Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 74 (1996)

Rubrik: Plenary session 2: Eurocode 1: basis of design

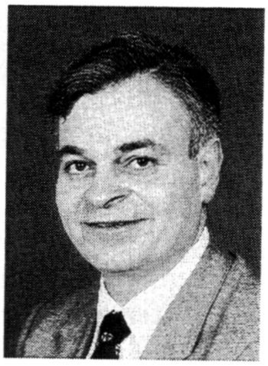
Plenary Session 2

Eurocode 1: Basis of Design

Leere Seite
Blank page
Page vide

ENV 1991-1: EUROCODE 1: Part 1: Basis of Design
Introduction, Development and Research Needs

Haig GULVANESSIAN
Professor & Civil Engineer
Building Research Establishment
Garston, Watford, UK



Haig Gulvanessian, born in 1941, is a graduate of the University of London. He is Head of the Structural Design Division at BRE and a Visiting Professor at Imperial College, London. He is Technical Secretary for the EC1 Committee and Chairman of the CEN/TC250 Interim Ad-Hoc Group on Basis of Design

Summary

This paper describes the concept and contents of ENV 1991:1 Eurocode 1 "Basis of Design". It also describes possible developments for the document that may be undertaken during its transposition to EN status, and suggests topics that may require research to aid the continued future developments of the document into the next millennium.

1 ENV 1991-1 - Eurocode 1:Part 1: Basis of Design

1.1 Introduction

Eurocode 1:Part 1: Basis of Design (hereafter called ENV 1991-1) was produced by a CEN/TC 250 Project Team (see Appendix A) and was published as an ENV in 1994 [1]. This part of the paper provides a brief description to ENV 1991-1. A theoretical background to ENV 1991-1 is given by Vrouwenvelder [2], and Gulvanessian and Holicky [3].

1.1.1 Objectives of ENV 1991-1
ENV 1991-1 describes the principles and requirements for safety, serviceability and durability of structures. It is based on the limit state concept used in conjunction with the partial factor method.

It will be used, for direct application, for the design of new structures, together with
- the other parts of Eurocode 1 and
- the design Eurocodes (Eurocodes 2 to 9).

ENV 1991-1 also give guidance for the aspects of structural reliability relating to safety, serviceability and durability for design cases not covered by Eurocodes 1 to 9 (eg other actions, structures outside the scope of the Eurocodes, other materials);

- different degrees of reliability required at national, regional or local level.

The required reliability relating to structural safety and serviceability may be achieved by the suitable combination of:
a) Measures relating to design which include serviceability requirements; the representative values of actions; the choice of partial factor; the consideration of durability; the consideration of the degree of robustness; the amount and quality of preliminary investigations of soils and possible environmental influences; the accuracy of the mechanical models used and the stringency of the detailing rules.
b) Measures relating to quality assurance to reduce the risk of hazards in gross human errors; design; and execution.

1.3.3 Design Situations

A relevant design situation is selected taking account of the circumstances in which the structure may be required to fulfil its function. ENV 1991-1 classifies design situations as follows:
- persistent situations (Conditions of normal use);
- transient situations (Temporary conditions eg during execution);
- accidental situations; and
- seismic situations.

1.3.4 Design Working Life

The design working life is the assumed period for which a structure is to be used for its intended purpose with anticipated maintenance but without major repair being necessary. Table 1, taken from the UK NAD [4] for ENV 1991-1 is a variation the table provided in ENV 1991-1.

Class	Notional design working life (years)	Examples
1	1-5	Temporary structures
2	25	Replaceable structural parts, e.g. gantry girders, bearings
3	50	Buildings and other common structures, other than those listed below
4	100	Monumental buildings, and other special or important structures
5	120	Bridges

Table 1. Design working life classification

1.3.5 Durability

The design should ensure that the durability of a structure or part of it in its given environment is such that it remains fit for use during the design working life given appropriate maintenance. Interrelated factors which should be considered to ensure an adequately durable structure are:
- the intended and possible use of the structure;
- the required performance criteria;

1.1.2 Intended Users
Due to the scope and objectives of ENV 1991-1 the document will be used by more categories of users than the other Eurocodes. The categories include:
- code drafting committees;
- clients (eg for the formulation of their specific requirements on reliability level and durability);
- designers and contractors (as for the other Eurocodes); and
- public authorities (eg to set safety criteria).

1.2 Assumptions

The following assumptions are associated with the validity of the design principles of ENV 1991-1:
- The choice of the structural system and the design of a structure is made by appropriately qualified and experienced personnel.
- Execution is carried out by personnel having the appropriate skills and experience.
- Adequate supervision and quality control is provided is provided during execution of the work, ie in design offices, factories, plants and on site.
- The construction materials and products are used as specified in Eurocodes 1 to 9 or in the relevant supporting material or product specification.
- The structure will be adequately maintained.
- The structure will be used in accordance with the design assumptions.
- Design procedures are valid only when the requirements for the materials, execution and workmanship given in Eurocodes 2 to 6 and 9 are also complied with.

1.3 Requirements

1.3.1 Fundamental Requirements
The fundamental requirements stipulate that :
a) a structure shall be designed and executed in such a way that it will, during its intended life with appropriate degrees of reliability and in an economic way:
- remain fit for the use for which it is required (serviceability requirement); and
- sustain all actions and influences likely to occur during execution and use (safety requirement); and

b) a structure shall be designed and executed in such a way that it will not be damaged by events such as fire, explosion, impact or consequences of human errors, to an extent disproportionate to the original cause (robustness requirement). ENV 1991-1 gives ways of avoiding or limiting potential damage.

1.3.2 Reliability Differentiation
An appropriate degree of reliability for the majority of structures is obtained by design and execution according to Eurocodes 1 to 9, and appropriate quality assurance measures. A different level of reliability may be adopted (reliability differentiation) for structural safety or serviceability and this may depend on:
- the cause and mode of failure;
- the possible consequences of failure in terms of risk to life, injury, potential economic losses and the level of social inconvenience;
- the expense and procedures necessary to reduce the risk of failure; and

- the expected environmental influences;
- the composition, properties and performance of the materials;
- the choice of the structural system;
- the shape of members and the structural detailing;
- the quality of workmanship, and level of control;
- the particular protective measures; and
- the maintenance during the intended life.

1.3.6 Quality Assurance
Appropriate quality assurance measures should be taken in order to provide a structure which corresponds to the requirements and to the assumptions made in the design. These measures should include organisational measures and controls at the stages of design, execution, use and maintenance.

1.4 Principles of Limit State Design

1.4.1 Ultimate and Serviceability Limit States
Eurocodes use the concept of limit state design. Limit states are states beyond which the structure no longer satisfies the design performance requirements. ENV 1991-1 makes a distinction between ultimate limit states and serviceability limit states.

Ultimate limit states are those associated with collapse or with other forms of structural failure and concern
- the safety of the structure and its contents; and
- the safety of people.

Serviceability limit states correspond to conditions beyond which specified service requirements for a structure or structural element are no longer met and concern
- the functioning of the construction works or parts of them;
- the comfort of people; and
- the appearance.

The serviceability requirements should be determined in contracts and/or in the design.

1.4.2 Limit State Design
Limit state design is carried out by:
- setting up structural and load models for relevant ultimate and serviceability limit states to be considered in the various design situations and load cases; and
- verifying that the limit states are not exceeded when the design values for actions, material properties and geometrical data are used in models.

Design values are generally obtained by using the characteristic or representative values (see 1.5) in combination with partial and other factors (see 1.10).

1.5 Actions

An action (F) is:
- a direct action, ie force (load) applied to the structure; or
- an indirect action, ie an imposed or constrained deformation or an imposed acceleration

caused, for example, by temperature changes etc.

Actions are classified
- by their variation in time, permanent actions (G); variable actions (Q); and accidental actions (A);
- by their spatial variation, fixed actions (eg self-weight); free actions (eg wind and snow loads);
- by their nature and/or structural response, static actions; dynamic actions.

The characteristic values of an action is its main representative value.

The self weight of a structure can, be represented by a single characteristic value (G_k), provided the variability of G is small, and be calculated on the basis of the nominal dimensions and the mean unit mass. If the variability of G is not small, ENV 1991-1 stipulates the use of two values; an upper value ($G_{k,sup}$) and a lower value ($G_{k,inf}$). Ostlund [5] provides more information on this.

A variable action has the following representative values. The characteristic value (Q_k); the combination value ($\psi_0 Q_k$) ; the frequent value (ψ_1 Q_k); and the quasi-permanent value (ψ_2 Q_k). Values for ψ_0 , ψ_1 and ψ_2 are given in ENV 1991-1.

1.6 Material Properties

Properties of materials (including soil and rock) or products are represented by characteristic values which correspond to the value of the property having a prescribed probability on not being attained in a hypothetical unlimited test series. They generally correspond for a particular property to a specified fractile of the assumed statistical distribution of the property of the material in the structure.

1.7 Geometrical Data

Geometric data are represented by their characteristic value, or in the case of imperfections by their design value.

1.8 Structural Analysis

ENV 1991-1 provides principles which are common for structures of different type and material and the Section on Structural Analysis provides guidance on the modelling of static actions, dynamic actions and fire actions.

1.9 Design Assisted by Testing

Where calculation rules or material properties given in Eurocodes 1 to 9 are not sufficient or where economy may result from tests on prototypes, part of the design procedure may be performed on the basis of tests. ENV 1991-1 requires that tests are set up and evaluated in such a way that the structure has the same level of reliability to all possible limit states and design situations as achieved by design based on calculation procedures specified in Eurocodes 1 to 9.

1.10 Verification

In the partial factor method, it has to be verified that for all relevant design situations, the limit states are not exceeded when design values for actions, material properties and geometrical data are used in the design models. In particular ENV 1991-1 stipulates that
a) the effects of design actions do not exceed the design resistance of the structure at the ultimate limit state; and
b) the effects of design actions do not exceed the performance criteria for the serviceability limit state.

Actions are combined so that they produce the most unfavourable effect on the structure for the limit state being considered; actions which cannot occur simultaneously, for example, due to physical reasons, should not be considered together in combination.

For the ultimate limit state there are three types of combination of actions as follows:
- Fundamental (persistent and transient) situations
- Accidental situations
- Seismic situations.

ENV 1991-1 provides the partial factors for ultimate limit states in the persistent, transient and accidental situations for the following cases:
- Case A, which is for loss of static equilibrium
- Case B, which is for failure of the structure or structural elements, including those of the footing, piles, basement walls etc., governed by strength of structural material
- Case C which is for failure of the ground.

The design should be verified for each case A, B and C separately as relevant.

For the serviceability limit states there are three types of combinations .
- The characteristic (rare) combination used mainly in those cases when exceedance of a limit state causes a permanent local damage or permanent unacceptable deformation.
- The frequent combinations used mainly in those cases when exceedance of a limit state causes local damage, large deformations or vibrations which are temporary.
- The quasi-permanent combinations used mainly when log term effects are of importance.

ENV 1991-1 also provides simplified verifications for consideration of both the ultimate and serviceability limit states which may be used for particular cases.

2 Possible Changes to ENV 1991-1 at Transposition to EN 1991-1.

CEN/TC250, the committee responsible for the development of the Eurocodes set up an interim Ad-Hoc Group (see Appendix A) whose objectives were to formulate the method of working and terms of reference and objectives for use by a future Project Team for the transposition of ENV 1991-1 into an EN. This part of the paper provides a summary of the Ad-Hoc Group's recommendations [6]. It is intended that voting to transpose the document into an EN takes place in late 1998.

2.1 Layout and Organisation

The Ad-Hoc Group has recommended (figure 1) that the EN 1991-1 has;
- General sections applicable to all structures within the fields of application of the Structural Eurocodes defining requirements and criteria:
- Separate application parts, which will be derivations from the General sections specific for each structural type (eg buildings, bridges, towers and masts etc).

