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General principles on reliability for structures

Principes généraux de la fiabilité des constructions

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Contents

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Foreword

ISO (the International Organization for Standardization) is a worldwide federation of national standards bodies (ISO member bodies). The work of preparing International Standards is normally carried out through ISO technical committees. Each member body interested in a subject for which a technical committee has been established has the right to be represented on that committee. International organizations, governmental and non-governmental, in liaison with ISO, also take part in the work. ISO collaborates closely with the International Electrotechnical Commission (IEC) on all matters of electrotechnical standardization.

Draft International Standards adopted by the technical committees are circulated to the member bodies for voting. Publication as an International Standard requires approval by at least 75 % of the member bodies casting a vote.

International Standard ISO 2394 was prepared by Technical Committee ISO/TC 98, *Bases for design of structures,* Subcommittee SC 2, *Reliability of structures.*

This second edition cancels and replaces the first edition (ISO 2394:1986), which has been technically revised.

Annexes A to F of this International Standard are for information only.

Introduction

This International Standard constitutes a common basis for defining design rules relevant to the construction and use of the wide majority of buildings and civil engineering works, whatever the nature or combination of the materials used. However, their application to each type of material (concrete, steel, timber, masonry, etc.) will require specific adaptation to ensure a degree of reliability which, as far as possible, is consistent with the objectives of the code drafting committees for each material.

This International Standard is intended to serve as a basis for those committees responsible for the task of preparing national standards or codes of practice in accordance with the technical and economic conditions in a particular country, and which take into account the nature, type and conditions of use of the structure and the properties of the materials during its design working life. It will also provide a common basis for other International Standards (e.g. ENV 1991-1 EC1) dealing with load-bearing structures. Thus it has a conceptual character and it is of a fairly general nature.

It is important to recognize that structural reliability is an overall concept comprising models for describing actions, design rules, reliability elements, structural response and resistance, workmanship, quality control procedures and national requirements, all of which are mutually dependent.

The modification of one factor in isolation could therefore disturb the balance of reliability inherent in the overall concept.

It is therefore important that the modification of any one factor should be accompanied by a study of the implications relating to the overall reliability concept.

General principles on reliability for structures

1 Scope

This International Standard specifies general principles for the verification of the reliability of structures subjected to known or foreseeable types of action. Reliability is considered in relation to the performance of the structure throughout its design working life.

The general principles are applicable to the design of complete structures (buildings, bridges, industrial structures, etc.), the structural elements making up the structure and the foundations.

This International Standard is also applicable to the successive stages in construction, namely the fabrication of structural elements, the transport and handling of the structural elements, their erection and all work on site, as well as the use of the structure during its design working life, including maintenance and repair.

To allow for the differences in design practice between different countries, the national standards or codes of practice may be simpler or more detailed in comparison with this International Standard.

Generally the principles are also applicable to the structural appraisal of existing constructions or assessing changes of use. However in some respects this is associated with special aspects of the basic variables and calculation models. Such aspects are considered in clause 10.

NOTE — When this International Standard is applied in a particular country for the development of its standards, it is admissible not to use those clauses which are not in accordance with the regulations of that particular country.

2 Definitions

For the purposes of this International Standard, the following definitions apply.

NOTE — An alphabetical index of the definitions is given in annex H.

2.1 General terms

2.1.1 structure: Organized combination of connected parts designed to provide some measure of rigidity.

2.1.2 structural element: Physically distinguishable part of a structure.

EXAMPLES: Column, beam, plate.

2.1.3 structural system: Load-bearing elements of a building or civil engineering works and the way in which these elements function together.

2.1.4 compliance: Fulfilment of specified requirements.

2.1.5 life cycle: Total period of time during which the planning, execution and use of a construction works takes place. The life cycle begins with identification of needs and ends with demolition.

2.2 Terms relating to design in general

2.2.1 design situation: Set of physical conditions representing a certain time interval for which the design demonstrates that relevant limit states are not exceeded.

2.2.2 persistent situation: Normal condition of use for the structure, generally related to its design working life.

NOTE — "Normal use" includes possible extreme loading conditions due to wind, snow, imposed loads, earthquakes in areas of high seismicity, etc.

2.2.3 transient situation: Provisional condition of use or exposure for the structure.

EXAMPLE: During its construction or repair, which represents a time period much shorter than the design working life.

2.2.4 accidental situation: Exceptional condition of use or exposure for the structure.

EXAMPLES: Flood, land slip, fire, explosion, impact or local failure, which represent in most cases a very short time period (apart from situations where a local failure may remain undetected during a longer period).

2.2.5 serviceability: Ability of a structure or structural element to perform adequately for normal use under all expected actions.

2.2.6 failure: Insufficient load-bearing capacity or inadequate serviceability of a structure or structural element.

2.2.7 reliability: Ability of a structure or structural element to fulfil the specified requirements, including the working life, for which it has been designed.

2.2.8 reference period: A chosen period of time which is used as a basis for assessing values of variable actions, time-dependent material properties, etc.

2.2.9 limit state: A state beyond which the structure no longer satisfies the design performance requirements.

NOTE — Limit states separate desired states (no failure) from undesired states (failure).

2.2.10 ultimate limit state: A state associated with collapse, or with other similar forms of structural failure.

NOTE — This generally corresponds to the maximum load-carrying resistance of a structure or structural element but in some cases to the maximum applicable strain or deformation.

2.2.11 serviceability limit state: A state which corresponds to conditions beyond which specified service requirements for a structure or structural element are no longer met.

2.2.12 irreversible limit state: A limit state which will remain permanently exceeded when the actions which caused the excess are removed.

2.2.13 reversible limit state: A limit state which will not be exceeded when the actions which caused the excess are removed.

2.2.14 structural integrity (structural robustness) : Ability of a structure not to be damaged by events like fire, explosions, impact or consequences of human errors, to an extent disproportionate to the original cause.

2.2.15 design working life: Assumed period for which a structure or a structural element is to be used for its intended purpose without major repair being necessary.

2.2.16 maintenance: Total set of activities performed during the design working life of a structure to enable it to fulfil the requirements for reliability.

2.2.17 reliability class of structures: Class of structures or structural elements for which a particular specified degree of reliability is required.

2.2.18 basic variable: Part of a specified set of variables representing physical quantities which characterize actions and environmental influences, material properties including soil properties, and geometrical quantities.

2.2.19 primary basic variable: Variable whose value is of primary importance to the design results.

2.2.20 limit state function: A function g of the basic variables, which characterizes a limit state when $g(X_1, X_2, ..., X_n) = 0$: $g > 0$ identifies with the desired state and $g < 0$ with the undesired state.

2.2.21 reliability index, β: A substitute for the failure probability p_f, defined by $\beta = -\Phi^{-1}(p_f)$, where Φ^{-1} is the inverse standardized normal distribution.

2.2.22 partial factors format: Calculation format in which allowance is made for the uncertainties and variabilities assigned to the basic variables by means of representative values, partial factors and, if relevant, additive quantities.

2.2.23 reliability element: Numerical quantity used in the partial factors format, by which the specified degree of reliability is assumed to be reached.

2.2.24 element reliability: Reliability of a single structural element which has one single dominating failure mode.

2.2.25 system reliability: Reliability of a structural element which has more than one relevant failure mode or the reliability of a system of more than one relevant structural element.

2.2.26 model: Simplified mathematical description or experimental set-up simulating actions, material properties, the behaviour of a structure, etc.

NOTE — Models should generally take account of decisive factors and neglect the less important ones.

2.2.27 model uncertainty: Related to the accuracy of models, physical or statistical.

NOTE — Further information is given in annexes D and E.

2.2.28 statistical uncertainty: Uncertainty related to the accuracy of the distribution and estimation of parameters.

2.2.29 assessment: Total set of activities performed in order to find out if the reliability of a structure is acceptable or not.

2.3 Terms relating to actions, action effects and environmental influences

2.3.1 action

1) An assembly of concentrated or distributed mechanical forces acting on a structure (direct actions).

2) The cause of deformations imposed on the structure or constrained in it (indirect action).

2.3.2 permanent action:

1) Action which is likely to act continuously throughout a given reference period and for which variations in magnitude with time are small compared with the mean value.

2) Action whose variation is only in one sense and can lead to some limiting value.

2.3.3 variable action: Action for which the variation in magnitude with time is neither negligible in relation to the mean value nor monotonic.

2.3.4 accidental action: Action that is unlikely to occur with a significant value on a given structure over a given reference period.

NOTE — Accidental actions are in most cases of short duration.

2.3.5 fixed action: Action which has a fixed distribution on a structure, such as its magnitude and direction are determined unambiguously for the whole structure when determined at one point on the structure.

2.3.6 free action: Action which may have an arbitrary spatial distribution over the structure within given limits.

2.3.7 static action: Action which will not cause significant acceleration of the structure or structural elements.

2.3.8 dynamic action: Action which may cause significant acceleration of the structure or structural elements.

2.3.9 bounded action: Action which has a limiting value which cannot be exceeded and which is exactly or approximately known.

2.3.10 unbounded action: Action which has no known limiting values.

2.3.11 representative value of an action: A value used for the verification of a limit state.

NOTE — Representative values consist of characteristic values, combination values, frequent values and quasi-permanent values, but may also consist of other values.

2.3.12 characteristic value of an action: Principal representative value.

NOTE — It is chosen either on a statistical basis, so that it can be considered to have a specified probability of not being exceeded towards unfavourable values during a reference period, or on acquired experience, or on physical constraints.

2.3.13 combination value: Value chosen, in so far as it can be fixed on statistical bases, 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.

2.3.14 frequent value: Value determined, in so far as it can be fixed on statistical bases, so that:

— the total time, within a chosen period of time, during which it is exceeded is only a small given part of the chosen period of time; or

— the frequency of its exceedance is limited to a given value.

2.3.15 quasi-permanent value: Value determined, in so far as it can be fixed on statistical bases, so that the total time, within a chosen period of time, during which it is exceeded is of the magnitude of half the period.

2.3.16 design value of an action, F_d: Value obtained by multiplying the representative value by the partial factor γ_F.

2.3.17 load arrangement: Identification of the position, magnitude and direction of a free action.

2.3.18 load case: Compatible load arrangements, set of deformations and imperfections considered for a particular verification.

2.3.19 load combination: Set of design values used for the verification of the structural reliability for a limit state under the simultaneous influence of different actions.

2.3.20 environmental influence: Mechanical, physical, chemical or biological influence which may cause deterioration of the materials constituting a structure, which in turn may affect its serviceability and safety in an unfavourable way.

2.4 Terms relating to structural response, resistance, material properties and geometrical quantities

2.4.1 characteristic value of a material property: An *a priori* specified fractile of the statistical distribution of the material property in the supply produced within the scope of the relevant material standard.

2.4.2 characteristic value of a geometrical quantity: A quantity usually corresponding to dimensions specified by the designer.

2.4.3 design value of a material property: Value obtained by dividing the characteristic value by a partial factor γ_M or, in special circumstances, by direct assessment.

2.4.4 design value of a geometrical quantity: Characteristic value plus or minus an additive geometrical quantity.

2.4.5 conversion factor: Factor which converts properties obtained from test specimens to properties corresponding to the assumptions made in calculation models.

2.4.6 conversion function: Function which converts properties obtained from test specimens to properties corresponding to the assumptions made in calculation models.

3 Symbols

NOTE — The symbols used generally are listed below. Those which are not general and which are used only in one clause (and are explained there) are not listed.

3.1 Main letters

A Accidental action

C Serviceability constraint

F Action in general

F_0 Basic action variable

F_r Representative value of an action

G Permanent action

Q Variable action

R Resistance

S Action effect

W Action model variable

X Basic variable

Y Model output variable

a Geometrical quantity

Δa Additive geometrical quantity

f Material property

p_f Probability of failure

p_{fs} Specified value of p_f

t Time

β Reliability index

γ Partial factor

γ_f Partial factor for actions

γ_F Generalized partial factor for actions taking account of model and geometric uncertainties

γ_G Partial factor for permanent actions

γ_Q Partial factor for variable actions

γ_m Partial factor for material properties

γ_M Generalized partial factor for resistance properties taking account of material, model and geometric uncertainties

γ_D Partial factor for model uncertainties

γ_n Factor by which the importance of the structure and the consequences of failure are taken into account

θ Parameter which contains model uncertainties

θ_S Value for action effects

θ_R Value for resistance

φ Function of action variables

Ψ_0 Factor for determining combination values of actions

Ψ_1 Factor for determining frequent values of actions

Ψ_2 Factor for determining quasi-permanent values of actions

$g(X,t)$ Limit state function

3.2 Subscripts

i Basic variable (mainly action) number i

j Action number j

k Characteristic value

d Design value

4 Requirements and concepts

4.1 Fundamental requirements

Structures and structural elements shall be designed, constructed and maintained in such a way that they are suited for their use during the design working life and in an economic way.

In particular they shall, with appropriate degrees of reliability, fulfil the following requirements:

— they shall perform adequately under all expected actions (serviceability limit state requirement);

— they shall withstand extreme and/or frequently repeated actions occurring during their construction and anticipated use (ultimate limit state requirement);

— they shall not be damaged by events like flood, land slip, fire, explosions, impact or consequences of human errors, to an extent disproportionate to the original cause (structural integrity requirement).

The appropriate degree of reliability should be judged with due regard to the possible consequences of failure and the expense, level of effort and procedures necessary to reduce the risk of failure (see 4.2).

The measures that can be taken to achieve the appropriate degree of reliability include:

— choice of structural system, proper design and analysis;

— implementation of a quality policy;

— design for durability and maintenance;

— protective measures.

These items will be treated in 4.3 to 4.5.

4.2 Reliability differentiation of structures

The expression "with appropriate degrees of reliability" used in 4.1 means that the degree of reliability should be adopted taking account of:

— the cause and mode of failure implying that a structure or structural element which would be likely to collapse suddenly without warning should be designed for a higher degree of reliability than one for which a collapse is preceded by some kind of warning in such a way that measures can be taken to limit the consequences;

— the possible consequences of failure in terms of risk to life, injury, potential economic losses and the level of social inconvenience;

— the expense, level of effort and procedures necessary to reduce the risk of failure;

— the social and environmental conditions in a particular location.

Differentiation of the required degrees of reliability may be obtained by the classification of whole structures or by the classification of structural elements. Thus, as an example, degrees of reliability may be selected according to the consequences of failure as follows:

a) risk to life low, economic, social and environmental consequences small or negligible;

b) risk to life medium, economic, social or environmental consequences considerable;

c) risk to life high, economic, social or environmental consequences very great.

The required reliability related to structural safety or serviceability may be achieved by suitable combinations of the following measures.

a) Measures relating to design:

  — serviceability requirements;

  — the choice of the values of action variables;

— the choice of the degree of reliability for the design calculations;

— consideration of durability;

— consideration of the degree of structural integrity (robustness), see 4.3;

— the amount and quality of preliminary investigations of soils and possible environmental influences;

— the accuracy of the mechanical models used;

— the stringency of the detailing rules.

b) Measures relating to quality assurance to reduce the risk of hazards in:

— gross human errors;

— design;

— execution.

4.3 Structural design

A failure of a structure or part of it may occur due to:

— an extremely unfavourable combination of actions, material properties, geometrical quantities, etc., all of which are associated with ordinary use and other ordinary circumstances;

— effects of exceptional but foreseeable actions or environmental influences, for example, collision or extreme climatic influences;

— consequences of an error, such as lack of information, omission, misunderstanding and lack of communication, negligence, misuse, etc.;

— influences that are not foreseen.

