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General principles on reliability for structures

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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.

The procedures used to develop this document and those intended for its further maintenance are described in the ISO/IEC Directives, Part 1. In particular the different approval criteria needed for the different types of ISO documents should be noted. This document was drafted in accordance with the editorial rules of the ISO/IEC Directives, Part 2 (see www.iso.org/directives).

Attention is drawn to the possibility that some of the elements of this document may be the subject of patent rights. ISO shall not be held responsible for identifying any or all such patent rights. Details of any patent rights identified during the development of the document will be in the Introduction and/or on the ISO list of patent declarations received (see www.iso.org/patents).

Any trade name used in this document is information given for the convenience of users and does not constitute an endorsement.

For an explanation on the meaning of ISO specific terms and expressions related to conformity assessment, as well as information about ISO's adherence to the WTO principles in the Technical Barriers to Trade (TBT), see the following URL: Foreword — Supplementary information.

The committee responsible for this document is ISO/TC 98, *Bases for design of structures*, SC 2, *Reliability of structures*.

This fourth edition cancels and replaces the third edition (ISO 2394:1998), which has been technically revised.

Introduction

The present fourth edition of this International Standard is intended to reflect advances in the common basis for decision making related to load-bearing structures relevant to the construction industry. Advances range from the development of systematic and rational treatment of risk to implementation of reliability-based design through codes and standards.

Compliance with this International Standard should therefore promote harmonization of design practice internationally and unification between the respective codes and standards such as for actions and resistances for the respective structural materials.

The principles and appropriate instruments to ensure adequate levels of reliability provide for special classes of structures or projects where the common experience base need to be extended in a rational manner.

In particular, a risk framework has been introduced which is scenario based, facilitates unified modelling approaches over different applications, accounts for consequences of both a direct and indirect nature, and has emphasis on robustness.

Whereas requirements to safety and reliability in the previous edition of this International Standard took their basis in efficiency requirements of a heuristic character, these are now based on risk considerations and socio-economics. This, in turn, facilitates a more relevant use of the International Standard in the context of sustainable societal developments and adaptation for application of the International Standard in different nation states in accordance with economic capacity and preferences.

The present International Standard, thus, enables the possibility to regulate, verify, and document the adequate safe performance of structures and also to consider them in a broader sense as part of societal systems. The International Standard provides for approaches at three levels, namely the following:

— risk informed;

— reliability based;

— semi-probabilistic.

The methodical basis for this edition of ISO 2394 is described in the Probabilistic Model Code[8] and Risk Assessment in Engineering — Principles, System Representation and Risk Criteria[9] by the Joint Committee on Structural Safety (JCSS), and EN 1990 (2007), where the reader will find additional information of relevance for its use.

Informative Annexes are included to this International Standard as a support to its users in the interpretations and use of the principles contained in its clauses.

General principles on reliability for structures

1 Scope

This International Standard constitutes a risk- and reliability-informed foundation for decision making concerning design and assessment of structures both for the purpose of code making and in the context of specific projects.

The principles presented in this International Standard cover the majority of buildings, infrastructure, and civil engineering works, whatever the nature of their application and use or combination of the materials used[1)]. The application of this International Standard will require specific adaptation and detailing in special cases where there are potentially extreme consequences of failure[2)].

This International Standard is intended to serve as a basis for those committees responsible for the task of preparing international standards, national standards, or codes of practice in accordance with given objectives and context in a particular country.

The present International Standard describes how the principles of risk and reliability can be utilized to support decisions related to the design and assessment of structures and systems involving structures over their service life. Three different but related levels of approach are facilitated, namely, a risk-informed, a reliability-based, and a semi-probabilistic approach.

The general principles are applicable to the design of complete structures (buildings, bridges, industrial structures, etc.), the structural elements and joints making up the structures and the foundations. The principles of this International Standard are also applicable to the successive stages in construction, the handling of structural elements, their erection, and all work on-site, as well as the use of structures during their design working life, including maintenance and rehabilitation, and decommissioning.

Risk and reliability are concepts accounting for and describing actions, structural response, durability, life-cycle performance, consequences, design rules, workmanship, quality control procedures, and national requirements, all of which are mutually dependent.

The application of this International Standard necessitates knowledge beyond what is contained in the Clauses and the Annexes. It is the responsibility of the user to ensure that this knowledge is available and applied.

2 Terms and definitions

2.1 General terms

2.1.1
structure
organized combination of connected parts including geotechnical structures designed to provide resistance and rigidity against various actions

2.1.2
structural member
physically distinguishable part of a structure, e.g. column, beam, plate, foundation

1) The present International Standard is completely general from the perspective of basic principles and can be applied for any structure below, on, and over the surface of the Earth.

2) This concerns, for example, structures of nuclear power plants and offshore oil and gas facilities in highly sensitive environments.

2.1.3
system
bounded group of interrelated, interdependent, or interacting members forming an entity that achieves a defined objective in its environment through interaction of its parts and interactions of its parts with the environment

2.1.4
structural system
load-bearing members of a building or civil engineering structure and the way in which these members function together and interact with the environment

2.1.5
requirement
demand with respect to structural aspects like safety for people and environment, functionality, usage, and commitment of resources and cost efficiency

2.1.6
compliance
fulfilment of specified requirements

2.1.7
life cycle
life cycle incorporates initiation, project definition, design, construction, commissioning, operation, maintenance, refurbishment, replacement, deconstruction, and ultimate disposal, recycling, or re-use of the structure (or parts thereof), including its components, systems, and building services

2.1.8
reliability
ability of a structure or structural member to fulfil the specified requirements, during the working life, for which it has been designed.

Note 1 to entry: Reliability is often expressed in terms of probability.

Note 2 to entry: Reliability covers safety, serviceability, and durability of a structure.

2.1.9
structural safety
ability (of a structure or structural member) to avoid exceedance of ultimate limit states, including the effects of specified accidental phenomena, with a specified level of reliability, during a specified period of time

2.1.10
durability
capability of a structure or any structural member to satisfy with planned maintenance the design performance requirements over a specified period of time under the influence of the environmental actions

2.1.11
exposure events
events which may cause damage or otherwise affect the performance indicators for the structure

2.1.12
assessment
total set of activities performed in order to verify the reliability of an existing structure

2.1.13
upgrading
modifications of an existing structure, construction works, and procedures to improve its structural performance or facilitate its use for new purposes

2.1.14
repair (of a structure)
restoring the condition of a structure that has been damaged or deteriorated

2.1.15
rehabilitation
repairing or upgrading of an existing structure

2.1.16
monitoring
frequent or continuous, normally long-term, observation or measurement of structural conditions or actions or structural response

2.1.17
inspection
on-site examination within the scope of quality control and condition assessment aiming to assess the present condition of a structure

2.1.18
constituent events
events associated with damage or failure of structural members or parts of these, such as individual cross sections and joints

2.1.19
reliability-based design
design procedure that is subjected to prescribed reliability level of the structure

2.1.20
member reliability
reliability of a single structural member which has one single dominating failure mode

2.1.21
system reliability
reliability of a system of more than one relevant structural member or a structural member which has more than one relevant failure mode

2.1.22
population
set of entities for which the same probabilistic descriptions (mean values, etc.) are valid

2.1.23
outcome space
set of all possible outcomes of a random phenomenon

2.1.24
constituent
component or ingredient contributing to a certain performance

2.1.25
performance indicator
parameter describing a certain property of the structure or a certain characteristic of the structural behaviour

2.1.26
structural performance
qualitative or quantitative representation of the behaviour of a structure (e.g. load bearing capacity, stiffness, etc.) related to its safety and serviceability, durability, and robustness

2.1.27
resistance
ability of a structure (or a part of it) to withstand actions without failure

2.1.28
quality control
activities to control quality of design, execution, use, and decommissioning of a structure

2.1.29
damage
unfavourable change in the condition of a structure that can affect the structural performance unfavourably

2.1.30
collapse
development of failure mechanisms in a structure to a degree involving disintegration and falling (parts of) structural members

2.1.31
deterioration
process that adversely affects the structural performance including reliability over time

Note 1 to entry: Deterioration may be caused by naturally occurring chemical, physical, or biological actions, normal or severe environmental actions, repeated actions such as those causing fatigue, wear due to use, and improper operation and maintenance of the structure.

2.1.32
serviceability
ability of a structure or structural member to perform adequately for a normal use under all expected actions

2.1.33
scenario description
determination of different sequences of events which affect the performance indicators, taking into account the likelihood of occurrence

2.1.34
consequence class
categorization of the consequences of structural failure

2.1.35
reliability class
class of structures or structural members for which a particular specified degree of reliability is required

2.1.36
reliability differentiation
socio-economic optimization of the resources to be used to build construction works, taking into account all the expected consequences of failures and the cost of the construction

2.1.37
hazard scenario
set of situations, transient in time, that a system might happen to undergo and which may endanger the system itself, the people, and the environment

2.1.38
risk-informed design
design optimized with due consideration of the total risks, including loss of lives and injuries, damages to the qualities of the environment, and monetary losses

Note 1 to entry: Risk-based design is presently not generally accepted by all national standards and codes.

2.1.39
safety plan
plan specifying the performance objectives, the hazard scenarios to be considered for the structure, and all present and future measures (design, construction, or operation, e.g. monitoring) to ensure the safety of the structure

2.1.40
risk
effect of uncertainty on the objectives

Note 1 to entry: From the view point of the decision theory, risk is the expected value of all undesirable consequences, i.e. the sum of all the products of the possible consequences of an event and the corresponding probabilities.

2.1.41
marginal lifesaving cost
increment of cost associated with saving one additional life through additional safety measures

2.1.42
risk screening
investigation into and classification of risks identified for all the hazard situations

2.1.43
Life Quality Index
LQI
indicator of the societal preference and capacity for investments into life safety expressed as a function of GDP, life expectancy at birth, and ratio between leisure to working time

2.1.44
utilisation plan
plan containing the intended use (or uses) of the structure and listing the operational conditions of the structure including maintenance requirements and the corresponding performance requirements

2.1.45
reliability target
specified average acceptable failure probability that is to be reached as close as possible

Note 1 to entry: Reliability targets are generally model dependent and need to be set for each case considered based on the models used.

2.1.46
robustness
damage insensitivity
ability of a structure to withstand adverse and unforeseen events (like fire, explosion, impact) or consequences of human errors without being damaged to an extent disproportionate to the original cause

2.1.47
hazard
unusual and severe threat, e.g. a possible abnormal action or environmental influence, insufficient strength or stiffness, or excessive detrimental deviation from intended dimensions

2.2 Terms related to design and assessment

2.2.1
design/assessment situations
set of physical conditions representing a certain time interval for which it shall be demonstrated that relevant limit states are not exceeded

2.2.2
persistent design situation
normal condition of use for the structure

2.2.3
transient design situations
provisional condition of use or exposure for the structure, for example, during its construction or repair, representing a time period much shorter than the design working life

2.2.4
accidental design situations
design situation involving possible exceptional conditions for the structure in use or exposure, including flooding, fire, explosion, impact, mal-operation of systems, or local failure

2.2.5
seismic design situation
design situation involving the exceptional conditions when the structure is subjected to a seismic event

2.2.6
failure
insufficient load-bearing capacity or inadequate serviceability of a structure or structural member, or rupture or excessive deformation of the ground, in which the strengths of soil or rock are significant in providing resistance

2.2.7
limit states
state beyond which a structure no longer satisfies the design criteria

2.2.8
ultimate limit states
limit states concerning the maximum load-bearing capacity

2.2.9
design criteria
quantitative formulations describing the conditions to be fulfilled for each limit state

2.2.10
serviceability limit states
limit state concerning the criteria governing the functionalities related to normal use

2.2.11
irreversible limit states
limit states which will remain permanently exceeded when the actions which caused the exceedance are no longer present

2.2.12
reversible limit states
limit states which will not be exceeded when the actions which caused the exceedance are no longer present

2.2.13
condition limit state
well-defined and controllable limit state without direct negative consequences, which is often an approximation to a real limit state that cannot be well defined or is difficult to calculate

Note 1 to entry: In applications relating to durability aspects, the condition limit state is often referred to as the durability limit state.

2.2.14
limit state function
function $g(X_1, X_2, ..., X_n)$ of the basic variables, which characterizes a limit state when $g(X_1, X_2, ..., X_n) = 0$

2.2.15
basic variables
variables representing physical quantities which characterize actions and environmental influences, material and soil properties, and geometrical quantities

2.2.16
design service life
assumed period for which a structure or a structural member is to be used for its intended purpose with anticipated maintenance, but without substantial repair being necessary

2.2.17
model uncertainty
basic variable related to the accuracy of physical or statistical models

2.2.18
aleatory uncertainty
inherent variability typically associated with the loading environment, the geometry of the structure, and the material properties

2.2.19
epistemic uncertainty
lack of knowledge that, in principle, can be reduced by measurements or improved theories

Note 1 to entry: The exact borderline between aleatory and epistemic is not always unambiguous.

2.2.20
hierarchical modelling of uncertainty
random variable is a function of other random variables

2.2.21
probabilistic methods
verification methods in which the relevant basic variables are treated as random variables, random processes, and random fields, discrete or continuous

2.2.22
reliability index
β
substitute for the failure probability $\beta = -\Phi^{-1}(p_f)$ where Φ^{-1} is the inverse standardized normal distribution

2.2.23
target reliability (index)
reliability (index) corresponding to acceptable safety or serviceability

2.2.24
semi-probabilistic or partial factors methods
verification method in which allowance is made for the uncertainties and variability assigned to the basic variables by means of representative values, partial factors and, if relevant, additive quantities

Note 1 to entry: Partial factors may be related to individual random variables or global variables.

2.2.25
structural model
idealisation of the structure, physical, mathematical, or numerical, used for the purposes of analysis, design, and verification

2.2.26
static system
idealisation of the structure, used for the purposes of static analysis, design, and verification

2.2.27
levels of verification
levels of the verification used to assess the compliance with the objectives for all design/assessment situations

Note 1 to entry: The following levels are commonly recognised: the risk level, the probabilistic reliability level, and the semi-probabilistic level.

2.2.28
First/Second Order Reliability Methods
FORM/SORM
numerical methods used for determination of the reliability index β

2.2.29
reliability elements
numerical quantities used in the partial factors format, by which the specified target reliability is assumed to be reached

Note 1 to entry: The reliability elements include partial factors and load combination factors.

2.2.30
characteristic value
value specified preferably on statistical bases, so it can be considered to have a prescribed probability of not being exceeded

Note 1 to entry: For variable actions, the characteristic value corresponds to either of the following:

— an upper value with an intended probability of not being exceeded or a lower value with an intended probability of being achieved, during some specific reference period;

— a nominal value, which may be specified in cases where a statistical distribution is not known.

2.2.31
reference period
period of time used as a basis for assessing the design value of variable and/or accidental actions

2.2.32
nominal value
value fixed on a non-statistical basis, for instance, on acquired experience or on physical constraints

2.2.33
alternative load path
ALP
alternative for a load to be transferred from a point of application to a point of resistance

2.2.34
consequence reducing measures
measure aiming at reducing the direct and indirect consequences of failure and thus the total risk

2.2.35
key element
structural member upon which the ultimate limit state performance of the structure depends

2.2.36
code calibration
determination of the reliability elements in a given code format in order to reach the reliability target

2.2.37
load testing
test of the structure (or part of it) by loading to evaluate its behaviour or properties or to predict its load bearing capacity

2.3 Terms related to actions, action effects, and environmental influences

2.3.1
action
assembly of concentrated or distributed forces acting on a structure (direct actions), displacements or thermal effects imposed to the structure, or constrained in it or environmental influences that may cause changes with time in the material properties or in the dimensions of a structure

2.3.2
individual action
single action
action which can be assumed to be independent in time and space of any other action acting on the structure

2.3.3
permanent action
action which is likely to act continuously throughout the design working life and for which variations in magnitude with time are small compared with the mean value

2.3.4
variable action
action which is likely to act during a given design working life and for which the variation in magnitude with time is neither negligible nor monotonic

2.3.5
accidental action
action which is unlikely to occur with a significant value during the design working life of the structure

2.3.6
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 of the structure

2.3.7
free action
action that may have an arbitrary spatial distribution over the structure within certain limits

2.3.8
dynamic action
action that may cause significant acceleration of the structure or structural members

2.3.9
static action
action that will not cause significant acceleration of the structure or structural members

2.3.10
bounded action
action that has a limiting value which cannot be exceeded and which is exactly or approximately known

2.3.11
design value of an action
$\boldsymbol{F}_{\mathbf{d}}$
value of an action used in semi-probabilistic verification calibrated to the reliability target

Note 1 to entry: In the partial factor method, the value is obtained by multiplying the representative value by the partial factor γ_F.

2.3.12
effect of actions
effect of actions (or action effect) is a result of actions on a structural member (e.g. internal force, moment, stress, strain) or on the whole structure (e.g. deflection, rotation)

2.3.13
prestress
force applied intentionally to a structure by imposing deformations

2.3.14
geotechnical action
action transmitted to the structure by the ground, fill, or groundwater

2.3.15
seismic action
action caused by earthquake ground motions

2.3.16
imposed load
load resulting from occupancy in buildings

2.3.17
construction load
load specifically related to execution activities

2.3.18
environmental influences
physical, chemical, or biological influences which may deteriorate the materials constituting a structure, which in turn may affect its serviceability and safety in an unfavourable way

2.3.19
load arrangement
identification of the position, magnitude, and direction of a free action

2.3.20
representative value of an action
one of the following quantities of an action: the characteristic value, nominal value, combination value, frequent value, and quasi-permanent value

2.3.21
combination value
values determined in such a way that the probability of action effect caused by several combination values being exceeded is approximately the same as the probability of the design value being exceeded by a single action

Note 1 to entry: The 'combination value' may be expressed as the characteristic value reduced by a factor Ψ_0.

2.3.22
action model
model describing the magnitude, position, direction, duration, etc. of the action

Note 1 to entry: Sometimes, there is an interaction between the components. There may, in certain cases, also be an interaction between the action and the response of the structure.

2.3.23
frequent value
value determined in such a way that either the total time, within a chosen period, during which it is exceeded is only a given small part of the chosen period of time or the frequency of its exceedance is limited to a given value

Note 1 to entry: This 'frequent value' may be expressed as the characteristic value reduced by a factor Ψ_1.

2.3.24
quasi-permanent value
value determined in such a way that the total time, within a chosen period, during which it is exceeded is of the magnitude half the period

Note 1 to entry: This 'quasi-permanent value' may be expressed as the characteristic value reduced by a factor Ψ_2.

2.3.25
load case
compatible load arrangement, set of deformations, and imperfections considered for a particular verification of a specific limit state

2.3.26
load combination
design values of the different actions considered simultaneously in the verification of the reliability of a structure for a specific limit state

2.3.27
fundamental combination of actions
combination of permanent actions and variable actions (the leading action plus the accompanying actions) in persistent and transient design situations used for verification of ultimate limit states in general

2.3.28
accidental combination of actions
combination for accidental design situations, involving either an explicit accidental action (e.g. fire or impact) or the situation following an accidental event

2.3.29
characteristic combination of actions
combination of permanent and variable actions used for verification of irreversible serviceability limit states

2.3.30
frequent combination of actions
combination of permanent and variable actions used for verification of reversible serviceability limit states

2.3.31
quasi-permanent combination of actions
combination of permanent and variable actions used for verification of long-term effects of serviceability limit states

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

2.4.1
material model
model describing relations between internal forces or stresses and deformations including strain rates

Note 1 to entry: The parameters of such relations are modulus of elasticity, yield limit, ultimate strength, etc. which generally are considered as random variables. In some cases, they are time or space dependent. Often, there is a correlation between the parameters.

2.4.2
stiffness
property relating deformation to the action causing it

2.4.3
characteristic value of a material property
priori specified fractile of the statistical distribution of the material property in the relevant supply

2.4.4
design value of a material property
value of material property used in semi-probabilistic methods obtained by dividing the characteristic value by a partial factor γ_M or, in special circumstances, by direct assessment

2.4.5
conversion factor or function
factors or functions which convert properties obtained from test specimens to those in in calculation models

2.4.6
characteristic value of a geometrical quantity
quantity usually corresponding to dimensions (nominal values) specified by the designer

2.4.7
geometrical properties
geometrical data (dimensions, angles, etc.) describing the structure and the structural members

Note 1 to entry: A structure can generally be described by a model consisting of one-dimensional member, two-dimensional members and three-dimensional members. The geometrical quantities which are included in the model generally refer to nominal values.

2.4.8
design value of a geometrical quantity
characteristic value adjusted by an additive or multiplicative quantity

3 Symbols

3.1 General

The following symbols listed below are used generally throughout the document. Symbols which are used only in one section are explained there and not listed here. All the symbols are based on ISO 3898.

