifications are being carried out show signs of failure.

Following assessment range has been arranged with the contract owner:

Preliminary verification:

- verification of available documentation and other data
- preliminary inspection
- preliminary verification
- decision on immediate measures
- recommendation for detailed evaluation

1. Work range for detailed evaluation:
   - detailed documentation finding and verification

- detailed inspection, sampling and testing
- load determination
- determination of construction properties
- construction analysis
- verification

2. Possible additional inspection
3. Report on evaluation results
4. Assessment and decision
5. Proposal of measures

3.4.2 Description of structure

Designation: Offices of municipal authority

Address: Normální Street 1, Popelín, postcode 00 000

Description: The building is situated in original in-line housing development, the load-bearing system is wall-longitudinal, the load-bearing peripheral brickwork is directly connected to the neighbouring buildings. The roof construction is saddle-shaped; the load-bearing construction of the roof frame is formed by classic purlin frame with upright stools.

The actual building consists of the ground floor, first, second, third floor and the attic. The load-bearing peripheral and inner walls are made of bricks, at the bottom the walls are made of stones with original mortar of undefined strength. Other walls are probably made of bricks.

History: The load-bearing constructions are original without any interventions during the lifetime period.

Utilization: The utilization of the building has been the same since the beginning. The building was designed and always used as office space.

3.4.3 Investigation

3.4.3.1 Examined documents

1. The original project documentation from time of the construction by the studio OPR in April 1956
2. Project documentation for the building permit of the building „Reconstruction and modification of store spaces – the office of municipal authority from 06/2004
3. Project „Drainage and hydro-insulation”, author Vodaři spol. s.r.o. in cooperation with Ing. J. Klapka.
4. The building journal of the reconstruction kept by the building company XYZ

3.4.3.2 Inspection items

1. Preliminary inspection of the building on 18.9.2007
   with the participation of:
   Ing. P. Nová – certified structural engineer, static office XYZ
   Ing. F. Starý – certified engineer, technical supervisor of the investor

2. Detailed inspection of the building on 5.11.2007
   with the participation of:
   Ing. A. Novák – certified structural engineer, expert witness

Ing. P. Nová – certified structural engineer, static office XYZ
Ing. F. Starý – certified engineer, technical supervisor of the investor
Ing. B. Malý – specialist in the field of founding constructions

3.4.3.3 Sampling and testing procedures

Samples were collected on 12.11.2007 based on the order of static office XYZ. 9 samples have been collected to find out the load-bearing capacity of the foundation ground, 6 samples to determine the load-bearing capacity of the brickwork and foundations. The samples have been collected and analysed by TAZUS with the seat in České Budějovice, Nemanická Street 8.

The testing and evaluation of results comply with ČSN ISO 13822 Principles of Construction Designing –Existing Structures Assessment.

The copy of laboratory protocols of testing and testing results and introduced in the annexes of the assessment.

3.4.4 Analysis

Based on the testing results a static calculation has been made that considers the load-bearing capacity of the existing foundation constructions with regard to the building modifications. It also considers the design of the new lintel including the weakened brick pillar. The calculation complies with ČSN ISO 2394 General Principles of Construction Reliability and ČSN ISO 13822 Principles of Construction Designing –Existing Structures Assessment.

The calculation result is a proposal to reinforce the brickwork around the new door opening using concrete B20 – C16/20 with a bracing of ground plan dimensions of 80mm x 600mm. The concrete is horizontally anchored to the existing brickwork using anchor centres. Such reinforced pillar compensates for increased weight due to the building modifications.

Detailed calculations are introduced in the annexe.

3.4.5 Verification

In the calculations safety and serviceability of the existing structure have been verified. For reliability assessment the remaining lifetime of the existing structure has been taken into account, as well as its actual state and the executed building modifications.

The verification has been based on the conception of limit states – bearing capacity and serviceability. The verification has been carried out based on the partial factor method, see current regulations. The partial factors have been modified with regard to the results of material testing and the quality of the contractor´s work during the building modifications.

3.4.6 Discussion of evidence

Based on inspections, conducted tests and calculations it can be assumed that failures are in the mounting of both ends of the arched girder on the pillar, which are in the mounting of both ends higher than the mounting of the given lintel.

It is possible to assume that the building modifications on the right hand side of the building (view from the Street) are not responsible for the failures.