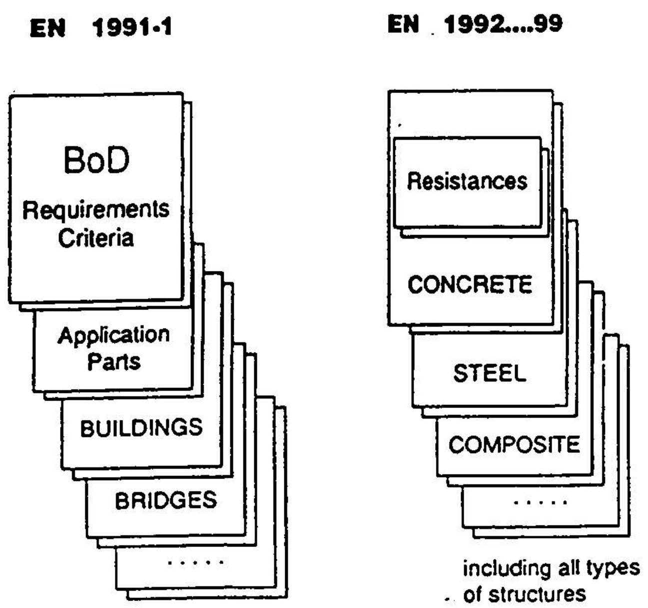


Figure 1: Layout and organisation of EN 1991-1:1998 and Design Eurocode

Figure 1. Proposed layout and organisation of EN 1991-1 and the EN design Eurocodes.

This recommendation will ensure that the document is more user-friendly; and application parts of various types of structures can be added at future dates without amendments to the main part of the document.

2.2 Scope and Contents

The Ad-Hoc Group recommended that the EN 1991-1 should include in addition to its present contents the material independent clauses from Chapters 2 of Eurocodes 2 to 9 from where these clauses will be removed.

This recommendation has the following merits:
a) One document will provide the framework of principles and partial combination factors for all types of structures and materials.
b) Technical decisions for all material independent matters will be taken by one committee and this will avoid contradictions between the various Eurocodes.
c) The same organisation and framework for all the Eurocodes.

2.3 Specific technical points for consideration for change

The Ad-Hoc Group have recommended to proceed in two steps. The first step, defined as the document for voting for EN in 1998 should only include improvements that the AD-Hoc Group feel are achievable within the timescale. The second step, defined as the first revision of the EN after five years, is to further develop the document. The proposed programme of work for the two steps is shown in Table 2.

In addition to the items in Table 2:
- the Ad-Hoc Group considered recommending the production of further guidance on Classification of Structures. The AD-Hoc group recognised that political and legal rules may inhibit a normative introduction of classification in Member States. However, considering that most failures occur due to deficiencies in the design and execution process of a project or poor maintenance and not because a partial factor was on the `low side' the Ad-Hoc Group have recommended the production of a new Informative annex to introduce a system where depending on the type of construction works and consequences of its failure, different design supervision and inspection classes can be recommended.
- the Ad-Hoc Group considered the need for guidance for the selection of γ_M values in the design Eurocodes and recommended in the long term for comprehensive guidance from which Eurocodes can select γ_M for appropriate components considering the conditions of quality control and the manufacturing process.

2.4 Experience in Use

The Ad-Hoc Group recommend that the various NADs to ENV 1991-1 and other experiences with the application of the document be considered when producing the EN 1991-1.

3 Future Research Needs

To aid future development of ENV 1991-1 and the related parts of the other Eurocodes, particular organisations (eg BRE, TNO, TU Munchen) intend to submit a proposal to the European Commission. The main objective of this proposal will be to create a network

Item	Current Situation and Need for Improvement	First Step (EN 1991-1:1998)	Second Step (EN 1991-1:5 yr revision)
Serviceability Limit State	The requirements relating to serviceability criteria are well defined but the verification rules need improvement.	The basic concepts for serviceability needs broadening to define more accurately the purpose of the appropriate verifications with regard to the fundamental requirement.	Develop material independent performance criteria for the application parts of Basis of Design.
Static Equilibrium	The guidance provided on static equilibrium is general and broader information on treating static equilibrium for all types of structures is required.	Definition of static equilibrium, with broadening of the concept of static equilibrium in EN 1991-1 to take account of all types of structures during the execution process and normal use.	Development of specific rules for all the application parts of EN 1991-1
Durability	The main requirement for defining durability is the design working life and the guidance provided needs improving.	Table 2.1 to be developed in order to provide useful information for the design of structures and structural components.	
Fatigue Verification	ENV 1991-1 provides an Annex for fatigue which should be brought into the main text.	Transfer Annex B of ENV 1991-1 to the main text with any appropriate rules from the design Eurocodes.	Development of rules for the application parts of ENV 1991-1: Basis of Design
Structural Analysis	The current guidance provided has to be broadened to be useful in design and to form a basis for harmonisation of the information in the design Eurocodes.	The information in the design Eurocodes on structural analysis should be brought together and harmonised into ENV 1991-1 with advice on the application of these methods.	
Annex A	Annex A is user-unfriendly and can lead to unsafe designs if used wrongly; it should clarified,improved and completed.	Completely re-edit and provide explanations for the between γ_m's in the design Eurocodes; and if possible harmonise with a common equation.	
Soil Structure / interaction	Application rules in Table 9.2 but ENV 1991-1 does not provide principles on this topic.	More precise explanation should be given for the field of application for cases B & C of Table 9.2.	Produce comprehensive rules together with CEN/TC250/SC7.

Table 2: Programme of work for items needing improvement

whose aims will be to have regular contact between the member states on
- the use of ENV 1991-1 in practice;
- exchange of information on prenormative research projects being carried out at present in the individual member states, CEB, CIB, ECCS, JCSS etc with the aim of avoiding repetition and pooling expertise; and
- to consider research needs and to develop proposals for research projects on topics that will aid the further development of ENV 1991-1.

Topics that will fall within the scope of the proposed network are
- durability and life cycle aspects, and designing and constructing to achieve durability;
- serviceability requirements
- background of partial factor and load combination rules;
- development of an alternative probabilistic model code;
- basis of design for the assessment of existing structures, repair and maintenance decisions;
- risk analysis procedures;
- advanced non-linear and dynamic structural analysis and code checking;
- the behaviour of structural systems and design for robustness; and
- supervision, inspection and quality control.
- soil structure interaction

4 Conclusions

It is intended to commence the work for the transposition of ENV 1991-1 from ENV into an EN in the near future. The TC 250 Ad-Hoc Group on Basis of Design has produced a recommended Action Plan assuming voting for the transposition takes place in late 1998.

Appendix A

The CEN/TC250 Project Team that produced ENV 1991-1 was, G Breitschaft (Convenor); H Gulvanessian; N Krebs Overson; J C Leray, R S Narayanan; L Ostlund; G Sedlacek; and T Vrouwenvelder

The CEN/TC 250 Interim Ad-Hoc Group, Basis of Design was, H Gulvanessian (Convenor); J A Calgaro, J Grumberg; T Hagberg; P Luchinger; and P Spehl.

References

[1] ENV 1991-1: Eurocode 1: Basis of Design and Actions on Structures: Part 1: Basis of Design. CEN 1994.

[2] Vrouwenvelder, A., Eurocode 1, Basis of Design, Background Information, IABSE Conference on Basis of Design and Actions on Structures, Delft, IABSE, 1996.

[3] Gulvanessian H and Holicky M. Handbook for Eurocode 1: Basis of Design. To be published by Thomas Telford Publications, 1996.

[4] UK National Application Document for ENV 1991:1. To be published by BSI, 1996.

[5] Ostlund L. Actions. IABSE Conference on Basis of Design and Actions on Structures, Delft, IABSE 1996

Eurocode 1, Basis of Design, Background Information

Ton VROUWENVELDER
Delft University of Technology
TNO BOUW
The Netherlands

Ton Vrouwenvelder was born in 1947 in The Netherlands and graduated as a Civil Engineer from Delft University of Technology. He has become a specialist in the fields of Structural Mechanics and Structural Reliability. In his present position he is deputy head of the Structural Division of TNO Bouw and part-time professor at Delft University, Department of Civil Engineering

Summary

This paper gives a review of the background philosophy of Eurocode 1, Basis of Design. The main ideas behind the various parts of the document are discussed. The emphasize is put on the reliability aspects and the partial factor method. Recommendations for future developments are given.

1. Introduction

Eurocode 1, Basis of Design [1], like most present day codes, is formulated in terms of the Limit State Approach and the Partial Factor Method. The main objectives of the code are to prevent the structure, with appropriate degrees of reliability, to reach states of collapse or inserviceability during its intended life time. In this paper we will address a number of the key words mentioned in this introduction. For more extended background information the reader is referred to [2] and [3]. A general introduction can be found in [4].

2. The limit state approach

The Eurocodes are based on the limit state approach. A limit state is the demarcation between desired and adverse states of the structure. Two main categories of limit states are distinguished: Ultimate limit states and Serviceability limit states.

Ultimate limit states are associated with collapse, either on the level of structural members or, in particular for accidental limit states, on the level of a structural system.

Serviceability limit states are associated with the usefulness of the structure. A further distinction is possible into reversible and irreversible limit states [5] [6]. In the first case the limit state is no longer exceeded when the actions are removed, as for instance may happen for large elastic deflections and excessive vibrations. For irreversible limit states the exceedance will remain even when the actions are removed, as is for instance the case for most cracks.

By definition, structural design is concerned with situations which may cause the exceedance of the limit states. In principle these situations may be classified as follows:

1. Extreme unfavourable combination of actions and structural properties,
2. Abnormal but foreseeable actions like collisions or explosions
3. Errors during design, execution or use
4. Influences that are not foreseen.

The first two categories are the situations that normally are considered in standard design. The usual procedure is to define a set of load cases and to verify that the limit state is not exceeded using design values for actions and resistance parameters. Guidance for this procedure can be found in Basis of Design and the other Eurocodes. In some cases, however, if the calculations are extremely difficult or if the theory is not well developed, the calculations may be replaced by empirical "deemed to satisfy rules". Examples are explosions for ordinary buildings or some durability effects.

The last two categories of hazardous situations are much more difficult to tackle. The possibility of errors should be reduced by a proper quality assurance system. The details of such a system are outside the scope of Basis of Design. Unforeseen actions require some kind of robustness of the structure. This vague requirement may be relieved for cases when in the normal design all kinds of hazard scenarios are taken into account explicitly.

3. Reliability aspects

According to Eurocode 1, Basis of Design, Section 2.1 limit states requirements "*shall be fulfilled with appropriate degrees of reliability*". This means that the degree of reliability should be adopted to suit the type and use of the structure as well as the design situation under consideration. The choice of the level of reliability should take into account both the possible consequences of failure and the amount of expense and effort required to reduce the failure probability.

Obvious examples of reliability differentiation are the distinction between the ultimate and serviceability limit states or between failure modes with warning and without warning. The statement in Basis of Design may also mean that one accepts other reliability levels for live load as for accidental and seismic actions or that the failure probability for an agricultural building is higher than for a large span bridge. Section 2.2 of the present version of Basis of Design offers these possibilities in principle. However, the idea is not elaborated and it is not clear where it has been applied in the Eurocodes or where should have been applied.

Once a level of reliability has been specified it may be achieved in various ways:

-prevention of hazards
-protection against hazards,
-ductile and redundant structural system behaviour
-structural strength on a member level

The Eurocodes deal primarily with the structural strength on a member level. Nonstructural measures and structural system behaviour are seldom considered in conjunction. According to Basis of Design, Annex A, the standard reliability corresponds formally to a life time reliability index ß = 3.8 for single mode, single member failure. This corresponds to a life time failure probability of 0.0001. For irreversible serviceability limit the target index ß = 1.5, corresponding to an exceedance probability of 0.07. These values have been established on the basis of example calculations in various countries. Human errors and unforeseen actions were not taken into account. The consequences of these numbers will be considered in section 5.