3.2 Latin upper case letters

A accidental action

A_{d} design value of an accidental action

A_{Ed} design value of seismic action

A_{Ek} characteristic value of seismic action

C serviceability constraint

F action in general

F_{k} characteristic value of an action

F_{rep} representative value of an action

F_{d} design value of an action

G permanent action

G_{d} design value of a permanent action

$G_{\mathrm{d,inf}}$ lower design value of a permanent action

$G_{\mathrm{d,sup}}$ upper design value of a permanent action

$G_{\mathrm{k},j}$ characteristic value of a permanent action j

$G_{\mathrm{k},j,\mathrm{inf}}$ lower characteristic value of a permanent action j

$G_{\mathrm{k},j,\mathrm{sup}}$ upper characteristic value of a permanent action j

P prestressing action

Q variable action

Q_{d} design value of variable action

Q_{k} characteristic value of a variable action

$Q_{k,1}$ characteristic value of the leading variable action 1

$Q_{k,j}$ characteristic value of a variable action j

R resistance

R_d design value of the resistance

R_k characteristic value of the resistance

S action effect

S_d design value of an action effect

$S_{d,dst}$ design value of destabilizing actions

$S_{d,stb}$ design value of stabilizing actions

X basic variable

Y model output variable

3.3 Latin lower case letters

a geometrical quantity

Δa additive geometrical quantity

m material property

p_f probability of failure

p_s probability of survival

p_{ft} target probability of failure

p_{fs} specified value of p_f

t time

3.4 Greek letters

β reliability index

β_t target reliability index

γ partial factor

γ_f partial factors for actions

γ_F generalized partial factors for actions taking account model and geometric uncertainties

γ_G partial factors for permanent actions

γ_Q partial factors for variable actions

γ_m partial factors for material properties

γ_M generalized partial factors for resistance properties taking account of material, model, and geometric uncertainties

γ_S partial factor for model uncertainties of action effects

γ_R partial factor for model uncertainties of resistance

γ_I factor by which the importance of the structure and the consequences of failure are taken into account

θ factor of model uncertainties

θ_S factor of model uncertainties for action effects

θ_R factor of model of uncertainties for resistance

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

3.5 Subscripts

i basic variable (action) number

j basic variable (action) number

k characteristic value

d design value

1 leading action

4 Fundamentals

4.1 General

In this Clause, the aims and requirements, conceptual basis, approaches, and documentation to ensure an adequate level of risk and reliability for structures are outlined with an emphasis on the principles. In the subsequent clauses, the most central of these principles are described in more detail. In the Annexes to this International Standard, additional guidance and information is provided with respect to aspects of special importance for its application including the role and use of quality control as an underlying prerequisite for its practical relevance.

4.2 Aims and requirements to structures

4.2.1 Fundamental requirements to structures

Structures shall be designed, operated, maintained, and decommissioned such as to support societal functionality and enhance sustainable societal development during their service life.

NOTE 1 Societal functionality encompasses the context of the structures with a focus on the functions structures provide for society. For example, an electricity mast is not just carrying electrical cables, but is providing electricity for industry and hospitals. The consequences of failure and thus also the requirements set for the performance of structures should be appreciated in this light.

In particular, the structures shall, with appropriate degree of risk and reliability, fulfil the following performance requirements:

— Function adequately under all expected actions throughout their service lives; providing service and functionality.

— Withstand extreme and/or frequently repeated and permanent actions, as well as environmental exposures occurring during their construction, anticipated use, and decommissioning; providing safety and reliability with respect to damage and failures.

— Be robust such as not to suffer severe damage or cascading failure by extraordinary and possibly unforeseen events like natural hazards, accidents, or human errors; providing sufficient robustness.

NOTE 2 Sustainability can be related to performance indicators such as the following:

— safety for people;

— reliability with respect to fulfilment of purposes;

— qualities of the environment;

— cost efficiency;

— minimized emission of CO_2;

— minimized consumption of natural resources;

— minimized use of energy.

The service life of a structure shall be accounted for and optimized with due consideration of the context of the structure and a holistic perspective to design and assessment. Fundamentally, decisions concerning service lives for structures shall be based on the duration of the need of the structure and the possibility to optimize service life benefits from the structure with a differentiated strategy for its individual components. Durability-based concepts for the design of structures are treated in ISO 13823.

4.2.2 Target performance level

The appropriate degree of reliability shall be judged with due regard to the possible consequences of failure, the associated expense, and the level of efforts and procedures necessary to reduce the risk of failure and damage.

To ensure acceptable levels of risk, safety, and reliability of structures within the context of structural design and assessment decision making, acceptance criteria and other requirements shall be formulated, assessed, and complied with. Some of these requirements shall relate to demands on safety for personnel and environment set by the society. Others shall relate to the reliability of the functionality of the structures as specified by the owners.

The fundamental principle of the marginal lifesaving costs for the regulation of life safety applies and is recommended. The use of the marginal lifesaving principle ensures that the safety for people using or otherwise exposed to a structure has a certain level, such that the costs associated with saving additional lives through additional safety measures exceed the corresponding marginal lifesaving costs.

NOTE 1 The marginal lifesaving costs principle facilitates differentiation of requirements to risk and reliability ensuring that efforts of lifesaving are directed on activities and situations for which the efficiency of lifesaving is the highest. This principle can be seen to be coherent with the general formulation of the ALARP principle as described in Annex G. To quantify the marginal lifesaving costs, a suitable method such as the Life Quality Index (LQI) applies. The same principle also forms the basis for the identification of target reliability levels for semi-probabilistic design codes, where target reliabilities are provided as a function of consequences of failure and safety improvement efficiency (see Annex G).

Depending on the characteristics of specific projects, structural performance requirements related to damages to the qualities of the environment, usage, and commitment of resources, as well as emissions to the atmosphere, can be relevant and shall be considered. In such cases, requirements shall be specified and fulfilled in terms of maximum annual frequencies of events of specified severities or expected values of total usage and/or emissions accordingly. These requirements shall then be seen and accounted for within an economic optimization as constraints or alternative objectives as described in 4.4.2.1.

NOTE 2 Requirements to the qualities of the environment, usage and commitment of natural resources and emissions to the atmosphere have as of yet not been formulated in general terms. However, in specific projects where the possible consequences in this respect are considerable, such as in offshore oil and gas exploitation in vulnerable environments, large dam project, etc., specific considerations with respect to such requirements should as a rule be part of the decision basis.

4.3 Conceptual basis

4.3.1 Decisions concerning structures

Decisions concerning structures shall be understood to encompass all decisions related to the process of designing, constructing, operating, maintaining, and decommissioning structures.

NOTE Decisions concerning structures affect the structures in terms of their resistances, deformations, movements, durability characteristics, safety, economy, material and energy consumption, etc. and thereby also their performances and their impact on sustainable developments. Typical decisions can, at the level of individual projects, include choices of the following:

— structural system;

— materials;

— cross-sectional properties;

— joints and structural detailing;

— inspection, testing, and monitoring in the laboratory and on site;

— active and passive measures of damage detection, prevention, and reduction;

— assessment, maintenance, and repair;

— retrofitting/strengthening;

— decommissioning;

— renewal.

The validity of all assumptions underlying decisions concerning structures, e.g. the relevance of and the uncertainty associated with available knowledge and information, the intended use, the service life, as well as the environmental and operational loads, should be controlled, ensured and, documented. Alternatively, it should be ensured that the performance of the structures is still adequate despite of possible violations of or deviations from assumptions.

Quality management plays a central role for the performance of structures and shall be completely integrated in the decision making processes related to design and assessment of structures (see also Annex A). In particular, for individual projects, the quality management and quality assurance shall address the following:

— quality plans;

— quality control of design;

— quality control of construction;

— quality control of materials, tools, and fabrication;

— quality control of worker qualifications, workmanship, and procedures;

— quality control of assumptions;

— documentation of quality control and quality management.

4.3.2 Structural performance modelling

Design decisions shall be assessed through the performance requirements listed in 4.2.2. Therefore, models should be established for these performance requirements, which allows for their quantification. The modelling of the performance requirements shall address all relevant issues concerning the intended use of the structures, the safety of people, as well as the qualities of the environment and economy throughout the entire life cycle of the structure (see also 4.2.2). Special considerations shall be given to the modelling of the interaction between the structure and its surroundings (i.e. any exposure the structure is subjected to and also the exposures which the structure might influence), dependencies between the structure and, for example, possible mechanical and electrical systems, as well as the influence of human and organizational errors. The models shall be established taking basis in a scenario description of the different sequences of events which affect the performance, taking into account their likelihood of occurrence and their consequences.

In the identification and description of the scenarios of events which are relevant, the following events shall be differentiated:

— Exposure events; actions, human errors and chemical environment.

— Constituent damage and failure events; direct consequences.

— Loss of function and/or cascading failures (progressive failure); indirect consequences.

NOTE 1 The exposure events represent all events which can cause damage or otherwise affect the performance indicators for the structure. Examples hereof include operational and environmental loads, aggressive chemicals, human errors, poor quality of design, materials and construction, etc. It is important to represent also scenarios of exposure events such as, for example, jointly acting loads and other environmental and chemical processes.

NOTE 2 The constituent damage and failure events are those associated with damage or failure of parts of the structure, such as individual cross sections and joints. The modelling of the performance of the individual constituents, that is, to which degree they are damaged or fail for a given scenario of exposures, is supported by the formulation and analysis of limit state functions (see Clause 5).

NOTE 3 Loss of function and/or cascading failures of the structure which can follow constituent damages and failures and lead to indirect consequences is related to robustness. The quantification of the associated risks is described in Annex F (see, for example, Formula F.4).

4.3.3 Uncertainty and treatment of knowledge

Decisions concerning structures shall account for all uncertainties of relevance for their performances such as inherent natural variability (aleatory uncertainty) and lack of knowledge (epistemic uncertainty).

Uncertainties shall be represented in the decision process through probabilistic models such as random variables, stochastic processes, and/or random fields. The probabilistic modelling shall address the representation of temporal and spatial dependency among the considered uncertainties and events. Moreover, possible non-ergodic phenomena, such as effects of climate changes and demographical developments shall be addressed in the modelling.

NOTE 1 The Bayesian probability theory forms the basis for full risk and reliability-based design and assessment (see 4.4.2 and Clause 6). Moreover, Bayesian probabilistic modelling forms the basis for semi-probabilistic design standards and regulations (see 4.4.3) through calibration (see Annex E).

The quantification of uncertainties and their probabilistic representation shall facilitate for the incorporation of both subjective information and available evidence.

NOTE 2 When a new structure is designed, the knowledge about several aspects is still at a very generic level. The associated uncertainties are therefore relatively large and are modelled taking into account data and experience accumulated over time which is representative for given quality control schemes and other specifications (see Annexes A and C). In the context of existing structures and in connection with testing, monitoring and inspection, and maintenance planning, it is important to take benefit from (Bayesian) updating of probabilistic models (See Annex B). In this manner, as more and more evidence in terms of observations are brought into the assessment, risks and reliabilities will gradually be updated.

For design of structures based on codified load and resistance factor or partial safety factor design (see 4.4.3), uncertainties shall be represented through design values and characteristic values together with specified design equations, load cases, and load combination factors. The characteristic values shall, when relevant, account for available information relating, for example, to loads and material properties. Semi-probabilistic approaches to design and assessment of structures are treated in detail in Clause 9.

4.4 Approaches

4.4.1 General

Design and assessment decisions shall take basis in information concerning their implied risks. When the consequences of failure and damage are well understood and within normal ranges, reliability-based assessments can be applied instead of full risk assessments. Semi-probabilistic approaches as a further simplification are appropriate when in addition to the consequences also the failure modes and the uncertainty representation can be categorized and standardized.

Risk- and reliability-based approaches shall be applied for the calibration of semi-probabilistic approaches, as well as for supporting design and assessment decisions for special structures and projects which are not covered by semi-probabilistic codes.

4.4.2 Risk-informed and reliability-based approaches

4.4.2.1 Risk-informed decisions concerning design and assessment

In a risk-informed design and/or assessment, the decisions shall be optimized with due consideration of the total risks, considering loss of lives and injuries, damages to the qualities of the environment, and monetary losses. The time horizon to be considered in the assessment of the total risks shall be determined on the basis of the duration of the functionality which the structure shall provide.

The assessment of the total risk shall take basis in a scenario representation (see also Annex F) and by probabilistic models of the exposures, the constituent damage, and failure events, as well as the direct and indirect consequences.

The performances of structures related to life safety and qualities of the environment, should be assessed with respect to their acceptability (see 4.2, Clause 7, and Annex G). Within these constraints, decisions shall be optimized on the basis of a maximization of the expected value of benefits; in this process, other indicators can be considered, insofar as these are consistent with the principles contained in this Clause. Acceptance criteria as described in 4.2.2 shall be considered as constraints to the optimization and should be included in the verification of the design and assessment decisions. The principle is illustrated in Figure 1.

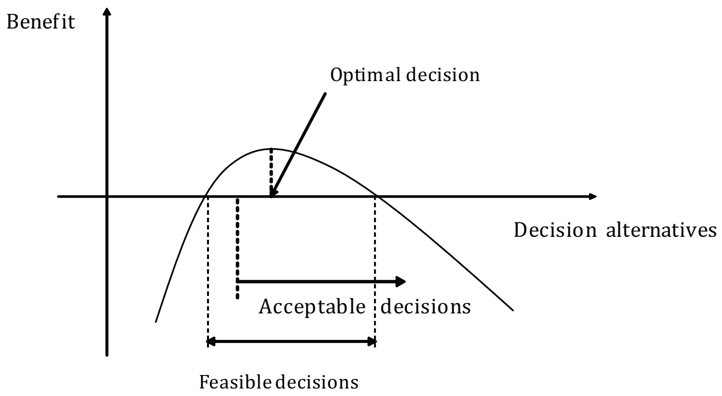


Figure 1 — Illustration of the optimization principle for the maximization of benefits

NOTE 1 The possible decision alternatives are arranged along the x-axis in Figure 1, corresponding to increasing reliability. The benefit indicated on the y-axis is representative for the expected net benefits associated with the different decisions over the considered period of time. In the illustration in Figure 1, the decisions are of a continuous nature; however, discrete decisions or combinations hereof can be organized and illustrated in a similar manner. From Figure 1, it is seen that only some decisions are feasible in the sense that they yield a positive net benefit. One of the feasible decisions is optimal but this decision might or might not be acceptable.

All anticipated future consequences shall be accounted for in the assessment of the risks. This includes both the consequences which are associated with uncertainty and the consequences which are deterministically related to decisions, such as committed future costs due to planned inspections and maintenance or compensation costs for lives which potentially can be lost during the lifetime of the structure (as explained in Annex B and Annex G).

In the assessment of the net present value of future costs, the interest rate to be used shall be chosen carefully. Considering decisions regarding structures which are made on behalf of society, the default annual discounting rate is the long-term annual economic growth rate and varies by country. The same applies when assessing the net present value of expenditures committed for lifesaving activities.

For structures where failure and damage can imply very serious consequences, a risk-based robustness assessment shall be undertaken as a part of the design and/or assessment verification.

NOTE 2 Annex F describes the methodology for risk-based robustness assessments. For operational purposes, it might be convenient to introduce a categorization of structures in accordance with their consequences of failure and, on the basis of this categorization, decide whether a risk-based robustness assessment is necessary or not. Annex F also contains a suggestion for such a categorization.

4.4.2.2 Reliability-based design and assessment

As an alternative to risk-based design and assessment of structures, a reliability-based approach can be chosen. This approach shall utilize an assessment and minimization of costs and/or minimization of committed resource usage subject to given reliability requirements for the structure. The reliability requirements shall be assessed on the basis of a full risk-informed assessment as described in 4.4.2.1 and will thus facilitate reliability differentiation in dependency of consequences of failure and costs of reliability improvements.

NOTE 1 The reliability requirements will, in general, depend on societal capacity to invest into lifesaving activities and will thus be specific from nation state to nation state. In Annex G, it is shown how reliability requirement as a function of failure consequences and costs of reliability improvements can be established for a given nation state.

The general principles for reliability based design and reliability informed decision making are specified in Clause 8.

4.4.3 Semi-probabilistic approaches

For structures for which the consequences of failure and damage are well understood and the failure modes can be categorized and modelled in a standardized manner, semi-probabilistic codes are appropriate as basis for design and assessment. Standards shall serve to ensure the quality of analysis, design, materials, production, construction, operation and maintenance, and documentation, and thereby explicitly or implicitly account for the uncertainties which influence the performance of the structures. The specifications given in standards should be developed such that they quantify all known uncertainties.

Semi-probabilistic design and assessment codes shall comprise a safety format prescribing the design equations and/or analysis procedures which shall be used for the verification of design and assessment decisions. The safety format shall also cover the load combinations which should be considered, as well as the scheme for calculating design values for actions and action effects, material properties, and other parameters associated with uncertainty of relevance for the design.

NOTE 1 The principles for the modelling of limit states, resistances, loads, and actions with due consideration to the relevant uncertainties are described in Clause 5 and Clause 6.

The design values applied for verification of design and assessment decisions shall be related explicitly to structural reliability using principles of reliability analysis as described in Clause 9. When developing design and assessment codes, design values shall be calibrated such that the level of reliability achieved for a structure of a certain type and use designed according to the code is close to the prescribed target nominal reliability (See 4.2, 8.3, and Annex G).

NOTE 2 In the text above, reference is made to nominal reliabilities with the aim to emphasize that the actual structural reliability can deviate from the reliability which is modelled and quantified. Such deviations are mainly caused by exclusion of human and organizational errors in the modelling.

For design and assessment of structures based on load and resistance factors or partial factors, the system performance shall be ensured, depending on the consequences of system failure, either through risk-based robustness assessments or through robustness provisions. The latter includes critical member design, structural ties, and structural segmentation and depends on the structural system and the consequences of system failure (See Annex F).

4.5 Documentation

Decisions related to the design of structures, as well as their verification with respect to acceptance criteria, shall be documented in a manner that is tractable and transparent for all involved stakeholders. This concerns design and assessment of individual structures, as well as development and calibration of design codes.

The documentation shall include all relevant information utilized for the design and assessment of the structures, including site-specific data, test results, models of the performance indicators, inspection results, information regarding damages, as well as maintenance and repairs, acceptance criteria and their verifications, quality control schemes and results, etc.

In addition, all relevant assumptions shall be identified, discussed with respect to their significance for the risk and reliability of the structure, and documented. This also includes assumptions concerning the use of the structures, envisaged maintenance, as well as possible requirements to the performance specified by the owner of the structure.

NOTE More details and suggestion on the documentation of decision related to structures can be found in Annex A, where also a birth certificate is suggested in certain cases.

5 Performance modelling

5.1 General

5.1.1 Structural performance and limit state concept

In order to assess the structural performance of a structure, the possible structural responses shall be assessed and divided into two domains consisting of desirable and undesirable states. The boundary between these domains is called the limit state and entering the undesirable domain is defined as failure. The limit state concept is elaborated further in 5.3.

NOTE 1 In general, this limit state concept is very helpful in assessing the performance of a structure. In some cases, however, there is a more gradual transition from desirable to undesirable behaviour with increasing costs of malfunctioning.

NOTE 2 The desirable performance can depend on the stakeholders like founders, owners, residents, operators, users, neighbours (if construction interferes with them), and contractors. Other stakeholders are the government and the society. Generally, the latter group is primarily interested in issues like safety and sustainability.

5.1.2 Performance and performance indicators

The performance of a structure relates to the structure as a whole or parts of it. In order to assess the performance, one shall select a set of quantitative performance indicators, which express physical states that can be used in relation to the performance requirements. Performance indicators can be defined on various levels of abstraction for the following:

— structural characteristics (e.g. stiffness/flexibility, load bearing capacity);

— response parameters (e.g. internal forces, stresses, deflections, accelerations, crack sizes);

— utilization factors;

— functionalities (e.g. safety for people, energy consumption, robustness, usability, availability, failure probabilities).

Models shall be set up to establish the relation between the various levels (see Clause 6).

5.1.3 Basic performance requirement and design situations

The performance requirements of Clause 4 shall be met for all relevant design/assessment situations, which can be classified as

— persistent situations, which refer to conditions of normal use of the structure,

— transient situations, which refer to temporary conditions of the structure, in terms of its use or its exposure, and

— accidental situations, which refer to exceptional conditions of the structure or its exposure.

In the elaboration of the performance requirements for the various design situations, spatial aspects, time variability, and deterioration shall be taken into account where relevant.

5.1.4 Levels of verification

In order to verify whether the structure is in compliance with the objectives for all design/assessment situations, one of the following levels shall be chosen:

a) Risk based: It shall be proven that, provided human safety aspects are taken care of in consistency with Clause 4 and/or local public law and codes, the sum of all costs (building costs, maintenance etc.) and economic risks (with respect to failure or malfunctioning) is at a minimum.

b) Reliability based: The structure shall fulfil a set of reliability requirements formulated as maximum admissible failure probabilities or minimum values for the reliability levels.

c) The semi probabilistic level: The structure shall fulfil a set of inequalities using certain design values of the basic variables.

The risk-informed level of verification is to be considered as the highest level. Lower levels of verification shall be calibrated to the higher levels using code calibration principles as per Clause 9.

NOTE Usually this calibration is performed by code committees allowing the designer to use level 3 verification methods; only for special structures a reliability or risk-based verification will be performed.

At all levels, use shall be made of limit state concepts and performance models (see 5.2 and 5.3).

5.2 Performance model

5.2.1 General

To establish the relation between exposures and structural properties on the one hand and the performance indicators on the other, a set of models shall be used.

Models shall be established to represent the structure itself, the environment (soil, water, air), as well as all relevant interactions of a mechanical, physical, chemical, biological, or anthropogenic nature. In many cases, it might be relevant to consider models for a system consisting of several structures, environmental entities, non-structural elements, devices and machinery, control systems, etc.

The degree of accuracy of models shall be chosen such that it is adequate for the application at hand; the corresponding uncertainties in the models shall be identified and defined as measurable quantities.

NOTE In general, it might be helpful to start with a risk-screening procedure followed by the definition of a set of hazard scenarios. The term hazard scenario refers to a set of situations, transient in time, that a system might happen to undergo and which can endanger the system itself, people, and the environment.

5.2.2 Time-dependent aspects

Performance models should consider ergodic, as well as non-ergodic (random and systematic), time variability of loads and structural properties.

In particular, one shall consider the following:

— time variability when analysing load effects due to simultaneous actions;

— dynamic effects when inertia forces are significant;

— degradation mechanisms, which can be of a mechanical nature (like fatigue, load duration effects), a physical/chemical nature (corrosion, chloride ingress,) or a combination thereof (stress corrosion).

In the last case, it might be necessary to include inspection, monitoring, and maintenance in the model.

NOTE See Annex B.

5.2.3 System aspects

The verification of the performance requirements shall consider and include the following:

— all relevant failure modes;

— interaction between failure modes;

— interactions between the structure and its environment (wind, water, soil, use);

— non-structural elements (partitions, ceilings, finishing, relevant electric, hydraulic, and mechanical devices);

— inspection and repair activities, quality management;

— functionality;

— environmental aspect (energy consumption, noise production);

— sustainability aspects (impact on human health, social property, biodiversity).