NOTE — The term "exceptional" refers to circumstances and/or actions that are present only during a small portion of the life and/or with a low probability. Depending on the type of structure, these actions may or may not be considered explicitly in design.

No structure can be expected to function adequately in all cases if exceptional actions or exceptionally low resistance occur, but the foreseeable scope of damage should be limited to an extent not disproportionate to the original cause. Thus measures should be taken to counter such events. The measures would basically consist of one or more of the following:

a) Designing and maintaining the structure according to the rules given in the following clauses for conditions associated with ordinary use and other ordinary circumstances.

b) Design of the essential load-bearing members of a structure for specified exceptional actions which may be caused by accidents or similar occurrences.

NOTE — The intention of such design criteria is that they should also cover the effect of the majority of unforeseen events.

The structural layout should be checked to identify "key" structural elements, whose failure would cause the collapse of more than a limited portion of the structure close to the element in question. Where such structural elements are identified and the structural layout cannot be revised to avoid them, the design should take their importance into account.

c) Protection against foreseeable actions and elimination of errors.

A careful check should be made and appropriate action taken to ensure that there is no inherent weakness in the structural layout and that adequate means exist to transmit all loads safely to the foundations.

Protective measures should be introduced, for example, safeguarding against vehicular impact by the provision of additional protection such as bollards.

The probability of gross design and construction errors should be reduced by appropriate quality assurance and/or quality control measures, as described in 4.4.

d) Design of the structure in such a way that local damage does not lead to immediate collapse of the whole structure or a significant part of it.

As a design assumption, the following may be chosen to represent local damage. The structure shall be designed and detailed so that any load-bearing elements other than the "key" structural elements can be removed without causing the collapse of more than a limited portion close to the element in question. In the event of the removal of a non-key structural element, a less than normal reliability for the remaining structure will be considered acceptable provided that the structure is repaired to normal reliability standard within a reasonably short period of time after damage.

4.4 Compliance

To achieve adequate confidence that the completed construction works fulfil the specified requirements for quality and, in particular, the fundamental requirements (4.1), an appropriate quality policy should be adopted and implemented by parties involved in the management of all stages of the life cycle of the construction works.

NOTE — For further details, see annex A and ISO 9000, ISO 9001, ISO 9002, ISO 9003 and ISO 9004.

This quality policy should comprise:

a) definition of quality requirements;

b) organization measures and controls at the stages of design and execution and during the use and the maintenance of the structure.

The quality management selected for implementing the quality policy should include consideration of:

— the type and use of the structure;

— the consequences of quality deficiencies (e.g. accidents resulting from structural failures); and

— the management culture of the involved parties.

In the structural design of the construction works, reliability is the most important aspect to consider to achieve quality. Standards for structural design should provide a framework to achieve structural reliability as follows:

— by providing requirements for reliability;

— by specifying the rules to verify the fulfilment of the requirements for reliability;

— by specifying the rules for structural design and associated conditions.

The conditions to be fulfilled concern, for example, the choice of structural system, level of workmanship and maintenance regime, and are normally detailed in the structural design standards. The conditions should also take into account the variability of material properties, the quality control and the criteria for material acceptance. They also include consideration of the use of information technology with regard to the design and the execution process including supply chains with regard to delivering of material and testing of material.

NOTES

1 For example the conditions are given as "assumptions" in Eurocode 1: Part 1, Basis of Design.

2 See also annex A.

4.5 Durability and maintenance

Durability is a necessary condition for the fulfillment of the requirements for reliability. The durability of the structure and the structural elements in their environment shall be such that they remain fit for use during their design working lives, given appropriate maintenance. This is also applicable to fatigue. Examples of design working life are given in table 1.

Table 1 — Notional classification of design working life

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

Maintenance is the total set of activities performed during the life of the structure to enable it to fulfil the requirements for durability. Maintenance includes the performance of regular inspections, inspections on special occasions (e.g. after an earthquake), the upgrading of protection systems and repair of structural elements.

Durability shall be secured by:

— either a maintenance programme, or

— design such that deterioration will not lead to the failure of the structure, in those cases when the structure cannot or is not expected to be subjected to maintenance.

In the first case above, the structure shall be designed and constructed in such a way, or provided with protection so that no significant deterioration is likely to occur within the period between successive inspections. The need for relevant parts of the structure being available for inspection — without complicated dismantling — shall be considered in the design.

To ensure an adequately durable structure, the following inter-related factors shall be considered:

— the intended use of the structure;

— the required performance criteria (e.g. appearance);

— the expected environmental conditions;

— the composition, properties and performance of the materials;

— the structural system;

— the shape of members and the structural detailing;

— the quality of workmanship, and level of control;

— the particular protective measures;

— the maintenance during the design working life.

The rate of deterioration may be estimated on the basis of theoretical or experimental investigation and experience.

5 Principles of limit states design

5.1 Limit states

5.1.1 General

The structural performance of a whole structure or part of it should generally be described with reference to a specified set of limit states which separate desired states of the structure from undesired states.

The limit states are divided into the following two categories:

a) ultimate limit states, which correspond to the maximum load-carrying capacity or, in some cases, to the maximum applicable strain or deformation;

b) serviceability limit states, which concern the normal use.

The effect of exceeding a limit state may be irreversible or reversible. In the irreversible case, the damage or malfunction associated with the limit state being exceeded will remain until the structure has been repaired. In the reversible case, the damage or malfunction will remain only as long as the cause of the limit state being exceeded is present. As soon as this cause ceases to act, a transition from the undesired state back to the desired state occurs.

5.1.2 Ultimate limit states

Ultimate limit states include:

a) loss of equilibrium of the structure or of a part of the structure, considered as a rigid body (e.g. overturning);

b) attainment of the maximum resistance capacity of sections, members or connections by rupture (in some cases affected by fatigue, corrosion, etc.) or excessive deformations;

c) transformation of the structure or part of it into a mechanism;

d) instability of the structure or part of it;

e) sudden change of the assumed structural system to a new system (e.g. snap through).

The effect of exceeding an ultimate limit state is almost always irreversible and the first time that this occurs it causes failure.

NOTE — For simplicity, a state prior to structural collapse may be considered as an ultimate limit state, e.g. reduced structural system following an accidental action.

5.1.3 Serviceability limit states

Serviceability limit states include:

a) local damage (including cracking) which may reduce the working life of the structure or affect the efficiency or appearance of structural or non-structural elements; repeated loading may affect the local damage, e.g. by fatigue;

b) unacceptable deformations which affect the efficient use or appearance of structural or non-structural elements or the functioning of equipment;

c) excessive vibrations which cause discomfort to people or affect non-structural elements or the functioning of equipment.

In the cases of permanent local damage or permanent unacceptable deformations, exceeding a serviceability limit state is irreversible and the first time that this occurs it causes failure.

In other cases, exceeding a serviceability limit state may be reversible and then failure occurs as follows:

a) the first time the serviceability limit state is exceeded, if no excess is considered as acceptable;

b) if the excess is acceptable but the time when the structure is in the undesired state is longer than specified;

c) if the excess is acceptable but the number of times that the serviceability limit state is exceeded is larger than specified;

d) if a combination of the above criteria or of some other relevant criteria occur.

These cases may involve temporary local damage (e.g. temporarily wide cracks), temporary large deformations and vibrations.

Design criteria for serviceability limit states are generally expressed in terms of limits for acceptable deformations, accelerations, crack widths, etc.

NOTE — These limits should generally be considered random and may be described by statistical methods. They are, however, normally introduced into codes with specified limiting values.

5.2 Design

5.2.1 Design procedure

All relevant limit states should be considered in the design.

For each specific limit state, the relevant basic variables should be identified, i.e. the variables which characterize:

— actions and environmental influences;

— properties of materials and soils;

— geometrical parameters.

Models which describe the behaviour of a structure should be established for each limit state. These models include mechanical models which describe the structural behaviour, as well as other physical or chemical models which describe the effects of environmental influences on the properties of the materials.

Where calculation models are available, the limit state can be described with aid of a function, g, of the basic variables $X = X_1, X_2, ..., X_n$ so that

$$g(X_1, X_2, ..., X_n, t) = 0 \qquad \text{... (1)}$$

Equation (1) is called the limit state equation, and

$$g(X_1, X_2, ..., X_n, t) \geqslant 0 \qquad \text{... (2)}$$

identifies the desired state.

In principle, the purpose of design calculations (or prototype testing) is to ensure an adequate degree of reliability. To verify this, calculations are performed according to a chosen design format. In this International Standard two possible design formats are treated:

— a probabilistic format (clause 8), and

— a partial factors format (clause 9).

The partial factors format is the format which is intended to be used for design calculations in normal cases. The probabilistic format may be convenient for special design problems and may be used for the calibration of partial factors.

In addition to design calculations, detailing is an important part of the design procedure. Thus assumptions made in the calculation models must be incorporated in the workshop drawings, instructions, etc., by appropriate structural provisions and detailing.

5.2.2 Design situations

Actions, environmental influences and, in many cases, the expected properties of a structure vary with time. These variations, which occur throughout the life of the structure, should be considered by selected design situations, each one representing a certain time interval with associated hazards, conditions and relevant structural limit states. Separate reliability checking is required for each design situation with due regard to the different consequences of failure.

The design situations are classified as:

— persistent situations;

— transient situations;

— accidental situations.

Persistent and transient situations are considered to act with certainty. Accidental situations by definition occur with a relatively low probability during the design working life.

Whether loads, such as snow loads, earthquakes, etc., are associated with other transient or accidental situations will depend on local conditions.

6 Basic variables

6.1 General

The calculation model for each limit state considered should contain a specified set of basic variables, representing physical quantities which characterize actions and environmental influences, material and soil properties, and geometrical quantities.

If the uncertainty of a basic variable is judged to be important, e.g. by experience or by a sensitivity study, it shall be represented as a random variable.

The uncertainties generally consist of a systematic part (bias) and a random part.

The uncertainties are caused by:

— inherent random variability, which is the unpredictable variability in time or among the typical structures and geographical regions under consideration;

— insufficient data and/or imprecise knowledge.

Random variables should be described by probability distributions, which often should be considered as conditional. In many cases these distributions are characterized by main parameters such as mean, standard deviation, skewness and coefficient of correlation in the case of multi-dimensional distribution. A probabilistic model should be based on a statistical analysis of available data. It is important to separate and identify the different statistical populations in order not to use erroneous types of distributions. The data should, when possible, be examined to eliminate measurement errors, scale effects, etc.

Probabilistic models for basic variables can be used directly within the probabilistic format (see clause 8). Within the partial factors format, basic variables are represented by their design values (see clause 9), which, where possible, should be derived from the probabilistic models.

NOTE — Further details are given in annex E.

6.2 Actions

6.2.1 General

An action is:

— an assembly of concentrated or distributed mechanical forces acting on the structure (direct actions), or

— the cause of deformations imposed on the structure or constrained in it (indirect actions).

An action is considered to be one single action if it can be assumed to be statistically independent, in time and space, of any other action acting on the structure.

NOTE — In reality, actions that are introduced simultaneously are often statistically dependent to a certain extent, for example, climatic actions (wind, snow, temperature). This dependency is usually taken into account by specific provisions.

An action is often characterized by two or more basic variables. For example, the magnitude and direction of an action can both be basic variables.

Sometimes an action may be introduced as a function of basic variables, each of which represents some underlying physical property. Such a function is called an action model. An example is earth pressure, which may depend on the vertical pressure from the earth and the angle of friction; both are random variables.

Variables of natural actions are determined by environmental conditions. Variables of actions due to human activity are determined by normal human behaviour, gross human errors, etc.

6.2.2 Classification of actions according to the variation of their magnitude with time

Actions are classified according to their variation in time as

— permanent actions (G),

— variable actions (Q),

— accidental actions (A).

Permanent actions are those which are likely to act continuously throughout a given reference period and for which variations in magnitude with time are small compared with the mean value; or the variation of which is only in one sense and can lead to some limiting values.

Variable actions are those for which the variation in magnitude with time is neither negligible in relation to the mean value nor monotonic.

Accidental actions are those that are unlikely to occur with a significant value on a given structure over a given reference period.

NOTE — Accidental actions are in most cases of short duration.

Variable and accidental actions can be described by random and/or non-random functions of space and time.

Probabilistic models for the extremes of variable and accidental actions should always refer to a given reference period.

NOTE — Examples of permanent, variable and accidental actions are given in annex B.

6.2.3 Classification of actions according to their variation in space

Actions are classified according to their variation in space as

— fixed actions,

— free actions.

Actions which cannot be defined as belonging to either of these two groups may be considered to consist of a fixed part and a free part.

The treatment of free actions requires the consideration of different load arrangements.

NOTE — In certain cases, e.g. for traffic loads, it is necessary to distinguish, among free actions, those that are moving or not, and what are the limits to their freedom. Such distinctions are taken into account by the model itself or by specific provisions for use.

6.2.4 Classification of actions according to the structural response

Actions are classified according to the way in which a structure responds to the action as:

— static actions, i.e. not causing significant acceleration of the structure or structural elements;

— dynamic actions, i.e. causing significant acceleration of the structure or structural elements.

NOTE — In most cases, the dynamic actions can be treated as static actions by taking into account the dynamic effects by an appropriate increase in the magnitude of the quasi-static component or by the choice of an equivalent static force. When this is not the case, corresponding dynamic models are used to assess the response of the structure; inertia forces are then not included in the action model but are determined by analysis.

6.2.5 Bounded and unbounded actions

Bounded actions are those which have a limiting value which cannot be exceeded and which is exactly or approximately known. Such a limiting value can be attained, or nearly attained, with a significant probability during the design situation under consideration. The other actions are called unbounded actions.

6.2.6 Other classifications of actions

Other classifications, most of them material-dependent, are to be considered in special cases (e.g. classification according to duration for assessing the effects of creep, and classification for fatigue verifications).

6.3 Environmental influences

Environmental influences may have a mechanical, physical, chemical or biological character and may deteriorate the material of a structure which, in turn, may affect safety and serviceability in an unfavourable way.

Environmental influences are in many respects similar to actions and can be classified in a similar way, especially concerning their temporal variability. Thus environmental influences can be classified as permanent, variable and accidental.

NOTE — An example of a permanent influence is the chemical effect of chlorides in sea water on concrete. The effect of humidity on the strength of wood is an example of a variable influence.

The effects of environmental influences are strongly material dependent and therefore the characteristics of these influences have to be specified for each particular type of material. In many cases involving chemical and biological deterioration, the presence of moisture is a key factor.

Environmental influences should, where possible, be described by the use of numerical values in the same way as for actions. In many cases this is difficult and therefore environmental influences are often classified with respect to their aggressiveness against a specific material. Often two or more environmental influences give a combined effect which is more severe than the sum of the effects of the single influences. In such cases the environment as a whole should be classified with regard to its aggressiveness.

NOTE — In some cases, however, an environmental influence can be described by numerical values and models for their effects on a specific material can be established. In such cases the degradation of the material, after different periods of time, can be estimated by calculations. One example is the carbonation of the concrete cover for reinforcement.

6.4 Properties of materials

Properties of materials including soils should be described by measurable physical quantities and should correspond to the properties considered in the calculation model. These properties may vary in time, depend on temperature, humidity, load history and other influences. They also depend on specified conditions concerning manufacturing, supply and acceptance criteria.

Generally, the properties and their variation should be determined from tests on appropriate test specimens. The tests should be based on random samples which are representative of the population under consideration.

By means of appropriately specified conversion factors or functions, the properties obtained from test specimens should be converted to properties corresponding to the assumptions made in calculation models. The uncertainties of the conversion factors should be considered. Possible conversion effects that shall be considered are size-effects, time-effects, and the effects of temperature, humidity, etc.