4. Working life

The working life is an important design parameter. It makes quite a difference whether a structure is meant for 100 years or only for a few months. Fatigue, corrosion and other deterioration mechanisms may play a role only in the long term. Further, in the case of purely economical optimisation, it can be proven that the design values of variable actions may depend on the intended working life. This aspect is fully neglected in the present version of Basis of Design. It should be noted that the dependence of the design loads on the intended working life time is no longer valid when human safety aspects are dominant.

In general, present knowledge is insufficient to enable a sharp prediction of the structural condition during its design working life. The behaviour of materials and structures over extended periods of time can only be estimated roughly. The best option to deal with this uncertainty is to design in such a way that inspection and repair can be carried out. Lack of sufficient durability may then be conceived as a serviceability problem. Consequently one may adopt the corresponding lower reliability levels. If, however, an important structural member cannot be inspected, the effects of deterioration should be included in the Ultimate Limit State analysis [7]. In EC3, Part 2, the fatigue rules have been based in these ideas.

5. The Partial factor method

According to the partial factor method, design values for loads and resistance's are determined via respectively:

$$X_d = \gamma_F \Psi_i X_k \quad \text{and} \quad X_d = X_k / \gamma_m$$

X_d = design value
X_k = characteristic value
γ = partial factor (if present)
Ψ_i = reduction factor for variable actions (if present; i may be 0, 1 or 2)

In the case of the resistance, X may be a single variable or the resistance of a whole element. The characteristic value is normally aimed at the 5 percent lower (and sometimes upper) fractile.

Characteristic values of permanent actions are equal to the mean value. If the variability is large, upper and lower fractiles may be defined. For variable actions one usually takes values with a return period equal to a reference period of 50 years. The ψ values are intended to give reductions for the design values of variable actions to be used in the various ULS and SLS load combinations. For more information see [2], [3] and [8].

According to the theory of structural reliability [9] [10], design values should follow from

$$P(X < X_d) = \Phi(-\alpha\beta)$$

Φ = distribution function of the standardised normal distribution
α = probabilistic influence factor
β = reliability index.

As an example, for a Gaussian distribution this leads to:

$$X_d = \mu(X) - \alpha\,\beta\,\sigma(X)$$

Similar expressions exist for other distributions. Once the design value has been established, the partial factor and ψ factors may follow from $\gamma\,\psi = X_d / X_k$ for loads and $\gamma = X_k / X_d$ for the resistance.

The mean μ and standard deviation σ in the expression for X_d should follow, as far as possible, from experiments and field observations. This seems to be the only rational background for structural design decisions. Unfortunately, in most cases only a limited and not fully adequate set of data is available. In those cases data need to be supplemented by engineering judgement.

In the absence of a more detailed risk study, the value $\beta = 3.8$, mentioned earlier, may be used. According to ISO-2394 the values of α may follow from Table 1. These values are also presented in EC1, Basis of Design, Annex A. Of course one might take all α-values equal to 1.0, but that is extremely conservative. In Table 1 a distinction is made between load and resistance parameters and between dominant and non other variables. In the case of combining time variant loads the above theory is not enough. One needs a more refined analysis on the basis of stochastic processes.

Table 1: Influence coefficients α for design values [9]

	dominant	others
resistance parameters	$\alpha = +0.8$	$\alpha = +0.32$
action parameters	$\alpha = -0.7$	$\alpha = -0.28$

As a simple example, consider a building element having a coefficient of variation σ/μ = 0.10. If we may assume a normal distribution, the design value follows from:

$$X_d = \mu(X) - 0.8 * 3.8 * 0.10 * \mu(X) = 0.70\ \mu(X)$$

The 5 percent characteristic value is equal to:

$$X_k = \mu(X) - 1.64\,\sigma(X) = 0.84\,\mu(X)$$

This leads to $\gamma_m = X_k / X_d = 0.84 / 0.70 = 1.20$.

In Eurocode 1 the values of γ_F and Ψ_0, however, have not been determined according to the above theory. Most values have been found by (rough) calibration to earlier design methods that have proved to be successful, e.g. the Allowable Stress Methods. Theoretical studies [2] give the impression that the γ values presented in Basis of Design are conservative as far as permanent loads and live loads are concerned. The partial factors for wind and snow on the other hand, are relatively low: theoretical calculations indicate values above 2.0 where Basis of Design prescribes 1.5. Another way of looking at these results is that the target $\beta = 3.8$ is not met for structures where wind or snow is dominant. From the economical point of view this might be an acceptable safety differentiation. Finally the Ψ_0 factors are in general conservative. A detailed discussion is presented in [2].

6. Load combinations according to Eurocode 1, Basis of Design

According to EC1, Basis of Design, Table 9.1 one has to check the Ultimate Limit state for the persistent/transient design situation and for the accidental and seismic design situations. For each design situation the table gives the design values to be used for the permanent load, the variable loads and the accidental actions. As far as the variable loads are concerned there is a subdivision into "dominant" and "other" actions. As each action may be dominant for some design aspect, each variable load should be considered in turn as the dominant action. As an example, for the persistent/transient design situation we have:

permanent load:	$G_d = \gamma_G \; G_k$
prestressing:	$P_d = \gamma_P \; P_k$
dominant variable load:	$Q_d = \gamma_Q \, Q_k$
other variable loads:	$Q_d = \gamma_Q \, \psi_0 \, Q_k$
accidental loads:	$A_d = 0$

In a similar way, according to Table 9.4, one has to check the reversible and irreversible Serviceability Limit State by means of the frequent and characteristic combinations respectively. The two combinations typically have different reliability levels. The quasi permanent combination is intended for long term effects.

The γ and ψ values necessary to determine the design values for the loads, can be found in the Tables 9.2 and 9.4. Especially Table 9.2 has been the subject of many lively discussions. Note that Table 9.2 is only relevant for the Ultimate Limit State as for the Serviceability Limit states all γ values have been put equal to 1.0. Note further that Table 9.2 is intended for buildings and silos only, and not, for instance, for bridges. The information for other structures will be implemented later.

In the Table a distinction is made between the cases A, B and C.

<u>Case A</u> deals with the check for *static equilibrium*. The essential characteristic of this verification is that no strength properties of either building elements or soil is involved. Examples are cantilevering beams if supports cannot take tensile forces (see Figure 2, upper case) and underwater structures like tunnels or docks. The analysis of these limit states require a more detailed and subtle analysis than the usual case with failure. The point is that a part or the total of the structural weight in those cases acts as resistance. So it is important to sort out which part acts as a resistance and which part acts as a load. The

partial factors in these cases should reflect the fact that only the relative differences between these parts are of importance, and not the overall deviations.

Case B is the "normal case" where the *strength of a structural element or cross section* is verified for a certain load case. Note that according to the footnote 3 in Table 9.2 no distinction is made between favourable and unfavourable parts of the permanent loads (e.g. the self weight) as long as it belongs to one source. So in the case of Figure 1, when designing the field AB, it is not necessary to distinguish between the favourable effect of part BC and the unfavourable effect of part AB. One simply takes 1.35 (or 0.9) for both parts.

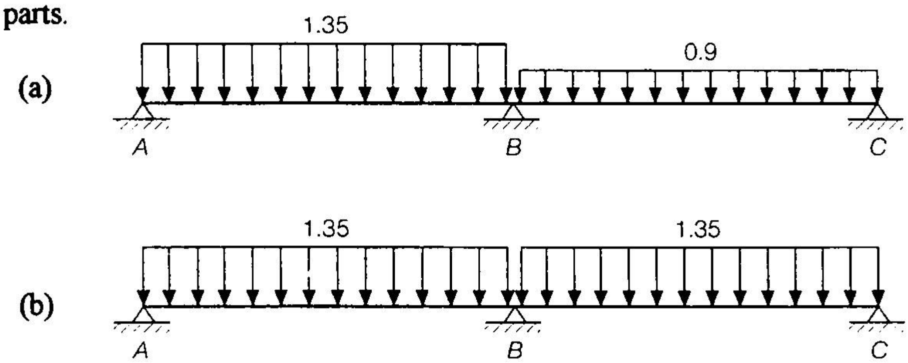


Figure 1: Factors to be used in case B for the permanent loads.
(a) permanent loads from two sources; (b) permanent load from one source

Basis of Design, however, gives no guidance on what should be regarded as "one source" and what should be regarded as two sources. One source obviously is:

- self weight from one material
- ground water pressures from ground water in one soil layer
- water pressure from water in one hydraulic system

What, on the other side to do, if the load effect results from favourable and unfavourable contributions of different building materials. Hans Denver from DGI asked this question to a number of European Colleagues. The differences of opinion were astonishing.

Another important issue in case B is the choice of the partial factor for the unfavourable permanent loads in combination with extreme other permanent and variable loads. In the present version this factor equals 1.35. For some countries this requirement was found to be too strong and a note to equation (9.10) was added. According to this note one may reduce the factor for permanent loads by $\xi = 0.85$ if one of the variable loads is dominant. As a simple example consider the case of one permanent and one variable load, both unfavourable. According to the standard text of Basis of Design one should check for:

$$1.35\, G_k + 1.5\; Q_k$$

If we include the note attached to (9.10) we have:

$$1.35\, G_k + 1.5\, \Psi_0 Q_k \quad \text{and} \quad 1.15\, G_k + 1.5\, Q_k$$

It turns out that for most structures the effective reduction is about 10 %. Note that the factor ξ may be interpreted as a Ψ_0 value for permanent loads.

Case C finally, is relevant if failure in the *soil* plays a role. The difficulty in soil is that it may contribute to the load as well as to the resistance, but in an even more complicated way than the self weight in case A. In addition, soil may have a relatively high degree of uncertainty. According to the theory behind the Partial Factor Method one should then increase the partial factors for soil (make them "dominant", see Table 1) and reduce them on the loading side (make them all "others"). This is what effectively has been done in Table 9.2. However, proper backing from reliability calculations is lacking at the moment.

Note

The distinction between the cases A, B and C may lead to some difficulties in practice. For instance, consider the case of the cantilever beam (Figure 1). In case A one should use a factor 1.10 for the cantilever part and a factor 0.9 for the resisting part. This may lead to the conclusion that the structure is not sufficiently stable and an anchoring device is required. However, if one makes a verification for the strength of the anchor according to case B, one should use the same factor of 1.35 for both parts of the structure. As a result, an anchor of zero strength would be sufficient. This of course is a contradiction. Similar problems may exist between the cases B and C. A possible solution could be to consider cases A and C also in the verification of structural strength. This item needs definitely some more attention before the code is transformed to an EN.

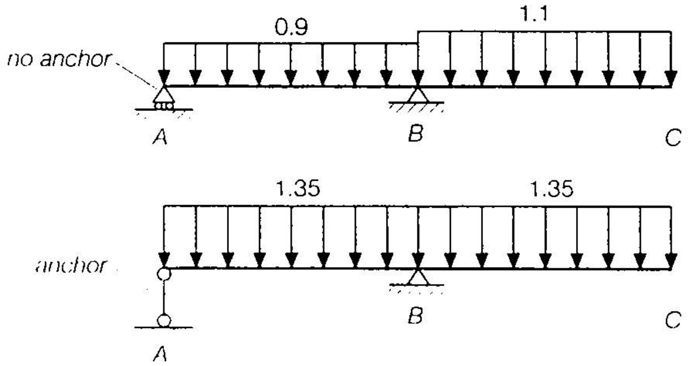


Figure 2: Cantilevered beam with and without anchor at A.

7. Conclusions

The present version of Eurocode 1, Basis of Design, has achieved an important goal. The basic principles of structural design have been harmonised for a large number of countries and (maybe even more important) for a large number of materials (concrete, steel, masonry, timber, aluminium) and disciplines (fire, geotechnics, earth quake, bridge design).