An adequate system level for the assessment procedure shall be selected; proper attention shall be given to a correct formulation of the physical and functional boundary conditions.

NOTE The latter is often achieved by code committees defining a set of constraints; for environmental and sustainability aspects, these constraints are still under development.

5.3 Limit states

5.3.1 Ultimate limit state

Ultimate limit states pertain to the following undesirable states (non-exhaustive):

— loss of equilibrium of the structure or part of it considered as a rigid body;

— instantaneous attainment of the maximum capacity of cross sections, members or connections by yielding, rupture or excessive deformations;

— failure of members or connections caused by fracture, fatigue, or other time-dependent accumulation effects;

— instability of the structure or part of it;

— sudden change of the assumed structural system to a new system (e.g. snap through, large crack formation);

— foundation failure.

The exceedance of an ultimate limit state is almost always irreversible and the first time that this occurs causes failure. The ultimate limit state can be the result from a single extreme action event or from a deterioration process over time followed by a (less) extreme action event.

Ultimate limit states can refer to structural elements, as well as to global structural systems; the latter including the effects of robustness (see Annex F).

5.3.2 Serviceability limit states

Serviceability limit states deal with loss of intended functionality related to normal use and can in particular pertain to the following undesirable states (non-exhaustive):

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

— excessive vibrations which cause discomfort to people or affect non-structural elements or the functioning of equipment;

— local damage affecting the appearance, the efficacy, or functional reliability of the structure;

— local damage (including cracking) which can reduce the durability of the structure or make the structure unsafe for use.

NOTE The last category is often referred to as the durability limit state (as for instance in ISO 13823).

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

In other cases, the exceedance of a serviceability limit state can be reversible and then failure occurs:

— The first time the serviceability limit state is exceeded, if no exceedance is considered as acceptable.

— If exceedance is acceptable but the time when the structure is in the undesirable state is longer than accepted.

— If exceedance is acceptable but the number of times that the serviceability limit state is exceeded is larger than specified.

— If a combination of the above criteria occur.

These cases can involve temporary local damage (e.g. temporarily wide cracks or leakage), temporary large deformations, and vibrations. Limit values for the serviceability limit state should be defined on the basis of their consequences.

5.3.3 Condition limit states

The condition limit states can correspond to the following situations:

— An approximation to the real limit state that is either not well defined or difficult to calculate. Examples are the use of the elastic limit as ultimate limit state and the use of depassivation as a limit state for durability (often also referred to as the initiation limit state).

NOTE 1 Theoretically, the requirement for sufficient durability is already covered by the fact that safety and serviceability are required for a certain period of time. However, for practical reasons it might be helpful to add specific durability related limit states (as in ISO 13823 and ISO 13822) or limit states referring to certain (non-critical) conditions. The reliability requirement for the condition limit state should be consistent with the original ultimate limit state.

— Local damage (including cracking) which can reduce the durability of the structure or affect the efficiency or appearance of structural or non-structural elements.

— Additional limit state thresholds in the case of a continuously increasing loss function.

NOTE 2 As mentioned in Note 1 of 5.1.1, a limit state presupposes the occurrence of a sudden loss when the exposure or circumstances only change little. If however in some case the losses occur gradually, a solution might be to introduce a subdivision of the undesired outcome space corresponding to several loss levels. As an example, it can be useful to define it like the limit states of initial damage, repair, and collapse in earthquake analysis.

5.3.4 Limit state function

For each specific limit state model, which describes the behaviour/performance of a structure, where available, a limit state function shall be established and the relevant basic variables shall be identified.

The limit state function is denoted as $g(\mathbf{X})$, where $\mathbf{X} = (X_1, X_2, ..)^T$ are the basic (random) variables.

Formula (1)

$$g(\mathbf{X}) = 0 \tag{1}$$

is called the limit state equation and the inequality

$$g(\mathbf{X}) < 0 \tag{2}$$

identifies the undesirable domain.

The set of basic variables **X** shall be defined in such a way that it contains all necessary random input information for the model; in general, the set of basic variables will consist of the following:

a) physical quantities, which characterize actions, environmental influences, material and soil properties, and geometrical dimensions;

b) model parameters that specify the model itself;

c) parameters that describe requirements related to the performance of the structural system (serviceability limits).

The limit state function can also contain explicit deterministic parameters and the time, t.

NOTE 1 For ultimate limit states and irreversible serviceability limit states, the minimum value $g(..)$ during the anticipated working life usually is decisive; for reversible limit states, a number of crossings or a certain (cumulated) time spent in the failure domain can be acceptable.

NOTE 2 The serviceability limit state can be defined by introducing a serviceability constraint, for instance, a displacement limit that is equal to c times the span L of a structural member. The constraint constant c (e.g. $c = 1/500$) in those cases can be interpreted as the design value of an underlying random variable. In that case, c can also be introduced as a random basic variable.

NOTE 3 In a component analysis where there is one dominating failure mode, the limit state condition can normally be described by a single equation according to Formula (1). In a system analysis, where more than one failure mode can be governing, there are several such equations. In a general system, the various failure modes can be represented in the form of mixtures of logical series and parallel systems.

6 Uncertainty representation and modelling

6.1 General

6.1.1 Types of uncertainty

Basic variables as defined in 5.3.4 can be representing one or more sources of uncertainty, as for instance inherent natural variability, statistical uncertainties, measurement uncertainties, uncertainties such as related to the precision of new information, and model uncertainties. All main sources of uncertainty shall be identified.

NOTE 1 The physical uncertainties are typically uncertainties associated with the loading environment, the geometry of the structure, and the material properties and are often referred to as aleatory uncertainties.

NOTE 2 Uncertainties arising from insufficient information, for example, due to a small number of materials tests or idealized models, are often referred to as epistemic uncertainty.

NOTE 3 A random variable can represent both aleatory and epistemic uncertainties. Furthermore, uncertainty can change nature in different phases of the lifetime of a structure. For example, a material property is considered as aleatory uncertainty before the construction of a structure; once the structure is constructed, it can be considered as epistemic uncertainty.

NOTE 4 In geotechnical design, the predominant sources of uncertainties are the soil properties and the calculation model uncertainty. These main sources of uncertainties are characterized in Annex D. Furthermore, Annex D presents the probabilistic modelling and incorporation of uncertainty into reliability analysis and design of geotechnical structures.

6.1.2 Treatment of uncertainty

All uncertainties that are considered to be of importance for the reliability assessment of a structural system shall be considered in the analysis using the theory of probability.

NOTE In the structural reliability analysis, no distinction should be made between the treatment of aleatory and epistemic uncertainties. This differentiation is introduced only for the purpose of setting focus on how uncertainty can be reduced by additional testing or more detailed research. It can also be a relevant issue in risk communication and the updating of structural reliability (see Annex B).

6.1.3 Interpretation of probability

In structural reliability analysis, the Bayesian interpretation of probability should be considered as the most adequate basis for the consistent representation of uncertainties, independent of their sources. It facilitates the joint consideration of purely subjectively assessed uncertainties, analytically assessed uncertainties and evidence as obtained through observations.

NOTE 1 In the Bayesian probability theory, the values for probabilities are estimated on basis of an adequate combination of data, theoretical arguments, and judgment. There is no distinction in the treatment between aleatory and epistemic uncertainty. If large amounts of data are available, the Bayesian interpretation coincides with the frequentistic one.

NOTE 2 Considering the probabilistic modelling of a random basic variable X, the principle in the Bayesian probability analysis is to model one or more of the parameters $\boldsymbol{\theta}$ (e.g. the mean or standard deviation) of the probabilistic model (e.g. the probability distribution function) as random variables themselves. The frequentistic or subjective information available about X can then be used in the probabilistic modelling of $\boldsymbol{\theta}$.

6.1.4 Probabilistic models

Depending on the nature of the reliability problem, basic variables can be represented as random variables, random processes, and random fields, discrete as well as continuous.

The probabilistic models shall describe the characteristics of the uncertainties of individual random basic variables, but also accommodate the consideration of the dependencies among them.

NOTE 1 Dependencies often arises due to causality, spatial and/or temporal correlations and/or ergodic states of underlying phenomena, as well as statistical and/or model uncertainties that commonly affect the uncertainties of the individual variables.

NOTE 2 Although in some cases it is necessary to model uncertain phenomena through random processes/fields, it is often convenient and acceptable to focus only on extremes and represent them through extreme value distributions, as far as ultimate limit states are concerned.

The probabilistic models should also accommodate the updating of the models by additional information, which can become available with, for example, experiments, tests, inspection, and monitoring. The Bayesian updating formula provides the operational basis for this.

NOTE Detailed information can be found in Annexes B and C.

6.1.5 Population/outcome space

The description of uncertain quantities by probabilistic models shall correspond to well-defined outcome spaces (or populations) and the results of the reliability analyses are only valid for the same sets.

NOTE 1 The word population seems to be more appropriate for aleatory variables, while outcome space is used for epistemic uncertainties.

The basis for the definition of an outcome space or population is in most cases given from the physical phenomenon defining or influencing the variable. Factors which can characterize the population are the following:

— Nature and origin of a random quantity.

— Spatial conditions (e.g. the geographical region considered).

— Temporal conditions (e.g. the design service life). The choice of a population shall be made with due consideration to the objective of the analysis and the amount and nature of the available data.

NOTE 2 It can, for instance, be convenient to subdivide a population into sub-populations (micro-zonation). When the results are used for design in a national or international code, it might be necessary or convenient to put the sub-populations together to the large population again in order not to get too complicated rules. This implies that the variability within the population is increased.

6.1.6 Hierarchical modelling of uncertainty

A hierarchical modelling is recommended for modelling different kinds of actions and materials, where applicable. The hierarchical model assumes that a random quantity X can be written as a function of several variables, each one representing a specific type of variability:

$$X_{ij\mathrm{k}} = f(Y_i, Y_{ij}, Y_{ij\mathrm{k}}) \tag{3}$$

The variables Y_i, Y_{ij}, $Y_{ij\mathrm{k}}$ represent various origins, time scales of fluctuation, or spatial scales of fluctuation.

NOTE For instance, Y_i can represent the constant in time variability, Y_{ij} a slowly fluctuating time process, and $Y_{ij\mathrm{k}}$ a fast-fluctuating time process. In the case of wind, we can have Y_i as the (model) uncertainty in the wind pressure coefficient C_p, Y_{ij} the mean hourly wind speed v_h, and Y_{ijk} the zero mean gust process $v(t)$:

$$X_{ij\mathrm{k}} = \frac{1}{2}\rho C_p \left(v_h^2 + 2v_h v\right) \tag{4}$$

where

ρ is the air density.

For concrete strength, Y_i can represent the building-to-building variation, Y_{ij} the floor-to-floor variation in building i, and $Y_{ij\mathrm{k}}$ the point-to-point variation on floor j.

6.2 Models for structural analysis

6.2.1 General

The decisive factors for describing the physical behaviour of the structural systems can be categorized as

— actions and environmental influences,

— geometrical properties,

— material properties and/or properties of structural elements, and

— active and passive control measures.

Models shall enable to describe the behaviour of structural systems up to the limit state under consideration. Models should generally be regarded as simplifications which take account of decisive factors and neglect the less important ones. Uncertainties in the models themselves are treated in 6.4.

The reliability of active and passive control measure (e.g. sprinkler installations or active dampers) shall also be modelled and incorporated in the overall risk analysis, but specific guidance on this is outside the scope of this International Standard.

6.2.2 Actions and environmental influences

6.2.2.1 General

Action descriptions shall be based on suitable mathematical models, describing the temporal, spatial, and directional properties of the action across the structure. The choice of the level of richness of details shall be guided by a balance between the quality of the information available and a reasonably accurate modelling of the action effect. The choice of the level of realism and accuracy in predicting the relevant action is, in time, guided by the sensitivity of the implied design decisions to variations of this level and the economical weight of these decisions. Thus, the same action phenomenon can give rise to different action models dependent on the effect and structure under investigation.

NOTE The environment, in which structural systems function, gives rise to internal forces, deformations, material deterioration, and other short-term or long-term effects. The causes of these effects are termed actions. The environment from which the actions originate can be of natural or man-made character, for example, snow, earthquake, fire. The following concepts are useful to characterize the aspects of actions:

— An action is an assembly of concentrated or distributed forces acting on the structure. This kind of direct action is also denoted as load.

— An action is the cause of imposed displacements or thermal effects in the structure. This kind of action is often denoted by indirect action.

Similar requirements hold for environmental influences that can cause changes with time in material properties or structural dimensions.

6.2.2.2 Classifications

Actions can be classified according to a number of characteristics. With respect to the type of the actions, the following non-exhaustive list shall be considered:

— Self weight of structural and non-structural components.

— Imposed loads in buildings, e.g. loads from persons and equipment.

— Loads caused by industrial activities, e.g. silo loads.

— Loads caused by transport: traffic, liquids in pipelines, cranes, impact, etc.

— Climate actions, e.g. snow loads, wind loads, outdoor temperature, etc.

— Indoor temperatures, fire.

— Hydraulic loads, e.g. water and ground water pressures.

— Geotechnical actions from soil or rock, including earth pressures, earth slides and earthquakes, sub-soil vibrations, settlements.

— Loads specific for the production and construction stages.

This classification does not cover all possible actions, but most of the common types of actions can be included in one or more classes. Some of the classes belong as a whole either to uncontrollable actions or to controllable actions. Other actions can belong to both, e.g. water pressure.

With respect to the variations in time, actions shall be considered as

— permanent actions,

— variable actions, and

— accidental actions.

Actions shall be classified as permanent actions if the variations in time around the mean is small and slow (e.g. self-weight, ballast, earth pressure) or monotonically change to a limiting value (e.g. prestressing, imposed deformation from construction processes, effects from temperature, moisture variation, or settlements).

Actions shall be classified as variable actions if the variations in time are frequent and large (e.g. all actions caused by the use of the structure and by most of the external actions such as wind and snow).

Actions shall be classified as accidental actions if the magnitude is considerable but the probability of occurrence for a given structure is small relative to the anticipated time of use. Frequently, the duration is short (e.g. impact loads, explosions, earthquakes, and snow avalanches).

NOTE 1 In this International Standard, an accidental action is not necessarily related to an accident.

NOTE 2 For some structures or at some locations, impacts or earthquakes can happen so often that they should be considered as a variable action.

As far as the spatial fluctuations are concerned, it is useful to distinguish between fixed and free actions. Fixed actions have a given spatial intensity distribution over the structure. They are completely defined if the intensity is specified in a particular point of the structure (e.g. earth or water pressure). For free actions, the spatial intensity distribution is variable (e.g. regular occupancy loading).

6.2.2.3 Action model

Where possible, complete action models shall be considered, consisting, in general, of several constituents which describe the magnitude, the position, the direction, the duration, etc. of the action. Sometimes, interactions between the components shall be taken into account. There can in certain cases also be an interaction between the action and the response of the structure.

In many cases, two kinds of variables shall explicitly be distinguished, i.e. F_0 and ω describing an action F, that is

$$F = \varphi(F_0, \omega) \tag{5}$$

where

F_0 is a reference action variable that is directly associated with the event causing the action and should be defined so that it is, as far as possible, independent of the structure (for example, for snow load, F_0 is the snow load on ground, on a flat horizontal surface);

ω is a conversion factor or model type parameter appearing in the transformation from the basic action to the action F, which affects the particular structure; the parameter ω can depend on the form and size of the structure, etc. (for the snow load example, ω is the factor, which transforms the snow load on ground to the snow load on roof, which depends on the roof slope, the type of roof surface, etc.);

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

The short-term, as well as long-term, time variability, is normally included in F_0 whereas ω can often be considered as time independent. A systematic part of the space variability of an action is in most cases included in ω, whereas a possible random part can be included in F_0 or in ω. Formula (4) should be regarded of principal character. For one action there can be several variables F_0 and several variables ω.

Any action model contains a set of parameters and variables that shall be evaluated before the model can be used. In probabilistic modelling, all action variables are in principle assumed to be random variables, random processes, or random fields, while other parameters can be time or spatial coordinates, directions, etc. Sometimes, parameters can themselves be random variables, for example, when the model allows for accommodating statistical uncertainty due to small sizes of samples on the basis of which the probabilistic model is developed.

NOTE 1 An action model often includes two or more variables of different character as described by Formula (4). For each variable, a suitable model should be chosen so that the complete action model consists of a number of models for the individual variables.

NOTE 2 It can often be convenient to define action models that describe explicitly the extreme values for a certain period of time, for instance, the distribution of the yearly maximum wind load or the extreme traffic load in the design working life.

6.2.3 Geometrical properties

The geometry of a structure shall be adequately described. In general, use can be made of standard elements like one-dimensional elements (beams, columns, cables, arches, etc.), two-dimensional elements (slabs, walls, plates, etc.), and three-dimensional elements (e.g. shells, layered half spaces). 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 overall structural behaviour or element capacity is sensitive to such imperfections, these shall be included in the model.

6.2.4 Material properties

6.2.4.1 General

Material models consisting of relations between forces or stresses on the one hand and deformations on the other (i.e. constitutive relationships) shall be formulated. The variables in such relations are the modulus of elasticity, the yield limit, the ultimate strength, etc. which generally shall be considered as uncertain quantities. Sometimes they are time dependent or space dependent. There is often a correlation between the parameters, e.g. the modulus of elasticity and the ultimate strength of concrete.

Other material properties, for example, resistance against material deterioration, can often be treated in a similar way. However, the principles are strongly dependent on type of material and the property considered.

6.2.4.2 Characterization

Material properties shall be defined as the properties of material specimens of defined size and conditioning, sampled according to given rules, subjected to an agreed testing procedure, the results of which are evaluated according to specified procedures.

The main characteristics of the mechanical behaviour are described by the one-dimensional stress-strain σ-ε diagram. As an absolute minimum for structural design, the following material properties shall be considered for both tension and compression:

— modulus of elasticity;

— strength of material.

Other important parameters in the one-dimensional σ-ε diagram, such as

— yield stress,

— limit of proportionality,

— strain at rupture and strain at maximum stress, and

— angle of friction, cohesion

might be as relevant.

The strain at rupture is a local phenomenon and the value obtained can heavily depend on the shape and dimensions of the test specimen. Additional to the one dimensional σ-ε diagram, information about a number of other quantities and effects is of importance, such as the following:

— multi-axial stress-strain relations;

— multi-phase materials (soil containing air and water);

— duration and strain rate effects;

— temperature effects;

— humidity effects;

— effects of notches and flaws;

— effects of chemical substances.

Possible dependencies between properties of a material shall be taken into account.

6.2.4.3 Material model

Structural materials shall be modelled by functions and parameters (material properties) describing (generalized) stress-strain relationships with a detailing as relevant; corresponding model uncertainties shall be taken into account.

Material properties vary randomly in space and time: The following discrepancies between measured and real properties can exist and shall be accounted for:

— Systematic deviations identified in laboratory testing by relating the observed structural property to the predicted property, suggesting some bias in prediction.

— Random deviations between the observed and predicted structural property, generally suggesting some lack of completeness in the variables considered in the model.

— Uncertainties in the relation between the material incorporated in the structural sample and the corresponding material samples.

— Different qualities of workmanship affecting the properties of (fictitious) material samples, i.e. when modelling the material supply as a supply of material samples.

— The effect of different qualities of workmanship when incorporating the material in actual structures, not reflected in corresponding material samples.

— Uncertainties related to alterations in time, predictable only by laboratory testing, field observations, etc.

— Uncertainties related to inspection procedures during or after fabrication.

6.2.5 Responses and resistances

6.2.5.1 Classification

The following mechanical models shall be used where relevant:

— Models describing static response.

— Models describing time dependent dynamic response.

— Models describing time dependent degradation mechanisms.

Interaction of the structure with the environment (in particular, air, water, and soil) shall be accounted for where relevant.

6.2.5.2 Models for static response

In analysis, appropriate models shall be selected for the relation between stresses, forces, or moments and corresponding deformations (or deformation rates). These models can vary and depend on the purpose and type of calculation. The following considerations shall be taken into account:

— In many cases, the elastic-plastic behaviour model, with plastic zones developing in the locations with the highest stresses, can be considered as sufficient.

— Advanced models can include softening behaviour based on general material degradation, as well as explicit or averaged crack formations, as well as phenomena like creep, relaxation, consolidation, etc.

— The theory of elasticity can be regarded as a simplification of a more general theory and can 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. The theory of elasticity can 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.) can be used, provided that the deformations which are needed to ensure plastic behaviour occur before the ultimate limit state is reached. Thus, the theory of plasticity shall be used with care to determine the load carrying capacity of a structure if this capacity is limited by brittle failure, instability, and repetitions of free variable actions (shake down mechanism).

— 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. The effects of such deformations are generally denoted geometrically nonlinear or second order effects. In this type of analysis also, initial imperfections need to be considered.

6.2.5.3 Models for dynamic response

Dynamic response of a structure is caused by a relatively fast fluctuation of the magnitude, position, or direction of an action. However, a sudden change of the stiffness or resistance of a structural element can also cause dynamic behaviour.

The models for dynamic response consist, in general, of the following:

— stiffness model, including geometrical and physical linear and nonlinear behaviour;

— damping model, including material, geometrical damping, artificial and active damping;

— mass model, including structural mass, content mass, and possible masses of surrounding media.

In the modelling of the dynamic behaviour, the interaction with air, water, soil, and possibly adjoining structures shall be considered.

Due to the short duration of dynamic (peak) loads, structures can survive dynamic loads above the static load bearing capacity. If that is the case, special attention shall be given to possible deformation limitations. In seismic analysis, this is part of the so-called capacity design approach.

NOTE In practice, dynamic calculations are often replaced by a quasi-static calculation where the dynamic effect is represented by a dynamic amplification factor on the static load.