NOTE — For soils, as for existing structures, the materials are in most cases not produced but are found on the site. Therefore, the values of the properties have to be determined for each project. A detailed investigation based on tests may then give more precise and complete information than a purely statistical approach, especially with respect to systematic trends or weak spots in the spatial distributions. However, fluctuations in homogeneous materials and the limited precision of the tests and of their physical interpretations can be treated by statistical methods. For these materials the extent of investigation is an element of the structural reliability, which is often difficult to quantify. At the design stage the other materials have generally not yet been produced or at least have not yet been identified. Therefore corresponding statistical parameters have to be deduced from an existing population which is considered similar to it, and these parameters should be checked afterwards for quality. The identification of sufficiently homogeneous populations (e.g. good division of the production in batches) and the size of samples are then elements of the structural reliability.

6.5 Geometrical quantities

Geometrical quantities describe the shape, size and overall arrangement of structures, structural elements and cross-sections. In design, account should be taken of the possible variation in geometrical quantities. The magnitudes of such variations are determined by the conditions concerning the level of workmanship at manufacturing and erection on the site.

The variability of most of the geometrical quantities can be considered small or negligible in comparison with the variability associated with actions and material properties. Such geometrical quantities can be assumed to be non-random and as specified on the drawings.

Where the deviation of certain geometrical quantities from specified values may have a significant effect on the behaviour and resistance of a structure, the geometrical quantities should be considered either explicitly as random variables or implicitly in the models for actions or structural properties.

NOTE — As a design assumption, it should be considered that the tolerance limits will be exceeded in only a small proportion of cases.

Many geometrical quantities, even those considered non-random, are substituted in verifications by idealized simplified values (e.g. effective spans, or effective flange widths) for modelling a structure or its environment.

Unintentional eccentricities, inclinations and curvatures affecting columns and walls are the most usual geometrical quantities to be taken into account as basic variables. They are usually specified in material-dependent codes. These imperfections are commonly defined by models consisting of simplified assumptions regarding the "imperfect shape" and by basic variables quantifying the degrees of imperfection. Where appropriate, these imperfections may be increased in order to cover the uncertainty of such models (and possibly other uncertainties such as some non-homogeneous material properties), and are then denoted as equivalent geometrical imperfections.

Tolerances specified on the basis of calculations or experience should be checked by actual measurements of structural elements.

7 Models

7.1 General

Calculation models shall describe the structure and its behaviour up to the limit state under consideration, accounting for relevant actions and environmental influences. Models should generally be regarded as simplifications which take account of decisive factors and neglect the less important ones.

One can often distinguish between:

— action models;

— structural models which give action effects (internal forces, moments, etc.);

— resistance models which give resistances corresponding to the action effects.

However, in some cases it is not possible or convenient to make this distinction, for example, if the instability or loss of equilibrium of an entire structural system is studied or if interactions between loads and structural response are of interest.

For structural models, the following response classes should be considered:

— dynamic versus static response;

— elastic versus non-elastic (plastic) response;

— geometrically linear versus geometrically nonlinear response;

— time-independent versus time-dependent behaviour (e.g. creep).

For resistance models, the following subdivisions can be made:

— local strength models, element strength models and system strength models;

— instantaneous strength models and models including cumulative effects (e.g fatigue, cumulative deflection, etc.).

The choice of which model is relevant for a certain design situation depends on the loading characteristics, the material properties and the geometry of the structure.

Calculation models should preferably be based on experimental quantitative verification of individual assumptions governing the relation between action and action effect and between action effect and resistance.

7.2 Types of models

7.2.1 Action models

A complete action model describes several properties of the action, such as its magnitude, position, direction, duration, etc. In some cases there is an interaction between the different properties which should be considered. Sometimes interactions between actions and the response of the structure occur (e.g. for certain wind oscillations and soil structure interaction) which should also be taken into account.

The magnitude, F, of an action can generally be modelled according to the symbolic expression

$$F = \phi\,(F_0, \omega) \qquad \ldots (3)$$

where

$\phi(\ )$ is an appropriate function;

F_0 is a basic action variable, often with time- and space-dependent variations (random or non-random) and is generally independent of the structure;

ω is a random or non-random variable which may depend on the structural properties and which transforms F_0 to the action F.

For example, the variable F_0 can be defined:

— in the case of selfweight, by the dimensions and the mass density;

— in the case of snow load, by the snow load on the ground;

— in the case of wind load, by the reference velocity at 10 m above the ground.

The variable ω can be defined:

— in the case of snow load, by the conversion factor which transforms the snow load on the ground to the snow load on roofs;

— in the case of wind load, by a variable in the velocity/pressure relationship.

The details of the action model that is required for the design depend on the type of analysis to be carried out. In the case of a static analysis without time-dependent or cumulative effects, normally only the maximum and minimum values occurring during some reference period are of importance. Only if several time-variable actions have to be combined will a more detailed description prove necessary.

When dynamic behaviour is of importance, a more detailed description of the process may be required. The dynamic action model shall describe the time variation of the action in a way that is sufficiently detailed and precise to give reasonably accurate calculation results. The description of the action may be given either in the time domain or the frequency domain, whichever is the more convenient. Uncertainties in the action history can be dealt with by describing the action as a non-random function of time having a selected number of random parameters, or as a random process. Random processes are often piece-wise stationary.

In some cases dynamic actions may depend on the material properties and stiffness of the structure, as for instance in the case of collisions. In such cases one might want to specify the circumstances (masses, initial velocities) rather than giving action values. However, based on upper boundary considerations (e.g. by assuming a stiff structure), one might translate these circumstances into equivalent static actions.

In many cases it is not always possible to choose beforehand numerical values of the action parameters in such a way that the final result will be on the safe side. Therefore, if the action parameters cannot be very well defined, it may be necessary to perform several calculations with different assumptions concerning the action model.

If an action causes considerable fatigue in a structure, it is necessary to describe the action effect (local stress) in terms of one of the following characteristics:

— a complete history of stress fluctuations, often in statistical terms; or

— a set of stress cycle specifications and the corresponding number of cycles.

Uncertainties concerning the magnitude of these actions should be considered in the same way as for other kinds of variable actions.

NOTE — More details about action models are given in annex F.

7.2.2 Models describing the geometrical properties of the structure

A structure can generally be described by a model consisting of one-dimensional elements (beams, columns, cables, arches, etc.), two-dimensional elements (slabs, walls, shells, etc.) and three-dimensional elements.

The geometrical quantities which are included in the model generally refer to nominal values, i.e. the values given in drawings, descriptions, etc. Normally, the geometrical quantities of a real structure differ from their nominal values, i.e. the structure has geometrical imperfections. If the structural behaviour is sensitive to such imperfections, these shall be included in the model.

In many cases the deformation of a structure causes significant deviations from nominal values of geometrical quantities. If such deformations are of importance for the structural behaviour, they have to be considered in the design in principally the same way as imperfections. The effects of such deformations are generally denoted geometrically nonlinear or second-order effects.

7.2.3 Models describing material properties and static response

In almost all design calculations some assumptions concerning the relation between forces or moments and deformations (or deformation rates) are necessary. These assumptions can vary and depend on the purpose and type of calculation. The most general relationship assumed in design follows elastic behaviour under low action effects (when the overall structural response is considered to be elastic), developing into plastic behaviour in certain parts of the structure at high action effects. In other parts of the structure, intermediate stages occur. Such relationships may be used generally. However the use of any theory taking into account in-elastic or post-critical behaviour may have to take into account repetitions of variable actions.

The theory of elasticity may be regarded as a simplification of a more general theory and may generally be used provided that forces and moments are limited to those values for which the behaviour of the structure is still considered as elastic. However, the theory of elasticity may also be used in other cases if it is applied as a conservative approximation.

Theories in which fully developed plasticity is assumed to occur in certain zones of the structure (plastic hinges in beams, yield lines in slabs, etc.) may also be used, provided that the deformations which are needed to ensure plastic behaviour occur before the ultimate limit state is reached. A second condition is that the actions influencing these deformations should not be repeated frequently. Thus theory of plasticity should be used with care to determine the load-carrying capacity of a structure, if this capacity is limited by:

— brittle failure, or

— failure due to instability.

In cases when action effect models and resistance models are applied separately in design calculations, both these kinds of models should in principle be mutually consistent. However, in many cases this principle may be modified or simplified. Thus, for example, a bending moment (action effect) in a continuous beam may be calculated according to the theory of elasticity and the resistance according to a theory of plasticity. In other cases, especially for second-order and other non-linear effects, such calculations cannot be applied unless special precautions are taken.

7.2.4 Models for dynamic response

In most cases the dynamic response of a structure is caused by a rapid variation of the magnitude, position or direction of an action. However, a sudden change (decrease) of the stiffness or resistance of a structural element may also cause dynamic behaviour. Thus, for example, the removal of a structural element mentioned in 4.3 d) may produce dynamic effects.

Dynamic analysis can be performed in the time domain and in the frequency domain. If the load is described in statistical terms, it is also the statistical description of the response that is looked for. Based on this description, one may calculate the probability of having some limit state being exceeded in a given reference period.

Structural properties may be time-dependent as well as time-independent. In a full probabilistic analysis, these effects will be taken into account.

The models for dynamic analysis consists of:

— a stiffness model,

— a damping model, and

— an inertia model.

The stiffness model is principally the same as for static analysis. Due to dynamic influences there might be an increase in stiffness; although repetitions may also cause deterioration and a decrease in stiffness. For nonlinear material models, there is normally a strain rate dependent increase in yield strength.

Inertia forces result from acceleration of the mass of the structure, non-structural masses and the added mass of the surrounding fluid, air or soil. These additional mass contributions originate from interactions of the structure with its environment. It may be necessary to perform dynamic analysis, considering different mass contributions.

Damping may be the result of many different types of mechanism. The most important mechanisms are:

— material damping, for instance from elastic nature or from plastic material behaviour;

— damping due to friction in connections;

— damping due to nonstructural members;

— geometrical damping;

— material damping in the soil;

— aerodynamic and hydrodynamic damping.

The latter mechanisms are again examples of interaction between the structure and the environment. In particular cases these damping terms may also be negative, leading to an energy flux from the environment to the structure. Examples are galloping, flutter and, to some extent, vortex-induced response.

A particular example of the first category mentioned above is concerned with the dynamic response in severe earthquakes. In this case it may be necessary to take into account cyclic degradation and corresponding hysteric energy dissipation.

Practical design will not always require a full dynamic analysis, even if important dynamic effects are present. For many cases it is possible to make simplifications. The most common procedure is to calculate the quasi-static response and to multiply this by a dynamic amplification factor, which is a function of the dominating natural frequency and the relative damping. For special classes of buildings, even further simplifications are possible.

7.2.5 Models for fatigue

If a structure is subjected to such kinds of actions which may cause fatigue, it shall be verified that the reliability with respect to fatigue is sufficient. The models which can be used for the calculation of the resistance with regard to fatigue are strongly dependent on the type of materials in the structure. This means that no general rules on such models can be given. In many cases the models can be based on empirically known relations between the resistance and the number of load cycles, or on fracture mechanics. Due regard should be given to the effects of inspection and maintenance.

NOTE — See annex C.

7.3 Model uncertainties

A calculation model is a physically based or empirical relation between relevant variables, which are in general random variables:

$$Y = f(X_1, X_2, \dots X_n) \qquad \dots (4)$$

where

Y is the prediction by the model;

$f(\ )$ is the model function;

X_i are the basic variables.

The model $f(\ )$ may be complete and exact, so that, if the values of X_i are known in a particular experiment (from measurements), the outcome Y can be predicted without error. In most cases the model will be incomplete and inexact. This may be the result of lack of knowledge, or a deliberate simplification of the model for the convenience of the designer. The real outcome Y' of the experiment can be written as:

$$Y' = f'(X_1 \dots X_n,\ \theta_1 \dots \theta_m) \qquad \dots (5)$$

θ_i are referred to as parameters which contain the model uncertainties and are treated as random variables. Their statistical properties can in most cases be derived from experiments or observations. For resistance models the mean of these parameters should in principle be determined in such a way that, on average, the calculation model correctly predicts the test results.

NOTE — For further information see annex D.

In most cases, models specially formulated for design purposes are based on assumptions (normally on the safe side) which do not reflect the conditions occurring in reality. In such cases the evaluation of model uncertainties according to the principles described above should be taken into account. An example of such an assumption would be neglecting the tensile strength of concrete in calculating the bending resistance of a reinforced concrete beam.

NOTE — The lower estimate of resistance does not always lead to the safe side. For example, verification of shear failure is preferably carried out by the higher estimate of bending strengths of both ends of the member.

7.4 Design based on experimental models

In those cases where no adequate calculation model is available, part of the design procedure may be performed on the basis of experimental models. The setup and evaluation of the tests should be performed in such a way that the structure, as designed, has at least the same reliability with respect to all relevant limit states and load conditions as structures designed on the basis of calculation models only. Conditions which are not met during the test (e.g. long-term behaviour) should be taken into account separately.

Experimental models may be used to evaluate:

— loads on the structure (e.g. wind tunnel tests);

— structural response under loading or accidental event;

— strength or stiffness of a structure or structural element.

NOTE — Checks on material properties or other control tests are not considered as design based on experimental models.

Before testing, one should set up, as far as possible, a calculation model covering the relevant range of the variables and clearly indicate the unknown coefficients or quantities that should be evaluated from the tests. If this is not possible, a set of preliminary tests should be carried out.

Relevant basic variables such as actions, material properties and geometrical properties, even when not explicitly present in the calculation model, should preferably be measured directly or indirectly for every test. If the values of the random variables are measured, the sample need not necessarily be representative; in these cases one may, for instance, select procedures to attain values in the vicinity of the estimated design value. If the values of the random variables in the test are not measured, one should ensure that they follow from a representative sample.

The evaluation of test results should be done on the basis of statistical methods. In principle the tests should lead to a probability distribution for the selected unknown quantities, including the statistical uncertainties. Based on this distribution, one may derive design values and partial factors to be used in the partial factors format.

NOTE — Further details are given in annex D.

Where the evaluation of tests gives results which are incompatible with experience, the detailed reasons for deviation should be looked for and recorded.

8 Principles of probability-based design

8.1 General

It is assumed in this clause that the basic variables (see 6.1) are considered as random variables and are treated with probabilistic procedures.

Such procedures give, if the structure and the loads are given, a well-defined probabilistic measure of reliability (e.g. probability of failure). In the majority of cases this value should be considered only as a reference value. However, the value can be used for consistent comparisons between various design situations and hence for calibrations with regard to a specified degree of reliability. The degree of reliability can be differentiated according to the consequences of failure as indicated in 4.2.

A probabilistic design means that a structure is designed so that, for example, the probability of failure, p_f, does not exceed a specified value, p_{fs}, over some specified period of time:

$$p_f \leq p_{fs} \qquad \text{. . . (6)}$$

Failure is associated with the transition of a limit state from the desired state to the undesired state according to clause 5. With reference to equations (1) and (2), the undesired state is defined by the limit state function:

$$g(\underline{X}) < 0 \qquad \text{. . . (7)}$$

where $\underline{X}$ are the basic variables which are relevant to the problem.

In general the basic variables which describe variable actions and environmental influences should be described with the aid of random processes. In many cases, however, a description as a random variable with a probability distribution function for the maximum within a given reference period may be sufficient. Other basic variables (such as material subject to corrosion) may also be time dependent.

The model uncertainty parameters, θ, according to 7.3, are treated as random variables in principle in the same way as the basic variables.