Of course, the harmonisation is not perfect:

- In many cases there still is a lack of uniformity between the various disciplines which cannot be justified from any theoretical point of view.
- Some important items have not been touched, for instance: non-linear calculations, geometrical imperfections, serviceability limit state requirements, durability and working life, reliability differentiation, relation with nonstructural safety measures, influence of working life on design loads, fatigue design, etc.
- Some load cases need to be further developed.

- The Load Combinations and Partial Factors for other structures than buildings (bridges, towers, agriculture structures) should be included.
- Numerical values are, in many cases, more the result of a compromise than of rational and scientific thinking on the basis of experiments and observations. The background of many numbers cannot be properly justified in technical terms, they are an unclear mixture of engineering judgement and experience.

This means that there is still work to be done, first for the transposition to EN and also later. This is especially of importance as also the building industry is not a static one. New materials will be put on the market and new types of buildings will be designed. This means that is will be more and more difficult to rely on intuition and experience built up in the past. In the years to come Basis of Design has to move into a more rational ad professional direction.

References

[1] ENV 1991-1: Eurocode 1: Basis of Design and Actions on Structures: Part 1: Basis of Design. CEN 1994.

[2] JCSS Working Document, Basis of Design Background Documentation, ECCS Publication no 94, Brussels, March 1996.

[3] Gulvanessian, H. and Holicky, M., Handbook for Eurocode 1: Basis of Design. To be published by Thomas Telford Publications, 1996.

[4] Gulvanessian, H., ENV 1991-1: Eurocode 1: Part 1: Basis of Design, Introduction, Development and Research Needs, IABSE Conference on Basis of Design and Actions on Structures, Delft, IABSE, 1996.

[5] Lüchinger, P., Basis of Design Serviceability Aspects. IABSE Conference on Basis of Design and Actions on Structures, Delft, IABSE, 1996.

[6] Ostlund L, Structural Performance Criteria, JCSS Working Document, Published by IABSE, 1991.

[7] Siemes, T. and Rostam, S., Durable safety and serviceability - a performance based design format. IABSE Conference on Basis of Design and Actions on Structures, Delft, IABSE, 1996.

[8] Ostlund, L., Actions. IABSE Conference on Basis of Design and Actions on Structures, Delft, IABSE, 1996.

[9] ISO, General Principles on Reliability for Structures, IS-2394, 1986.

[10] Madsen H O, Krenk S and Lind N C, Methods of Structural Safety, Prentice Hall, 1996.

Basis of Design
Serviceability Aspects

Paul LüCHINGER
Dr. sc. techn., Civil Engineer
Dr. Lüchinger Partners
Zürich, Switzerland

Paul Lüchinger, born 1943, is a graduated of the Federal Institute of Technology Zürich, Switzerland. For several years he undertook research and developement in structural engineering. Since a long time, he is a consulting engineer in responsible position, and is the Vice-Chairman of the CEN Committee TC 250/SC 1.

Summary

A structure is expected to remain fit for the use during the intended working life and to sustain all actions and influences likely to occur with an appropriate reliability and in an economic way. The client focusses his attention on the requirements relating to serviceability. The serviceability aspects concern the funtionality and appearance of the structure as well as the comfort of user which are verified with criterias such as deformations, vibrations, stress or crack control. Considering the particular aspect of serviceability the criterias should be defined individually and recorded in the contract. Structural design codes may recommend indicative values.

1. Fundamental Requirements

Fundamental requirements have been identified and generally accepted which structures have to meet. According to these fundamental requirements a structure shall be designed and executed in such a way that it will remain fit for use for which it is required and sustain all actions and influences likely to occur during its intended working life with appropriate degrees of reliability and in an economic way (figure 1).

Several preventive measures in design and execution process are appropriate to meet the above requirements. Design procedures according to these two fundamental requirements implies that due regard is given to structural safety and serviceability, including durability, in both cases.

Adequate safety with respect to a hazard is ensured provided that the hazard is kept under control by appropriate measures or the risk is limited to an acceptable value. By the way, absolute safety is not achievable. Ultimate limit states are those associated with collapse, or with other forms of structural failure which may endanger the safety of people.

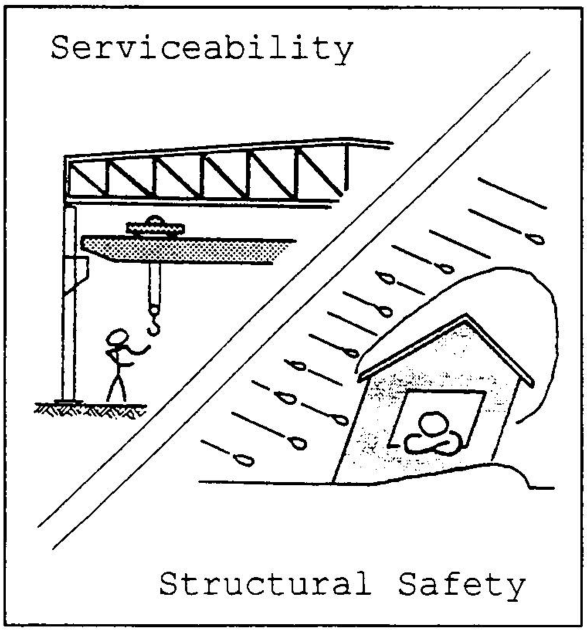


Fig. 1. Fundamental requirements: Serviceability and Structural Safety.

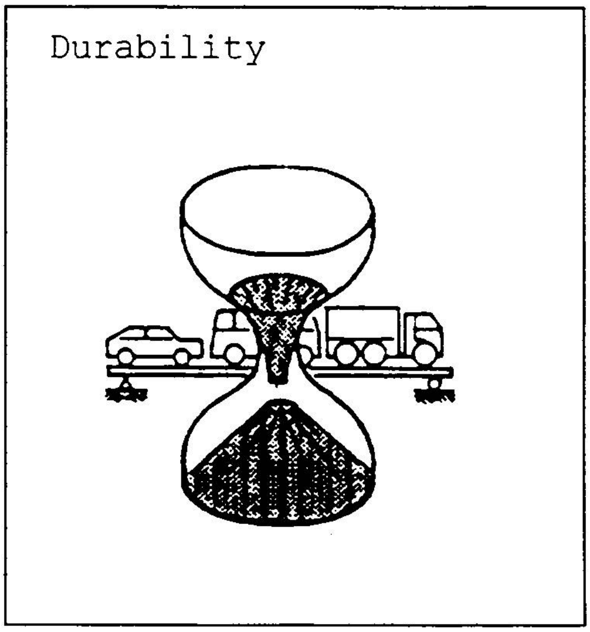


Fig. 3 Durability of Structures is related to its intended Working Life.

Serviceability for the intended use is obtained if the structure behaves within fixed limits. The requirements concerning serviceability aspects (figure 2) are related to:

- the funtion of the structure or parts of it with regard to, for example, watertightness, building physics, boundary conditions of non-loadbearing elements, building services or equipment,
- the comfort of user,
- the appearance of the structure, where, for example, the presence of water, cracking or deflection may be deemed unsatisfactory.

Service states which constitute a hazard for the structure, for example resonance or loss of resistance due to corrosion or fatigue, shall be included in the safety considerations.

Durability relates to the prevention of detoriation of material under the condition of planned inspections and adequate maintenance.The durability of a structure in its environment shall be such that it fullfills its funtion during the design working life (figure 3).

Construction is not an end in itself. A new project for a construction work starts always on the initial impulse of the client. It lies in his responsibility to give a clear vision and a definition of the project and to identify the purpose and the intended use of a structure. The service requirements may be influenced by economic considerations relating to the costs of construction, inspection, and maintenance. In consideration of the particular characteristics of the service requirements the serviceability aspects are clearly situated in the centre of interest of the client.

In consequence of these considerations the service requirements have to be determined in the contract as well as the design working life, during which the proper function of the structure is expected to be ensured. The service requirements are based on the consensus of the client with

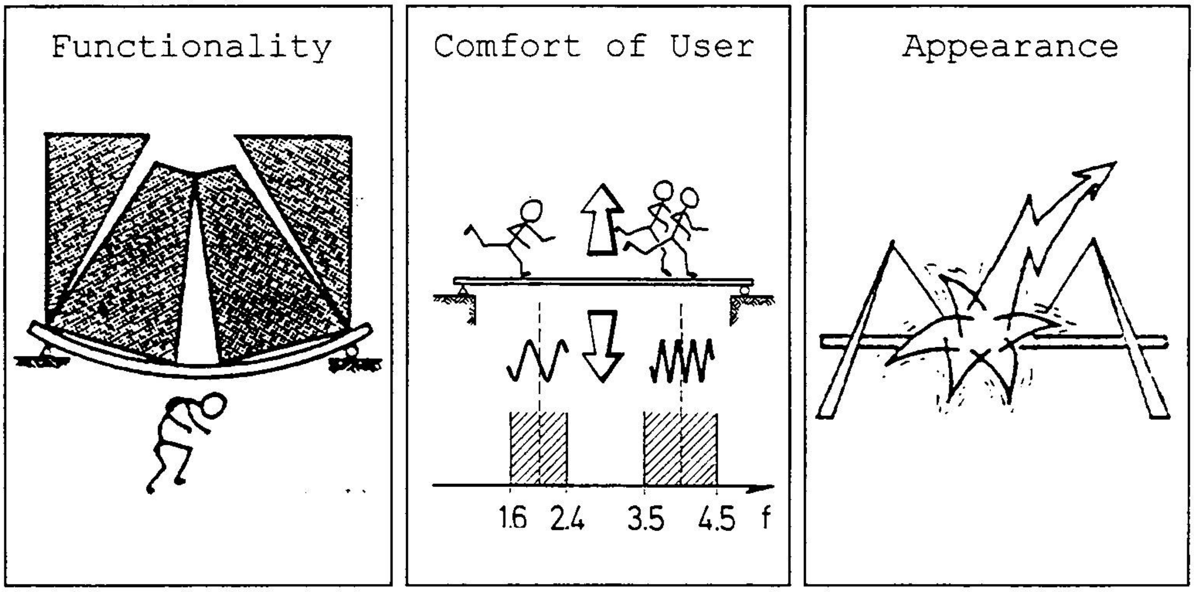


Fig. 2. Serviceability Aspects: Funtionality, Comfort of User, and Appearance.

the designer. The design codes of practice may only give indicative instructions and recommendations.

In contrast to the serviceability the structural safety is related to risk of life. The level of reliability in relation of structural safety concerns the public interest and is normally detremined by acceptance at national level.

2. Preventive Measures

The conditions of use are a matter of convention. The fundamental requirements related to the functionality, the comfort, or appearance are to be specified in advance. The required reliability relating to serviceability may be achieved by one or a suitable combination of different measures. The choice of suitable materials, the appropriate design and detailing are as important as the clearly specified conditions of use.

An example of a concrete structure shall clarify the choice of different measures to meet the requirements with regard to functionality. The clarification basins of a sewage plant with a length of approximately 100 m which are situated in a field where the ground water is used for the water supply of a town are expected to be watertight for understandable reasons. The following measures have been taken to fullfill the required good function of the structure:

- The conditions of use and in consequence the actions to be considered, e.g. the relevant level of ground water, have been verified and specified in the beginning of the design process.
- The basins are designed without any dilatation joint along the whole length. A special sliding layer beyond the foundation minimizes the ground friction.
- The concrete mix is designed with special respect to low shrinkage deformations of the structure.

- The reinforcement and prestressing are designed and detailed in such a way, that the crack pattern due to imposed deformations, for example due to temperature variation, does not reduce the function of the basins.
- The basins are executed in especially arranged stages in order to minimize the differential shrinkage between the different structural components.

The combination of all measures ensured the projected behaviour under the predicted conditions of use. In addition the durability of the structure was improved by means of the above measures.