6.2.5.4 Models for degradation and damage accumulation

Where relevant, the influence of damage on stiffness and strength of the structural elements shall be considered.

Degradation of material and structural properties can be caused by energy related to the following:

— mechanical effects (fluctuating actions, long-term loading, settlements, erosion, wear);

— physical effects (temperature, moisture, UV-light);

— chemical processes (fire, corrosion, alkali silica reaction);

— biological processes (corrosion, rotting of timber) origin.

In the case of fatigue failures caused by fluctuating actions, two types of models are distinguished:

— S-N model based on experiments.

— Fracture mechanics model.

Special care is to be given to possible "weak links" where strain concentrations can occur.

NOTE Reference is made to ISO 13823.

6.3 Models for consequences

In a risk-based approach, models for the direct and indirect consequences of structural failure shall be set up in order to evaluate the risk and enable optimization of safety measures and comparison to possible constraints.

Consequences shall be determined by modelling at least:

— The prior and post-failure behaviour of the structure.

— The amount of primary and secondary structural damage.

— Possible mitigating factors like warning systems.

— Human self-rescue actions.

— Professional rescue and mitigating actions like fire brigades.

— Repair and rebuilding activities.

— Loss of the structural functionality.

— Environmental losses.

When performing consequence analysis techniques like scenario analyses, fault and event tree analyses are recommended. Uncertainties in possible scenarios shall be accounted for.

Consequences shall be expressed numerically with respect to the extent of human fatalities and injuries and/or environmental damage and economic loss. In some cases, just a classification of consequences can be sufficient.

Models for estimating human fatalities usually consist of two parts: (1) the quantification of the total number of people at risk and (2) the probability that an exposed person can actually be killed or injured.

6.4 Model uncertainty

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

$$Y = f(X_1, X_2, ..., X_n) \tag{6}$$

where

Y	is the model output;
f	is the model function;
$X_1, X_2, ..., X_n$	are basic variables.

The model $f(\cdot)$ can be complete and exact, so that, if the outcomes of X_i are known in a particular situation (e.g. from measurements), the outcome Y can be predicted precisely. This, however, is not the normal case. Usually, the model will be incomplete and inexact. This might be the result of lack of knowledge, or a deliberate simplification of the model, for the convenience of the designer. To incorporate these aspects, Formula (6) should be changed into

$$Y = f'(X_1, X_2, ..., X_n, \theta_1, \theta_2, ..., \theta_m) \tag{7}$$

Here f' is the model function f extended with the variables $(\boldsymbol{\theta}_1, \boldsymbol{\theta}_2, ... \boldsymbol{\theta}_\mathrm{m})^T = 0$ in order to include the model uncertainty. The $\boldsymbol{\theta}$ variables represent the model uncertainties and are treated as random variables. Their statistical properties shall as far as possible be derived from experiments, observations of calculations from more accurate models. The mean of these parameters should be determined in such a way that, on average, the analysis model correctly predicts the test results.

6.5 Experimental models

In those cases where no adequate calculation model is available or existing models are considered as being too conservative, part of the design procedure can 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 can be used to evaluate or check assumptions about

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

— structural response under loading or accidental event, and

— strength or stiffness of a structure or structural element.

NOTE 1 Standard checks on material properties or other control tests are not considered as design based on experimental models. These tests are intended to check the assumptions already made in design.

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. The samples of these basic variables need not necessarily be representative; one can, 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 are taken from a representative sample.

The test results should be evaluated 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 can derive design values and partial factors to be used in the partial factors format.

NOTE 2 Further details are given in Annex C.

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

6.6 Updating of probabilistic models

In the case of relatively high uncertainties in actions, structural properties, and/or models, the possibility of updating procedures shall be considered in order to accomplish a more economical design or assessment solution.

Updating can be based on quality assurance procedures during and after construction, as well as lifetime inspection and monitoring planning. Given observations, so-called posterior distributions for random variables can be obtained. Sometimes it is more effective to calculate directly posterior failure probabilities. Uncertainties in inspection procedures shall always be taken into account.

Inspections can also be set up after events like fire or earthquake.

The economic efficiency of inspection activities can be obtained by decision making on the basis of so-called pre-posterior analyses.

NOTE See also Annexes A and B.

7 Risk-informed decision making

7.1 General

Decisions with respect to the design, construction, use/operation, assessment, repair, strengthening, maintenance, renewal, and decommissioning of structures shall take basis in risk assessments, ensuring that benefits are optimized and at the same time that risk to life and qualities of the environment are managed in accordance with societal preferences as described in 4.2.2.

Risk assessments shall be performed in accordance with ISO 13824 and as further elaborated in the subsequent clauses.

NOTE Risk-informed decision making as compared to reliability-informed decision making includes directly and explicitly all consequences associated with the decisions, including consequences caused by structural failures but also in terms of the benefits achieved from the operation of the structures. Risk-informed decision making is thus more consistent with available information and comprises a richer basis for the optimization of decisions concerning structures over their life cycle.

7.2 System identification

As a first task in risk-informed decision making concerning structures, the system shall be identified. In particular,

— the spatial and temporal boundaries of the system shall be fixed and documented,

— the possible and relevant exposure events, constituents, direct consequences, and indirect consequences shall be identified as described in 4.3.2, and

— the various possible measures of reducing the risks shall be identified with reference to their costs, their effect on exposure events, and the direct and indirect consequences. Moreover, consideration shall be given to identify measures of risk reduction which are relevant before possible damages of the constituents, during the damage evolution and after the damage evolution.

NOTE 1 It is recommended to undertake the system identification in a joint effort, in terms of a risk screening, involving the owner of the structure, as well as a range of subject matter experts covering the relevant knowledge for the considered structure in the given context. As the number of possible different scenarios can be very large, it is important to identify which scenarios are irrelevant and thus can be excluded in a detailed risk assessment.

NOTE 2 Risk-reducing measures can be of passive (e.g. increasing cross-sectional dimensions, implementing structural ties, increasing the concrete cover thickness) or active nature (e.g. structural monitoring, inspections and maintenance strategies, and smoke and fire detection systems). Moreover, risk-reducing measures such as quality control, *in situ* soil investigations, materials testing, observation of environmental conditions, etc. can also be identified which do not change the system but rather establishes improved knowledge about the performance of the system. Improved knowledge facilitates the optimization of risk-reducing measures involving physical changes of the system.

7.3 System modelling

The system modelling shall address and describe the uncertainty associated with the system as specified in 4.3.3. This includes a Bayesian probabilistic representation of the exposure events, all relevant states of damages and failures of the constituents of the system, the direct and indirect consequences, as well as the effects of possible risk-reducing measures. The performances of the constituents shall be modelled according to the principles specified in Clause 5.

NOTE Dependencies between different exposure events, as well as among the performances of constituents, can significantly influence the risks and should be carefully considered in the modelling.

In the modelling of the benefits associated with different measures of risk reduction, discounting shall be applied for all consequences, as well as costs which might occur in the future.

7.4 Risk quantification

The risk associated with the identified possible measures of risk reduction shall be assessed as the expected value of the sum of the direct and indirect consequences, where the expectation operation is taken over all uncertainties affecting the performance of the system. Additional indicators of risk such as probable maximum loss, value at risk, etc. can be assessed in addition as found relevant in a given context.

For each considered measure of risk reduction a, the corresponding risk $R(a)$ comprises a number n_E of contributions arising out of the possible consequence events which can result from the decision:

$$R(a)=\sum_{i=1}^{n_E} P_i C_i \tag{8}$$

where

P_i and C_i are the probability and the consequence associated with event i, respectively. Both the number of possible events, their probabilities, as well as the associated consequence, generally depend on the decision a.

The probabilities P_i in Formula (8) shall be assessed according to the principles provided in Clause 8. When representing the events associated with consequences probabilistically, the principles of uncertainty modelling and the concept of limit states described in Clause 5 and Clause 6 shall be applied.

NOTE Based on the assessment of the direct and the indirect risks, an assessment of the robustness of structures can be undertaken for the support of design decisions (see 4.3.2 and Annex F).

7.5 Decision optimization and risk acceptance

The optimization of decisions for risk reduction shall take basis in a ranking of their associated benefits. The general principles for optimizing decisions on the basis of information about risks are provided in 4.4.2.1.

The assessment of the acceptability of decisions with regards to implications on life safety shall follow the principles described in 4.2.2 (see also Annex G for more information).

NOTE Risk-informed decision making can be applied directly as basis for decisions concerning structures throughout their entire life cycle; however, it can also be applied for the purpose of setting maximum acceptable nominal failure probabilities for structures and thereby support reliability-based approaches, as well the formulation and regulation of semi-probabilistic safety formats.

8 Reliability-based decision making

8.1 General

The general basis for reliability based decision making is described in 4.4.2.2. Decisions with respect to the design, repair, strengthening, maintenance, operation, and decommissioning of structures shall fulfil given requirements to reliability, or equivalently, requirements to the probability of failure.

It is assumed that the uncertainty models in Clause 6 can be represented by stochastic variables or stochastic fields/processes and the limit state approach in Clause 5 can be used to model the relevant events, e.g. failure events. Further, it is assumed that the reliability can be assessed by time-variant or time-invariant reliability methods such as FORM/SORM and Monte Carlo Simulation techniques.

During the life cycle of a structure, decisions of different types and based on varying degrees of information have to be made. This includes decisions relating to the following:

— Design with respect to ultimate and serviceability limit states.

— Planning of tests and quality control to be performed during the design process and execution of the structure. This includes e.g. soil investigations, material tests, coupon tests, subcomponent tests, full-scale tests, proof load tests, numerical tests, and quality control procedures.

— Planning of inspections and monitoring during operation as criteria for future repair and maintenance.

— Decisions on life extension and removal/replacement of the structure at the end of the design lifetime.

A reliability-based decision implies that the probability of failure, p_f, does not exceed a specified target value, p_ft (see 8.4):

$$p_\mathrm{f} \leq p_\mathrm{ft} \tag{9}$$

The reliability requirement shall be verified for all relevant design situations related to the decisions described above. The design situations shall be sufficiently severe and varied so as to encompass all conditions that can reasonably be foreseen to occur during the construction and use of the structure. This typically refers to loading conditions subject to normal operation, extreme environmental loads, exceptional and accidental loads, during the different phases of the life of the structure.

Failure is associated with a limit state according to Clause 5 and, in the case of a time-invariant reliability problem, the undesired state is defined by the limit state equation:

$$g(\mathbf{x}) \leq 0 \tag{10}$$

where

$\mathbf{x}$ is a vector containing the realizations of the basic variables X which are relevant to the problem.

In general, the basic variables which describe variable actions and environmental influences should be described using stochastic processes or random fields. In many cases, however, a description as a random variable with a probability distribution function for the maximum within a given reference period or domain can be sufficient. Other basic variables (such as material or geometry characteristics) can also be time or location dependent.

The model uncertainty parameters 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:

$$p_{\mathrm{f}} = P\left[g(\mathbf{X}) \leq 0 \right] \tag{11}$$

In the case of time-dependent variables, the problem is time variant and in principle first-excursion or out-crossing approaches should be applied to assess the probability of failure. However, in some cases, the time-variant problem can readily be transformed to time-invariant problems (see ISO 13822).

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

Due to the dependence upon time, p_{f} shall be referred to a certain a priori specified period of time, the reference period. Lifetime probabilities can be used if economic consequences are determining. If a failure can be expected to endanger people, other reference periods might be used, typically one year.

The failure probability, p_{f}, is related to the reliability index β through the definition:

$$\beta = -\Phi^{-1}(p_{\mathrm{f}}) \tag{12}$$

where

Φ^{-1} is the inverse standard normal probability distribution function.

Reliability methods can be applied to calibrate the partial factors format outlined in Clause 9. In some cases, a probability based method can be applied in a direct design and decision making.

The reliability obtained by Formula (11) gives a well-defined probabilistic measure of the reliability (e.g. probability of failure). This value can be used for consistent comparisons between various design situations for decision making and for calibrations with regard to a specified, required degree of reliability.

NOTE In general, quality management systems for construction works shall be risk-based and according to an integral approach, encompassing human errors, design errors, and execution errors (see Annex A).

The degree of reliability can be differentiated according to the consequences of failure as indicated in 4.3.

8.2 Decisions based on updated probability measures

If results from tests, quality control, inspections, condition monitoring, structural health monitoring, etc. are available, the probability of failure used in Formula (11) should be updated using Bayesian statistical methods. See Annex B and Annex C for assessment of existing structures.

The updated probability of failure should be used for decision making with respect to repair, maintenance, and possibly upgrading and as basis to plan future inspections ensuring that the reliability requirement in Formula (9) is satisfied during the entire life cycle of the structure.

8.3 Systems reliability versus component reliability

The performance of structural systems, as well as systems involving structures, can be described in terms of scenarios of events of failures of constituents as described in 4.3.2. The scenarios can describe a sequence of failures of individual failure modes of a structural system, e.g. from first damage to total collapse.

Traditionally, reliability-based decision making for design and assessment is primarily applied to components and individual limit states (serviceability — and ultimate failure). Systems behaviour is of concern because systems failure is usually the most serious consequence associated with failure of a structure. It is therefore of interest to assess the probability of system failure following an initial component 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 component reliability requirements should depend upon the systems characteristics. See also requirements to robustness in Annex F.

A reliability assessment should therefore be carried out to establish the probability of scenarios which might lead to structural systems failure including identification and consideration of the following:

— The sequences of failures of individual constituent failure modes leading to system failure modes.

— The ability of the structure to allow for redistribution of internal forces in given failure states.

— Possible means of ensuring that reliable redistribution of internal forces is facilitated (tying).

— Possible means of ensuring that unreliable redistribution of internal forces is avoided (segmenting).

— Dependencies between failures of individual constituent failure modes and system 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.

8.4 Target failure probabilities

The target failure probabilities, i.e. p_{ft}, should be chosen taking into account the consequence and the nature of failure, the economic losses, the social inconvenience, effects to the environment, sustainable use of natural resources, and the amount of expense and effort required to reduce the probability of failure. If there is no risk of loss of human lives associated with structural failures, the target failure probabilities can be selected solely on the basis of an economic optimization. If structural failures are associated with risk of loss of human lives, the marginal lifesaving costs principle applies and is recommended, and this can be used through the LQI as shown in Annex G. In all cases, the acceptable failure probabilities should be calibrated against well-established cases that are known from past experience to have adequate reliability.

NOTE Tentative target reliabilities related to one-year reference period and ultimate limit states can be found in Annex G.

When dealing with time-dependent structural properties, the effect of the quality control and inspection and repair procedures on the probability of failure should be taken into account. This can 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.

For serviceability limit states, target failure probabilities shall be consistent with the objective to limit the loss of functionality and/or the occurrence of damage to economically acceptable levels.

For reversible serviceability limit states, there can also be requirements on the frequency of passing the limit state (see Clause 5).

8.5 Calculation of the probability of failure

8.5.1 General

Fundamentally, the calculation of the probability of failure shall take basis in all available knowledge, and the uncertainty representation shall include all relevant causal and stochastic dependencies, as well as temporal and spatial variability. The appropriate choice of method for the calculation of the failure probability depends on the characteristics of the problem at hand and especially on whether the problem can be considered as being time-invariant and whether the problem concerns individual failure modes or systems.

8.5.2 Time-invariant reliability problems

In case the problem does not depend on time (or spatial characteristics), or can be transformed such that it does not, e.g. by use of extreme value considerations, three types of methods can in general be used to compute the failure probability p_f namely the following:

a) First/Second Order Reliability Methods (FORM/SORM).

b) Simulation techniques, e.g. crude Monte Carlo simulation, importance sampling, asymptotic sampling, subset simulation, and adaptive sampling.

c) Numerical integration.

8.5.3 Transformation of time-variant into time-invariant problems

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

— failures caused by extreme values, and

— failures caused by the accumulation and progression of effects over time.

In the case of failure due to extreme values, a single action process can , provided stationarity, be replaced by a random variable representing the extreme characteristics (minimum or maximum) of the random process over a chosen reference period, typically one year. If there is more than one stochastic process involved, they should be combined, taking into account the dependencies between the processes.

In the case of failures due to cumulative deterioration (fatigue, corrosion, etc.), the total history of the load up to the point of failure might be of importance. In such cases, the time dependency can be accounted for by subdividing the considered time reference period into intervals and to model and calculate the probability of failure as the probability of failure of the logical series system comprised by the individual time intervals.

NOTE Examples are provided in ISO 13822.

8.5.4 Out-crossing approach

An exact and general expression for the failure probability of a time varying process on a time interval $(0,t)$ can be derived from integration of the conditional failure rate $h(\tau)$ according to Formula (13):

$$p_f(0,t) = 1 - \exp\left[-\int_0^t h(\tau)\, d\tau\right] \tag{13}$$

The conditional failure rate is defined as is the probability that failure occurs in the interval $(\tau, \tau + d\tau)$, given no failure before time τ. When the failure threshold is high enough, it can be assumed that the conditional failure rate $h(\tau)$ can be replaced by the average out-crossing intensity $\nu(\tau)$:

$$\nu(t) = \lim_{\Delta \to 0} \frac{P(g(X(t)) > 0 \cap g(X(t+\Delta) \leq 0)}{\Delta} \tag{14}$$

If failure at the start ($t = 0$) explicitly is considered:

$$p_f(0,t) = p_f(0) + \left[1 - \exp\left[-\int_0^t \nu(\tau)\, d\tau\right]\right] \tag{15}$$

in which $p_f(0)$ is the probability of structural failure at $t = 0$. The mathematical formulation of the out-crossing rate $\nu(\tau)$ depends on the type of loading process, the structural response, and the limit state. For practical application, Formula (15) might need to be extended to include several processes with different fluctuation scales and/or time invariant random variables.

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

8.6 Implementation of probability-based design

Design decisions can, in accordance with 8.1 to 8.5, be directly based on probabilistic analysis, facilitating that specified requirements to the reliability are satisfied. Such an approach can be applied subject to the availability of

— uncertainty models,

— reliability methods, and

— expertise in probabilistic analysis.

In many, if not most, cases it is, however, possible to simplify the design process by means of semi-probabilistic design methods.

9 Semi-probabilistic method

9.1 General

Semi-probabilistic methods can be applied as an alternative to risk- and reliability-based decision making for a wide range of decisions of relevance for design, repair, strengthening, maintenance, operation, and decommissioning of structures. The semi-probabilistic safety formats shall, however, be formulated such that they facilitate the identification of acceptable and feasible decisions concerning new structures, as well as existing structures.

Basis shall be taken in generic information concerning loads, materials, operational conditions, workmanship, inspection and maintenance, monitoring, and quality control in such a manner that the risk and the reliability of the structure are adequate and ensured in conformance with the principles given in 4.4 and 8.4.

For semi-probabilistic safety formats, all relevant information concerning possible limitations and assumptions with respect to their validity and application should be specified. This includes specification of

— legal, temporal and geographical limitations (e.g. project, altitude, and expiry/revision date),

— types of structures (such as building structures, offshore structures, foundation structures, etc.),

— types of materials (such as concrete, steel, timber, composites, and soil),

— types of loads (such as permanent, wind, snow, traffic, earthquakes, and waves) and relevant load combinations for variable and permanent loads including consideration of stabilizing and destabilizing permanent loads, and

— types of uses (such as hospitals, offices, storage, and energy production/distribution).

NOTE Semi-probabilistic methods usually reduce random variables to a set of design values with a specific use: extreme favourable or unfavourable values, load combination values, serviceability level values, etc. Calibration to higher levels of analysis can take place on the basis of an individual value, as well as for a complete set of design rules.

9.2 Basic principles

Semi-probabilistic safety formats shall comprise

— consequence class categorizations (see 8.4 and Annex G),

— design situations (see 8.1),

— design equations, and

— design values.

Design equations shall be formulated according to the principles given in Clause 5 and Clause 6, taking basis in the modelling of structural performance in terms of limit state functions and the probabilistic modelling of uncertainties. The principal form of design equations is given in Formula (16), which for all relevant failure modes of the structures shall facilitate that values of structural parameters, such as cross-sectional properties $\boldsymbol{z}$, can be determined uniquely and such that these are conforming with given requirements to risk and reliability (see also 8.4).

$$G(\boldsymbol{z}) = R_{\mathrm{d}}(\boldsymbol{z}) - S_{\mathrm{d}}(\boldsymbol{z}) > 0 \tag{16}$$

where

$\boldsymbol{z}$ is a vector of design parameters (e.g. the cross-sectional dimensions);

$R_{\mathrm{d}}(\boldsymbol{z})$ is the design value for the resistance;

$S_{\mathrm{d}}(\boldsymbol{z})$ is the design value for the action effect.

The resistance is assumed to be obtained by the following general model (see Annex C):

$$R = b\ \theta\ R(X, a) \tag{17}$$

where

$R(X, a)$ is the resistance model as defined in a relevant materials standard;

X are the material properties;

a are the geometrical parameter(s) (the design parameters $\boldsymbol{z}$ are generally a subset of a);

θ is the model uncertainty related to resistance model (can be determined using the method in the Annex C);

b is the bias in resistance model (can be determined using the method in Annex C).

Design equations shall in general be formulated for failure modes involving in principle both failure of individual cross sections of the structures, as well as for failure modes involving the failures of several cross sections of the structures.

The design values for the various actions and materials characteristics entering the design equations should be determined such as to account for the characteristics of the uncertainties associated with the loads and resistances which are of relevance for the given design situation.

9.3 Representative and characteristic values

9.3.1 Actions

A permanent action has often a unique characteristic value. When the action refers to the self-weight of the structure, its value G_{k} should be obtained from the specified values of geometrical quantities and the mean unit weight of the material. However, in some cases, it might be necessary to define two values, one upper and one lower characteristic value of a permanent action.