For most ultimate limit states and for some serviceability limit states the probability of failure can be written:

$$p_f = p\,[\,g(\underline{X}) < 0\,] \qquad \text{. . . (8)}$$

In the case of time-dependent variables, the minimum of $g(\underline{X})$ with respect to time should be considered.

For some special ultimate limit states and for many serviceability limit states, the first excess of a limit state does not mean failure. In such cases failure occurs, according to 5.1, only if some additional conditions apply and the failure criteria have to be formulated for each particular case.

Due to the dependence upon time, p_f should be referred to a certain *a priori* specified period of time, the reference period. Lifetime probabilities may be used if economic consequences are determining. If a failure can be expected to endanger people, other reference periods might be used.

The failure probability, p_f, may be substituted by a reliability index, β, through the definition:

$$\beta = -\Phi^{-1}(p_f) \qquad \text{. . . (9)}$$

where Φ^{-1} is the inverse standardized normal distribution.

NOTE — Equation (9) is a definition. Nothing is said about the accuracy of p_f and β. For further discussion see annex E.

The probabilistic method may primarily be applied to calibrate the partial factors format outlined in clause 9. Under the special circumstances described in 8.5, the probabilistic method may be applied in a direct design to a specified degree of reliability.

8.2 Systems reliability versus element reliability

From a probabilistic point of view, an element can be considered to have one single dominating failure mode. A system may have more than one failure mode and/or consist of two or more elements, each one with a single failure mode.

Probabilistic structural design is primarily applied to element behaviour and limit states (serviceability — and ultimate failure). Systems behaviour is of concern because systems failure is usually the most serious consequence of structural failure. It is therefore of interest to assess the likelihood of systems failure following an initial element failure. In particular, it is necessary to determine the systems characteristics in relation to damage tolerance or structural integrity with respect to accidental events. The element reliability requirements should depend upon the systems characteristics.

A systems analysis should therefore be carried out to establish:

— the redundancy (alternative load-carrying paths);

— the state and complexity of the structure (multiple-failure modes).

NOTE — Systems reliability analysis should, however, be carried out with due recognition of the uncertainties inherent in the methods currently available and should therefore be used with caution.

8.3 Specified degrees of required reliability

Specified maximum acceptable failure probabilities should depend on the consequence and the nature of failure, the economic losses, the social inconvenience, and the amount of expense and effort required to reduce the probability of failure. They should be calibrated against well-established cases that are known from past experience to have adequate reliability. Hence, the specified failure probability should depend on the reliability class (see 4.2).

The specified failure probabilities, p_{fs}, which are relevant for ultimate and serviceability limit state design, should reflect the fact that criteria for such limit states do not account for gross errors. These probabilities are not directly related to the observed failure rate, which is mainly caused by gross errors.

When dealing with time-dependent structural properties, the effect of the inspection and repair procedures on the probability of failure should be taken into account. This may lead to adjustments to specified values, conditional upon the results of inspections. Specified failure probabilities should always be considered in relation to the adopted calculation and probabilistic models and the method of assessment of the degree of reliability.

Specified failure probabilities should always be defined for some reference period. Depending on the type of limit state, this could be the design working life, a period of one year or an arbitrary point in time.

For reversible serviceability limit states, there may also be requirements on the frequency rate of passing the limit state (see 5.1.3).

NOTE — For further information, see annex E.

8.4 Calculation of failure probabilities

8.4.1 General

An important special case of the failure function (8.2) where all variables X are time-invariant is considered (see 8.4.2). In this case the variables X are random variables, and not random processes.

When the reliability problem is time-dependent, it is often possible to transform it to a time-invariant one in terms of random variables. See 8.4.3.

8.4.2 Time-invariant reliability problems

Three types of method may in general be used to compute p_f when X are time-invariant variables, namely:

a) analytical methods, for example, FORM/SORM (First/Second Order Reliability Methods),

b) Monte Carlo simulation, and

c) numerical integration.

8.4.3 Transformation of time-variant into time-invariant problems

Two classes of time-dependent problems are envisaged, namely those associated with

— overload failure, and

— cumulative failure.

In the case of overload failure, a single action process may be replaced by a random variable with a mean value equal to its expected maximum value over a chosen reference period. If there is more than one random action process, they should be combined, taking into account the scales of fluctuation of all action processes.

NOTE 1 Further details are given in annex F.

In the case of cumulative failures (fatigue, corrosion etc.), the total history of the load up to the point of failure is of importance.

NOTE 2 Failure could be the combined result of a cumulative damage process and another load with a relatively high value.

8.5 Implementation of probability-based design

The probabilistic method may be directly applied to achieve designs with degrees of reliability close to the specified values.

Such an approach could be used contingent upon standardized

— uncertainty measures,

— reliability methods.

Instead of using a direct probabilistic method, the two following simplifications may be used:

a) design value method, and

b) partial factor method.

In both cases the methods are calibrated in such a way that for certain ranges of structural layouts, actions, etc., a design is obtained that is sufficiently close to the design obtained by the direct probabilistic method.

NOTE — Design value methods and code calibration are outlined in annex E. The partial factors format is dealt with in clause 9.

9 Partial factors format

9.1 Design conditions and design values

The format of partial factors separates the influences of uncertainties and variabilities originating from different causes by means of design values assigned to basic variables. With reference to 5.2.1, the design condition is expressed in terms of design values, e.g. as:

$$g(F_d, f_d, a_d, \theta_d, C, \gamma_n) \geq 0 \qquad \text{. . . (10)}$$

where

F_d are design values of actions;

f_d are design values of material properties;

a_d are design values of geometrical quantities;

θ_d are design values of the variables θ which account for the model uncertainties according to equation (5);

C are serviceability constraints;

γ_n is a coefficient by which the importance of the structure and the consequences of failure, including the significance of the type of failure, are taken into account. The value γ_n could be made dependent on the specified degree of reliability of the actual structure or structural element.

Equation (10) should be regarded only as a symbolic description of the principles. Each symbol in equation (10) may represent a single variable or a vector containing several variables.

The basic variables are separated into:

— primary basic variables, and

— other basic variables.

The primary basic variables are those whose values are of primary importance for the design results. They should be specified in those codes which treat actions and structures of specific materials.

NOTE — For the ultimate limit state of a prestressed concrete beam, for example, the strengths of concrete and steel are primary basic variables but the moduli of elasticity are non-primary basic variables. Actions are normally primary basic variables.

The design values of the primary basic variables F, f, a and θ are obtained in the following way:

$$F_d = \gamma_f \; F_r \qquad \text{. . . (11)}$$

$$f_d = \frac{f_k}{\gamma_m} \qquad \text{. . . (12)}$$

$$a_d = a_k \pm \Delta a \qquad \text{. . . (13)}$$

$$\theta_d = \gamma_D \text{ or } 1/\gamma_D \qquad \text{. . . (14)}$$

where

F_r are the representative values of actions (see 9.2);

f_k are characteristic values of material properties (see 9.3);

a_k are characteristic values of geometrical quantities (see 9.4);

γ_f are partial factors for actions;

γ_m are partial factors for materials;

Δa are additive geometrical quantities;

γ_D are partial factors for model uncertainties.

γ_f takes account of:

— the possibility of unfavourable deviations of the action values from the representative values, and

— the uncertainty in the action model.

γ_m takes account of:

— the possibility of unfavourable deviations of the material properties from the characteristic values, and

— uncertainties in the conversion factors.

Δa takes account of:

— the possibility of unfavourable deviations of the geometrical parameters from the characteristic (specified) values including the importance of variations in a, the tolerance specifications for a and the control of the deviations from a; and

— the cumulative effect of a simultaneous occurrence of several geometrical deviations.

γ_D takes account of uncertainties of models as far as can be found from measurements or comparative calculations.

For basic variables other than the primary ones, the partial factors are, *a priori*, set to unity and the additive quantities to zero; i.e. the design values are equal to the characteristic values. In some cases mean values may be used.

The partial factors for actions may include the effect of uncertainties of an action effect model. In a similar way the partial factors for resistance may include the effect of uncertainties in the geometrical parameters and in the resistance models. In such cases the notations γ_f and γ_m should be substituted by γ_F and γ_M respectively.

The values of the partial factors depend on the design situation and the limit state considered.

If the deformation capacity is governing the design, equation (10) has to be given a different form and part of the variables have to be substituted by other kinds of variables. This may, for example, be the case in design for seismic situations.

9.2 Representative values of actions

A **permanent action** has often a unique characteristic value. When the action refers to the selfweight of the structure, the value G_k should be obtained from the specified values of geometrical quantities and the mean unit mass of the material. However, in some cases it may be preferable to define two values, one upper and one lower characteristic value of a permanent action.

A **variable action** has the following representative values:

— the characteristic value Q_k,

— the combination valuey $\Psi_0 Q_k$

— the frequent value $\Psi_1 Q_k$

— the quasi-permanent value $\Psi_2 Q_k$

The **characteristic value** is chosen so that it can be considered to have a specified probability of being exceeded towards unfavourable values during a chosen reference period.

The **combination values** are 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.

The **frequent value** is determined so that:

— the total time, within a chosen period of time, during which it is exceeded, is only a small given part of the chosen period of time; or

— the frequency of its excess is limited to a given small value.

NOTE — There may in some cases be two or more different frequent values for the same load associated with different design situations.

The **quasi-permanent** value is determined so that the total time, within a chosen period of time during which it is exceeded, is of the magnitude of half the chosen period.

An **accidental action** may have a unique characteristic value A_k

9.3 Characteristic values of properties of materials including soils

Properties of materials are defined for some relevant volume of material and are represented by their characteristic values. For a produced material, the characteristic value should in principle be presented as an *a priori* specified fractile of the statistical distribution of the material property being supplied, produced within the scope of the relevant material standard. For soils and existing structures, the values should be estimated according to the same principle and so that they are representative of the actual volume of soil or the actual part of the existing structure to be considered in the design.

9.4 Characteristic values of geometrical quantities

For geometrical quantities, the characteristic values a_k usually correspond to dimensions specified by the designer.

9.5 Load cases and load combinations

Load cases are the specific spatial arrangements of free actions which (together with the fixed actions) are introduced into calculations.

The free actions should be arranged so that they produce the most unfavourable effect on the structure for the limit state considered.

A combination of actions is a set of design values used for the verification of the structural reliability for a limit state under the simultaneous influence of different actions.

The basic principle of combination of actions is the following:

— one or a few actions are considered as dominating and are introduced into the combination with extreme design values;

— all other actions are introduced with more likely values.

NOTE — In annex G an example is given of a system for combination of actions which is based on these principles but developed in more detail.

Actions which cannot occur simultaneously (e.g. because of physical reasons) should not enter together into a combination.

9.6 Action effects and resistances

In many cases the basic variables and the factors θ which describe the uncertainties of the calculation models can be separated into groups so that some groups give action effects:

$$S(F, f, a, \theta_S)$$

and other groups give resistances:

$$R(F, f, a, \theta_R)$$

In the expression for S, the material properties, f is a primary basic variable only in special cases, for example, calculations according to a second-order theory. In the expressions for R, the actions F are of importance only in very special cases.

Thus design values, S_d and R_d, can be defined as:

$$S_d = S(F_d, f_d, a_d, \theta_{Sd}) \qquad \text{. . . (15)}$$

$$R_d = R(F_d, f_d, a_d, \theta_{Rd}) \qquad \text{. . . (16)}$$

Then equation (10) can be written

$$g(S_d, R_d) \geq 0 \qquad \text{. . . (17)}$$

As for equation (10), equation (17) should be regarded only as a symbolic description. Each symbol S and R may represent several action effects and resistances respectively.

In the simplest case, equation (17) can be written

$$R_d \geq S_d \quad \text{. . . (18)}$$

Equations (17) and (18) can be applied in the ultimate limit state and the serviceability limit state. For the serviceability limit state, concerning, for example, deflections, the design condition is often of the type

$$S_d \leq C \quad \text{. . . (19)}$$

where C is a serviceability constraint, for example, acceptable deflection.

9.7 Verification for fatigue

See annex C.

9.8 Calibration

γ, Ψ and ξ values should be calibrated either by direct comparison or by probabilistic methods, in both cases combined with judgement.

10 Assessment of existing structures

10.1 Relevant cases

An existing structure shall be subjected to the assessment of its actual reliability when one or more of the following actions are to be taken:

a) rehabilitation of an existing constructed facility during which new structural members are added to the existing load-carrying system;

b) adequacy checking in order to establish whether the existing structure can resist loads associated with the anticipated change in use of the facility, operational changes or extension of its design working life;

c) repair of an existing structure which has deteriorated due to time-dependent environmental effects or has suffered damage from accidental actions (e.g. an earthquake);

d) where the reliability of the structure is in doubt (e.g. for earthquakes).

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

10.2 Principles of assessment

Analysis and design during the assessment of an existing structure shall be based on the general principles outlined in clauses 1 to 9. Historic codes valid in the period when the original structure was designed, based on other principles, should be used only as guidance documents.

The assessment need not be performed for those parts of the existing structure which will not be affected by structural changes, rehabilitation, repair, change in use, or which are not obviously damaged or are not suspected of having insufficient reliability.

10.3 Basic variables

For the reliability requirements, the values of basic variables shall be taken as follows.

a) Dimensions of the structural elements: when the original design documents are available and no change in dimensions has taken place or other evidence of deviations are present, the nominal dimensions in accordance with the original design documents should be used in the analysis. These dimensions shall be verified by inspection to an adequate extent.

b) Load characteristics shall be introduced with values corresponding to the actual situation. When overloading has been observed in the past, it may be appropriate to increase representative values. When some loads have been reduced or removed completely, the representative values of the load magnitudes can be appropriately reduced and/or the partial factors can be adjusted.

c) Material properties shall be considered according to the actual state of the structure; when the original design documents are available and no serious deterioration, design errors or construction errors are suspected, the characteristic value in accordance with the original design should be used. If appropriate, destructive or non-destructive inspections should be performed and evaluated using statistical methods.

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

10.4 Investigation

An investigation is intended to update the knowledge about the present condition (state) of a structure with respect to a number of aspects. Often, the first impression of the condition of a structure will be based on a qualitative inspection. The description of possible damage of the structure will be in verbal terms such as: “none, minor, moderate, severe, destructive, unknown”. Very often decisions based on such observation will be made in a purely intuitive way by experts. A better judgement of the structure can be made on the basis of quantitative inspections. These should result in a set of values which characterize the properties or condition of the structural elements. For all inspection techniques one should have information on the probability of detecting some damage if present and the accuracy of the results.

A special type of investigation is proof loading. Based on such tests, one may draw conclusions with respect to:

— the bearing capacity of the tested member under the test load condition;

— other members;

— other load conditions;

— the behaviour of the system.

The inference in the first case is relatively easy; the probability density function of the load-bearing capacity is simply cut off at the value of the proof load. The inference for the other conclusions is more complex. Note that the number of proof load tests need not to be restricted to one. One might decide to test one member under various loading conditions and/or a sample of structural elements. In order to avoid unnecessary damage to the structure due to the proof load, it is recommended to increase the load gradually and to measure the deformations. Measurements may also give a better insight into the behaviour of the system. Proof loads can in general not deal with duration effects. These effects should be compensated for by calculation.

Given the result of an investigation, there is a need to update the properties and reliability estimates of the structure. Two different routes can be distinguished:

a) by updating the multivariate probability distribution of the individual variables; this method can be used to derive updated design values to be used in the partial factors format and for comparing action effects directly with limit values (cracks, displacements);

b) by a formal updating of the structural failure probability.

In principle the result of all observations (qualitative inspection, calculations, quantitative inspection, proof loading) should be treated in one of these two ways.

10.5 Assessment in the case of damage

In the case of the assessment of a damaged structure, the following stepwise procedure is recommended.