The specifications of the technical requirements represent the first step of quality assurance. Clearly written and unambiguous specifications indicate the level of quality requirements to all participants involved in the design and execution process. In the second step procedures for design, production, execution which are under control and adjusted to the particular aspects of every project, guarantee that the requirements are successfully met.

Analysis of many damages observed in practice clearly demonstrate the fact ,that inadequate behaviour of structures under service conditions and a considerable number of damages to so called secondary structural elements are caused by insufficient transfer of informations and poor operational arrangements. These damages could have been avoided easily if the requirements and conditions would have been known to everybody. The continuity of the flow of information is asked. Hence, the measures related to organistaion and management are at least as important as the measures to be respected on the technical level, especially in consideration of the servicability limit states.

3. Design Concept

Apropriate design and detailing are two important of different possible measures to assure the fundamental requirements. The limit states design concept is generally appreciated and used in structural engineering practice. Limit states are states beyond which the structure no longer satisfies the design performance requirements. In the limit state design concept the design procedure follows a red line on a path with several steps. The various steps are indicated in table 1.

Both kinds of verification, the verification of ultimate limit states as well as the verification of serviceability limit states, are based on the same design concept and follows comparable design steps

4. Design Situations and Effects of Actions

In accordance with 'Basis of Design' the various design situations shall be considered and critical load cases identified. Seviceability aspects are mostly related to persistant design situations which refer by definition to the conditions of normal use. However, under certain circumstances serviceability requirements may also to be met under accidental situations. For example, it is highest interest that a hospital is not so heavily damaged under well defined seismic actions to get lost of its funtion as a part of a life line.

The serviceability requirements are related to conditions of use which may represent, in consideration of the nature of the specified requirement, a quasi-permanent, frequent, or rare situation. The relevant types of design situations and load cases have to be defined individually for each structure under consideration of the specified conditions of use.

The experiance shows, that most requirements concerning the funtionality of the structure such as with regard to boundary conditions of non-loadbearing elements, finishes, building services, equipment, or building phisics are related to quasi-permanent design situations and load cases.

	Structural safety:	**Serviceability:**
1.	*Identification of design situations*	
	Hazards	**Conditions of use**
	• persistant situations • transient situations • accidental situations • seismic situations	• persistant situations
2.	*Specification of requirements*	
	• **Resistance** • **Stability** • **Rotation Capacity**	• **Function of structure** • **Comfort of user** • **Appearance of structure**
3.	*Choice of measures*	
	• **Definition of the structural concept** • **Choice of suitable materials** • **Appropriate design and detailing** • **Professionel execution in accordance with project** • **Specified quality management**	
4.	*Specification of performance criterias*	
	• **Strength** • **Strain Rates, etc.**	• **Deformations,** • **Vibrations** • **Stress Limitations, etc.**
5.	*Identification of design situations, load cases and load arrangements*	
	Ultimate Limit States	**Serciceability Limit States**
6.	*Determination action effects*	
	E_d	$E_{d,ser}$
7.	*Verifications of performance criterias under a given action effect*	
	$S_d (E_d) \leq R_d$	$C_d (E_{d,ser}) \leq C_{lim}$

Tab. 1. Steps in Limit States Design Procedure

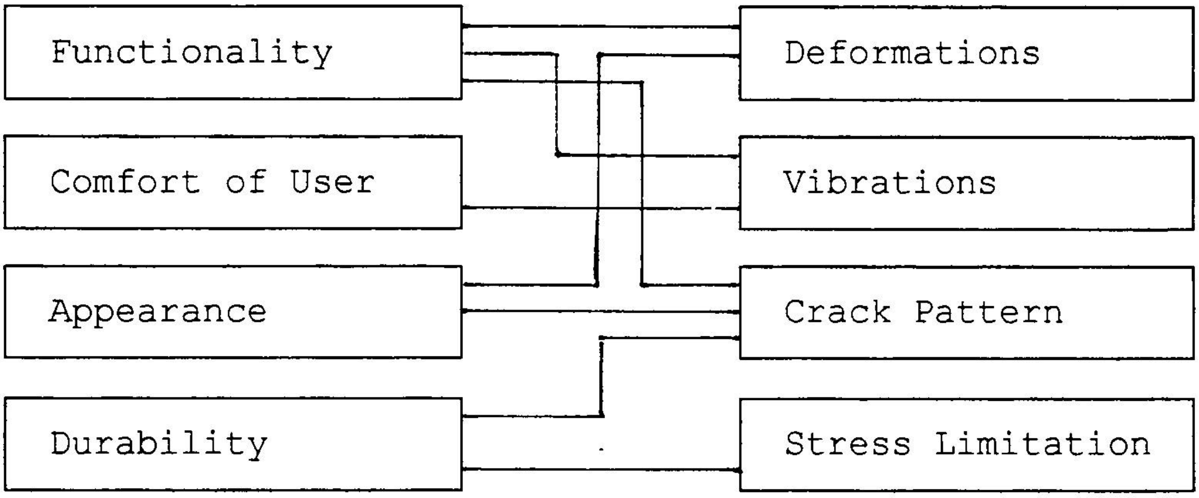


Fig. 4. Relation of Structural Requirements and Performance Criterias.

Again the serviceability requirements with respect to the appearance of the structure are usually to be considered under quasi-permanent design situations and load cases.

The comfort of user, however, is mainly touched under frequent actions during normal use.

A local damage to the structure itself or to a non loadbearing element may reduce the durability or influence the appearance in a unsatisfactory manner. In such a case the distinction between reversible and irreversible serviceability limit states has to be made. Irreversible serviceability limit states might be linked to rare load cases.

In each critical load case identified for the verification of a specific serviceability limit state only compatible load arrangements, sets of deformations, and imperfections which may occur simultaneously have to be considered. In addition only those portions of actions have to be taken into account which are relevant for the verification under consideration. However long-term structural behaviour effects have to be taken into account, such as creep, relaxation, and shrinkage effects.

5. Performance Criterias

Usually, the fundamental requirements, such as funtionality of the structure, comfort of user or appearance, can not be verified directly. Rather, performance criterias are identified as representative for the structural behaviour by means of which the requirements are controlled. Serviceability limit states which may require consideration include:

- Deformations and displacements,
- Vibrations,
- Crack pattern,.
- Stress limitations.

Whereas the deformations, displacements, and vibrations may be determined in the design process, the crack pattern is measured in crack width and crack spacing. To avoid damage including permanent or accumulative deformations, which may influence the effective function of the structure adversely, the maximum stresses or stress ranges are limited.

The performance criterias have to be well assigned to the various requirements (figure 4). Deformations and displacements may affect the appearance of the structure as well as the effective function including the functioning of equipment and services and including damage to non-loadbearing elements or finishes.

In special cases excessive deformations may increase the loads acting on a structure. For example, if water is retained on a roof due to its deformations, the load effect of water increases. Such bumping effects have to be considered under the aspect of structural safety.

Vibrations may cause discomfort of the user or may cause damage to the structure or the materials which it supports. Excessive dynamic effects in structures succeptible to vibrations have to be related to ultimate limit states.

Large crack widths and an unsatisfactory crack pattern may disturb the appearance of the the structure and, hence, irritate the user. In addition the effective funtion are no longer ensured with unacceptable crack width.

6. Verifications

The distinction of the various fundamental reqirements with respect to serviceability limit states asks for a comprehensive analysis of the relevant design situations including the identification of the load cases in due relation to and consistant with the specified requirements such as funtionality, comfort of user, and appearance.

The serviceability is verified in comparing the calculated actual performance criterias under consideration with limiting values (table 1). As an example, when considering a limit state of serviceability the requirement of which is controlled by deformations it shall be verified that:

$$w_d(E_{d,ser}) \leq w_{lim}$$

where:

$w_d(E_{d,ser})$ is the deflection due to the design value of the action effects under the considered conditions of use and the relevant load cases.

w_{lim} is the limiting value of the deflection for the specified requirement.

All the different parts of the deflection are to be taken into account if necessary in accordance with the specified requirement. A deflection may be expressed as the sum of the following parts (figure 5):

$$w = w_1 + w_2 + w_3 + w_4$$

where:

w_1 is the camber,e.g. the form of structural steelwork as fabricated or the camber of concrete structures obtained by falsework and formwork

w_2 is the deflection under the permanent actions including the long-term deflection.

w_3 is the deflection due to the quasi-permanent value of a variable action including the corresponding long-term deflection.

w_4 is the deflection due to the short-term (frequent or characteristic) value of one variable action.

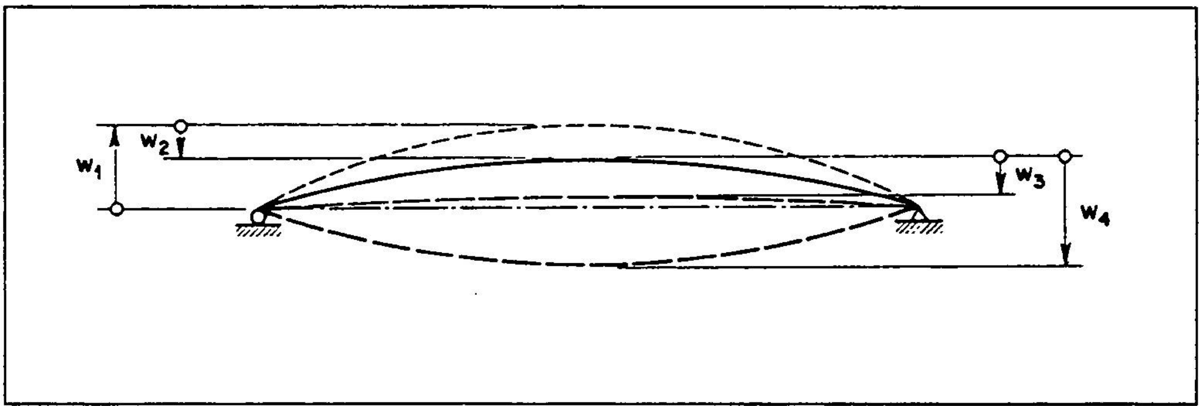


Fig. 5. The different parts of the deflection of a beam.

The verification of the other performance criterias such as vibrations or stresses are applied by analogy.

Regarding the strict correspondance between the conditions of use on one side and the requirements on the other side the action effects, the load cases, and the limiting values of the performance criterias are to be obtained in an similar consistancy. For example, the limitation of the deflection of a floor slab may end up on a different level, if the requirement takes into consideration the function of finishes with brittle behaviour or if the requirement concerns the comfort of user.

Since the requirements and conditions of use are determined in agreement between the client and the designer, the analysis of the design situations and load cases as well as the determination of the limits of the relevant performance criterias have to be considered individually in each design process. The structural design codes can only give indicative values and recommandations of general interest, unless otherwise specified.

Durable safety and serviceability - a performance based design format

Ton SIEMES
Civil Engineer
TNO Building and Construction Research
Delft
Netherlands

Ton Siemes, born in 1947, got his civil engineering degree from Delft University in 1973. Shortly after that he joined TNO, where he is working on durability problems of building materials and structures.

Steen ROSTAM
Civil Engineer
COWI
Copenhagen
Denmark

Steen Rostam, born in 1943, got his civil engineering degree in 1969 and his PhD degree in 1977 from the Technical University of Denmark. In 1978 he became associate professor. Now he is head of the department on Assessment and Rehabilitation from COWI. Since 1978 he has chaired the CEB-activities on durability.

Summary

The paper gives a framework for a performance based procedure for the design of structures for durability. This procedure is in principle a modification of the structural limit state design. This implies that the performances are given as limit state functions and expressed in terms of reliability. At this moment several activities have been started for the further development of the performance based durability design. It is expected that within a few years the first results will be available for the building industry.