A variable action has often the following representative values Q_{rep}:

— characteristic value Q_{k};

— combination value $\Psi_0 Q_{\mathrm{k}}$;

— frequent value $\Psi_1 Q_{\mathrm{k}}$;

— quasi-permanent value $\Psi_2 Q_k$.

The characteristic value for a variable action 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 (see Annex E).

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, and

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

NOTE There might, 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 can have a unique characteristic value A_k which is also used as design value.

9.3.2 Resistances

Properties of materials are defined for some relevant volume of material and are represented by their characteristic values X_k. For a produced material, the characteristic value should in principle be presented as an a priori specified quantile 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.

Material properties to be used in nonlinear analyses can either be based on design values, characteristic values, or mean values provided a consistent safety concept is used that results in a design with the required target reliability.

9.4 Safety formats

9.4.1 General

Semi-probabilistic safety formats can take basis in different approaches. Common for the different approaches is that the risk shall be ensured to be acceptable through adequate choices of design situations, design equations, and representative values.

Other formats can be considered as long as they provide an adequate level of risk and/or level of reliability as the direct use of risk and reliability methods.

The design values entering into the design equations shall be chosen such as to ensure that an adequate and sufficient level of reliability is achieved for all relevant failure modes of the considered structures.

The partial factor format shall as, a general rule, be utilized as basis for the definition of semi-probabilistic safety formats.

9.4.2 Partial factor method

9.4.2.1 Actions

For a specific load case i, the design values of the effects of actions E_d can be expressed in general terms as:

$$E_{d,i} = \gamma_S S\left[G_{d,i}; Q_{d,i}; \boldsymbol{a}_d\right] \tag{18}$$

where

$\boldsymbol{a}_d$ is a vector containing the design values of the geometry;

γ_S is a partial factor taking account of uncertainties

— in modelling the effects of actions;

— in some cases, in modelling the actions.

NOTE In a more general case, the effects of actions depend on material properties.

Design values for permanent and variable actions are

$$G_d = \gamma_G G_k \tag{19}$$

$$Q_d = \gamma_Q Q_{rep} \tag{20}$$

where

γ_G, γ_Q are the partial factors for permanent and variable actions, respectively.

9.4.2.2 Resistances

The design value of the resistance, R_d, can be determined by different models:

Model 1 where a design value of the resistance is determined using design values of the material resistance parameters:

$$R_d = \frac{R(\boldsymbol{X}_d, \boldsymbol{a}_d)}{\gamma_R} \tag{21}$$

where

$\boldsymbol{a}_d$ are design values for the geometry;

$\boldsymbol{X}_d$ are design values for resistance parameters;

γ_R is a partial factor related to the model uncertainty for the resistance model – including possible uncertainty related to the transformation from laboratory to real structure and bias in the resistance model.

Design values for material resistance parameters are determined as

$$X_{\mathrm{d}} = \eta \frac{X_{\mathrm{k}}}{\gamma_{\mathrm{m}}} \tag{22}$$

where

η is the conversion factor taking into account load duration effects, moisture, temperature, scale effects, etc.;

X_{k} is the characteristic value of the resistance parameter generally defined by the 5 % quantile;

γ_{m} is the partial factor for material property.

For geometrical quantities, the design values $\boldsymbol{a}_d$ usually correspond to dimensions specified by the designer.

If more than one resistance parameter is used in the resistance model, then design values are applied for each resistance parameter in Formula (21).

The partial factor γ_{m} depends on the uncertainty of the resistance parameter(s) and γ_{R} depends on the uncertainty of the resistance model, including bias:

$$\gamma_{\mathrm{R}} = \frac{\gamma_{\theta}}{b} \tag{23}$$

where

γ_{θ} is a partial factor depending on the model uncertainty.

Model 2 where a characteristic value of the resistance is obtained using characteristic values of the material resistance parameters:

$$R_{\mathrm{d}} = \frac{R(\eta\, X_{\mathrm{k}}, a_{\mathrm{k}})}{\gamma_{\mathrm{M}}} \tag{24}$$

where

γ_{M} is a partial factor related to uncertainty of the resistance parameters $\mathbf{X}$ through the resistance function $R(\mathbf{X},\mathbf{a})$.

The total uncertainty of the resistance depends on the model uncertainty θ and the uncertainty related to the resistance parameters $\mathbf{X}$ through the resistance function $R(\boldsymbol{X},\boldsymbol{a})$. The material partial safety factors are correspondingly obtained from

$$\gamma_{\mathrm{M}} = \frac{\gamma_{\theta}\gamma_{\mathrm{R}}}{b} \tag{25}$$

where

γ_{R} is a partial factor depending on the uncertainty related to the resistance parameters $\mathbf{X}$ through the resistance function $R(X,a)$;

γ_{θ} is a partial factor depending on the model uncertainty.

Model 3 where a design value of the resistance is determined using a characteristic value of the resistance estimated based on tests:

$$R_d = \frac{R_k}{\gamma_M} \tag{26}$$

where

R_k is the characteristic value for the resistance estimated based on tests (see the Annex C) (R_k is generally defined by the 5 % quantile);

γ_M is the partial factor related to the uncertainty of the resistance obtained based on tests including statistical uncertainty.

The load partial factors and the material partial factors γ_m, γ_R, and γ_θ should be calibrated such that failure probabilities for the relevant failure modes are close to the target reliability level in 8.4.

Whereas the representative values in the design equations are determined in terms of quantile values which are selected according to convention, the partial factors and the load combination values can be determined by means of calibration.

The partial factors and the representative values shall be determined such that they take into account both the aleatory and epistemic uncertainties of relevance for the considered design situation and failure modes. Typically, representative values for variables of importance for action effects are selected as upper quantile values whereas representative values of importance for resistances are selected as lower quantile values.

The calibration shall be undertaken by choosing the partial factors and load combination values such that when the semi-probabilistic safety format is applied on a set of structures, the difference between the achieved probability of failure and the maximum acceptable probability of failure is minimized over the entire set of structures. The procedure for the calibration of partial safety factor based codes is outlined in detail in Annex E.

NOTE The load and resistance factor design (LFRD) method follows basically the same principles as the partial factor method.

9.4.3 The design value method

The design value method takes basis in a direct check of the relevant design situations and corresponding design equations using design values for the basic variable which are determined on the basis of reliability assessments. The design values can be determined using simplified methods of direct use of First Order Reliability Methods (FORM) as shown in Annex E.

9.5 Verification in case of cumulative damage

In case of limit states involving a cumulative, non-decreasing damage measure, a reliability verification shall take place for the last year of the planned service life:

$$R_\mathrm{d}\left(\boldsymbol{X},\boldsymbol{a},n\right) > S_\mathrm{d} \tag{27}$$

where

N	is the number of years for the planned service life;
S_d	is the design value of the extreme action effect in the last year based on the annual reliability target;
$R_\mathrm{d}(\boldsymbol{X},\boldsymbol{a},n)$	is the design value of the resistance at the end of the lifetime, based on the original state, all action effects during the lifetime and the cumulative damage. Design values for $\boldsymbol{X}$ and $\boldsymbol{a}$ can be based on a target reliability related to the service life.

Using inspection programs the requirement can be released (see Annex B).

NOTE A cumulative damage measure can, for example, be used to model degradation of structures (due to corrosion, fatigue, etc.).

Annex A
(informative)

Quality management

A.1 Objectives

The objective of this annex is the validation of assumptions made in the risk- and reliability-based decision-making process by means of quality management, quality assurance, and quality control. In general, quality management, quality assurance, and quality control shall be in accordance to the ISO 9000-series.

In general, the construction works should

a) meet defined requirements, uses, or purposes,

b) satisfy customer expectations,

c) comply with applicable standards and specifications, and

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

In case of construction works at least a minimum basic level of quality management, assurance and control is required.

A.2 Definitions

A.2.1 General definitions related to quality management, quality assurance, and quality control

General definitions related to quality, quality management, quality assurance, and quality control shall be in accordance with ISO 9000:2005.

A.2.2 Definitions relating to Annex A

A.2.2.1 Sampling plan: A sampling plan is a detailed procedure of how samples should be obtained with respect to quality control. This can consist of a description of the type of information/measurements needed for the quality control, the sampling time interval, how and by whom this information has to be obtained. Sampling plans should be designed in such a way that the resulting data will contain a representative sample of the entity or its characteristic of interest.

A.2.2.2 Quality level: A qualitative or quantitative degree of quality which is aimed for or achieved.

A.2.2.3 Acceptable quality level (AOQ): The poorest level of quality that, for the purpose of compliance control, is considered satisfactory as a process average.

A.2.2.4 Limiting quality (LQ): The poorest level of quality of a characteristic of an entity that is considered acceptable to pass the compliance control.

A.2.2.5 Fraction defectives: Percentage of a distribution of a characteristic of an entity beyond a specified value.

A.2.2.6 Average outgoing quality (AOQ): Fraction defectives of the unknown distribution of a characteristic of an entity beyond a specified value multiplied by the corresponding acceptance probability associated to that distribution when using the applied compliance control.

A.2.2.7 Average outgoing quality limit (AOQL): Maximum allowed value of the average outgoing quality (corresponding to the maximum average fraction defectives of a characteristic of an entity as a result of the applied compliance control).

A.3 Quality management

Quality management shall encompass an integral risk-based approach on quality assurance and control. Also, human errors, design errors, and execution errors shall be considered as part of the integral quality management system.

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

Quality assurance in order to achieve an adequate confidence that the design fulfils the specified requirements for quality implies that the following 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 and/or service life design — Specification for suppliers — Preliminary specifications for execution and maintenance — Choice of intervening parties with appropriate qualifications for personnel and organization

[a] Aspects of quality management activities which can result in Bayesian updating of characteristics of entities on the basis of quality control (see A.5.5).

Table A.1 *(continued)*

Stages of the life cycle	Activities
Design	— Specification of performance criteria for materials, components, and assemblies — Specifications of service life and/or life cycle performances — Confirming acceptability and achievability of performance — Specification of test options (prototype, *in situ*, etc.) — Specification for materials
Tendering	— Reviewing design documents, including performance specifications [a] — Acceptance of requirements (contractor) — Acceptance of tender (customer)
Execution and inspection	— Control of procedures and processes[a] — Sampling and testing[a] — Correction of deficiencies[a] — Evaluation, correction and/or updating of design assumptions[a] — Certification of work according to compliance tests specified in the design documentation[a]
Completion of the construction works and handover to the customer	— Commissioning — Verification of performance of completed building (e.g. by testing under operational loads)[a]
Use and maintenance	— Monitoring performance[a] — Inspection for deterioration or distress[a] — Investigation of problems[a] — Assessment of structural performance[a] — Certification of work[a]
Rehabilitation or demolition	— Establishing appropriate levels of performance for existing structures and construction works with respect to rehabilitation — Assessment of necessity of rehabilitation or demolition[a] — Activities for design, tendering, execution, completion, use, and maintenance similar to above in case of rehabilitation[a] NOTE Demolition is outside the scope of this International Standard.

[a] Aspects of quality management activities which can result in Bayesian updating of characteristics of entities on the basis of quality control (see A.5.5).

In case of quality assurance with respect to structural calculations, these should in general typically specify

— design criteria, including discussion and description of the design basis, as well as the assumptions made,

— list of dead, live, and/or environmental loads,

— specifications for materials used,

— geotechnical information (if relevant), and

— drawings as-built for structural elements.

Considering the relevance according to a simplified sensitivity analysis, an advanced structural reliability calculation with respect to load-structure interactions can include the following:

— Vertical load analysis and design of roof structures, floor structures, frames or trusses, columns, walls, and foundations.

— Lateral load analysis and design for seismic and wind action.

— Dynamic analysis.

In case of the use of software tools, the user should know the principles underlying the software in order to produce meaningful calculation results. A sound engineering judgment is needed for deciding which features should be accurately modelled or simplified. Software calculations should be embedded in a tailored safety philosophy. Structural reliability calculations using simplified structural models should be used as a control.

In case lack of quality of specific quality management activities has a disproportionate effect on the structural robustness, quality assurance and control of these specific activities or structural details shall be intensified. Risk-based methods can be used to indicate the activities where a potential disproportionate effect of lack of quality might endanger structural robustness.

For structures for which the consequences of failure and damage are high, i.e. Consequence Class 3-5 defined in Annex F, a Structural Certificate shall be issued. It is the responsibility of the owner that this certificate is established, safely kept, and regularly updated.

The Structural Certificate shall be issued at the time when the structures is handed over to its owner and shall include the following information:

— Owner-specified requirements to the geometry, materials, use, and performance of the structure.

— References to the documentation for the design and construction of the structure, whether based on risk, reliability or semi-probabilistic approaches.

— Documentation of assumptions with respect to strategies and procedures for condition control, inspection, maintenance, and repair.

— Documentation of the quality control undertaken concerning materials production, design, and construction.

— Documentation of the structure "as-built" — commissioning — together with an assessment of possible non-conformities and how these have been treated.

— Documentation on performed condition control, inspections, and maintenance, as well as repairs and other modifications.

— Documentation of evacuation plans and other loss reduction activities for relevant types of accidents and incidents.

— Documentation on a running "fit for purpose" assessment which is updated after each planned and performed condition control activity.

A.5 Quality control

A.5.1 General

Applying quality control implies that the following actions should be taken:

— Collection of information.

— Judgement based on this information.

— Decision based on the judgement.

A.5.2 Control procedure

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

— 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;

— Conformity control, which is control during or after the construction process or of the result of a production process; the purpose of this control is to ensure that the result of a production process conforms to the given specification.

NOTE In case of construction works, conformity control is often best performed before or during the construction process, e.g. in case of placement of reinforcement.

As both control procedures have different objectives, methods for performing either production or conformity control should in general also be differentiated.

The control procedure, as well as possible non-conformity actions, should be specified in advance.

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.

With respect to compliance control, a distinction can be made between the following:

— Control by attributes, when a unit is in a state of 'good' or 'bad' and the decision yields 'accepted' or 'not accepted'.

— Control by variables, when a unit can be evaluated according to a scale of measurement.

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 can be regarded as homogeneous (both in space and time) with regard to the properties which are the subject of the control. Batching and sampling of the products should be executed according to a specified sampling plan. The judgement of the results should normally be made with regard to either

— a given level of confidence and/or a given interval of confidence,

— a specified performance of the operating characteristic associated to the compliance control, either in terms of acceptance probabilities at the acceptable quality level (AQL) and limiting quality (LQ) or in terms of a specified value of the average outgoing quality limit (AOQL), and

— by applying Bayesian techniques.

In case of control by attributes, the acceptance rules are specified as an acceptable number of defectives c in a random test sample of size n. In case of control by variables, it is verified whether a compliance function consisting of one or more test statistics based on n random test samples lies within an acceptable region. The acceptable region can consist out of one or more boundaries.

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;

— independent external party control of design and/or execution;

— control and supervision by the client's organization.

The choice of the required control steps mentioned shall depend on the required quality inspection level, which can depend on a quality level differentiation (see A.6).

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 far 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.

A.5.5 Filtering effects of quality control

In general, quality control of an entity has a favourable effect on its characteristics due to the fact that the existence of quality requirements (such as quality management, production, and compliance control) compels one to deliver high-quality products. This favourable filtering effect has an influence on the uncertainty representation (Clause 6), probability-based decision making (Clause 7), and structural robustness assessment (Annex F). Hence, the beneficial effect of quality control might be included in risk-based approaches.

One or more aspects of the quality control, i.e. quality management, quality assurance, or quality control (either by production control or compliance control), can be taken into account in the uncertainty representation by using Bayesian techniques. In case of conformity control, operating characteristics can be considered as a likelihood function in case of Bayesian updating. In the latter case, updating of the parameters or the hyperparameters of probabilistic models can be calculated as follows:

$$f''_{\overline{B}}(\beta) = \frac{P_a(\beta) f'_{\overline{B}}(\beta)}{\int P_a(\beta) f'_{\overline{B}}(\beta)\, d\beta} \tag{A.1}$$

with $\boldsymbol{\beta}$ a vector with parameters or hyper-parameters of a probabilistic model of the statistical population to be updated, $f'_{\overline{B}}(\beta)$ and $f''_{\overline{B}}(\beta)$ the prior resp. posterior distribution function of the vector $\boldsymbol{\beta}$, and $P_a(\boldsymbol{\beta})$ the operating characteristic of the conformity control, i.e. the probability of acceptance of a population characterized by a probabilistic model with parameters or hyperparameters $\boldsymbol{\beta}$.

A tentative indication of the quality management activities, for which such a Bayesian updating of characteristics of entities might be considered, is indicated in Table A.1.

In case the quality control is taken into account in the uncertainty representation and/or probability-based decision making, it should however always be ensured that the quality control will be adequately executed.

In case of the application of new materials or construction methods, intensified quality control together with a quantification of its influence on risk-based or reliability-based methods can justify a risk-based or reliability-based design when incorporating material and model uncertainties. In case of lack of prior information, vague priors or low-informative expert judgment based priors can be used in the above-mentioned Bayesian approach.

A.6 Quality level differentiation

Three possible quality levels (QL) are shown in Table A.2. The quality levels can be linked to the quality management, quality assurance, and quality control measures described in A.3, A.4, and A.5, respectively. All three components (quality management, quality assurance, and quality control) should be considered together when assigning a quality level.

The quality level differentiation can directly be linked to the consequence classes described in F.1. As such, they can directly be related to a differentiation in structural applications.

NOTE This quality level differentiation can also be related to a differentiation in reliability index β which takes account of accepted or assumed statistical variability in action effects, resistances, and model uncertainties.

In case of buildings, engineering works and engineering systems where high consequence for loss of human life or economic, social, or environmental consequences are involved, that is, public buildings where consequences of failure are high (e.g. a concert hall, grandstand, high-rise building, critical bearing elements, etc.), a quality level QL3 has to be applied. The choice of the required quality level can be based on reliability-based methods.

Table A.2 — Quality levels (QL)

Quality Level (QL)	Consequence class (see Annex F)	Description	Control organism for specification of requirements and checking
QL1	1–2	Basic quality level	Self-control: specification of requirements for quality management, assurance, and control, as well as the checking performed by the person who has prepared the stage of the life cycle involved.
QL2	3	Increased quality level	Specification of requirements for quality management, assurance, and control, as well as the systematic checking performed by self-control, as well as by different persons than those who prepared the stage of the life cycle involved and in accordance with the procedure of the organization. Increased effort with respect to supervision and inspection during the construction of the structural key elements.
QL3	4–5	Extensive quality level associated to extended measures for quality management, inspection, and control	Besides self-control and systematic control, independent party control shall also be executed: specification of requirements for quality management, assurance, and control, as well as the checking performed by an organization different from that which has prepared the stage of the life cycle involved. Intensive supervision and inspection during construction of the structural main bearing system by well-qualified people with an expert knowledge (e.g. with respect to design and/or execution of structures).

Annex B
(informative)

Lifetime management of structural integrity

B.1 Introduction

Structural integrity management (SIM) is a continuous lifetime process which should ensure, with an appropriate degree of reliability that a structure satisfies the aims and objectives specified in 4.2.1 over its entire working life, from construction to demolition. The process is essential since even if a structure has been designed and constructed according to the above aims and objectives, there is no guarantee that it will continue to fulfil them in the future.

Integrity of a structure can be impaired with time by degradation and damage caused by various actions and environmental influences, its use and/or loading can be changed, or its service life might need to be extended beyond the design working life. In addition, there can be errors in design and construction. Under such circumstances, inspections and maintenance are essential for detecting unexpected flaws, damage, and degradation. They are to be followed by an appropriate evaluation and possible repair or upgrading which help the structure to maintain its integrity. The necessity to carry out such actions over the service life of the structure should be taken into account during its design and construction.

NOTE The term inspections include any activity aiming to collect information about the performance of structures over their service life and thus also should be understood to include what is commonly referred to as monitoring.

In the following, a generic SIM process will be described in the context of a risk/reliability-based approach. The main phases of the process such as data collection, evaluation, development of inspection strategy and programme will be considered. Special attention will be paid to updating of structural reliability assessment using new inspection data.

B.2 Main phases of structural integrity management process

A SIM process includes the following phases (ISO 19902):

— Data collection, in particular by inspection.

— Evaluation of data followed, if necessary, by structural assessment and remedial actions.

— Development of inspection strategy.

— Development of a detailed inspection program.

The phases of the process are shown in Figure B.1 and considered in more detail further in the annex.

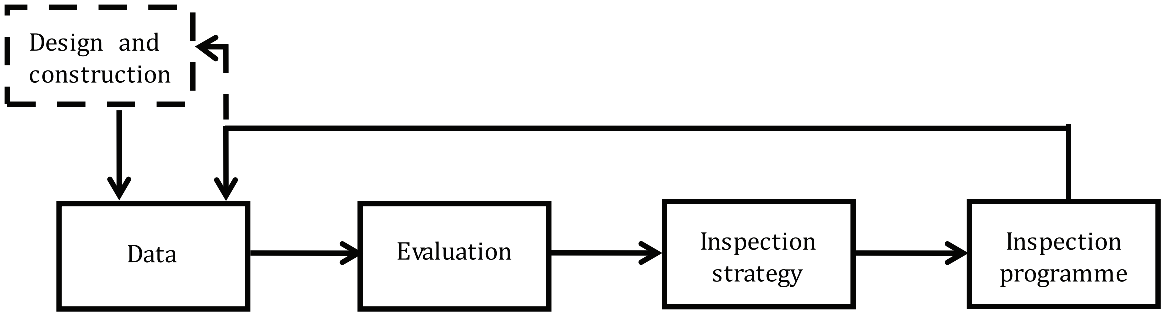


Figure B.1 — Phases of a structural integrity management process

The dashed arrow in Figure B.1, which goes back from a SIM process to design and construction, stresses the importance of planning this process during the design of a structure. This includes identifying components of the structure, which are critical for its integrity and can be damaged and/or deteriorate over its service life. The design should ensure that there is a reasonably easy access to such components for their inspection and maintenance. It should also be considered that such components might need to be repaired or replaced over the structure service life. Thus, as far as it practicable, the design should allow performing these actions with relative ease. If it becomes clear during initial planning of the SIM process that the above conditions are not met changes might need to be made in the original design.