The failures could be caused by temporary increase or decrease in the level of ground water, which based on the documentation lies above the foundation joint of the building. Also, the failures could be caused by increased load on the arched girder.

It is not possible to entirely exclude that the projection of the joint between the pillar under the base of the arched girder and the wall of the passageway could have occurred during making of the opening in the supporting wall. In this case the failures would be definitive and would not endanger the structural stability of the building.

Still two options for increasing the bearing capacity of the brickwork on the spot of the building modifications have been proposed.

3.4.7 Review of intervention measures

To strengthen the brickwork two variants have been proposed. First, the existing brickwork could be exchanged for brickwork with higher load bearing capacity. Second, the existing brickwork could be strengthened using reinforced concrete. Cost estimation has been made for both variants in cooperation with a building company. After consulting the investor and the building company the second variant has been chosen.

3.4.8 Conclusions

Documented failures of the supporting wall in the passageway are not serious and do not endanger the structural stability of the building. Building modifications have not disrupted the static conception of the building.

Based on the entry information, obtained data and the calculations there is no causal link between the failures of the transverse supporting wall neighbouring the passageway and the executed building modifications.

3.4.9 Recommendation

With regard to the fact that the cracks were discovered during the building modifications, it cannot be excluded that the cracks had developed before the work on the building modifications, or whether they were caused by them. This is similar with every building where modifications are being carried out. In this case the observed faults do not endanger the structural stability of the building.

As it is impossible to exclude other causes of the failures than the above mentioned building modifications, it is recommended to watch closely the cracks, e. g. using plaster bands. If the cracks and consequently the failures appear not to be definitive, i. e. on the bands appear identical cracks. It is recommended to appoint an expert to investigate the whole building.

3.4.10 References

Textbooks:

- Konstrukce pozemních staveb - Poruchy, údržba, rekonstrukce a modernizace budov: I. díl - SNTL Praha 1985, II. díl VUT Brno 1984
- Konstrukce pozemních staveb - Vady, poruchy, údržba a změny staveb. Cvičení - VUT Brno 1984
- Konstrukce pozemních staveb 60 - Poruchy a rekonstrukce staveb, part I and II-ČVUT Praha 1994
- D. Pume, F. Čermák a kol. - Průzkumy a opravy stavebních konstrukcí - ARCH Praha 1993
- Eichler: Mechanika zemin a zakládání staveb
- Rukověť znalce oboru 35 "Stavebnictví", Diagnostika vad a poruch v zakládání staveb obytných, průmyslových a zemědělských
- T. Vaněk: Rekonstrukce staveb
- Vyhláška MMR č. 268/2009 Sb. o obecných technických požadavcích na výstavbu
- J. Witzany: Poruchy a rekonstrukce zděných budov, ČKAIT Praha 1999
- Ing. P. Linhart a kol. - Rekonstrukce staveb v obraze
- R. Drochytka, J. Bydžovský - Stavební vady od A do Z

- O. Makýš - Technologie renovace budov
- Z. Bažant, L. Klusáček - Statika při rekonstrukci objektů

Standards:
- ČSN ISO 13822 Principles of Designing Constructions–Existing Structures Assessment. June 2005
- ČSN ISO 2394 General Principles of Construction Reliability. November 2003

3.4.11 Annexes

Fig. 1 building drawings
Fig. 2 laboratory protocols on testing and testing results
Fig. 3 static calculations
Fig. 4 photographs
Fig. 5 records from inspections
Fig. 6 copy of records from the building journal
Fig. 7 records from negotiations on inspection results, calculations and modification proposals
Fig. 8 crack pattern

4 CONCLUDING REMARKS

The above mentioned example of a report shows an option how to process a report when assessing existing structures (or in other words when assessing existing structures from a static point of view), which is done during restorations, when a building's usage changes, or in cases of other interventions.

I recommend to the author of an assessment of existing structures to create a report according to the above mentioned citation of Annexe G (informative) of ČSN ISO 13822 *Principles of Construction Designing – Existing Structures Assessment* from July 2005.

I think that a majority of clients who require an assessment of existing structures will at first familiarise themselves with the report where they can easily find the results and conclusions of performed calculations. Also the processors of further necessary project documentation can familiarise themselves with the concrete calculations of the author of the assessment of existing structures.

A report is the fastest way the client can familiarise himself with the state of his building.

REFERENCES

ČSN ISO 13822 *Principles of Construction Designing–Existing Structures Assessment* from July 2005.

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