1. Present durability design approach

A lack of durability can cause serious safety and serviceability problems for structures. Despite this designers have at this moment, considerably more attention for load and resistance based structural design than for durability design. The recent history has however shown that due to a lack of durability serious collapses and other types of damages can occur with tremendous amounts of damages. Some examples of them are:

- The Ynys-y-Gwas bridge in Wales (UK) collapsed [1] on 4 December 1984. This concrete bridge was a single, 18 meters span segmental post-tensioned structure. The cause of the collapse was serious corrosion of the post-tensioned tendons.
- In Germany the outer roof of the Berlin Congress Hall collapsed on 21 May 1980 due to hydrogen-induced stress corrosion [2]. One person died. Another was badly injured.

- In 1990 a reinforced concrete gallery in Wormerveer (NL) collapsed. This was caused by chloride induced corrosion in a crack that was caused by poor construction.
- In Melle (B) a prestressed concrete bridge collapsed during the passage of a truck. The driver died. The cause was a crack that opened during the passage of high loads. Chloride could penetrate through this crack and reach the post tensioned tendons.
- In Uster (CH) the roof of a swimming pool collapsed in 1985 [3]. The roof was made of stainless steel that was supposed to be resistant to the present damp, chloride contaminated atmosphere and the high temperatures. Nevertheless the steel corroded and 12 people were killed.
- Further it can be mentioned that woodrot in foundation piles have caused serious damages in the buildings that they carry.

This list of accidents with structures makes clear that durability should have serious attention from structural engineers during design and construction. Both the ultimate and the serviceability limit state must be analyzed.

The design approach with respect to durability of structures is in the existing building codes to a large extent empirical. It is mainly based on deem-to-satisfy rules. For instance for concrete these rules relate to the minimum concrete cover, the water-cement ratio or the maximum crack width. For steel rules have been given for the maintenance. For timber the various wood species have been classified in durability classes. If these rules are met, it is assumed that an acceptably long but unspecified lifetime will be achieved.
A major part of testing the durability of metallic and organic coatings is based on standard tests that do not represent the environment in which the building components will be exposed [4]. It is obvious that the result is a bad correlation between test results and the durability in practice.
The design rules in the present codes are not related to the performance of the structure. They are not yet formulated as limit states as is usual for structural designs.

The problem is not exclusively related to large structures. Metal anchors for façade plates, metal ties for cavity walls, steel angles for supporting the outer leaves of masonry cavity walls are relatively small structural components for which the safety is directly coupled to their durability. Indeed these are examples of serious safety and durability problems.
These examples show a main safety and durability problem in the building industry. The problem arises from the combination of:

- a long intended service life (say more then 50 years)
- failure will lead to exceeding an ultimate limit state for which high reliability indexes apply (Eurocode 2 requires in these cases β-values of 3.8)
- safety is to a large amount dependent on the durability
- inspection or monitoring is not usual or even impossible
- hardly any pre-warning before collapse.

In Figure 1 a sketch of a probability density function for the service life is given based on an intended service life of 50 years and a β-value of 3.8. The end of the service life is defined as the moment of collapse. The area under the curve between 0 and 50 years equals 10^{-4} whereas the whole area under the curve equals 1. Without further mathematical prove this implies that the structure must have a mean service life of some hundred years.

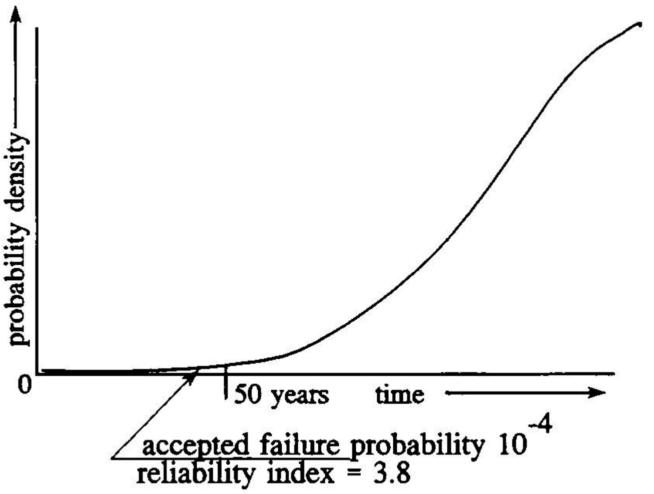


Fig. 1. Example of a probability density function of a service life for an intended service life of 50 years and a reliability index β-value = 3.8.

The performance based design that is illustrated in Figure 1, will provide the basis for an objective assessment of the durability. It must be based on realistic and quantified environmental and material models capable to predict the future behaviour of the structure. In this way it describes the performance in relation to time. This offers the following benefits:

- durability design can be based on the same principles as structural design (safety, serviceability, limit states and reliability)
- objective designs based on the total life cycle costs will be possible
- a reduction of the consumption of materials and energy by optimal use of the materials
- basis for design engineers, contractors and maintenance engineers to make tailor made designs, especially if the design is based on specific material properties
- realistic performance test procedures to establish the building material properties and the building component behaviour
- methodology can be extended to new materials and new applications; nowadays much attention is paid to a sustainable development and as a consequence waste materials or secondary materials will be used more and more.

2. Durability in Eurocode 1

In Eurocode 1: 'Basis of design and actions on structures' (ENV 1991-1, 1994) an attempt has been made to give fundamental requirements with respect to the durability of structures. It is stated that structures should be designed with appropriate levels of reliability for safety and serviceability, including durability. For this purpose a 'design working life' is defined, being the period for which the structure has to be used for its intended purpose with anticipated maintenance but without major repairs being necessary. For the design working life a classification is given varying from 1 to 5 years for temporary structures to 100 years for monumental buildings, bridges, and other civil engineering structures.

It is assumed in design that the durability of a structure or part of it in its environment is such that it remains fit for use during the design working life. The structure should be designed so that the deterioration does not impair the durability and performance of the structure. The environmental conditions have to be appraised at the design stage to assess their significance in relation to durability and to enable adequate provisions to be made for protection. The degree of deterioration may be estimated from calculations, experimental investigations, experience from earlier structures, or a combination of these considerations.

Although it is important that in Eurocode 1 attention is paid to durability the practical significance is still restricted. The main reason for that is the lack of a strict design format and of objective requirements. In some material related Eurocodes more elaborated requirements are given. These are however implicit and not related to performances or to the fundamental durability requirements in Eurocode 1.

All necessary ingredients for a performance based durability design procedure are nevertheless present in Eurocode 1. These ingredients are the performance based structural design method, levels of reliability, and reference periods (design working life).
In the performance based structural design [5] both the resistance R and the load S are considered to be time independent. In many loading situations this is not realistic. The limit state function should then be rewritten as a time dependent limit state function:

$$R(t) - S(t) \geq 0 \qquad (1)$$

A special case for this limit state function occurs if either R or S is not time dependent. Relationship (1) applies for all t in the time interval (0,T). T is the reference period(intended reference period or design working life). Even if the loading or the capacity of a structure is time dependent,the limit state functions for designing structures are rarely formulated in this way (with an exception for fatigue). They are often simplified to time independent quantities.

Well-known simplifications are:

- assuming that the material strength during the service life period is either equal to the short term strength or to the long term strength
- for (semi) static loads one characteristic maximum value, related to the reference period, is defined
- for fluctuating loads (like wind, traffic or waves) one characteristic maximum fluctuation in the reference period is defined.

The durability design can be presented in two different, but theoretically equivalent, formats. These are the 'intended service period design' and the 'lifetime design'. In the intended service period design the condition is that the limit state may, with a certain reliability, not be reached within the intended service period. The format for the intended service life design is largely comparable with the format for the conventional structural design. In the lifetime design the reliability of the structure is related to the probability that the design lifetime will be exceeded. The lifetime ends at the moment that the limit state is exceeded.

The concept of the intended service period can be expressed in a design formula:

$$P_{f,T} = P\{R(t) - S(t) < 0\}_T \leq P_{target} = \Phi(-\beta) \qquad (2)$$

in which:

$P_{f,T}$ - the probability of failure of the structure within T
T - intended service period.
P_{target} - the accepted maximum value of the probability of failure
Φ - standard normal distribution function
β - reliability index (this value is normally given in codes instead of the failure probability)

Probably it will be possible to simplify this relationship in a later stage to a design format equivalent to the conventional design formulas.

Example 1:
A steel rod with a diameter of $ø$ = 20 mm and a tensile strength f_a = 500 N/mm^2 is loaded with an axial tensile force F =105 kN. Due to corrosion the diameter reduces with a speed s = 0.1 mm/year. In this case the design formula for the period t = 0 to T years is:

$$P_{f,T} = P\{R(t) - S(t) \leq O\}_T = P\{\tfrac{1}{4}\ \pi\ (ø - s\ t)^2 f_a\ - F \leq 0\}_T$$

$$= P\{\ [\tfrac{1}{4}\ \pi\ (20 - 0.1\ t)^2 - 150.10^3 \leq 0]\ \}_T \leq P_{target} = \Phi(-\beta)$$

In some structural codes indications have been given for values of T and P_{target}. For example in Eurocode 1 for the ultimate limit state (collapse): T = 50 years and $\beta = 3.8$ corresponding with $P_{target} = 7.10^{-5} \approx 10^{-4}$.

For the lifetime design relationship (1) must be transformed to a lifetime function. This can be done by writing it as an explicit function of the time:

$$L = t\{R,S\} \tag{3}$$

In which:
L - the lifetime of the structure.

The reliability of the structure can be introduced by limiting the probability of exceeding a target value:

$$P_f = P\{L \leq T\} \leq P_{target} = \Phi\ (-\beta) \tag{4}$$

A possible, more practical form for a design formula is:

$$t_d \geq t_{target} \tag{5}$$

In which:
t_d - design value of the lifetime
t_{target} - target lifetime.

Example 2:
For the same steel rod that was presented in example 1 the life time function is:

$$P_f = P\{\ L \leq T\ \} = P\{\varnothing - \sqrt{\{[\ F\ /\ ¼\ \pi\ f_a]\ /\ s\}} \leq T\}$$

$$= P\{\ [20 - \sqrt{\{[105000\ /\ ¼\ \pi\ 500\ ]\ /\ 0.1\ \}} < P_{target} = \Phi\ (-\beta)$$

In Figure 2 the durability design procedure is illustrated. It shows the similarities and differences between the service period design and the lifetime design. The illustration makes clear that for both approaches the same information is used. Consequently they will lead to exactly the same result. Moreover the lifetime distribution in Figure 2 shows that the margin between the mean service life $\mu(L)$ and the target value T can be very large. This margin depends on the type of the distribution, the scatter and the target failure probability (As the target failure probability is very small; it is almost impossible to draw the figure in the right scale!).