B.3 Data collection

Up-to-date data on the structure are essential for a SIM process. The data should include information on the structure original design, construction, inspections, structural assessments, modifications, changes of use, strengthening, repairs, and accidents. In some cases, information about structural parameters such as strain, stress, deflection, vibration, temperature, pressure, etc. can be collected continuously through monitoring. Monitoring can be economically justified on the basis of the cost–benefit analysis of the structure with and without monitoring. All the collected data should be stored by the owner or operator throughout the entire service life of the structure and transferred to a new owner if the change of ownership takes place.

B.4 Evaluation and structural assessment

B.4.1 Data evaluation

When new information on the structure becomes available, all relevant data need to be evaluated using initially engineering judgement and then, if found necessary, more detailed analysis in order to determine whether structural assessment or updating the inspection strategy is needed. Note that decision about updating the inspection strategy can be made without detailed structural assessment, while structural assessment might not necessarily lead to updating the inspection strategy.

An assessment of a structure is usually required when either

a) purpose/use of the structure has changed compared to the original design or previous assessment, or

b) properties of the structure have deviated from those adopted in the original design or previous assessment.

Situations when the purpose/use of the structure has changed include

— increase in loading/actions,

— change in use,

— extension of the structure life beyond its design working life, and

— increase in the target reliability level due to increased importance of the structure to society.

Situations when properties of the structure have deviated include

— design error,

— defects or damage of the structure during construction,

— degradation of the structure (e.g. due to corrosion, fatigue, etc.),

— structure has been damaged by an accidental event or overload,

— modification of the structure, and

— change in design requirements due to revision of design codes.

Carrying out a structural assessment can require collection of more data by further inspection.

It can be decided without an assessment that updating the inspection strategy is needed when, e.g. there is evidence that a degradation process takes place but no associated damage has yet been observed, environmental conditions have changed, similar structures have exhibited unsatisfactory performance, etc.

B.4.2 Structural assessment

An assessment should determine whether a structure fits intended purposes through the performance indicators listed in 4.1 of the standard or remedial measures are needed. In the assessment exposure events, vulnerability and robustness of the structure are considered in line with 4.2.2, i.e. in the same way as in structural design. However, there is a fundamental difference between the assessment and design situations with regard to uncertainties.

In design, uncertainties arise from the prediction a priori of load and resistance parameters of a structure, which does not exist at that time. These uncertainties represent the variability of a large population of structures caused mainly by the differing quality of materials and construction practices and variability of site-specific loads. They are assessed based on given specifications for manufacturing and construction of the structure along with generic data on statistical characteristic of loads and environmental parameters.

In assessment, an existing structure can be inspected/tested so that load, resistance, and environmental parameters can be measured on-site. However, this does not mean that the uncertainties can be completely resolved because of uncertainties associated with in-service inspection/testing. It is essential to remember that inspection methods have a limited resolution. Thus, inspection results can only be considered as indicators of the real condition of a structure. The issue is to which degree the indication of a certain condition is related to the real condition. For this purpose, the concept of the Probability of Detection (PoD) is very useful. PoD provides a quantification of the quality of inspection methods through the probability of detection of a defect of a given size or extent.[1]

Uncertainties associated with in-service inspection/testing include

— measurement error,

— inherent variability of a measured parameter,

— model uncertainty when a parameter of interest cannot be measured directly so that a relationship between it and the corresponding measured parameter is needed, and

— statistical uncertainty due to a limited number of measurements.

Uncertainties associated with inspection should be taken into account in a structural assessment. This can be done by the use of probabilistic methods either explicitly in reliability analysis or for updating the characteristic and design values of basic variables. Because of different nature of uncertainties associated with the design and assessment situations, applicability of the characteristic values of basic variables and the partial safety factors from design codes shall be carefully investigated.

A conservative design does not usually lead to a significant increase in structural cost, while a conservative assessment can result in unnecessary and costly repairs or replacement. Thus, in order to reduce the model uncertainty, more refined structural models (e.g. finite element models) compared to those in design codes can be used for the assessment of existing structures.

The process of collecting information, assessing structural performance through analysis and devising repair and strengthening activities is a decision process which aims to identify the most effective investigations and modifications required to satisfy new requirements to the use of the structure and/or to remove any doubts regarding its current condition and future performance. It is important that this process is optimized with due consideration of the total service life costs of the structure.

A generic assessment process can be organized in accordance to the flowchart shown in Figure B.2 (ISO 13822:2010, Annex B). Further information on structural assessment can be found in ISO 13822.

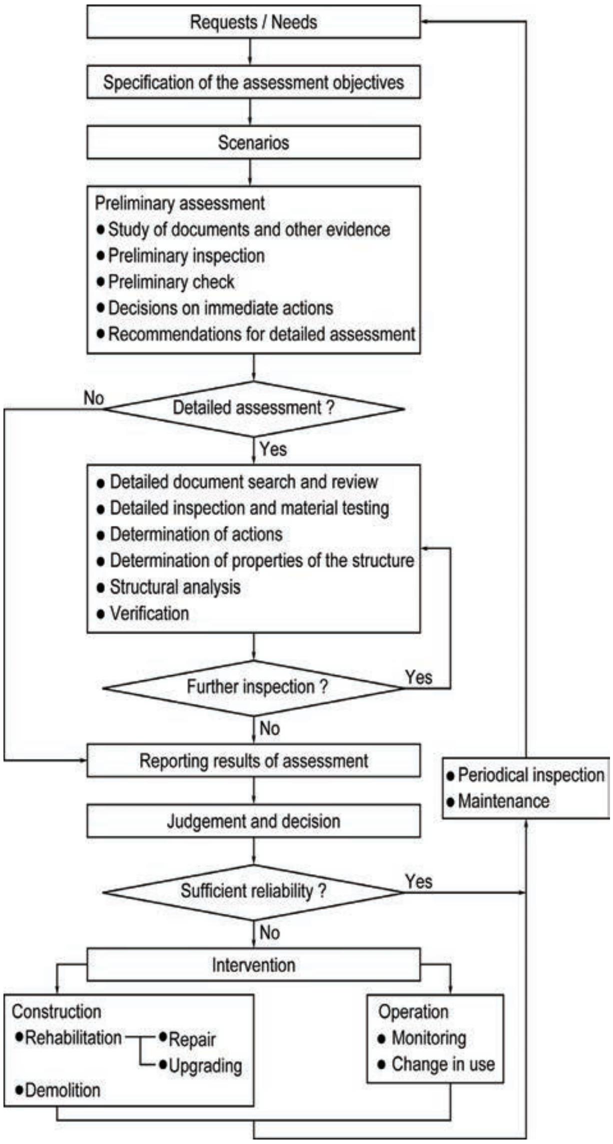


Figure B.2 — Flowchart for a generic assessment process

B.4.3 Updating information

Updating information on properties of a structure is an essential part of an assessment. Two complementary approaches to updating can be considered:

a) Checking performance of the whole structure (or its structural elements) by proof-load testing or by using information about its past performance.

b) Collecting data about individual basic variables by conducting in-service inspections. Updating previously available uncertain information using new inspection data are usually done by a Bayesian method. Updating can be carried out using either a single observation — event updating or multiple observations — distribution updating. In the first case, a parameter controlling “failure” is measured. If the corresponding observation of the safety margin H [= $g(X)$], is greater than zero, the updated probability of failure is then:

$$p_{\mathrm{f}}^{U} = p\left[g(X) \leq 0 \,|\, H > 0\right] = \frac{p\left[g(X) \leq 0 \cap H > 0\right]}{p\left[H > 0\right]} \tag{B.1}$$

Efficiency of such updating depends on how closely the measured parameter is related to failure.

To illustrate updating in the second case consider a basic variable X whose probability density function depends on a set of parameters **θ** (e.g. mean and standard deviation). Denote a prior distribution of **θ** as $f'_{\theta}(\theta)$. This distribution is based on information available on **θ** before an in-service inspection. It should be noted that even if the prior information is rather poor, it is very important to take it into account as it could reduce significantly uncertainty on φ and, subsequently, on X. Assume that $\boldsymbol{x}' = (x_1, x_2, .., x_n)^T$ is a vector, which includes results of n measurements made during the in-service inspection. According to Bayes theorem, the posterior distribution $f''_{\theta}(\theta)$ of φ is then

$$f''_{\theta}(\theta) = \frac{L(\theta|x') f'_{\theta}(\theta)}{\int_{\theta} L(\theta|x') f'_{\theta}(\theta)\; d\theta} \tag{B.2}$$

where

$L(\theta|\,\boldsymbol{x}')$ is a likelihood function, which is proportional to the conditional probabilities $f_{X|\theta}(x_i|\theta),\quad (i = 1,2,..n)$ of making the measurements as

$$L(\theta|\,\boldsymbol{x}') \propto \prod_{i=1}^{n} f_{X|\theta}(x_i|\theta) \tag{B.3}$$

The following predictive distribution of X is then used to estimate the updated probability of failure.

$$f_X(x) = \int_{\theta} f_{X|\theta}(x|\theta) f''_{\theta}(\theta) d\theta \tag{B.4}$$

Proof load testing can be used to verify the resistance of an existing structure (or its components). The observation that a structure has survived a proof load test indicates only that the minimum resistance of the structure is greater than the applied load effect — it does not reveal the actual resistance of the structure, nor does it provide a meaningful measure of structural safety. However, results of proof load testing can be analysed using a probabilistic (or reliability) approach. If a structure has survived a known proof load then the original cumulative distribution function, $F'_R(r)$, of the structure resistance is simply truncated at this known load effect, QPL, so that the updated distribution function of the resistance, $F_R''(r)$, is given by

$$F_R''(r) = \frac{F_R'(r) - F_R'(Q_{PL})}{1 - F_R'(Q_{PL})} \tag{B.5}$$

Satisfactory structural performance during T years in service means that the structural resistance is greater than the maximum load effect over this period of time. The updated distribution function of structural resistance at time T is then given by

$$F_R''(r) = \frac{\int_0^r F_Q^T(r) f_R'(r) dr}{\int_0^\infty F_Q^T(r) f_R'(r) dr} \tag{B.6}$$

where

F_Q^T is the cumulative distribution of the maximum load effect over T years;

$f_R'(r)$ is the probability density function of the resistance prior to loading.

B.5 Inspection strategy

It is highly desirable that in-service structural inspection strategy be initially developed for a structure during its design based on generic data for similar structures under similar loading and environmental conditions. Information obtained by modelling the effects of potential degradation processes over the design working life of the structure can also be used. It is important to optimize the inspection schedule regarding the life cycle cost of the structure.

The ability of different inspection methods (visual and NDE) to detect damages and defects depends on the nature of the latter. Whenever it is possible to identify possible causes of damage or potential defects, it is also possible to devise appropriate inspection strategies, i.e. where to inspect, with which method, and how often. However, it is also essential to schedule periodic inspections to detect possible unforeseen or simply unknown phenomena which can damage structural integrity. This is especially important at the beginning of the structure life when errors made in design and construction can reveal themselves. In this context, a baseline inspection to determine the initial condition of the structure after the completion of construction is needed. In later stages, such phenomena could be related to damages due to accidents. An unscheduled inspection might be needed after an accident based on the accident report. In such cases, a visual inspection covering the total extent of the structure is usually useful.

The inspection strategy should be periodically updated throughout the service life of a structure, usually through amendments following the receipt of new data, results of the structure assessments, and other data or information relevant to the SIM process. Inspection frequency and extent should be justified by cost-benefit analysis.

Risk-based inspection (RBI) planning can be used to optimize inspection strategy. The methodology takes into account risks associated with failures of components of a structure and usually involves the use of probabilistic models of structural deterioration and inspections, which are combined via Bayesian updating. The optimization is based on minimising the total expected cost over the service life of the structure, which includes the costs of the structure failure, inspections, and repair, while acceptable probabilities of failure serve as constraints.[17]

An inspection by itself obviously does not improve integrity of a structure. Thus, it is essential along with inspection strategy to develop plans for the structure maintenance, repair, and/or replacement, which are informed by inspection outcomes. A comprehensive strategy, which includes inspections, maintenance, and repair actions, can be developed by minimising the life cycle cost of a structure.[3] Figure B.3 illustrates the implementation of SIM over the lifetime of a structure under different scenarios.

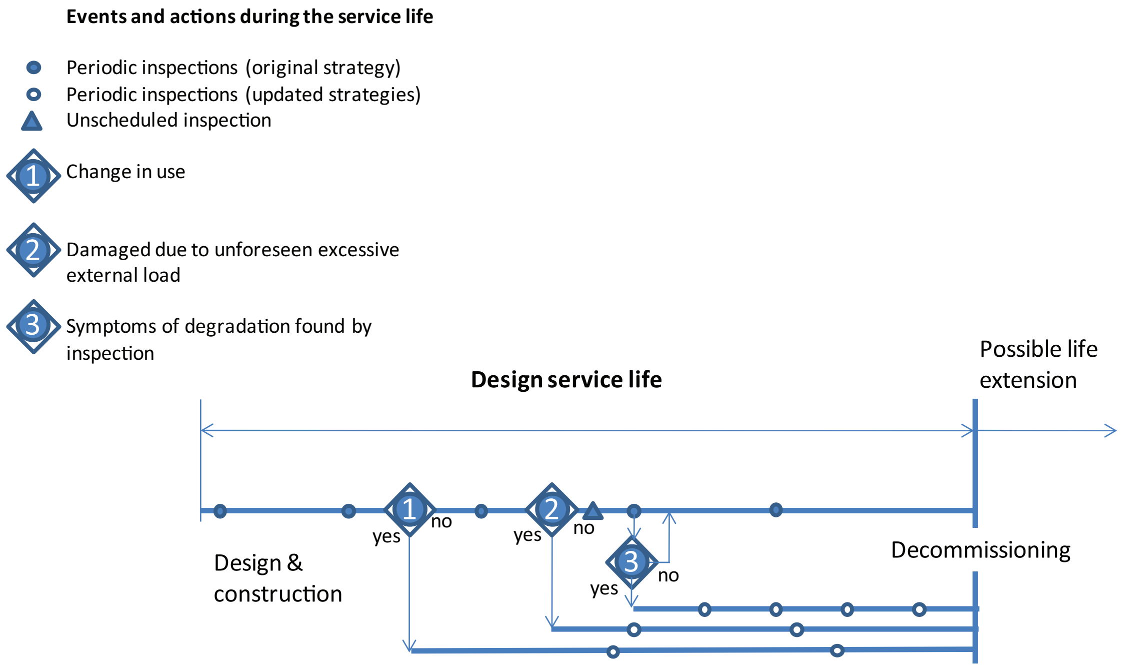


Figure B.3 — Events and actions associated with SIM over the lifetime of a structure

B.6 Inspection programme

The inspection programme is the detailed scope of work developed from the inspection strategy. The programme requires schedules, budgets, personnel profiles, and other procedures, in particular the selection of appropriate inspections methods and procedures, before it can be implemented.

Annex C
(informative)

Design based on observations and experimental models

C.1 Overview

Design based on experimental models (or, in short, design by testing) is a method for establishing design values of load parameters and/or 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 the following:

— 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 might be appropriate. For quality control, reference is made to Annex A.

C.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 or circumstances which might be critical for the considered element. Furthermore, an unambiguous definition of the limit state under consideration should be given.

Tested units should preferably be 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, might not always be evident. Development of the test program and evaluation of the obtained test results require appropriate theoretical knowledge, experience in testing, and engineering judgment.

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 C.6) or, in the absence of such a model, by direct evaluation (see C.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 news tests, unless and expansion of the obtained results to other element classes is possible, based on theoretical analysis.

C.3 Consideration of differences between reality and testing conditions

The conditions during testing can 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 the following:

— 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, can 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.

C.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 form the tests (e.g. required parameters and range of validity).

b) Description of all properties and conditions which can 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 might 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 can 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 might 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 compressive 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 can 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).

C.5 Direct evaluation of the test results

C.5.1 General

In this annex, 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 can be used to update a predefined prior distribution on the statistical parameters of the resistance. Guidance is given in C.5.3. If the partial factor format is used, the method of C.5.2 can be applied.

C.5.2 Partial factor design

The design value to be used in partial factor methods should be estimated from:

$$R_d = \eta_d \left\{ m_R - t_{vd} S_R \sqrt{1 + \frac{1}{n}} \right\} \tag{C.1}$$

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 C.1);

n is the number of tests;

η_d is the design value of the conversion factor.

Values for t_{vd} follow from Table C.1, where $v = n - 1$, $\beta_R = \alpha_d \beta$, where β is 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 E.5.2 and E.5.3).

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

a) The characteristic value R_k is defined, using the same Formula (C.1), 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 $R_k = \gamma_m R_k$; note that in this case, R_k can have 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.

Formula (C.1) 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 can replace the sample standard deviation by the distribution standard deviation and take $v = \infty$. For the processing of other types of prior information, the equations given in C.5.3 can be used.

The normal distribution can be regarded as a relatively conservative distribution. Other more favourable distributions like Lognormal or Weibull should only be used if there is evidence from many tests.

NOTE For many conventional building materials, this evidence can be considered as being present

The Bayesian method as presented here is 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 C.5.3, more information on this procedure is given.

Table C.1 — Values of t_{vd}

		Degrees of freedom, v								
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
1,65	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, $v = \infty$ should be used.										

Example 1 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 (v = 2):

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

NOTE The classical method would lead to: $R_k = m_R - 3{,}15\, S_R$ = 52,8 kN (see Table C.1). The result is almost the same.

C.5.3 Evaluation using full 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:

$$f''(\boldsymbol{q}) = C \times L(\bar{\boldsymbol{x}}|\boldsymbol{q})\, f'(\boldsymbol{q}) \tag{C.2}$$

where

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

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

$L(\bar{\boldsymbol{x}}|\boldsymbol{q})$ is the likelihood function for the parameters $\boldsymbol{q}$ given the observations $\bar{\boldsymbol{x}}$;

$\boldsymbol{q}$ 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:

$$f_R(r) = \int f_R(R|\boldsymbol{q}) f''(\boldsymbol{q}) d\boldsymbol{q} \tag{C.3}$$

where

$f_R(r|\boldsymbol{q})$ is the distribution of R for given values of $\boldsymbol{q}$:

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

This distribution for R can directly be used in a probabilistic design procedure (Clauses 7 and 8). It is also possible to derive design values (Clause 9) on the basis of Formula (C.3).

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:

$$f'(\mu,\sigma) = k\sigma^{-(v'+\delta(n')+1)} \exp\left(-\frac{1}{2\sigma^2}\left(v'(s')^2 + n'(\mu' - m')^2\right)\right) \tag{C.4}$$

where

$$\delta(n') = 0 \text{ for } n' = 0$$

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

This special choice allows a further analytical treatment of the integrals. The prior distribution, Formula (C.4), contains four parameters: m', n', s', and v'. The meaning of these parameters is explained below.

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

$$E(\sigma) = s' \tag{C.5}$$

$$V(\sigma)=\frac{1}{\sqrt{2\nu'}} \tag{C.6}$$

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:

$$E(\mu)=m' \tag{C.7}$$

$$V(\mu)=\frac{s'}{m'\sqrt{n'}} \tag{C.8}$$

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' Hypothetical sample value;

— ν' Hypothetical number of degrees of freedom for s';

The information about the mean requires two additional parameters:

— m' Hypothetical sample average;

— n' 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' can be chosen independently from each other.

NOTE 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 C.5.2. 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 $V(\mu)=0{,}14s'/m'$.

NOTE 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 Formula (C.6), corresponds to $\nu' = 5$. Such a model can 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 Formula (C.2), one can combine the prior information characterized by Formula (C.4) and a test result of n observations with sample mean m and sample standard deviations s. The result is a posterior distribution for the unknown mean and standard deviation of R, which is again given by Formula (C.4), but with parameters given by the following updating rules:

$$n''=n'+n \tag{C.9}$$

$$\nu''=\nu'+\nu+\delta(n') \tag{C.10}$$

$$m''n'=n'm'+nm \tag{C.11}$$

$$(\nu''(s'')^2+n''(m'')^2)=(\nu'(s')^2+n'(m)^2)+\nu s^2+nm^2 \tag{C.12}$$

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

Using Formula (C.2), the predictive value of R can be found from:

$$R = m'' - t_{v''} s'' \sqrt{(1 + \frac{1}{n''})} \tag{C.13}$$

Here $t_{v''}$ has a central t-distribution; values of $t_{v''}$ for given probabilities of exceeding the limits are given in Table C.1. Modifications for lognormal distributions of R are straightforward.

Example 2 Consider one more example 1, but assume previous test series have shown that:

— the sample mean is equal to 110 kN on average with a coefficient of variation of 30 %.

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

According to Formulae (C.5) to (C.8), this prior information leads to the following prior distribution parameters:

$$m' = 110 \text{ kN}, \quad n' = 0, \quad s' = 20 \text{ kN}, \quad v' = 1/\left(2V^2\right) = 1/\left(2 \times 0{,}3^2\right) = 5$$

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 Formulae (C.6) to (C.9) give the following parameters for the posterior distribution:

$$n'' = 0 + 3 = 3$$

$$v'' = 5 + 2 = 7$$

$$m'' = 100 \text{ kN}$$

$$7(s'')^2 + 3 \times 100^2 = 5 \times 20 + 0 \times 110^2 + 2 \times 15^2 + 3 \times 100^2$$

or

$$s'' = 18{,}7 \text{ kN}$$

Using Formula (C.13) and Table C.1, this leads to the following result for the 5 % characteristic value:

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

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 can even be larger.