10.5.1 Visual inspection

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

10.5.2 Explanation of the observed phenomena

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

10.5.3 Reliability assessment

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

10.5.4 Additional information

If the reliability according to 10.5.3 is insufficient, one may look for additional information from more advanced structural models, additional inspections and measurements or actual load assessment. The updating techniques about how to use this information is discussed in 10.4.

10.5.5 Final decision

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

a) accept the present situation for economical reasons;

b) reduce the loads on the structure;

c) repair the building;

d) start demolition of the structure.

NOTE — Solution a) can be motivated by the fact that the costs for additional reliability are much higher for existing structures than for structures under design. This justifies the choice implying that higher reliability is required for a structure to be designed. However, if human safety is involved, limits to economical optimization should be set.

Effectively lower acceptance levels can be set by reducing specified β values for probabilistic design and reducing γ values in the partial factors format.

Annex A
(informative)

Quality management and quality assurance

A.1 Objectives

The objective of this annex is to provide general guidance for the implementation of a quality management system for construction works, and in particular for the application of the ISO 9000 series.

In general the construction works should:

a) meet defined needs, uses or purposes;

b) satisfy client expectations;

c) comply with applicable standards and specifications; and

d) comply with statutory (and other) requirements of society.

A.2 Definitions

A.2.1 customer: Recipient of a construction works in a contractual situation.

A.2.2 quality: Totality of the characteristics of an entity (e.g. construction works) that bear on its ability to satisfy stated and implied needs (i.e. all kinds of explicit or implicit requirements).

A.2.3 requirements for quality: Expression of the needs or their translation into a set of quantitatively or qualitatively stated requirements for the characteristics of an entity to enable its realization and examination.

A.2.4 conformity: Fulfilment of specified requirements.

A.2.5 quality policy: Overall intentions and direction of an organization (i.e. contractors, customer) with regard to quality, as formally expressed by top management.

A.2.6 quality management: All activities of the overall management function that determine the quality policy, objectives and responsibilities and implement them within the quality system.

A.2.7 quality loop: A conceptual model of interacting activities that influence quality at the various stages ranging from the identification of needs to the assessment of whether these needs have been satisfied.

A.2.8 quality control: Operational techniques and activities that are used to fulfil requirements for quality.

A.2.9 quality assurance: All the planned and systematic actions necessary to provide adequate confidence that an entity will fulfil the requirements for quality.

A.2.10 quality plan: Document setting out the specific quality practices, resources and sequence of activities relevant to a particular product, project or contract.

A.2.11 process: A set of interrelated resources and activities which transform inputs into outputs.

A.2.12 procedure: A specified way to perform an activity.

A.3 Quality management

Managing design quality implies that the following actions should be taken.

a) The various reliability aspects of quality are identified (e.g. structural safety, fitness for use, comfort, durability, aesthetics, cost, etc.).

b) These aspects are transformed into a set of requirements for quality (e.g. functional characteristics, thermal characteristics, structural safety, serviceability and robustness criteria, design working life, cost, etc.).

c) The main activities that contribute to obtaining the quality are identified (e.g. preliminary investigations, conceptual options, design situations, characteristics of actions, characteristics of materials, level of workmanship, limits of use, principles of maintenance). The various activities of the life cycle of the construction works that influence quality are identified. These activities can be interpreted as the quality loop of the construction works (see table A.1).

d) The considered activities are controlled by the management of the involved organizations.

Table A.1 can be considered to be a basis for preparing the quality plan.

A.4 Quality assurance

To achieve an adequate confidence that the design fulfils the specified requirements for quality, supplementary actions should be taken:

— the main factors intervening into the fulfilment of the specified requirements for quality should be considered in a quality plan (see table A.1);

— documents related to the control of factors that contribute to quality should be compiled and retained throughout the course of the life cycle of construction works.

Table A.1 — Quality management activities in the quality loop for construction works

Stages of the life cycle	Activities
Conception	• Establishing appropriate levels of performance for construction works and components • Specification for design • Specification for suppliers • Preliminary specifications for execution and maintenance • Choice of intervening parties with appropriate qualifications for personnel and organization
Design	• Specification of performance criteria for materials, components and assemblies • Confirming acceptability and achievability of performance • Specification of test options (prototype, *in situ*, etc.) • Specification for materials
Tendering	• Reviewing design documents, including performance specifications • Acceptance of requirements (contractor) • Acceptance of tender (customer)
Execution	• Control of procedures and processes • Sampling and testing • Correction of deficiencies • Certification of work according to compliance tests specified in the design documentation

Stages of the life cycle	Activities
Completion of the construction works and handover to the customer	• Commissioning • Verification of performance of completed building (e.g. by testing under operational loads)
Use and maintenance	• Monitoring performance • Inspection for deterioration or distress • Investigation of problems • Certification of work
Rehabilitation or demolition	Similar to above NOTES 1 Rehabilitation is not mandatory. 2 Demolition is outside the scope of this International Standard.

A.5 Quality control

A.5.1 General

Quality control consists of:

— collection of information;

— judgement based on this information;

— decision based on the judgement.

A.5.2 Control procedure

Regording the control procedure within manufacturing and construction, a distinction can be made between:

— production control, which is control of a production process; the purpose of this control is to steer a production process and to guarantee an acceptable result;

— compliance control, which is control of the result of a production process; the purpose of this control is to ensure that the result of a production process complies with the given specification.

A.5.3 Control criteria and acceptance rules

Control can be total or statistical. If the control is total, every produced unit is inspected. The acceptance rules imply that a unit is judged as being good (accepted) or bad (not accepted). Normally the criteria, if they are quantitative, refer to given tolerances.

A statistical control procedure generally consists of the following parts:

— batching the products;

— sampling within each batch;

— testing the samples;

— statistical judgement of the results;

— decision regarding acceptance.

A batch should be such that it may be regarded as homogeneous with regard to the properties which are the subject of the control. The judgement of the results should normally be made with regard to a given level of confidence and/or a given interval of confidence, or by applying Bayesian techniques.

A.5.4 Control process

Distinction can be made between the following different control steps depending on the person or organization supervising the control:

— individual self-checking;

— internal control;

— acceptance control handled by the project management.

There often exists an additional control, such as that initiated and executed by the public authority and based on building laws and/or codes.

Internal control is executed in the same office, factory or workshop where the work which is the object of the control is carried out. However, the work and the control are executed by separate bodies.

If a control process consists of several steps, it is important for the final result that the activities of these steps, as for as possible, are mutually independent, in a statistical sense, otherwise the efficiency of the control will decrease.

In many cases, it is necessary to set up a control plan which is part of the quality plan according to A.4.

Annex B
(informative)

Examples of permanent, variable and accidental actions

NOTE — The following examples include the most common types of actions. In certain cases there may be other types; such actions should be classified according to the basic definitions.

B.1 Permanent actions

These include the following:

a) the mass of the structures themselves (except possibly certain parts of this mass during certain phases of construction);

b) the mass of superstructures, including any permanent formwork or fixtures;

c) the forces applied by earth pressure, resulting from the mass of the soil at their final values;

d) the deformations imposed by the mode of construction of the structure at their final values;

e) the actions resulting from shrinkage of concrete and from welding;

f) the forces resulting from water pressure, where appropriate;

g) the actions resulting from support settlements and mining subsidence;

h) prestressing forces.

B.2 Variable actions

These include the following:

a) loads due to use and occupancy, imposed loads;

b) certain parts of the mass of structures themselves during certain phases of construction;

c) erection loads;

d) all moving loads and their effects;

e) wind actions;

f) snow loads;

g) ice formation;

h) earthquake actions[1)];

i) the effects of variable level of water surface, where appropriate;

j) temperature changes;

k) wave loads.

1) Earthquakes may be considered as either a variable action under specified conditions or an accidental action.

B.3 Accidental actions

These include the following:

a) collisions;

b) explosions;

c) subsidence of subsoil;

d) tornadoes in regions not normally exposed to them;

e) earthquake actions[1];

f) fire;

g) extreme erosion.

Annex C
(informative)

Models for fatigue

C.1 Introduction

In structures loaded by fluctuating actions, fatigue failures may occur at load levels which are significantly lower than the load levels at which failure may normally be expected. When such a fatigue failure is due to crack growth phenomena, the complete mechanism consists of three stages:

a) an initiation phase, in which cracks are formed;

b) a crack growth phase, in which stable crack growth takes place during every load cycle;

c) a failure phase, in which unstable crack growth occurs due to brittle fracture or ductile tearing, or in which the reduced cross-section fails due to general yielding.

When, during the crack growth phase, large alternating plastic zones are present, failure occurs after relatively few cycles and the mechanism is referred to as low cycle fatigue. When the plastic zones are small, the mechanism is called high cycle fatigue.

Two main methods of analysis can be distinguished.

— the S-N line approach, and

— the Fracture Mechanics approach.

Both methods will be discussed in some detail. The methods have been developed for steel structures, but the principles may also be useful for other materials.

C.2 S-N lines

The S-N line approach combines all three phases of the fatigue mechanism and is completely based on experiments. A number of test specimens are subjected to a series of constant amplitude load cycles until failure. Plotting the stress range, S, against the number of cycles at failure, N, gives the S-N line. The S-N line may or may not depend on the mean stress level. In order to deal with a realistic variable amplitude loading on a structure, a damage accumulative rule has to be applied. The most widely used is the linear damage rule of Palmgren-Miner. According to this rule, failure occurs if:

$$\sum \frac{n_i}{N_i} \geqslant D_c \qquad \cdots (C.1)$$

where

n_i is the number of applied load cycles with stress range level S_i;

N_i is the number of load cycles at failure for stress range level S_i;

D_c is the critical value for the damage ratio.

The stress range S_i is assumed to include the effects of local stress concentrations (e.g. at weld tips).

In order to find the number of stresses, n_i, for each stress range level, S_i, special counting procedures (e.g. rainflow counting) may be necessary. The Palmgren-Miner damage rule does not take sequence effects into account. In the ideal case, the critical value D_c equals 1,0, but in general it depends on the load history, the environment and material type.

C.3 Fracture mechanics

In the Fracture Mechanics approach, separate models are used for the three different stages.

a) The crack initiation stage is often modelled by the local strain approach; this is mainly used for small thin plated structures; in many other applications this stage can often be neglected.

b) The crack propagation stage can in many cases be governed by a crack growth model, where the crack size a_t after a time t is a function of the initial crack size, a_0, the history of the local nominal stress, $\sigma(\tau)$, and the fatigue resistance, R_f, which depends on local material properties and geometry:

$$a_t = f(a_0, \sigma(\tau), R_f) \quad (0 < \tau < t) \qquad \text{. . . (C.2)}$$

In most crack growth models, the stress history $\sigma(\tau)$ will be translated into a sequence of cycles. In addition to crack length, it may also be necessary to take crack depth and width into account.

c) The failure stage is often modelled using the concept of the critical crack size. Every stress level, $\sigma(\tau)$, can be associated with a critical crack size, $a_{crit,t}$, such that, considering all possible failure modes a stress equal to $\sigma(\tau)$ would lead to a failure if $a_t > a_{crit,t}$. The limit state function for fatigue failure can then be formulated as:

$$g(x) = \min.(a_{crit,t} - a_t) \quad (0 < t < T) \qquad \text{. . . (C.3a)}$$

or conservatively:

$$g(x) = \min.(a_{crit,t}) - \max.(a_t) \quad (0 < t < T) \qquad \text{. . . (C.3b)}$$

The minimum (maximum) should be taken for the total design working life, T.

Equations (C.3a) and (C.3b) are illustrated in figure C.1.

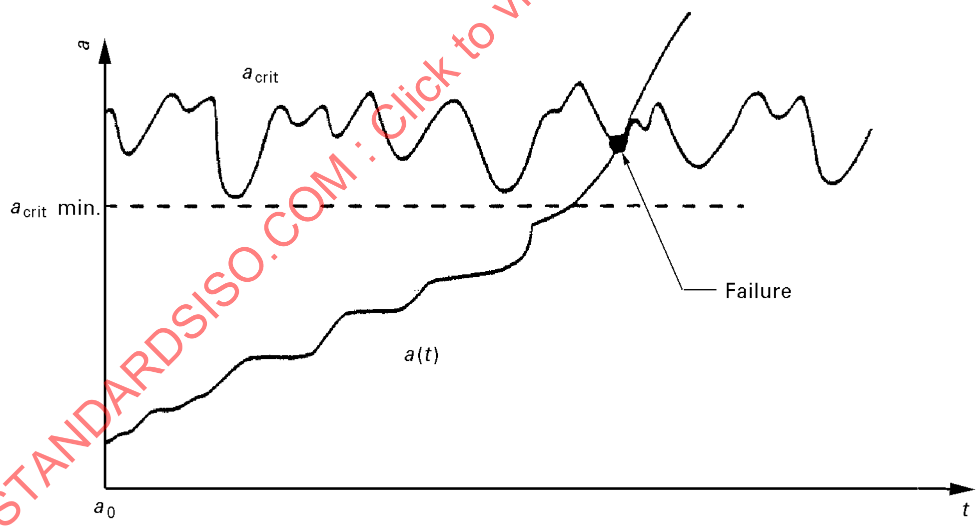


Figure C.1 — Illustration of equations (C.3a) and (C.3b)

C.4 Verification procedure in partial factor design

The safety format depends on the type of analysis.

In the case of damage accumulation methods in combination with S-N lines, the verification rule can be presented as:

$$\sum \frac{n_i}{N_i} < \frac{D_c}{\gamma_d} \qquad \text{. . . (C.4)}$$

$$N_i = N_i\left(\gamma_{\mathrm{Ff}} S_i, \frac{R_{\mathrm{fk}}}{\gamma_{\mathrm{Mf}}}\right) \qquad \text{. . . (C.5)}$$

where

n_i and S_i are the best estimates of the load history;

R_{fk} is the characteristic value of the fatigue resistance;

γ_{Ff} is the partial factor that deals with the uncertainties in the load level and load model;

γ_{Mf} is the partial factor that deals with the uncertainties in the material model;

γ_{d} is the partial factor that deals with the uncertainties in damage accumulation rule, the design working life and the consequences of failure.

For the Fracture Mechanics approach, the verification rule can be presented as follows, assuming the simplified method according to equation (C.3b):

$$a_{\mathrm{crit}}\left(\gamma_{\mathrm{F1}} S_{\mathrm{k}}, \frac{R_{\mathrm{k}}}{\gamma_{\mathrm{M1}}}\right) > a_T\left(a_{0\mathrm{k}}, \gamma_{\mathrm{F2}} S(\tau) \frac{R_{\mathrm{fk}}}{\gamma_{\mathrm{M2}}}\right) \qquad \text{. . . (C.6)}$$

where

S_{k} is the load effect due to the characteristic load or load cycle;

R_{k} is the relevant characteristic strength property (fracture toughness, yield strength) for the relevant temperature;

$a_{0\mathrm{k}}$ is the characteristic initial crack size;

$S(\tau)$ is the best estimate of the load history;

R_{fk} are characteristic values of the crack growth material properties;

γ are partial factors;

T is the design working life.

The level of the partial factors should depend on:

— the uncertainties and sensitivities of the random variables;

— the damage tolerance of the structure, that is the ability of the cracked structure to find alternative load paths;

— the inspected intervals and the probability of crack detection;

— the ability to effect repairs.

Annex D
(informative)

Design based on experimental models

D.1 Scope

Design based on experimental models (or, in short, design by testing) is a method for establishing design values of resistance properties for defined structural elements and materials. The method described in this annex is, to a large extent, based on a statistical evaluation of the test results consistent with the concept of probabilistic design and partial factor design.