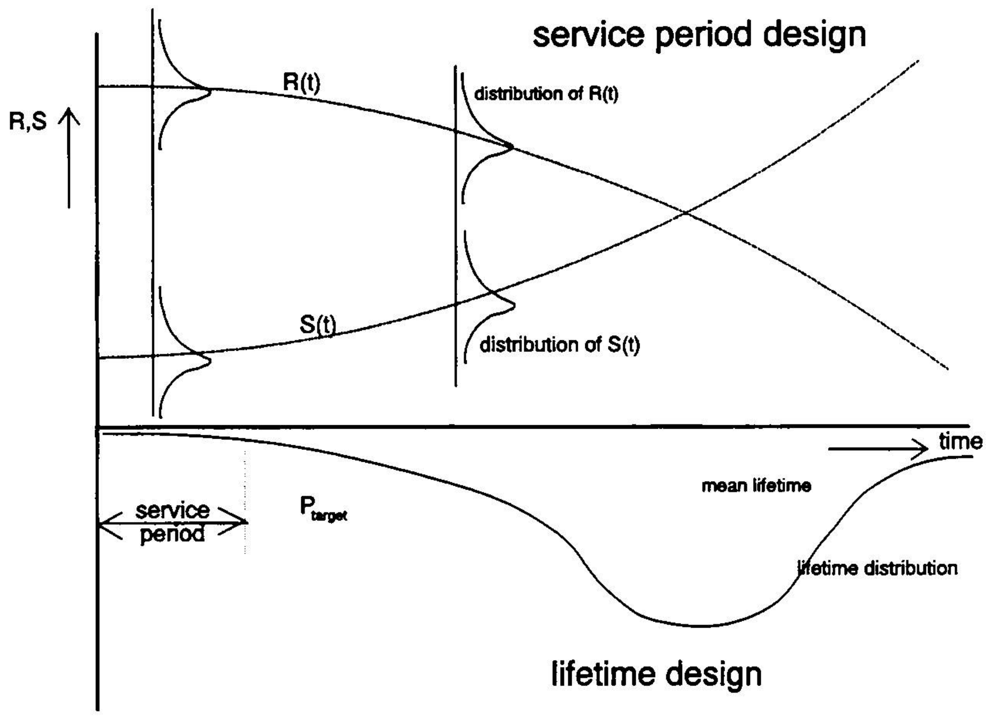


Fig. 2. Similarity between the service period design and the lifetime design

It is important to realise that both the intended service period design and the lifetime design are not necessarily restricted to the conventional ultimate and serviceability limit states. The processes (or mechanisms) involved can be of mechanical, physical, chemical, elector-chemical or biological nature. The limit states can refer to aspects such as structural safety, serviceability, functionality, comfort, aesthetics and so on. The accessory target reliability index depends on the type and the amount of damage.

In the limit state approach, as presented so far, the first exceedence of the limit state determines failure. Other types of criteria are however possible. Examples are: the number of exceedences of a limit state is restricted. Or: the duration of the exceedence of a limit state is restricted. The latter two examples may apply in reversible limit states. The two examples may be true for vibrations that cause discomfort. This paper deals for simplicity only with the conventional limit state approach. It may be expected that the extension in a later stage to other types of criteria will not meet fundamental problems.

The number of limit states (and their types of criteria) that have to be considered for one building is very large. That means there is a need for priorities and simplifications. In this paper no attention will be paid to that.

Some worked out examples of both concepts were presented some years ago in [6, 7, and 8]. These examples made it clear that the concepts can be used for various building materials, showing the generic character of these concepts. Rilem committee 130 CSL 'Calculation of the Service Life of Concrete' has prepared a report [8] that shows calculation examples for this approach for various degradations of concrete structures.

The conventional structural design is in practice restricted to performances with respect to safety and serviceability. This restriction is not necessary for the durability design (in fact there is neither a formal restriction in the conventional procedure). As long as it is possible to formulate a limit state, it will be possible to apply either the service period design or the lifetime design. It is obvious that the aimed reliability is related to the nature and the amount of damage that can be expected if the structure fails. If human lives are threatened or the economic losses are very high the acceptable failure probability must be very low. Higher values are acceptable for smaller damages. This principle is completely in line with the conventional approach for structural design. Possible performances for structures relate to:

- load bearing capacity (bending, axial, torsion, shear)
- stiffness (deflection, vibration)
- protection by coatings
- flat surface for walking, riding, driving or running
- fire protection
- heat storage
- sound insulation
- aesthetics.

3. Conclusions and further developments

The present deem-to-satisfy approach with respect to the design for durability, as is also present in the various parts of the Eurocode, gives no insight in the service life of a structure. In that sense existing methods are not objective and not generic for all building materials. A performance based durability design does not have these disadvantages. The Eurocodes strive in principle after such a type of design.

The format for a performance based durability design can be copied, to a certain extent, from the modern format for structural design as is also present in Eurocode 1. Characteristic for that format are limit states, reference periods and degrees of reliability. This structural format has to be extended to the time domain. It is obvious that this approach will offer a seamless connection between the structural and the durability design.

The performance based durability design can be extended from ultimate and serviceability limit states to non structural performances. This means it can also be applied for functional and aesthetical aspects of building materials and components.

At various occasions proposals have been made to extend the design procedure to durability aspects [e.g. 6, 7, and 8]. In the present work of CEB (Comité Euro-International du Béton) commission V 'Operation and Use' this approach has also been adopted with the aim to develop a reliability based Model Code for the durability design of concrete structures.

The amount of work that has to be done to achieve such a Model Code is tremendous. Therefore a consortium of companies that are convinced of the benefits of such an approach, has recently received a research grant from the Brite/Euram programme of the European Community. It is intended to develop the method to a practical design handbook for concrete structures including new performance tests for materials and components. Their research project is named DuraCrete. The actual research has been started in February 1996 and will end three years later.

During a workshop in December 1994 in Copenhagen an international group of about 25 experts in the field of durability of concrete has expressed that they would support the performance based design procedure. New initiatives for further research will also be taken by them.

The new developments will not be restricted to concrete structures. The intention is to extend the work to all structural materials, such as steel, aluminum, timber, masonry and plastics.

Both the service period and the lifetime concept correspond to the service life prediction method for building materials and building components of CIB/RILEM (9). This method gives a protocol for accelerated service life testing. One of the first steps in that protocol is establishing the functions R(t) and S(t). By extending the present protocol with the aspect that the lifetime is a stochastic quantity a full correspondence between the service period or the lifetime concept and the CIB/RILEM method can be achieved. In the CIB/RILEM method for predicting the service life of building materials and building components the reliability aspect must be introduced. In that case the method will correspond directly to the performance based durability design that has been presented in this paper.

4. References

1. Woodward R.J. and F.W. Williams, *Collapse of Ynys-y-Gwas bridge West Glamorgan*, 1988, Proceedings of the Institution of Civil Engineers, Part 1, Volume 84, August, pp. 635-669
2. Schaich J. and K. Kordina, H.J. Engell, *Partial collapse of the Congress Hall in Berlin - Causes of damage - report summary*, Beton und Stahlbetonbau, Vol. 75, No. 12, December 1980, pp. 281 - 294
3. Note in Krupp-VDM Berichte, Metal Times No. 8, 1992
4. Martin J.W. and S.C. Saunders, F.L. Floyd, J.P. Wineburg, NIST Building Series 172: *Methodologies for Predicting the Service Lives of Coating Systems*, 1994, pp. 68
5. Vrouwenvelder, A.C.W.M., Eurocode 1, *Basis of Design, Background Information*, Proceedings IABSE Colloquium Basis of Design and Actions on Structures- Background and Application of Eurocode 1, March 27-29 1996, Delft, The Netherlands.
6. Sentler L., Report TVBK-3018: *Service life predictions of concrete structures*, 1981, Department of Structural Engineering - Lund University of Technology, Lund, Sweden
7. Siemes A.J.M. and A.C.W.M. Vrouwenvelder, *Durability of buildings: a reliability analysis*, 1985, Heron, Volume 30, Number 3, Delft The Netherlands, 1985
8. Hergenröder M., *Zur statistischen Instandhaltungsplanung für bestehende Betonbauwerke bei Karbonatisierung des Betons und möglicher Korrosion der Bewehrung*, 1992, Technische Universität München, Germany
9. Sarja A. and E. Vesikari, Rilem report 14, *Durability Design of Concrete Structures - Calculation methods for service life design of concrete structures*, 1996, Chapman & Hall London, England (in print)
10. Masters L.W. and E. Brandt, *CIB W80/Rilem 71-PSL Recommendation: Prediction of service life of building materials and components*, 1983, Materials and Structures, Vol. 20, No. 115

Actions

Lars ÖSTLUND
Professor em.
Lund Institute of
Technology, Sweden

Lars Östlund, born 1921, got his civil engineering degree 1945. He has been professor in structural engineering 1965-1980. He was member of the project team "Basis of Design".

Summary

Some basic concepts concerning actions and action models are outlined and a probabilistic modelling of actions is discussed. With this as a background some of the basic principles given in Eurocode 1 "Basis of Design" are commented.

1. Action models

Actions are generally caused by some kind of event e.g. construction of a building, snow fall, trucks passing a bridge, collision, fire, etc. Thus a causal sequence from the event to the response of a structure can be written.

Causal event → Action → Action effect → Response of the structure

The transition from the causal event to the action implies in the design procedure consideration both to the data which describe the causal event and some of the data which characterize the object of the action (e.g. building, bridge). Thus in many cases one can distinguish between two kinds of action variables, F_o and W, and describe an action F in a very schematic way by the equation

$$F = \varphi\,(F^{o}, W) \tag{1}$$

where

$\varphi\,(\cdot)$ is a suitable function, often a simple product.

F^{o} is a basic action variable which is directly associated with the causal event and which should be defined so that it is as far as possible independent of the structure. F^{o} is often time dependent. For example, for snow load F^{o} is the snow load on ground, on a flat horizontal surface.

W is a kind of conversion factor appearing in the transformation from the basic action to that action F which affects the particular structure. W can normally be considered as time independent. For the snow load example W is the conversion factor which transforms the snow load on ground to the snow load on roof.

The basic data which give the numerical values of the action variables (F^o and W) can be obtained in different ways which may shortly be described as
- observation (e.g. snow load, wave data on the sea)
- calculation according to physical laws (e.g. self weight, dynamic forces from machines)
- choice (e.g. maximum lifted load in a crane, maximum wheel load on a bridge deck)
- judgement (e.g. accidental actions).

Often different ways are combined.

As far as possible the values of the action parameters should be described in statistical terms and be based on statistical data. Thus the background for the description of the basic action variable F^o could be a stochastic process or, if only the maximum value is of interest, a random variable. The factor W could in most cases be described by a random variable.

If the action parameters are determined by observation and/or calculation the procedures will normally include analysis of statistical data and the results can then be presented in statistical terms. If the action parameters are determined mainly by choice or judgement the procedures will generally not give results expressed in statistical terms. However action values determined in different ways are all treated in the same way in a design process according to Eurocode 1. Therefore in some cases, action data such as mean values, characteristic values, frequent values etc, which in principle have a statistical meaning, have to be determined in a fairly subjective way. Such values are denoted as nominal values. It is assumed that this does not prevent actions of different kinds to be treated according to unified principles in the design.

2. Uncertainties

The numerical values of the action variables are generally more or less uncertain. The sources of the uncertainties can normally be referred to one or more of the following categories:

- Inherent uncertainties i.e. uncertainties associated with the variability of the action characteristics themselves. Examples may be uncertainties concerning snow depths, wind speeds etc. In most cases there is nothing that can be done to decrease uncertainties of this kind, they have to be accepted and introduced in some way into the action models.
- Uncertainties which depend on lack of knowledge about the action variables or are associated with approximations made, for example, to simplify the design calculations. These uncertainties may be denoted "model uncertainties". They may be decreased through research activities or refined action models. Statistical uncertainties depending, for example, on a small number of observations are also referred to this category and in this case they may be decreased by increasing the number of observations.

- Uncertainties about the future development, for example, concerning traffic loads on bridges. This kind of uncertainties can be affected through research activities only to a limited extent.

The uncertainties should be considered in the design format. In a description of actions according to the principles given by eq. (1) the uncertainties in the action variables (F° and W) may be considered by defining these variables in a probabilistic way, for example, as random variables. The model uncertainties, i.e. the uncertainties in the function φ, may be considered by introducing an additional random variable θ which exclusively accounts for the model unceraintíes. It could, for example, be given the mean value 1 and a coefficient of variation V_θ.

In Eurocode 1, "Basis of Design" the uncertainties in the action variables are considered by using representative values (e.g. characteristic values) and partial factors. The model uncertainties may be considered by using a special partial factor γ_{Sd}.

In many cases the effect of the model uncertainties are included in the partial factor γ_F for the actions. Ideally values of γ_{Sd} should be presented in connection with the action models.

3. Probabilistic description of actions

A probabilistic description of actions is in many cases useful as a basis for the evaluation of representative values of actions and of partial factors.