C.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:

$$Y = \theta g(\boldsymbol{X}, \boldsymbol{W}) \tag{C.14}$$

where

X	is the vector of random variables;
W	is the set of measurable deterministic variables;
$g(\boldsymbol{X},\boldsymbol{W})$	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 $\boldsymbol{I} = 1,2,..n$ is carried out, where:

— values of $\boldsymbol{W}$ have been set to w_i, $i = 1,2,..n$;

— values of $\boldsymbol{X}$ have been measured as x_i, $i = 1,2,..n$;

— values of $\boldsymbol{Y}$ have been measured as y_i, $i = 1,2,..n$.

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

$$\theta_i = \frac{y_i}{g(x_i, w_i)} \tag{C.15}$$

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

$$m(\theta') = \frac{1}{n}\sum_{i=1}^{n}\theta_i' \tag{C.16}$$

$$s(\theta')^2 = \frac{1}{n-1}\sum_{i=1}^{n}(\theta_i' - m(\theta'))^2 \tag{C.17}$$

with θ_i', given by:

$$\theta_i' = \ln[y_i / g(x_i, w_i)] \tag{C.18}$$

The design value:

$$\theta_d = \exp[m(\theta')]\exp\left[\pm t_{vd}s(\theta')\sqrt{(1+\frac{1}{n})}\right] \tag{C.19}$$

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 C.1, 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 α_d = 0,8 if the uncertainty of R is dominating and α_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:

$$R_\mathrm{d} = \frac{1}{\gamma_\mathrm{d}} \eta_\mathrm{d} g(x_\mathrm{d}, w) \tag{C.20}$$

where

$\gamma_\mathrm{d} = 1/\theta_\mathrm{d}$ and η_d is the design value of the model uncertainty.

Annex D
(informative)

Reliability of geotechnical structures

D.1 Introduction

The purpose of Annex D is to consider application of the requirements of this International Standard to geotechnical design. Although the principles of reliability-based design are applicable to geotechnical engineering in general, it is essential to cater for the higher variability depicted by geotechnical design parameters and other site conditions. As noted in 6.1.1, Note 4, uncertainty representation is a critical link between this International Standard and geotechnical design as treated in this annex.

According to the Probabilistic Model Code (JCSS 2001), a characteristic feature of geotechnical structures is the dominating role of uncertainties of soil properties. This leads directly to the implementation of a diversity of methodologies in geotechnical design practice.

The emphasis in this Annex is on the identification and characterization of critical elements of the geotechnical reliability-based design process. These elements cannot be accounted for in existing deterministic geotechnical practice. The critical distinctive elements are the following:

a) Coefficients of variation (COVs) of geotechnical design parameters can be potentially large because geomaterials are naturally occurring and *in situ* variability cannot be reduced (in contrast, most structural materials are manufactured with quality control).

b) COVs for geotechnical design parameters are not unique and can vary over a wide range, depending on the procedure in which they are derived.

c) Because geotechnical design parameter characteristics are different from one site to another, it is common to conduct a site investigation at each site. For this reason, statistical uncertainty should be handled with much care.

d) It is common to conduct both laboratory and field tests in a site investigation. A geotechnical design parameter is typically correlated with more than one laboratory and/or field test indices. It is important to consider this multivariate correlation structure where possible because the COV of the design parameter reduces when consistent information increases.

e) Spatial variability of geotechnical design parameters cannot be readily dismissed because the volume of geomaterial interacting with the structure is related to some multiple of the characteristic length of the structure and this characteristic length (e.g. height of slope, diameter of tunnel, depth of excavation) is typically larger than the scale of fluctuation of the design parameter, particularly in the vertical direction.

f) There are usually many different geotechnical calculation models for the same design problem. Hence, model calibration based on local field tests and local experience is important. The proliferation of model factors, possibly site-specific, is to be expected because of the number of models and the number of calibration databases.

g) A geotechnical system, such as a pile group and a slope is a system reliability problem containing multiple correlated failure modes. Some of these problems are further complicated by the fact that the failure surfaces are coupled to the spatial variability of the soil medium.

D.2 Uncertainty representation of geotechnical design parameters

D.2.1 Estimation of geotechnical design parameters

The COV of a geotechnical design parameter is not an intrinsic statistical property. It depends on the site condition, the measurement method, and the transformation (correlation) model. Hence, the COV takes a range of values rather than a unique value. Some guidelines on the typical COVs of common design parameters as a function of soil type, measurement method, and transformation model are below.

D.2.2 Sources of uncertainties

The overall uncertainty underlying a geotechnical design parameter results from many disparate sources of uncertainties, as illustrated in Figure D.1. There are three primary sources of geotechnical uncertainties: (1) inherent variability, (2) measurement error, and (3) transformation uncertainty. The first results primarily from the natural geologic processes that produced and continually modify the soil mass *in situ*. The second is caused by equipment, procedural/operator, and random testing effects. The third source of uncertainty is introduced when field or laboratory measurements are transformed into design parameters using empirical or other correlation models.

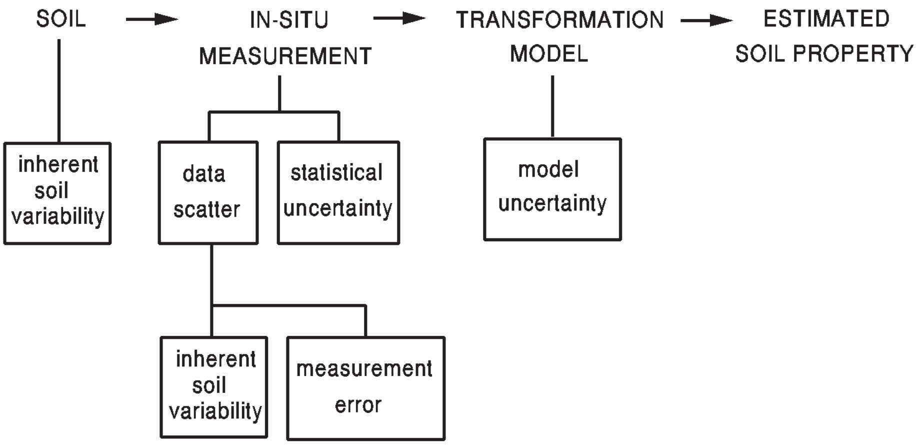


Figure D.1 — Sources of uncertainties contributing to overall uncertainty in geotechnical design parameter[13]

The various sources of uncertainty should be characterized separately as input to derive total uncertainty. The use of COVs generally reported in the literature based on total variability analysis can lead to overestimation of uncertainty. This is caused by the following: (i) soil data from different geologic units are mixed, (ii) equipment and procedural controls generally are insufficient, (iii) deterministic trends in the soil data are not removed, and (iv) soil data are taken over a long period.

D.2.3 Inherent variability

Spatial variability can be decomposed into a smoothing varying trend function and a fluctuating component. It can be quantified by modelling the fluctuating component as a homogeneous random function. The scale of fluctuation is also of importance, as discussed in D.2.7.

D.2.4 Measurement error

Equipment effects on measurement error results from inaccuracies in the measuring devices and variations in the equipment geometries and systems employed for routine testing. Procedural-operator effects originate from the limitations in existing test standards and how they are followed. Random testing errors refer to the remaining scatter in the test results. Measurement error is extracted from field measurements using simple additive probabilistic models or is determined directly from comparative laboratory test results.

D.2.5 Transformation of uncertainty

Transformation models are needed to relate test measurements to appropriate design properties. Uncertainty is introduced because most transformation models in geotechnical engineering are obtained by empirical or semi-empirical data fitting. The transformation model is typically evaluated using regression analyses and the spread of the data about the regression curve is modelled as a zero-mean random variable (ε). The standard deviation of ε is an indicator of the magnitude of transformation uncertainty.

Most transformation models were developed for a specific geomaterial type and/or a specific locale. Site-specific models are generally more precise than “global” models calibrated from data covering many sites. However, site-specific models can be significantly biased when applied to another site. This “site-specific” limitation is a distinctive and prevalent feature of geotechnical engineering practice. Geotechnical RBD should take cognizance of this limitation to avoid gross oversimplification of “ground truths”.

D.2.6 Total uncertainty

Total uncertainty comprises the combination of inherent soil variability, measurement error, and transformation uncertainty. These components can be combined consistently using a simple second-moment probabilistic approach or by more sophisticated uncertainty analysis methods. The characteristic design parameter governing a specific limit state needs to be identified when determining the total COV.

For ultimate limit state problems, the characteristic design parameter is typically a spatially averaged strength over the most critical failure path. In the presence of spatial variability, the COV of this spatially averaged strength is smaller than the COV of the point strength; the degree of COV reduction is a function of the scale of fluctuation discussed in D.2.7. It is inappropriate to apply the point COV to a problem where the COV reduction is significant, say because the scale of fluctuation is short relative to some characteristic length scale of the failure path length (e.g. height of slope, diameter of tunnel, depth of excavation). Alternatively random finite element methods can be used for ultimate limit state analysis, in which random fields are combined with deterministic finite elements in a Monte Carlo framework.

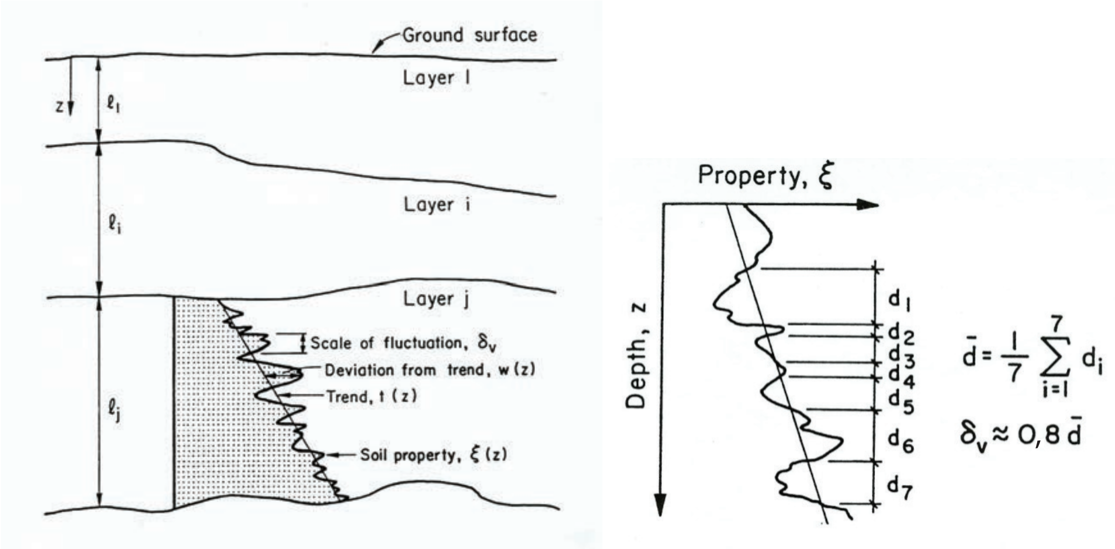
It suffices to note that it is not realistic to assign a single representative value to a geotechnical design parameter. For example, a COV of 30 % for undrained shear strength can be appropriate for good quality laboratory measurements or direct correlations from field measurements such as the cone penetration test. It might not be appropriate for indirect correlations based on the standard penetration test. The practical impact of this observation is that it is unlikely to be realistic to calibrate a single resistance or partial factor for application in the context of a simplified RBD format.

D.2.7 Scale of fluctuation

Decomposition of spatial variability according to D.2.3 into a smoothly varying trend $t(z)$ and a fluctuating component $w(z)$ is illustrated in Figure D.2. It is noteworthy that a physically homogeneous soil layer is not necessarily statistically homogeneous. A critical statistical parameter that is needed to describe inherent variability is the correlation distance or scale of fluctuation. The scale of fluctuation provides an indication of the distance within which the property values show relatively strong correlation.

NOTE A simple but approximate method of determining the scale of fluctuation is shown as an insert in Figure D.2.

The practical importance of considering a reasonable scale of fluctuation, i.e. a reasonably realistic spatial variability, in the estimation of COV has been highlighted in D.2.6. The assumption of independent soil parameters will produce an unconservative reduction of the point COV for the spatial average. The assumption of fully correlated soil parameters will not result in COV reduction for the spatial average, which is overly conservative. In addition to COV reductions, it is important to point out that failure mechanisms are related to spatial variability.



NOTE Revised from Phoon and Kulhawy 1999a.[13]

Figure D.2 — Random field model for inherent soil variability

D.3 Statistical characterization of multivariate geotechnical data

D.3.1 General

Multivariate information is usually available in a typical site investigation. Because bivariate correlations between soil parameters are more commonly available, for example, s_u-N and s_u-OCR , the multivariate normal distribution is a sensible and practical choice to capture the multivariate dependency among soil parameters in the presence of transformation uncertainties. The practical significance of considering multivariate data are that the COV of a design parameter typically decreases when it is estimated from more than one parameter. Hence, site investigation is not a cost item but an investment item because reduction of uncertainties through multivariate tests can translate directly to design savings through RBD. This important link between the quality/quantity of site investigation and design savings cannot be addressed systematically in a deterministic design approach.

D.4 Statistical characterization of model factors

D.4.1 General

Model uncertainty emanates from imperfections of analytical models for predicting engineering behaviour. Mathematical modelling of any physical process generally requires simplifications to create a useable model. Inevitably, the resulting models are simplifications of complex real world phenomena. Consequently, there is uncertainty in the model prediction even if the model inputs are known with certainty. The current state of the art in developing model uncertainty statistics in various fields including geotechnical engineering involves comparing predicted performance with measured performance.

Model uncertainty is generally represented in terms of a model factor. Model factors need to be characterized for the various geotechnical structures, such as shallow foundations, pile foundations, retaining structures, slopes, etc. The model factor applies to a specific set of conditions (e.g. failure mode, calculation model, local conditions and experience base, etc.). Therefore the proliferation of model factors can be expected.

The model factor is commonly defined as the ratio of the measured to the calculated capacity:

$$M = \frac{Q_m}{Q_p} \tag{D.1}$$

where

Q_m is the measured “capacity” determined from a load test;

Q_p is the capacity generally predicted using limit equilibrium models;

M is the model factor.

Robust model statistics can only be evaluated using: 1) realistically large-scale prototype tests, 2) a sufficiently large and representative database, 3) reasonably high-quality testing where extraneous uncertainties are well-controlled. With the possible exception of foundations, insufficient test data are available to perform robust statistical assessment of the model error in many geotechnical calculation models.

D.5 Implementation issues in geotechnical reliability-based design

D.5.1 General

The general principles in this International Standard are equally applicable to geotechnical reliability-based design (RBD). Appropriate levels of performance can be achieved for geotechnical structures by limit states procedures as provided for in Clause 5. This can be achieved alternatively at the risk-based level given in Clause 7, the reliability-based level given in Clause 8, or semi-probabilistic based design given in Clause 9. Representation of geotechnical uncertainties can be based on the principles provide in Clause 6.

D.5.2 Goal and challenges

The key goal in geotechnical RBD is to achieve a more uniform level of reliability than that implied in existing allowable stress design. With regard to the semi-probabilistic approach, it is important to highlight that reliability calibration is more challenging in geotechnical engineering. One key reason is the diversity of design scenarios that shall be considered in the calibration domain, such as the range of COV resulting from different soil property estimation methodologies. Another source of diversity is the number of different soil profiles encountered even within a city size locale.

The challenge of producing geotechnical RBD that is both realistic (handle wide range of design scenarios) and robust (achieve target reliability index with reasonable accuracy) is being addressed in this section. Some implementation details associated with probability based decision making (i.e. direct probability-based design, system reliability and reliability targets in Clause 8) and semi probabilistic methods (i.e. partial factor methods and characteristic values in Clause 9) are discussed below.

D.5.3 Reliability-based design method

Reliability-based design is particularly suitable for the representation of the multitude of conditions, failure modes, and uncertainties found in geotechnical design. Reliability-based design (Clause 8) can be directly applied to geotechnical practice. The required inputs for a reliability analysis are the probability distributions of the soil parameters and the loads. The identification of appropriate probability distributions and the selection of appropriate model parameters for these distributions (e.g. mean and COV) from limited data are subject to statistical uncertainties.

D.5.4 Semi-probabilistic method

Due to practical considerations, geotechnical reliability-based design is generally applied at the simplified level of the semi-probabilistic method, or more specifically the partial factor method (9.4.2). A distinctive feature in geotechnical design is that many geotechnical calculation models are relatively “simple” and there are usually many different calculation models for the same design problem because of the empiricism nature of our geotechnical heritage. In the calibration process, the "best" geotechnical (deterministic) calculation models should be used directly.

It is desirable to choose a "best" calculation model (minimum bias and least variability when compared against field measurements or perhaps laboratory tests/numerical simulations in the absence of field measurements) for reliability calibrations. Nonetheless, local practice should be respected in geotechnical engineering because the behaviour of geomaterials and construction/design experience are largely site specific. Hence, it is more realistic to define a "best" model as what is best practice within a locale. The upshot is that it is not possible to enforce a single physical model (e.g. CPT method) or a single probabilistic model (e.g. lumped entire capacity into a single lognormal variable) and a geotechnical RBD calibration process should be sufficiently general to accommodate the existing diversity of local practices. For RBD calibration, the only mandatory requirement is that the model bias should be quantified in some approximate way.

Because of the considerably larger COV for geotechnical design parameters (see D.2), it is sensible to divide the range of an influential design parameter, and its variability, into several segments or domains (see Figure D.3). Then calibration points within each segment or domain can be selected to ensure uniform coverage of the variables during the calibration process and to achieve a consistent design risk across the range in the input parameters and their variability.

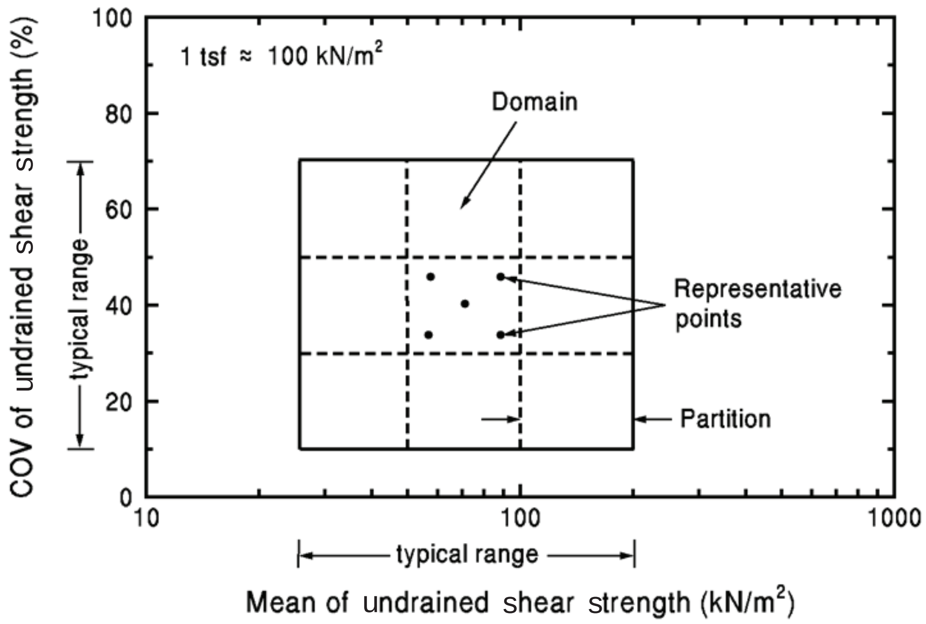


Figure D.3 — Partitioning of parameter space for calibration of resistance factors[10],[12]

D.5.5 Characteristic value

The concept of a "characteristic value" is intrinsically linked to semi-probabilistic formats, particularly the partial factor approach. In this approach, using the ultimate limit state as an example, a "characteristic value" for a soil parameter (e.g. undrained shear strength) is divided by a strength partial factor to produce a "design" value and the geotechnical capacity based on this design value should be larger than the design load (characteristic load multiplied by a load factor).

The soil parameter should be defined such that it is relevant to the limit state equation. For example, if a single undrained shear strength parameter appears in a slope stability equation, then the relevant physical definition is the spatial average along the most critical failure path. It is neither the undrained shear strength at a point in the soil mass nor a spatial average along a prescribed line in the soil mass. The emphasis in the geotechnical literature is on clarifying this physical aspect of the characteristic value, which is justifiably so.

It is necessary to make clear the physical meaning of the characteristic soil parameter before the uncertainty aspect could be rationally considered. For illustration, the characteristic undrained shear strength for the shaft friction of a pile is the spatial average along the length of the pile, while the characteristic undrained shear strength for the end bearing of a pile is the spatial average within a bulb of soil below the pile tip. When reliability analysis is carried out, the performance function will contain two random variables following two distinct probability distributions for these spatial averages.

When semi-probabilistic design is carried out, it would be necessary to select a single value characteristic of each probability distribution. This value can refer to the mean or to the lower 5 % quantile. The statistical estimation of these characteristic values is subject to the same statistical uncertainties underlying the probability distributions appearing in reliability analysis. Clearly, this statistical aspect of the characteristic value is distinctive from the physical aspect of the characteristic value.

In principle, partial factors can be calibrated to achieve a prescribed target reliability index for any statistical definition of the characteristic value. In practice, it is known that a partial factor calibrated for the mean value could change significantly if the COV of the input random variable changes. This limitation is less severe for a partial factor calibrated using say the lower 5 % quantile. Hence, if the COV of an input random variable varies over a wide range within the scope of design scenarios covered in a design code and if there is a practical need to simplify presentation of a partial factor as a single number rather than as a function of COV, the lower 5 % quantile definition is preferred (except in the case considering nonlinear responses where special considerations shall be made). It is useful to reiterate that the key function of a RBD code is to achieve a prescribed target reliability index (typically a function of limit state and importance of structure) over a range of commonly encountered design scenarios and not for a specific design scenario. The statistical definition of the characteristic value should be viewed within this broader context of what an RBD code intends to achieve rather than adherence to past practice or a component separate from reliability calibration. In other words, the ensuing design is produced by design values, which is the product of partial/resistance factors with characteristic values, not merely characteristic values alone. There are practical concerns regarding: (1) estimation of quantiles reliably from limited data and (2) quantiles falling below known lower bounds (e.g. residual friction angle) due to inappropriate choice of unbounded probability distribution functions. However, both concerns do not merely affect the characteristic value but fundamentally affect the reliability analyses underlying code calibration as well.