The scope of application covers:

— cases which cannot be treated by the information given in the Codes of Practice because adequate theoretical models or data are lacking;

— cases which are so particular that the data commonly applied for calculation do not reflect properly the actual circumstances (e.g. due to a particular production method);

— cases when the existing design formulae seem to lead to very conservative results and a direct limit states check is expected to bring about a more economical solution;

— derivation of new design formulae.

This annex does not cover non-destructive testing, quality control tests for specific materials (e.g. soil); some further elaborations and/or restrictions may be appropriate.

D.2 General considerations

In order to establish a relevant test arrangement, the experiments should be preceded by a qualitative preliminary analysis to find out zones which might be critical for the behaviour of the considered element. Furthermore, an unambiguous definition of the limit state under consideration should be given.

Tested units should preferably by produced in the same size and by the same technology as those of the units to be produced and built-in according to the testing and in relevant situations randomly chosen for testing.

The test procedure shall not be restricted to recording only the final values. Attention is to be paid to phenomena which occur when the considered limit state is exceeded, to the accompanying circumstances and to the mechanism of this limit state, as well as to the boundary conditions (e.g. to what extent they differ from those expected in the actual structure, to the loading conditions, etc.).

Circumstances which occur when the considered limit state is exceeded, particularly the failure mode which was decisive for failure, may not always be evident. Development of the test programme and evaluation of the obtained test results require appropriate theoretical knowledge, experience in testing, and engineering judgement.

The methods used for deriving design values from the tests should take into account the (generally) limited number of tests. The evaluation can be made on the basis of a pre-existing analysis model (see D.6) or, in the absence of such a model, by direct evaluation (see D.5). In addition to these statistical considerations, it should be noted that the general theories of structural behaviour and the set of commonly accepted design rules remain valid during design by testing.

Conclusions derived from a particular investigation refer to the properties and/or production technology associated with the scope of that investigation. Extensions of the conclusions require new tests, unless and expansion of the obtained results to other element classes is possible, based on theoretical analysis.

D.3 Consideration of differences between reality and testing conditions

The conditions during testing may differ from the conditions for the intended structure in its actual environment. Such differences should be accounted for by suitably determined conversion or modification factors.

The conversion factor η should be established by experimental or theoretical analysis based on a general structural theory and/or on experience. Some degree of arbitrariness is usually unavoidable.

Influences accounted for by η could include:

— size effects;

— time effects (normally tests are performed under short-term loading, whereas the load-carrying capacity and deflections of many materials depend on long-term effects);

— boundary conditions of the tested units (free or fixed, etc.);

— humidity conditions influencing the material properties.

Workmanship conditions, for instance production according to laboratory conditions instead of actual conditions, may influence structural properties considerably (e.g. properties of joints in assembled structures). If these effects are considered to be essential, corrections should be made or specimens from actual production quality should be used.

D.4 Planning

Prior to the execution of tests, a test plan should be drawn up by the designer and the testing organization. The plan should consider the objective of the test and all specifications necessary for the selection or production of the test specimens, the execution of the tests and the test evaluation. In particular, the test plan shall deal with the following items.

a) Scope of information required from the tests (e.g. required parameters and range of validity).

b) Description of all properties and conditions which may influence the behaviour at the limit state under consideration (e.g. geometrical parameters and their tolerances, material properties, parameters influenced by fabrication and erection procedures, scale effects, environmental conditions).

c) Modes of failure and/or analysis models with the appropriate variables.

d) Measurements of relevant properties of each individual test specimen to be carried out prior to the execution of the tests; examples of these relevant basic variables are environmental influences, material properties, and geometrical quantities.

e) Specifications of the properties of the specimen (e.g. specification for dimensions, material and fabrication of prototypes, sampling procedures, restraints).

f) The number of the specimen and the sampling procedure.

NOTE 1 If an analysis model is available and the values of all random variables are measured, the sampling procedure is not relevant. In all other cases one should ensure that the test specimens are selected from a representative sample. It may be necessary to account for populations from different producers (e.g. by the use of weighting factors).

NOTE 2 A design-point oriented sample is recommended if samples are small and/or when the failure mode may change as a function of the basic variables. In general this is strongly recommended for geometrical imperfections. For strength parameters, this concept needs to be assessed with care. For instance, there may be a difference between a poor sample of concrete grade 30 and an average sample of concrete grade 20, even if both have the same cubic strength.

g) Specifications about the loading and environmental conditions in the test (e.g. loading points, loading paths in time and space, temperatures, loading by deformation or force control). Loading paths shall be selected in such a way that they are representative of the anticipated scope of application of the structural member, that they account for possible unfavourable paths, and/or that they account for those paths which are considered in the analysis of comparable cases.

NOTE 3 Where structural properties are conditional on one or several effects of actions which are not varied systematically, then these effects should be specified by their design values. When they are independent of the other parameters of the loading path, design values related to estimated load combination values may be adopted.

h) Testing arrangements (including measures to ensure sufficient strength and stiffness of the loading and supporting rigs and clearance for deflections, etc.).

i) Observation points and methods for observation and recording (e.g. time histories of displacements, velocities, accelerations, strains, forces and pressures, required frequency and accuracy of measurements and measuring devices).

D.5 Direct evaluation of the test results

D.5.1 General

In this clause it is assumed that the resistance of a structural element or the strength of a material is directly evaluated from the test. It is further assumed that the strength of a specimen can be represented by a single quantity and that the failure mechanism under consideration is the critical one for all tests.

If the results are used in connection with a probabilistic design method, the test data may be used to update a predefined prior distribution on the statistical parameters of the resistance. Guidance is given in D.5.4.

If the partial factor format is used, either the classical method of D.5.2 or the Bayesian method of D.5.3 may be applied. Both methods are used in practice. Sometimes a mixture of both methods is used. In many cases the numerical values will not differ considerably. A recommendable procedure is to evaluate the tests using both methods and compare the results. If the results are close, the choice does not matter. If they are not close, one should have very strong arguments for not selecting the most unfavourable result.

D.5.2 Partial factor design: Classical approach

In this method the design resistance R_d is to be calculated by the formula

$$R_d = \frac{R_{k,est}}{\gamma_m} \cdot \frac{\bar{\eta}}{\gamma_{Rd}} \qquad \text{. . . (D.1)}$$

where

$R_{k,est}$ is the estimate of the lower characteristic value R_k of the resistance determined statistically from tests;

γ_m is the partial factor for the material;

$\bar{\eta}$ is the mean value of the conversion coefficient or modification factor;

γ_{Rd} is the model uncertainty coefficient.

The partial factor γ_m should be specified according to the values normally used for the material and failure mode under consideration. The decision as to whether or not there is sufficient resemblance between the test pieces at hand and the validity region of the selected partial factor is a matter of engineering judgement. In those cases where the discrepancy between the tests and the standard design situations is too large to select partial factors with sufficient confidence, the method described in D.5.3 should be used.

The model uncertainty coefficient γ_{Rd} is meant to cover the randomness of η-values with respect to the unknown differences between the testing conditions and the actual conditions. The value of γ_{Rd} has to be defined primarily by the researcher, taking into account the objectives of the testing, limit state requirements, failure mode and information about the production and the site conditions. The primary γ_{Rd} value may be corrected by the designer according to his judgement about the production and the site conditions. In general, it should be $\gamma_{Rd} \geqslant 1{,}0$.

The lower characteristic value $R_{k,est}$ is estimated from the test results, taking into account a confidence level of at least 0,75. In the absence of other information, the characteristic value is assumed to be the 0,05 fractile of a normal distribution. The characteristic value is estimated by:

$$R_{k,est} = m_R - k_s s_R \quad \text{... (D.2)}$$

where

m_R is the sample mean value;

s_R is the sample standard deviation;

k_S is the coefficient depending on the sample size.

The values k_S depend on the number of tests, n, and on the chosen confidence level. Table D.1 gives k_S-values for the 0,01, 0,05 and 0,10 fractiles and a confidence level 0,75. The values in table D.1 are based on the noncentral *t*-distribution[2)].

The standard deviation s_R in equation (D.2) is to be established from the test results. In some cases the standard deviation may be considered to be known *a priori*. In that case:

$$R_{k,est} = m_R - k_\sigma s_R \quad \text{... (D.3)}$$

where

m_R is the sample mean value;

σ_R is the distribution standard deviation;

k_σ is the coefficient depending on sample size.

The value of k_σ should be taken from table D.2.

NOTES

1 In the above procedure the normal distribution is used. This assumption can be regarded as a relatively conservative one. In reality, one may also consider Lognormal or Weibull distributions. Using these distributions, one may find more economic design values. It is stressed however, that such a choice should be based on evidence from many (similar) tests. During evaluation of those tests, special attention should be given to the shape of the distribution as a whole (especially its skewness) and the lower tail in particular.

2 In this approach the statistical uncertainty is only considered in the assessment of the characteristic value; in the step from characteristic value to design value no statistical uncertainty is included. This might be too optimistic in some cases.

2) More information may be found in ISO 12491:1997, *Statistical methods for quality control of building materials and components.*

Table D.1 — Values of k_S; σ_R unknown (Confidence level = 0,75)

Probability P	Number of tests, n 3	4	6	8	10	20	30	100	∞
0,10	2,50	2,13	1,86	1,74	1,67	1,53	1,47	1,38	1,28
0,05	3,15	2,68	2,34	2,19	2,10	1,93	1,87	1,76	1,64
0,01	4,40	3,73	3,24	3,04	2,93	2,70	2,61	2,46	2,33

Table D.2 — Values of k_σ; σ_R known (Confidence level = 0,75)

Probability P	Number of tests, n 3	4	6	8	10	20	30	100	∞
0,10	1,67	1,62	1,56	1,52	1,50	1,43	1,40	1,35	1,28
0,05	2,03	1,98	1,92	1,88	1,86	1,79	1,77	1,71	1,64
0,01	2,72	2,66	2,60	2,56	2,54	2,48	2,45	2,39	2,33

D.5.3 Partial factor design: Bayesian method

In the Bayesian method the design value may be estimated directly from the test data:

$$R_d = \eta_d \left\{ m_R - t_{vd} s_R \sqrt{\left(1 + \frac{1}{n}\right)} \right\} \qquad \text{. . . (D.4)}$$

where

m_R is the sample mean value;

s_R is the sample standard deviation;

t_{vd} is the coefficient of the Student distribution (table D.3);

n is the number of tests;

η_d is the design value of the conversion factor.

Values for t_{vd} follow from table D.3, where $\nu = n - 1$, $\beta_R = \alpha_d\, \beta$, where β is the the target reliability index and α_d the design value for the FORM (First Order Reliability Method) influence coefficient. Without further indication, one should use α_d = 0,8 if the uncertainty of R is dominating and α_d = 0,3 otherwise (see annex E, E.5.1 and E.6.3).

Equation (D.4) can be used directly within the design value method.

For use within the partial factor method, two ways are possible.

a) The characteristic value R_k is defined, using the same equation (D.4), but with β_R = 1,64; the partial factor follows from $\gamma_m = R_k / R_d$.

b) The γ_m value normally used for the type of material and failure mode is used; in this way, the characteristic value R_k is defined as $R_k = \gamma_m R_d$; note that in this case R_k may have a probability of exceeding the limit value different from 0,95.

Which method is chosen is a matter of presentation only. In both cases, the same design value is used in the verification procedure.

Equation (D.4) is based on a normal distribution for R and a non-informative prior distribution for both the standard deviation and the mean. If the standard deviation is known in advance, one may replace the sample standard deviation by the distribution standard deviation and take $\nu = \infty$. For the processing of other types of prior information, the formulae given in D.5.4 can be used. Note 1 in D.5.2 about the choice of the distribution is also applicable to the approach in this subclause.

The Bayesian method as presented in this subclause is very sensitive to the observed standard deviation σ_R, if this quantity is not known in advance. It might be advisable to eliminate excessively small and large values of the posterior standard deviation in order to avoid unsafe or uneconomic results. One possible way to achieve this is by choosing a proper prior distribution for the standard deviation, even in the absence of specific information. The mere fact that an engineer considers some technical solution feasible enough to put it to the test, can be used as an argument. In D.5.4 more information on this procedure is given.

Table D.3 — Values of t_ν

		Degrees of freedom, ν								
β_R	$\Phi(-\beta_R)$	1	2	3	5	7	10	20	30	∞
1,28	0,10	3,08	1,89	1,64	1,48	1,42	1,37	1,33	1,31	1,28
1,64	0,05	6,31	2,92	2,35	2,02	1,89	1,81	1,72	1,70	1,64
2,33	0,01	31,8	6,97	4,54	3,37	3,00	2,76	2,53	2,46	2,33
2,58	0,005	63,7	9,93	5,84	4,03	3,50	3,17	2,84	2,75	2,58
3,08	0,001	318	22,33	10,21	5,89	4,78	4,14	3,55	3,38	3,09
NOTE — If σ_R is known, $\nu = \infty$ should be used.										

EXAMPLE 1

As an example, consider a sample of n = 3 test pieces, having a sample mean m equal to 100 kN and a sample standard deviation s_R equal to 15 kN. The 5 % characteristic value is given by (ν = 2):

$$R_k = m_R - 2{,}92\ s_R\sqrt{1+\frac{1}{3}} = m_R - 3{,}37\ s_R = 100 - 3{,}37 \times 15 = 49{,}5 \text{ kN}$$

Note that the classical method would lead to: $R_k = m_R - 3{,}15 s_R$ = 52,8 kN (see table D.1). The result is almost the same.

D.5.4 Evaluation using probabilistic methods

In a full probabilistic treatment, the first step is the establishment of a so-called prior distribution function for the unknown distribution parameters of the resistance R. Such a distribution should reflect all the available prior information about these parameters. Given this prior distribution and given the statistical test data, a posterior distribution can be derived from:

$$f''(\underline{q}) = CL(\text{data}|\underline{q})f'(\underline{q}) \qquad \text{. . . (D.5)}$$

where

$f''(\underline{q})$ is the posterior distribution of $\underline{q}$;

$f'(\underline{q})$ is the prior distribution of $\underline{q}$;

$L(\text{data}|\underline{q})$ is the likelihood function;

$\underline{q}$ is the vector of distribution parameters (e.g. mean and standard deviation);

C is the normalizing constant.

Then, the updated distribution of R itself, given the prior information and the test data, is given by:

$$f_R''(R) = \int f(R|\underline{q})f''(\underline{q})\mathrm{d}\underline{q} \qquad \text{. . . (D.6)}$$

where

f(R|q) is the distribution of *R* for given values of *q;*

$f''_R(R)$ is the updated distribution of *R.*

This distribution for *R* can directly be used in a probabilistic design procedure. It is also possible to derive design values on the basis of equation (D.6).

We shall further consider the case that *R* has a normal distribution. The parameter vector of distribution parameters then contains the mean μ and the standard deviation σ. Let the prior distribution be given by:

$$f'(\mu,\sigma) = k\sigma^{-(\nu'+\delta\{n'\}+1)} \exp\left\{-\frac{1}{2\sigma^2}\left\{\nu'(s)^2 + n'(\mu - m')^2\right\}\right\} \qquad \text{. . . (D.7)}$$

where

$\delta(n') = 0$ for $n' = 0$

$\delta(n') = 1$ for $n' > 0$

This special choice allows a further analytical treatment of the integrals (D.5) and (D.6). The prior distribution (D.7) contains four parameters: m', n', s' and ν'. The meaning of these parameters is explained below.