The basic action variable (F° in eq. (1)) is often time dependent and in those cases a description as a stochastic process according to fig. 1 may be convenient.

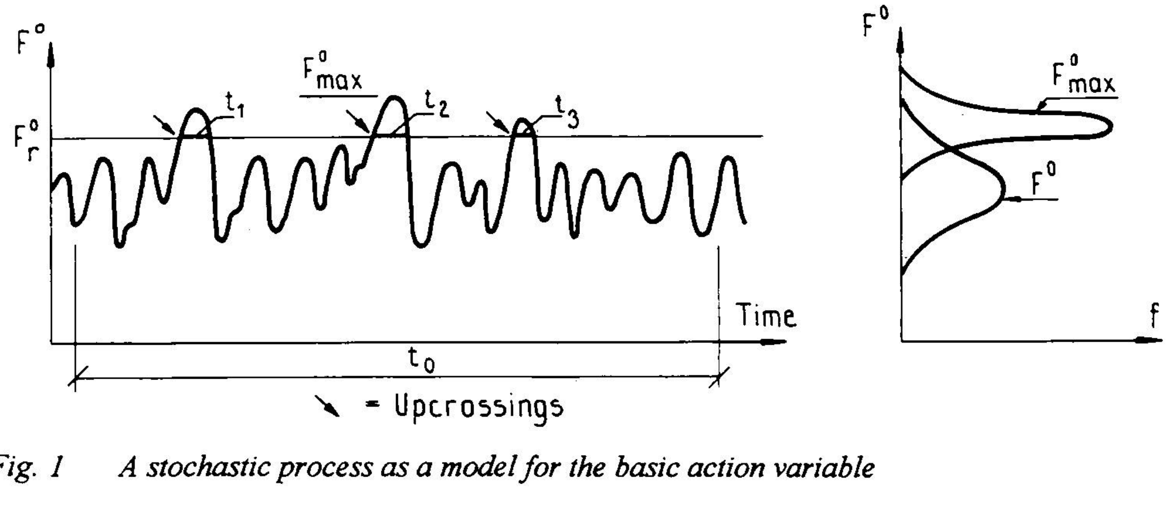


Fig. 1 A stochastic process as a model for the basic action variable

The following properties of such a process are of interest in connection with design problems.

1) Mean, standard deviation and fractiles for point-in-time values, i.e. F° in fig. 1.
2) Mean, standard deviation and fractiles for maximum values referred to a specified period of time t_o, i.e. F°_{max} in fig. 1.
3) Mean number of upcrossings (see fig. 1) per unit time for a specified level, F°_r, of the action variable, upcrossing rate.

4) The time for which the magnitude is above a specified level, F^o_r, of the action, the excursion time, i.e. t_1, t_2, t_3 in fig. 1.

In many cases a probabilistic description of an action can be simplified to a sequence of random variables according to fig. 2. This is especially the case if only the maximum value, F^o_{max}, within a specified period of time, t_o, is of interest. For the simple case, shown in figure 2, with n statistically independent action values occurring during the reference time, t_o, the relation between the probability distribution functions, F_Q for the point-in-time values and F_{Qmax} for the maximum values can be written

$$F_{Qmax}(Q) = [F_Q(Q)]^n \tag{2}$$

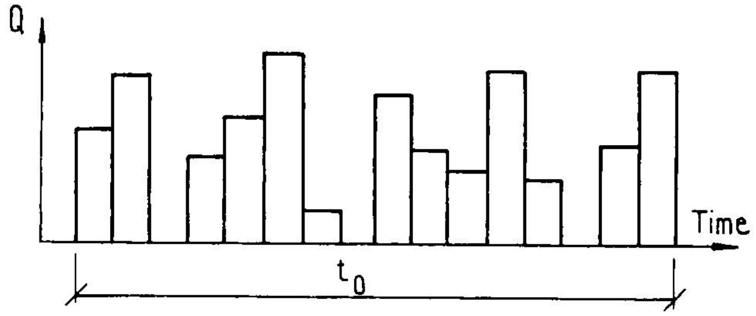


Fig. 2 A simple action-time diagram

Sometimes a description of the action variable F^o as deterministic is sufficient. This is especially the case if the magnitude of the action variable is of minor importance for a design problem.

The action variable W in eq. (1) should in principle be regarded as a random variable. In many cases the character of this variable is such that it has natural lower and/or upper limits. This may, for example, be the case for the conversion factor: snow load on ground to snow load on roof. Sometimes the variability of the action variable W is fairly unimportant and then W may be taken as deterministic.

4. Action values according to the partial factors format

4.1 Classification of actions

Important characteristics of actions could be
- their probability of occurrence
- their variability in time and space characterized, for example, by mean values and standard deviations
- other uncertainties of stochastic or non-stochastic character.

In "Basis of Design" these characteristics are considered through classifications. Thus with regard to the variation of their magnitude with time actions are classified as permanent actions, variabale actions and accidental actions. With regard to their spatial variation actions are classified as fixed actions and free actions.

The concept of *permanent actions* implies that their probability of occurrence at any arbitrary point-in-time is close to one and that their variability with time is small. One can distinguish between different kinds of permanent actions
- self weight of a structure
- weight of non structural components
- other kinds of permanent actions.

The self weight is normally fairly well defined and its uncertainties are small, the coefficient of variation is seldom more than 0.05. The weight of structural components, e.g. partition walls, have greater uncertainties often due to foreseen future alterations. Other permanent actions, e.g. settlements of foundations, may have very great uncertainties.

The probability of occurrence of *variable actions* at any arbitrary point-in-time is very different but the occurrences sometimes during the working life of a particular structure is generally close to one. The concept of variable actions implies that their variability with time is not small, in many cases it can be intense. The uncertainties of the magnitude may vary very much from one action to another.

The probability of occurrence of *accidental actions* sometimes during the working life of a particular structure is small. If it occurs the variability in time and space of an accidental action is generally great. The uncertainty of its magnitude is in most cases very great.

4.2 Representative action values

For the application in different design situations the actions have in "Basis of Design" been given one or more representative values of different kinds. Thus in ordinary cases the following kinds of values are used.

For permanent actions, one kind of values:
  Characteristic value
For variable actions, four kinds of values:
  Characteristic value
  Combination value
  Frequent value
  Quasi-permanent value.
For accidental actions, one kind of values:
  Design value.

As characteristic value for *permanent actions* one value may be chosen if the variability is small e.g. for most cases of self weight. In other cases, e.g. for the weight of non structural components, where the variability often is greater the use of upper and lower characteristic values may be justified.

For *variable actions* the characteristic value is associated with the probability distribution function for the maximum value, Q_{max}, occuring within the specified reference time. The other representative values are associated with the probability distribution function for the point-in-time values, Q, even if they are specified as the characteristic value multiplied with a factor ψ. This is illustrated in fig. 3.

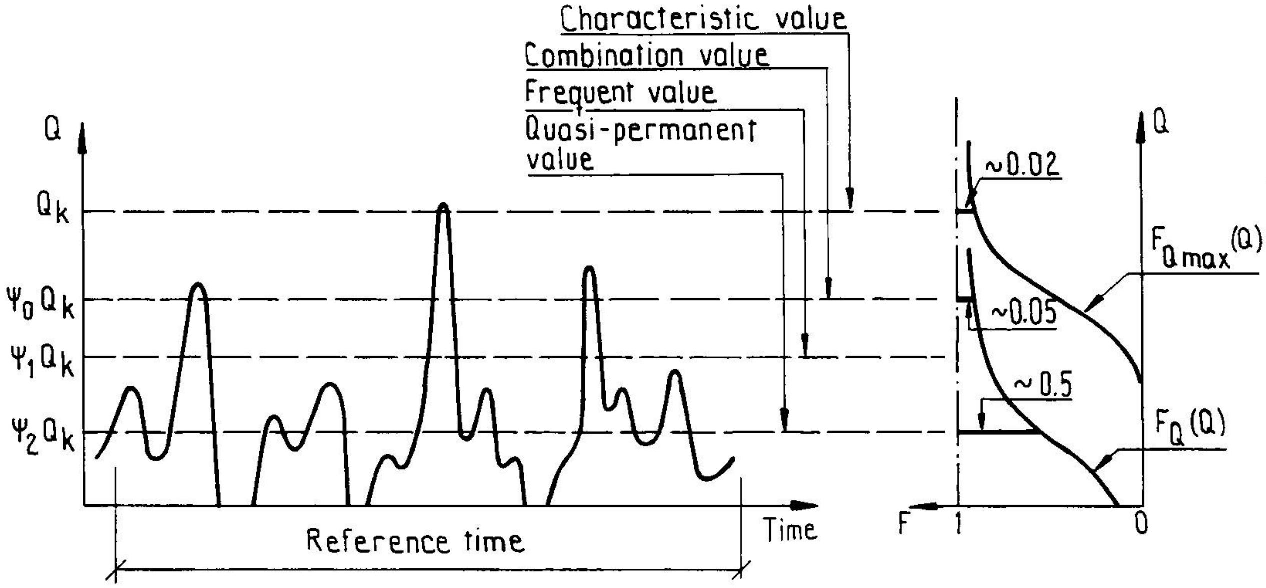


Fig. 3 Representative values of variable actions. (The numerical values are examples).

The characteristic value of a variable action is a comparatively high value which can be expected to be exceeded very seldom, in average once in a period of 50 years.

The combination value should be chosen so that the probability that the action effect values caused by the combination will be exceeded is approximately the same as when a single action is considered. Thus the combination value is defined according to a somewhat different principle in comparison with the other representative values. The portion of time when no load (or a very small load) occurs is often important.

The frequent value is defined with regard to the portion (η) of the time or to the number (ν) of times, when the load value is above the frequent value. With the numerical values proposed in "Basis of Design", i.e. $\eta = 0.05$ and $\nu = 300$ per year, it is obvious that the frequent value may be exceeded fairly often.

The quasi-permanent value is defined so that it can be expected to be exceeded about half the time. Thus the quasi-permanent value should be about the time average value.

Vind load could be taken as an example. There is a considerable part of the time during which the wind load is very small and absolutely unimportant. This does not affect the characteristic value which should be about the greatest value occuring during a 50 years period. It affects also the frequent value very little as this value should be chosen so that it is exceeded during about 400 hours every year in average. As one storm has a duration of 4 - 10 hours it can be concluded that the frequent value does not require a very strong wind. If winds of importance are assumed to occur only during 25 percent of the time this may have influence on the combination value at least for combinations with other short term loads. Further this is determining for the quasi-permanent value which should be zero for wind load.

4.3 Combinations of actions

The representative action values are introduced into the different types of combinations. The basic principle of combination of actions is to take **one** action (the dominating action) with a comparatively high value and combine it with the other actions (the non-dominating actions) which have lower values. Thus the kind of limit state and the type and purpose of the considered combination determine which kind of action value that should be chosen as dominating.

For *ultimate limit states* the "ordinary" combination", i.e. the combination applied in persistant or transient design situations, contains as value of the dominant action the characteristic value of either a permanent action or a variable action. This is shown in the expressions (9.10a) or (9.10b) respectively. The expression (9.10) is a simplified combination of (9.10a) and (9.10b).

The combination for accidental design situations (expression (9.11)) contains quite naturally the accidental action as dominating action.

For *serviceability limit states* the characteristic combination should be used for irreversible limit states when failure implies serious permanent damage. Consequently a high value, the characteristic value, should be chosen for the dominating variable action.

The frequent combination should be used for reversible limit states when, with certain limitations, passage of the limit state is considered as acceptable. Therefore the frequent value, which may be exceeded now and then, may be convenient to use as value of the dominating action. The frequent combination could also be used for irreversible limit states if the consequences of failure are not serious.

Finally the quasi-permanent combination is indented to be applied for long term problems. Thus the quasi-permanent action value should be chosen for the dominating as well as all other variable actions.

Leere Seite
Blank page
Page vide