The parameters s' and ν' characterize the prior information about the standard deviation. The expectation and the coefficient of variation of the standard deviation σ can asymptotically (for large ν') be expressed as:

$$E(\sigma) = s' \qquad \text{. . . (D.8)}$$

$$V(\sigma) = \frac{1}{\sqrt{2\nu'}} \qquad \text{. . . (D.9)}$$

The prior information about the mean is characterized by m', n' and s'. The expectation and the coefficient of variation of the mean μ can asymptotically (for large values of ν') be expressed as:

$$E(\mu) = m' \qquad \text{. . . (D.10)}$$

$$V(\mu) = \frac{s'}{m'\sqrt{n'}} \qquad \text{. . . (D.11)}$$

It is also possible to interpret the prior information as the result of hypothetical prior test series, one for the mean and one for the standard deviation. In that case, we have for the standard deviation:

s' is the hypothetical sample value;

ν' is the hypothetical number of degrees of freedom for s'.

The information about the mean requires two additional parameters:

m' is the hypothetical sample average;

n' is the hypothetical number of observations for m'.

In other words, m' and s' represent the best estimates for the mean and standard deviation. Through the choice of n' and ν', the uncertainty with respect to the estimates can be expressed.

Also note that for a test, we normally have $\nu = n - 1$, but that the prior parameters n' and ν' may be chosen independently from each other.

NOTES

1 If very little information is available, n' and ν' should be chosen equal to zero. In that case the final results will be equal to those of D.5.3. If past experience leads to an almost deterministic knowledge about the mean and standard deviation, n' and ν' could be given relatively higher values, for instance 50, corresponding to $V(\sigma) = 0{,}10$ or $V(\mu) = 0{,}14\ s'/m'$.

2 In many cases it seems reasonable to assume that there is very little or no prior information on the mean (so $n' = 0$), but that it is possible to obtain a fairly good estimate of σ'. As an example, let the coefficient of variation of σ be in the order of 30 %, which according to equation (D.9) corresponds to $\nu' = 5$. Such a model may be based on the result of many previous test samples, showing considerable variability in the mean but significantly less in the standard deviation. For concrete cubes this is very close to reality. When this option is selected we avoid the situation by which small samples lead to very uneconomical or very unsafe results.

Using equation (D.5) one may combine the prior information characterized by equation (D.7) and a test result of n observations with sample mean m and sample standard deviation s. The result is a posterior distribution for the unknown mean and standard deviation of R, which is again given by equation (D.7), but with parameters given by the following updating rules:

$$n'' = n' + n \qquad \text{. . . (D.12)}$$

$$\nu'' = \nu' + \nu + \delta\{n'\} \qquad \text{. . . (D.13)}$$

$$m''n'' = n'm' + nm \qquad \text{. . . (D.14)}$$

$$[\nu''(s'')^2 + n''(m'')^2] = [\nu'(s')^2 + n'(m')^2] + [\nu s^2 + nm^2] \qquad \text{. . . (D.15)}$$

where $\nu = n - 1$; $\delta(n') = 0$ for $n' = 0$ and $\delta(n') = 1$ otherwise.

Using equation (D.5) the predictive value of R can be found from:

$$R = m'' - t_{\nu''}s''\sqrt{\left(1+\frac{1}{n''}\right)} \qquad \text{. . . (D.16)}$$

Here $t_{\nu''}$ has a central t-distribution; values of $t_{\nu''}$ for given probabilities of exceeding the limits are given in table D.3. Modifications for lognormal distributions of R are straightforward (see also D.6).

EXAMPLE 2

Consider once more example 1, but assume previous test series have shown that:

— the sample mean is equal to 110 kN on average, but with very high scatter;

— the sample standard deviation is equal to 20 kN on average with a coefficient of variation V of 30 %.

According to equations (D.8) to (D.11), this prior information leads to the following prior distribution parameters:

$m' = 110$ kN, $n' = 0$, $s' = 20$ kN, $\nu' = 1/(2V^2) = 1/(2\times0{,}3^2) = 5$

Now combine this prior information with the same test results as in example 1 (three specimens with sample mean $m = 100$ kN and sample standard deviation $s = 15$ kN). Then equations (D.12) to (D.15) give the following parameters for the posterior distribution:

$n'' = 0 + 3 = 3$

$\nu'' = 5 + 2 = 7$

$m'' = 100$ kN

$7(s'')^2 + 3\times100^2 = 5\times20^2 + 0\times110^2 + 2\times15^2 + 3\times100^2$

or $s'' = 18{,}7$ kN

Using equation (D.16) and table D.3, this leads to the following result for the 5 % characteristic value:

$$R_k = 100 - 1{,}89 \times 18{,}7\sqrt{1 + \frac{1}{3}} = 100 - 2{,}17 \times 18{,}7 = 59{,}3 \text{ kN}$$

The change in characteristic values from 49,5 kN to 59,3 kN is due to the effect of the prior information. For design values the discrepancies may even be larger.

D.6 Evaluation on the basis of an analysis model

Assume that an analysis model for the structural property under consideration is available. Let the model be complete except for an unknown coefficient θ to be determined from the tests. Such a model can be written as:

$$Y = \theta\, g(\underline{X}, \underline{W}) \qquad \text{. . . (D.17)}$$

where

$\underline{X}$ is the vector of random variables;

$\underline{W}$ is the set of measurable deterministic variables;

$g(\,)$ is the model;

Y is the measurable output parameter of the model;

θ is the unknown coefficient, to be determined by the experiment.

The parameter θ is also referred to as the model uncertainty. In the absence of other information it will be assumed that θ has a lognormal distribution, which means that $\theta' = \ln \theta$ is normal.

Assume a series of experiments $i = 1, \ldots n$ is carried out, where:

– values of $\underline{W}$ have been set to $\underline{w}_i$;

– values of $\underline{X}$ have been measured as $\underline{x}_i$;

– values of Y have been measured as y_i.

From these results one may derive the following set of observations for the unknown coefficient θ:

$$\theta_i = \frac{y_i}{g(\underline{x}_i, \underline{w}_i)} \qquad \text{. . . (D.18)}$$

Mean and standard deviation for $\theta' = \ln \theta$ then follow from:

$$m(\theta') = \frac{1}{n}\sum_{i=1}^{n} \theta'_i \qquad \text{. . . (D.19)}$$

$$s(\theta')^2 = \frac{1}{n-1}\sum_{i=1}^{n} \{\theta'_i - m(\theta')\}^2 \qquad \text{. . . (D.20)}$$

with θ'_i given by:

$$\theta'_i = \ln \{y_i / g(\underline{x}_i, \underline{w}_i)\} \qquad \text{. . . (D.21)}$$

The design value θ_d, including the statistical uncertainties, is given by:

$$\theta_d = \exp\{m(\theta')\}\exp\left\{\pm t_{vd} s(\theta')\sqrt{\left(1+\frac{1}{n}\right)}\right\} \qquad \text{. . . (D.22)}$$

The factor $\exp\{m(\theta')\}$ is often referred to as the bias factor; if $m(\theta') = 0$, then $\exp[m(\theta')] = 1{,}0$ and the model is called unbiased.

Values for t_{vd} follow from table D.3, where $v = n - 1$, $\beta_R = \alpha_d \beta$, with β the target reliability index and α_d the design value for the FORM influence coefficient. Failing other indications, one should use $\alpha_d = 0{,}8$ if the uncertainty of R is dominating and $\alpha_d = 0{,}3$ otherwise (see annex E).

The design resistance R_d of the structural element designed by testing can then be calculated as follows:

$$R_d = \frac{1}{\gamma_d}\eta_d g(x_d, w) \qquad \text{. . . (D.23)}$$

Here $\gamma_d = 1/\theta_d$, and η_d is the design value of the model uncertainty.

Annex E
(informative)

Principles of reliability-based design

E.1 Introduction

The objectives of this annex are:

— to give some background information about this International Standard;

— to complete clause 8 with more detailed descriptions about principles and methods;

— to give recommendations concerning the application of probabilistic methods.

Probabilistic methods can, in principle, be used for all verification problems which can be described with the aid of mathematical relations when the set of random events can be identified. Their use can be divided into two main groups: calibration of safety elements (e.g. partial factors), and direct application for design purposes. The application for design purposes generally concerns advanced problems of such a character that makes the common verification methods less suitable. Design assisted by testing and the assessment of existing structure are two kinds of problems which are often amenable to probabilistic treatment.

This annex is mainly for the use of:

— those who have the task of producing national and international codes or recommendations;

— designers wishing to be informed about reliability based design;

— researchers in the field of probability based design.

The annex contains some general aspects of design based on probabilistic methods. It may be regarded as a state-of-the-art report. Clauses E.4 to E.7 apply mainly to ultimate limit states, but in many cases they are also applicable to irreversible serviceability limit states. They are generally not applicable to problems involving reversible serviceability limit states.

E.2 Uncertainty modelling

This clause treats the uncertainties of basic variables, i.e. actions, material properties and geometrical data. It is assumed that the basic variables also include random variables θ which are assumed to represent the model uncertainties (see 7.3) associated with analysis models.

E.2.1 Sources of uncertainties

According to 6.1, three types of uncertainties may be identified:

— inherent random variability or uncertainty;

— uncertainty due to inadequate knowledge;

— statistical uncertainty.

These types can be further subdivided as follows:

a) **Inherent random variabilities and uncertainties** can be divided into those uncertainties which can, and cannot, be affected by human activities. Many kinds of action parameters (e.g. snow load on ground, wind speed and earthquake ground motion intensity) belong to the second category. So do strength values (e.g. soil parameters). The first category concerns, for example, the uncertainties of strength values of steel or concrete or of the dimensions of steel beams. These uncertainties can be decreased by the use of more advanced production and quality control methods which, on the other hand, may cause costs to increase. Thus, within certain limits, the level of uncertainty can be chosen with regard to economic consequences. Therefore, the distinction between the two categories may be important if economic optimization is considered.

b) **Uncertainties due to inadequate knowledge** can also be subdivided into two categories. One category includes, for example, the model uncertainties of action effect models or resistance models for which knowledge can be increased (and thus uncertainty can be decreased) by research or other similar activities. Also measurement errors belong to this category of uncertainties. In the other category belong, for example, uncertainties which depend on future development. One example is the future development of the traffic loads on road bridges and imposed loads on floors. The possibility of decreasing these uncertainties by research or similar activities is very limited.

c) **Statistical uncertainties** are associated with the statistical evaluation of results of tests or observations. They may result from:

— lack of identification and separation of different statistical populations;

— a limited number of test results which cause uncertainties in the estimation of statistical parameters (e.g. mean and standard deviation);

— neglecting systematic variations of the observed variables (e.g. of climatic variables);

— excessive extrapolations of statistical information;

— neglecting possible correlations;

— using statistical distributions for describing uncertainties which are partly or not at all of a statistical character (compare E.2.2).

The statistical uncertainties can normally be decreased by increasing test and observational efforts.

E.2.2 Different ways to obtain basic data

The numerical values of the parameters which characterize the model and its uncertainties can be obtained in many different ways, such as:

a) observation or measurements

b) analysis

c) decision

d) judgement

Often, the basic data are obtained through a combination of these ways.

Some simple examples may be given as follows.

— The concrete tensile strength is often determined from measurement (of the compressive strength) and analysis (using some conversion function).

— The maximum load which should be lifted by a crane is determined by decision. Additional dynamic forces are determined in other ways.

— Traffic loads on bridges are often determined by observation combined with a judgement about future development. Decision making may also be important.

The basic variables which describe the uncertainties should be characterized by parameters such as the mean value, the standard deviation, correlations with other variables and also by their probability distributions. If the numerical values of these parameters are determined according to a) and b) above, the procedure normally includes analysis of statistical data and the results can be presented in statistical terms. If the values of the basic variables are determined mainly by decision making and/or judgement, the results can generally not be presented directly in statistical terms. However, if it is assumed (see 8.1) that it should be possible to treat all basic variables with probabilistic procedures, statistical parameters (mean value, standard deviation, etc.) have to be assigned also to those basic variables for which the determination of the values does not give statistical data. This must be achieved in a fairly subjective way which may also include the selection of deterministic values. Thus, for example, a possible overload above the allowed load on a floor in a store house could be considered by taking the allowed load as a mean and some expected overload as a standard deviation.

Those uncertainties which are due to gross measurement errors, scale effects, etc., should be eliminated as much as possible by quality assurance measures (see annex A). If this is done, two main kinds of uncertainties remain: model uncertainties and statistical uncertainties. If possible, these two kinds of uncertainties should be separated by statistical methods (see annex D).

E.2.3 The choice of probability distribution functions

Only in a few cases is the amount of available data such that a probability distribution function can be determined unambiguously. In most cases one has to select (among well-known analytic distributions) a distribution which has reasonable properties with regard to the particular basic variable under consideration. The following recommendations apply to most applications.

— For permanent action values and for arbitrary point-in-time values of variable actions, a Gaussian distribution may be convenient if the non-zero probability of negative values is not disturbing. A log normal distribution, a Weibull distribution, a gamma distribution or an extreme value distribution may also be convenient especially if the distribution is intended to represent a maximum value within a chosen reference time.

— For material properties and dimensions, a Gaussian distribution or a log-normal distribution may be convenient. The log-normal distribution is preferred if the non-zero probability of negative values associated with the choice of a Gaussian distribution is disturbing.

The choice of probability distribution functions should be made with caution. Possible bias should be considered. If the actual distribution has a multimodal character, a choice of one single distribution (among the well-known analytical distributions) may cause considerable errors.

E.3 Failure criteria

E.3.1 Ultimate limit states

It is assumed that the failure criteria for a structure are governed by a function $g(\underline{X})$ of the basic variables $\underline{X}$ so that:

$g(\underline{X}) > 0$ for the desirable state (safe set)

$g(\underline{X}) = 0$ for the limit state

$g(\underline{X}) < 0$ for the undesirable state (unsafe set).

This is illustrated in figure E.1 for a case with two basic variables X_1 and X_2; i.e. $\underline{X} = (X_1, X_2)$

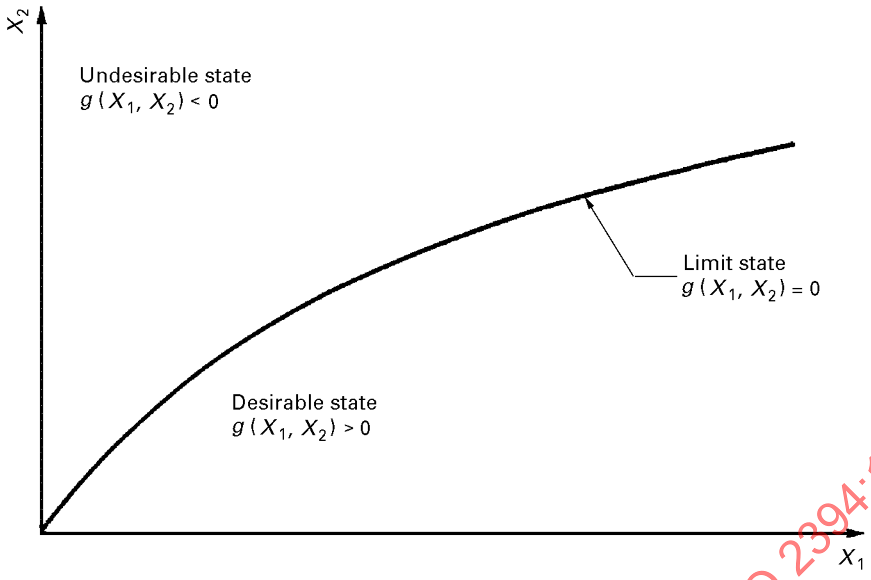


Figure E.1 — Illustration of the function $g(\underline{X})$

The basic variables $\underline{X}$ may be time dependent. For instance, extreme environment loads may vary with time. Structural material may deteriorate with time due to corrosion or other phenomena. The resistance may also decrease with time due to fatigue. In the general case, some of the variables $\underline{X}$ must be represented by stochastic processes. In particular, the time variability of $\underline{X}$ implies that maxima or minima of the components of $\underline{X}$ do not occur at the same time. The time dependency implies that the probability of failure is associated with a chosen reference time t_0.

The reliability (probability of survival or of no failure) of a structure is defined as

$$P_s = 1 - P_f \qquad \text{. . . (E.1)}$$

If the reliability of one element, or one cross-section of an element, is studied with regard to a particular failure mechanism and a particular load combination, the function $g(\underline{X})$ can often be described by one single expression derived from the mechanical behaviour. Then the analysis can be described as an **element analysis**.

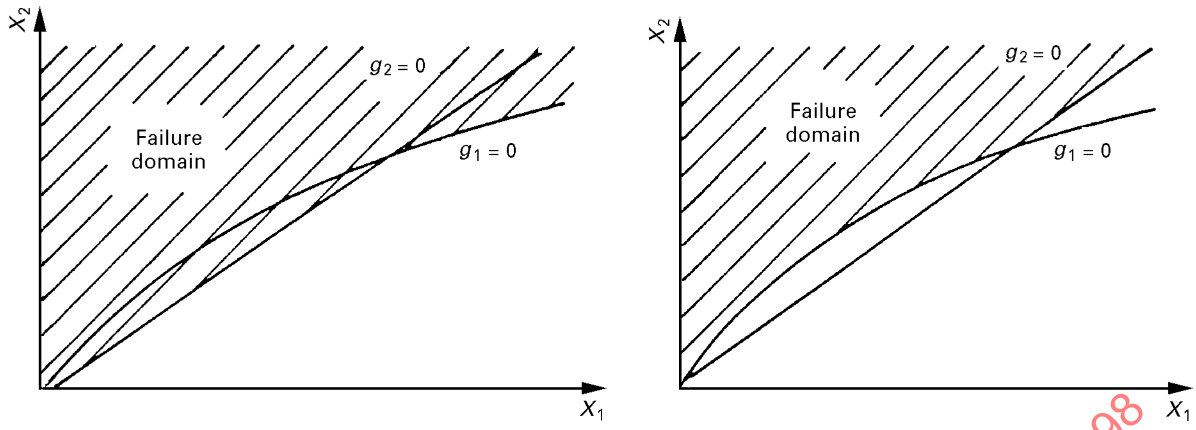
If more than one failure mechanism for an element or if more than one element is studied simultaneously, then the function $g(\underline{X})$ can be considered to be composed of several functions $g_1(\underline{X})$, $g_2(\underline{X})$ ... . This is illustrated in figure E.2 by an example with two functions $g_1(X_1, X_2)$ and $g_2(X_1, X_2)$ of two basic variables X_1 and X_2. Figure E.2 shows two extreme cases.

For the case in figure E.2a), the failure domain (undesirable state) is determined by

$$g_1(X_1, X_2) < 0 \quad \text{or} \quad g_2(X_1, X_2) < 0 \qquad \text{. . . (E.2)}$$

For the case in Figure 2b), the failure domain is determined by

$$g_1(X_1, X_2) < 0 \quad \text{and} \quad g_2(X_1, X_2) < 0 \qquad \text{. . . (E.3)}$$



a) b)

Figure E.2 – Failure domains (shaded) in two extreme cases

An analysis which takes account of several conditions $g_i(\underline{X}) < 0$ simultaneously is described as a **system analysis**. The definition of the system function $g(\underline{X})$ is strongly dependent on the characteristics of the system; i.e. if it is a "weakest-link system" [figure E.2a)] or a "redundant system" [figure E.2b)] or some combination of these two cases.

E.3.2 Serviceability limit states

For some serviceability limit states, the passage of a particular limit state from the desirable state to the undesirable state can be considered to occur under fairly distinct conditions. This means that the limit state, with reasonable approximation, can be considered as a mechanical reality. However, for many serviceability limit states the transition from the desirable state to the undesirable state occurs under more diffuse conditions. The transition implies a more or less slowly decreasing degree of serviceability. Thus, in principle, a degree of serviceability, μ ($0 \leq \mu \leq 1$) can be defined and can by introduced as a function of some serviceability parameter, λ (e.g. deflection of a beam or vibration intensity of a floor). This is illustrated in figure E.3, where it is assumed that there are two limit values of λ: λ_1 for which the structure is fully serviceable, and λ_2 for which the structure is completely unserviceable. In some cases it may be possible to express the degree of serviceability in economic terms.

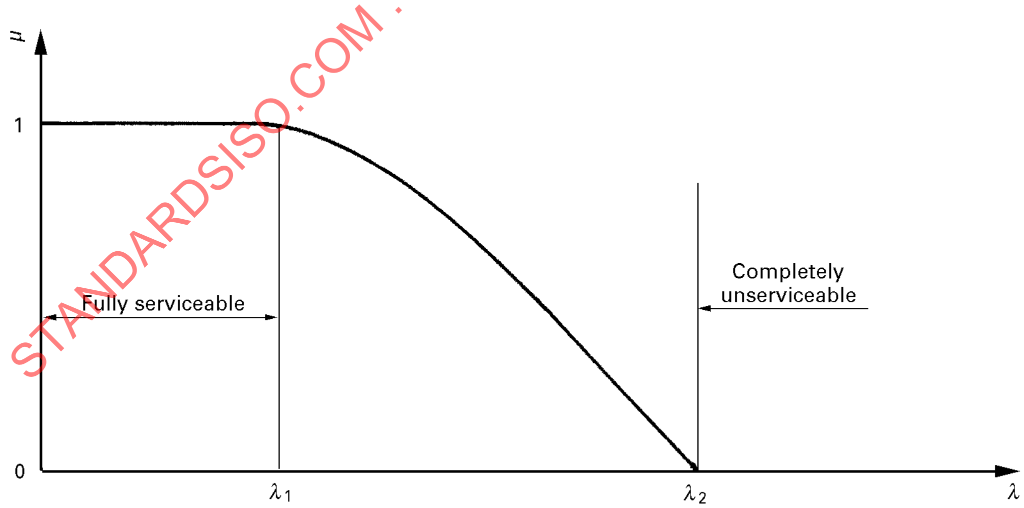


Figure E.3 – Degree of serviceability μ as a function of the serviceability parameter λ

E.4 Specified reliability levels

E.4.1 Safety of people

Structural reliability is important first and foremost if people may be killed or injured as a result of collapse. An acceptable maximum value for the failure probability in those cases might be found from a comparison with risks resulting from other activities. Taking the overall individual lethal accident rate of 10^{-4} per year as a reference, a value of 10^{-6} seems reasonable to use. The maximum allowable probability of failure of the structure then depends on the conditional probability of a person being killed, given the failure of the structure:

$$P(f\,|\,\text{year})\;P(d\,|\,f) \langle 10^{-6}\ \text{year}^{-1} \qquad \text{. . . (E.4)}$$

The probability $P(d\,|\,f)$ is the probability that a person present in the building at the time of collapse is killed. If a building is seldom visited by human beings, a further reduction factor may be introduced in equation (E.4).

Requirement (E.4) is presented as a requirement per year. This should be considered as an average over some reference period. In general, it is allowable to have a large failure rate in some part of the reference period and a smaller value in another part. The reference period need not necessarily be the lifetime of the structure, 10 to 20 years may often be reasonable. In general, one may accept deviations from the yearly average only for a much shorter period of time.

Equation (E.4) gives a minimum requirement for human safety from the individual point of view. In many cases authorities explicitly want to avoid accidents where large numbers of people may be killed. In that case, the additional requirement is of the type:

$$P(f\,|\,\text{year}) \langle A\,N^{-\alpha} \qquad \text{. . . (E.5)}$$

where N is the expected number of fatalities. The numbers A and α are constants, for instance A = 0,01 or 0,1 and α = 2. Modifications of the numerical values are possible in special cases (e.g. if there is an emergency evacuation plan).

E.4.2 Economic optimization

From an economic point of view, the target level of reliability should depend on a balance between the consequences of failure and the costs of safety measures. In a formal way, the objective may be to minimize the total lifetime cost, given by:

$$C_{tot}=C_b+C_m+\sum P_f C_f \qquad \text{. . . (E.6)}$$

where

C_b is the building cost;

C_m is the expected cost of maintenance and demolition;

C_f is the cost of failure;

P_f is the lifetime probability of failure.

The summation is over all (independent) failure modes and load combinations. This formula is highly simplified and may need further refinement before it can be used in practical applications. In addition to economic considerations, authorities may want to specify some minimum reliability level if the safety of human lives is involved. This may lead to a constrained optimization problem with equation (E.6) as object function and equation (E.4) and/or (E.5) as constraints.

Note that, alternatively, $\sum P_f C_f$ may be considered to be covered by insurance.

E.4.3 Examples of calibration

In general it is very difficult to apply the above principles directly in practice. The main point is that there is a substantial difference between the notational probability of failure in the design procedure and the actual failure frequency (which to a considerable extent is due to human errors). For this reason, target levels for reliability are often based on calibration. Using calibrated reliability values, one should keep in mind that they are related to a specific set of structural and probabilistic models. Using the calibrated values in connection with other models could lead to unintentionally high or low levels of reliability.

The numerical values of the reliability are often described on the basis of the reliability index β defined by $\beta = -\,\Phi^{-1}(P_f)$. The relationship between β and P_f is given in table E.1.

Table E.1 — Relationship between β and P_f

P_f	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}
β	1,3	2,3	3,1	3,7	4,2	4,7	5,2

Table E.2 gives an example of calibration life time target β-values, depending on the consequences of failure and the relative cost of safe design.

Table E.2 — Target β-values (life-time, examples)

Relative costs of safety measures	Consequences of failure			
	small	some	moderate	great
High	0	A 1,5	2,3	B 3,1
Moderate	1,3	2,3	3,1	C 3,8
Low	2,3	3,1	3,8	4,3

Some suggestions are:

A: for serviceability limit states, use $\beta = 0$ for reversible and $\beta = 1{,}5$ for irreversible limit states.

B: for fatigue limit states, use $\beta = 2{,}3$ to $\beta = 3{,}1$, depending on the possibility of inspection.

C: for ultimate limit states design, use the safety classes $\beta = 3{,}1$, 3,8 and 4,3.

These numbers have been derived with the assumption of lognormal or Weibull models for resistance, Gaussian models for permanent loads and Gumbel extreme value models for time-varying loads and with the design value method according to E.6.2. It is important that the same assumptions (or assumptions close to them) are used if the values given in table E.2 are applied for probabilistic calculations.

Finally, it should be stressed that a β-value and the corresponding failure probability are formal or notional numbers, intended primarily as a tool for developing consistent design rules, rather that giving a description of the structural failure frequency.

E.5 Calculation of failure probabilities

E.5.1 Time-invariant problems

A comparatively simple case is obtained if all the basic variables $\underline{X}$ can be considered as time-invariant. The probability of failure, P_f, can then be calculated from

$$P_f = \int_{\text{Failure domain}} f_{\underline{X}}(\underline{x})\,\mathrm{d}\underline{x} \qquad \text{. . . (E.7)}$$

where $f_{\underline{X}}(\underline{x})$ is the joint probability density function of the basic random variables $\underline{X}$ (and not random processes). The failure domains are in general given by intersections and unions of domains given by $g_{ij}(\underline{X}) \leq 0$. Here j is the member number and i is the failure mode number.

Failure probabilities may be computed by

— exact analytical methods

— numerical integration methods

— approximate analytical methods (FORM/SORM[3]) methods of moments)

— simulation methods

or a combination of these methods.

In some cases, equation (E.7) can be integrated analytically. When the number, n of random variables is small, say $n \leq 5$, various types of numerical integration may be conveniently applied.

The main steps in the approximative FORM method are:

— transform the variables $\underline{X}$ into a space of standard normal variables, $\underline{U}$, and a corresponding transformation of the failure surface $g(\underline{X}) = 0$ into $g^{u}(\underline{U}) = 0$;

— in the FORM method the failure function $g(\underline{U})$ is approximated by a tangent hyperplane at the design point, which is the point on $g(\underline{U})$ closest to the origin;

— the failure probability P_f according to FORM is then given by $P_f = \Phi(-\beta)$, where β is the distance from the origin to the design point.

The analytical method may be refined by approximating the failure surface $g(\underline{U}) = 0$ by a quadratic surface in the design point (SORM).

Simulation methods can be divided into

— zero-one indicator based methods, which are non-analytical, and operate in the original space of variables $\underline{X}$;

— conditional expectation methods which are semi-analytical methods.

Zero-one indicator methods comprise

— direct Monte-Carlo simulation with the sampling density taken as the original probability density;

— importance sampling where the Monte-Carlo technique is applied with a density (fictitious) function close to the design point;

— adaptive sampling in which importance sampling is applied with successive updating of the density function.

Conditional expectation methods consists of the following techniques:

— directional simulation (suitable for unions of events);

— axis orthogonal simulation (suitable for intersection of events).

3) FORM is an abbreviation for First Order Reliability Method. Sometimes FOSM, First Order Second Moment Method is used. SORM means Second Order Reliability Method.

E.5.2 Transformation of time-variant into time-invariant problems

Two classes of time-dependent problems are discussed, namely those associated with

— overload (first-passage) failure;

— fatigue or other cumulative failures.

The time dependence is due to variability over time of actions and/or strength (degradation). Time-dependent quantities in general need to be represented by stochastic processes.

In the case of a first-passage failure, a single action process may be replaced by a probability distribution representing the uncertainty over the given period for which the failure probability is to be calculated. The mean value may be taken to be the expected maximum value in the chosen reference period; and with a random uncertainty corresponding to that of the expected maximum.

In the case of fatigue failure, the failure function may be formulated in terms of SN-data and the Miner-Palmgren rule. The failure function will then be time-independent when it is referred to a given time period.

E.5.3 General problem

In general, calculation of the failure probability is concerned with determining

$$P_f = P\{\cup\cap g_{ij}(\underline{X}, t) < 0 \text{ for some } t \in [0, T]\} \qquad \text{. . . (E.8)}$$

where g_{ij} are the failure functions ("limit functions") in the space of the basic variables. In equation (E.8), $g_{i1} \le 0$, $g_{i2} \le 0$, etc. in general specify a failure sequence of a structure in a given failure mode (i). For instance, a stiffened panel subjected to lateral and axial forces may fail in two basic modes: 1) buckling, 2) bending. The time dependence may be related to loads; or resistance (e.g. due to strength degradation). Some of the variables $\underline{X}$ may be functions of time and spatial coordinates, and may involve differential or integral expressions.

E.6 Design value methods

E.6.1 General

It is assumed that the limit state considered can be specified by a calculation model in terms of one (or several) function(s) $g(...)$ of a set of variables X_1, X_2, ... X_n, comprising actions, material properties, etc., so that a condition for the structure not to fail of the form

$$g(X_1, X_2, \dots X_n) \ge 0 \qquad \text{. . . (E.9)}$$

can be associated with the limit state. The design requirement may then be written as:

$$g(x_{1d}, x_{2d}, \dots x_{nd}) \ge 0 \qquad \text{. . . (E.10)}$$

where x_{1d}, x_{2d}, ... x_{nd} are design values, defined in E.6.2.

E.6.2 Design values according to FORM

The design value x_{id} of variable X_i depends on:

— the parameters of the variable X_i

— the assumed type of distribution

— the target safety index β for the limit state and design situation of concern (see E.4.3)