D.5.6 Quantile-based design

It has been recognized that these code calibration methods can be unwieldy because the COVs of geotechnical design parameters and model factors are not constant and can vary over a wide range (D.2 and D.4), necessitating the segmentation of the calibration domain highlighted in D.5.3. For geotechnical design problems with variable COVs, although it is not possible to maintain a uniform target reliability index with constant partial factors, it might be possible to maintain such uniformity with fixed quantiles.

D.5.7 System reliability

Because the majority of geotechnical structures have multiple failure modes, most geotechnical RBD is in fact system reliability problems (8.3). For example, a simple gravity retaining wall has at least three failure modes: horizontal sliding along the base of the wall, overturning or rotation about the toe of the wall, and bearing capacity failure of the soil beneath the wall. These failure modes tend to interact among each other, because loads and capacities for different failure modes can be correlated. Consider, for example, self-weight of a gravity retaining wall, which is the major source of capacity against sliding and overturning failure modes, but at the same time, is also a major source of load for bearing capacity failure mode.

A pile foundation is a system of piles, which consists of several pile groups, each group consisting of a few individual piles. The evaluation of the reliability of the pile system requires the consideration of the reliability of the individual piles, the pile group effects, and the system effects arising from pile-superstructure interactions.

Many geotechnical structures form failure mechanisms in the surrounding soil mass in the ultimate limit state (e.g. slopes, tunnels, deep excavations). Each potential slip surface in the soil mass is a failure mode. The probability of failure associated with the “most likely” failure mode identified by FORM only provides a lower bound for the system probability of failure. In contrast to a pile foundation where the sliding surface is mostly restricted to the interface between soil and pile, the trajectory of a slip surface in a soil mass is coupled to the specific realization of a random field and can only be determined through finite element analysis or comparable numerical methods. This class of system reliability problems is complex, because of the coupling between mechanics and spatial variability. However, it is not uncommon in geotechnical engineering.

Annex E
(informative)

Code calibration

E.1 Introduction

In developing new codes or revising existing codes, calibration is generally required. Calibration here refers to the determination of the values of all the parameters in a given code format. Code calibration can be performed by judgment, fitting, optimization, or combination of these. In this annex, a code calibration approach is explained. The approach explained here explicitly takes basis in the principles of reliability of structures, which are described in the present document.

E.2 Probabilistic models for calibration

The probabilistic models for the basic random variables should be selected very carefully. Guidelines for the selection can be found in the JCSS Probabilistic Model Code (JCSS 2001). In general, the following main recommendations can be made.

Strength or resistance variables are often modelled by Lognormal distributions. This avoids the possibility of negative realizations. In some cases, it can be relevant also to consider Weibull distributions for material properties. This is especially the case if the strength is governed by brittleness, size effects, and material defects. The coefficient of variation varies with the material type considered. Typical values are 5 % for steel and reinforcement, 15 % for concrete compression strength, and 15 % to 20 % for the bending strength of structural timber. The characteristic value is generally chosen as the 5 % quantile.

Variable loads (imposed and environmental) can be modelled in different ways. The simplest model is to use a probabilistic variable modelling the largest load within the reference period (often one year). This variable is typically modelled by an extreme distribution such as the Gumbel distribution. The coefficient of variation is typically in the range of 20 % to 40 % but could be much larger than that especially for seismic load. The characteristic value can be chosen as the 98 % quantile in the distribution function for the annual maximum load.

Permanent loads are typically modelled by a Normal distribution since they can be considered as obtained from many different contributions. The coefficient of variation is typically 5 % to 10 % and the characteristic value is chosen as the 50 % quantile.

Model uncertainties are in many cases modelled by Lognormal distributions if they are introduced as multiplicative random variables and by Normal distributions if the they are modelled by additive random variables. Typical values for the coefficient of variation are 3 % to 15 % but should be chosen very carefully. The characteristic value is generally chosen as the 50 % quantile.

E.3 Code calibration as optimization problem

Ultimately structural design codes are established for the purpose of providing a simple, safe, and economically efficient basis for the design of ordinary structures under normal loading, operational and environmental conditions. Design codes thereby not only greatly facilitate the daily work of structural engineering but also provide the vehicle to ensure a certain standardization within the structural engineering process which in the end enhances an optimal use of the resources of society for the benefit of the individuals.

Therefore, code calibration can be seen as a decision optimization problem where the expected societal net benefits are maximized by optimizing the parameters in the design code. Within this formulation, all relevant benefits and consequences associated to the structures should be considered; i.e. benefits in general for the society, construction costs, maintenance costs during the design life, and failure costs. In principle, target reliabilities can also be optimized within the formulation. However, it is often found difficult to formulate an objective function in a satisfactory manner taking into account all the benefits and consequences that can arise during the life time of structures. Especially, it is arguable how non-monetary benefits and costs can be taken into account in the objective function; not least, life safety (see Annex G).

For this reason, more practically operational approaches are recommended. One of such approaches is explained in the following. The approach takes basis in the reliability-based optimization and is explained in the context of the code calibration for semi-probabilistic design formats.

E.4 Reliability-based code optimization

E.4.1 Design equations and limit state functions

Before explaining the procedure for the reliability-based code calibration, the correspondence between the design equations in a semi-probabilistic design code and limit state functions are provided. The limit state functions related to individual failure modes are written as:

$$g_j(\boldsymbol{x},\boldsymbol{p}_j,\boldsymbol{z})=0 \tag{E.1}$$

where

$\boldsymbol{x}$ is the realization of the basic random variables X;

$\boldsymbol{p}_j$ is a vector of the deterministic parameters;

z is a set of the design variables.

The application area for the code is described by the set I of L different vectors $\mathbf{p}_j$, $j=1,\ldots,L$. The set I can, for example, contain different geometrical forms of the structure, different parameters for the random variables, and different statistical models for the random variables. The deterministic design equation corresponding to the limit state function in Formula (E.1) is written:

$$G_j(\boldsymbol{x}_c,\boldsymbol{p}_j,\boldsymbol{z},\boldsymbol{\gamma}) \geq 0 \tag{E.2}$$

where

$\boldsymbol{x}_c$ is the vector of the characteristic values of the basic random variables;

$\boldsymbol{\gamma}$ is the vector of the partial factors.

The set of partial factors should not only take into account the aleatory uncertainty on random variability in nature but also epistemic uncertainties associated with, for example, statistical uncertainties on the distributions of the load and resistance and due to the simplification incurred by employed models.

E.4.2 Procedure

E.4.2.1 Steps

Code optimization is generally performed in the following steps:

Step 1: Definition of the scope of the code.

Step 2: Definition of the code objective.

Step 3: Definition of code format.

Step 4: Identification of typical failure modes and probabilistic models.

Step 5: Definition of a measure of closeness.

Step 6: Determination of the optimal partial factors for the chosen code format.

Step 7: Examination and determination of partial factors.

E.4.2.2 Step1: Definition of the scope of the code

The class of structures and the type of relevant failure modes to be considered are defined.

E.4.2.3 Step 2: Definition of the code objective

The code objective can be defined using target reliability indices or target probabilities of failure depending on the use and characteristics of the considered class of structure. These can be determined by referring to the reliability indices implicitly or explicitly assumed in existing codes or based on other criteria, e.g. based on the LQI concept (see Annex G). The code optimization is therefore reduced to the optimization only on the partial factors[3].

E.4.2.4 Step 3: Definition of the code format

The code format includes: (1) how many partial factors and load combination factors to be used; (2) whether load partial factors should be material independent; (3) whether material partial factors should be independent of load type; (4) how to use the partial factors in the design equations rules for load combinations. In general for practical use the number of partial factors should be as few as possible and these should be as general as possible. On the other hand, a large number of partial factors are needed to obtain economical and safe structures for a wide range of different types of structures.

E.4.2.5 Step 4: Identification of typical failure modes and probabilistic models

Within the class of structures considered in a code, typical failure modes are identified. Limit state equations and design equations are formulated and probabilistic models for the parameters in the limit state equations are selected. Also, the frequency at which each type of safety check is performed is determined.

E.4.2.6 Step 5: Definition of a measure of closeness

The partial factors γ are calibrated such that the reliability indices corresponding to L different vectors $\mathbf{p}_j$ are as close as possible to a target reliability β_{target}. Therefore, a measure of closeness is formulated as:

$$\sum_{j=1}^{L} w_j \left(\beta_j(\gamma) - \beta_{\text{target}}\right)^2 \tag{E.3}$$

where

$\beta_j(\boldsymbol{\gamma})$ is the reliability corresponding to the given partial factors γ;

w_j ($j = 1,2,...,L$) are factors indicating the relative frequency of the appearance or importance of the different design situations ($\sum_{j=1}^{L} w_j = 1$).

Instead of using the reliability indices in Formula (E.3), the probabilities of failure can be employed. Also, a nonlinear objective function giving relatively large weight to reliability indices smaller than the target compared to those larger than the target can be used. The above formulations can be easily extended to include a lower bound on the reliability for each failure mode.

3) Here, partial factors should be interpreted in a generic manner as factors that are introduced in design equations in order to take into account the uncertainties in the random variables.

E.4.2.7 Step 6: Determination of the optimal partial factors for the chosen code format

The optimization problem is then formulated: The measure of closeness defined by Formula (E.3) is to be minimized with respect to the partial factors γ. The optimal partial factors are obtained by numerical solution of the optimization problem in step 5.

In the optimization process, the reliability index $\beta_j(\boldsymbol{\gamma})$ for design scenario j given the partial factors γ is obtained by the following two steps.

Firstly, for given partial factors γ the design z^* is determined. If the number of design variables is one, the optimal design z^* can be determined as the solution to the design Formula (E.2). If the number of design variables is more than one, a design variable optimization problem can be formulated:

$$\begin{aligned}
&\min_{\boldsymbol{z}} C(\boldsymbol{z}) \\
&s.t. \\
&c_i(\boldsymbol{x}_c, \boldsymbol{p}_j, \boldsymbol{z}, {}^{\boldsymbol{3}}) = 0, \quad i = 1,2,\ldots,m_e \\
&c_i(\boldsymbol{x}_c, \boldsymbol{p}_j, \boldsymbol{z}, {}^{\boldsymbol{3}}) \geq 0 \quad i = m_e + 1,\ldots,m \\
&z_i{}^l \leq z_i \leq z_i{}^u \qquad i = 1,2,\ldots,N
\end{aligned} \tag{E.4}$$

$C(z)$ is the is the objective function, and c_i ($i = 1,2,\ldots,m$) are the constraints. The objective function $C(z)$ is often chosen as the weight of the structure. The m_e equality constraints in Formula (E.4) can be used to model design requirements (e.g. constraints on the geometrical quantities) and to relate the load on the structure to the response (e.g. finite element equations). Often equality constraints can be avoided because the structural analysis is incorporated directly in the formulation of the inequality constraints. The inequality constraints in Formula (E.4) ensure that the response characteristics such as displacements and stresses do not exceed codified critical values as expressed by the design Formula (E.2). The inequality constraints can also include general design requirements for the design variables. The lower and upper bounds, $z_i{}^l$ and $z_i{}^u$, to z_i in Formula (E.4) are simple bounds. N is the number of the design variables. Generally, the design Formula (E.2) is nonlinear and non-convex.

Secondly, the reliability index $\beta_j(\boldsymbol{\gamma})$ is estimated by FORM/SORM or simulation on the basis of the limit state function in Formula (E.1), using the design $\boldsymbol{z}^*$ obtained as the solution to the design variable optimization problem formulated as Formula (E.4).

These two steps are repeated until the partial factors that minimize the measure of closeness formulated as Formula (E.3) are found.

E.4.2.8 Step 7: Examination and determination of partial factors

A first guess of the partial factors is obtained by solving the optimization problem in step 6. On this basis, the final partial factors are determined taking into account current engineering judgment and tradition.

E.5 Examples for calibration of partial factors[4)]

E.5.1 Problem setting

In the following, a simple but representative limit state function is considered:

$$g = zRX_R - \left((1-\varphi)G + \varphi(\varphi_Q Q_1 + (1-\varphi_Q)Q_2)\right) \tag{E.5}$$

where

- R is the load beading capacity;
- X_R is the model uncertainty;
- z is the design variable;
- G is the permanent load;
- Q_1 is the variable load type 1, e.g. wind load;
- Q_2 is the variable load type 2, e.g. snow load;
- φ is the factor between 0 and 1, modelling the relative fraction of variable load and permanent load;
- φ_Q is the factor between 0 and 1, modelling the relative fraction of the type 1 and type 2 variable load.

See Table E.1 for the probabilistic models for these basic random variables. All basic random variables are assumed to be independent. The load combination is assumed to be modelled by the Ferry-Borges-Castanheta load model.[6] The numbers of repetitions of the variable loads are assumed to be:

Wind load: 360 times per year

Snow load: 10 times in 5 month period where snow load is assumed to occur

Imposed load: 1 time per 10 years

Table E.1 — Probabilistic models for the basic random variables

Variable	Distribution type	Mean	Coefficient of variation	Characteristic value
G	Normal distribution	1,00	0,10	50 %
Q_1, Q_2	Gumbel distribution	1,00	0,40	98 %
R	Lognormal distribution	1,00	0,05	5 %
X_R	Lognormal distribution	1,00	0,03	50 %

The corresponding design equation can be written in the form of:

$$G = zR_c / \gamma_m - \left(\varphi\gamma_G G_c + (1-\varphi)\gamma_Q(\varphi_Q Q_{1c} + (1-\varphi_Q)\psi Q_{2c})\right) = 0 \tag{E.6}$$

The subscript c of the symbols represents the characteristic value, γ_m is the partial safety factor for the resistance, γ_G is the partial safety factor for the permanent load, γ_Q is the partial safety factor for the variable loads, and Ψ is the load combination factor. The above formulation corresponds to the case where partial safety factors are introduced to both resistance and load terms, as well as a combination

4) A software tool is available for code calibration in accordance with the procedure described here; CodeCal developed by JCSS, see also www.jcss.byg.dtu.

of the loads are considered. In the case where only one variable load is dominant, the second variable load term nay be neglected; hence, set equal to $\varphi_Q = 1$.

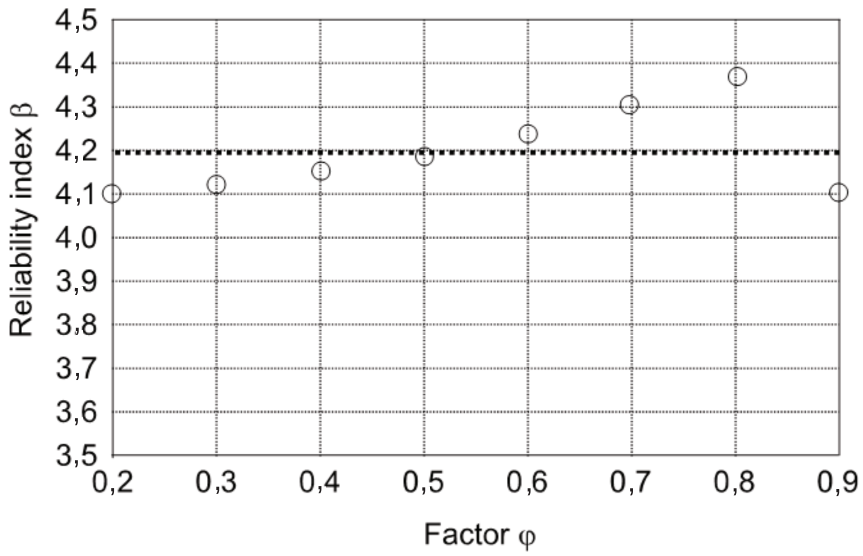
In the following, two cases for the calibration are considered where the target annual reliability index is chosen as $\beta_{\text{target}} = 4{,}2$, the values of φ between 0,2 and 0,9 with the interval of 0,1 are assumed to represent typical values and equal weights $w_j = 1/8$ ($j = 1,2,...,8$) for different values of φ are assumed. In the first case, only one variable load is dominant, whereas in the second case both of the two variable loads are relevant.

E.5.2 Case 1: One variable load and partial factors in both load and resistance terms

This case corresponds to the design Formula (E.6) with $\varphi_Q = 1$. The value of one of the three partial factors in Formula (E.6) can be chosen arbitrarily; here $\gamma_G = 1$ is chosen. Therefore, the design equation is reduced to:

$$G = zR_c / \gamma_m - \left(\varphi G_c + (1-\varphi)\gamma_Q Q_{1c}\right) = 0 \tag{E.7}$$

Following the procedure explained in E.4.2, the partial factors are obtained as $\gamma_m = 1{,}15$, $\gamma_Q = 1{,}65$. The achieved reliability indices for different values of φ are shown in Figure E.1.



Key

....	target reliability
○	achieved reliability

Figure E.1 — Reliability index as a function of φ

E.5.3 Case 2: Two variable loads and partial factors in both load and resistance terms

Load combination factors Ψ are determined for the following cases such that the target reliability index is $\beta_{\text{target}} = 4{,}2$:

Environmental load and non-dominating imposed load: $\psi_{I,E}$

Imposed load and non-dominating environmental load: $\psi_{E,I}$

Snow and non-dominating wind load: $\psi_{W,S}$

Wind and non-dominating snow load: $\psi_{S,W}$

Table E.2 shows the results for $\varphi_Q = 0{,}1$, 0,3, 0,5, 0,7, and 0,9.

Table E.2 — Load combination factors

	$\varphi_Q = 0{,}1$	$\varphi_Q = 0{,}3$	$\varphi_Q = 0{,}5$	$\varphi_Q = 0{,}7$	$\varphi_Q = 0{,}9$
$\psi_{I,E}$	0,75	0,6	0,3	0,25	0,0
$\psi_{E,I}$	0,6	0,45	0,3	0,05	0,0
$\psi_{W,S}$	0,9	0,6	0,1	0,0	0,0
$\psi_{S,W}$	0,7	0,45	0,1	0,0	0,0

See also Reference [4]. For more elaborated cases, the tutorial of CodeCal should be referenced. In case the calibration is performed on the basis of a classical approach, ISO 2394:1998, Annexes F and G should be referenced.

E.6 Calibration of design values in design-value methods

E.6.1 Design values according to FORM

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

— the parameters of the probability distribution of the variable X_i,

— the assumed type of distribution,

— the target reliability index β for the limit state and design situation of concern, and

— a factor α_i describing the sensitivity to variations in X_i with regard to attaining the limit state, according to the definition given in a FORM calculation.

For X_i with an arbitrary distribution $F(x_i)$, the design values are given by:

$$F(x_i) = \Phi(-\alpha_i \beta) \tag{E.8}$$

If X_i is assumed to be normally distributed, then

$$x_{id} = \mu_i (1 - \alpha_i \beta V_i) \tag{E.9}$$

Here, μ_i and V_i are the mean value and the coefficient of variation of the random variable X_i.

A lognormal distribution gives:

$$x_{id} = \xi_i \exp(-\alpha_i \beta v_i) \tag{E.10}$$

where

$$\xi_\mathrm{i} = \frac{\mu_i}{\sqrt{1 + \mathrm{V}_i^2}}$$

$$v_i = \sqrt{\ln\left(1 + \mathrm{V}_i^2\right)}$$

For a small value of V_i (e.g. $V_i \le 0{,}25$), $\xi_i \approx \mu_i$ and $v_i \approx V_i$.

E.6.2 Sensitivity factors according to FORM

If the random variables are independent, the factors α_i in a FORM analysis have the following properties:

$$-1 \le \alpha_i \le 1 \tag{E.11}$$

$$\sum \alpha_i^2 = 1 \tag{E.12}$$

The values of α_i should in principle be found from a number of representative FORM calculations. In principle, this would require many iterative calculations which, of course, is very inconvenient. However, based on experience, a set of standardized α_i values has been developed, which is presented in Table E.3. Note that the sum of squares can be greater than 1,0 as a result of conservatism. To limit the error in using Table E.3, it is usually required that $0{,}16 < \sigma_S/\sigma_R < 6{,}6$, where σ stands for the standard deviation and S is the dominating load and R is the dominating resistance parameter.

Table E.3 — Sensitivity factors

X_i	α_i
Dominating resistance parameter	0,8
Other resistance parameters	0,4 × 0,8 = 0,32
Dominating load parameter	−0,7
Other load parameters	−0,4 × 0,7 = −0,28
NOTE The principle of standardized α-values was already present in ISO 2394:1998, Annex B, where the same α-values as in Table E.3 were proposed.	

In applying Table E.3, one does not know in advance which variable should be regarded as "dominating". The only way to find this out is by making all variables "dominating" one at the time and see which one governs the design. Sometimes this can be done at the level of the code writer, sometimes it is the task of the designer (for instance, by checking various load cases).

EXAMPLE Consider the elementary case of one resistance parameter R and one load parameter S, both normally distributed. Assume that the target reliability index $\beta = 3{,}8$. Then from Formula (E.9):

$$R_\mathrm{d} = \mu_R - 3{,}04\sigma_R \text{ and } S_\mathrm{d} = \mu_s + 2{,}66\sigma_s$$

Now one should check $R_d > S_d$. If the load parameter S follows the lognormal distribution, Formula (E.10) is employed instead of Formula (E.9):

$$S_\mathrm{d} = \mu_S \exp(2{,}66 V_S)$$

Then one should check $R_\mathrm{d} > S_\mathrm{d}$.

E.7 Partial factor design for fatigue based on S-N lines

E.7.1 S-N lines

The S-N line approach is often used for design with respect to fatigue and 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 might or might 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:

$$\sum \frac{n_i}{N_i} > D_c \tag{E.13}$$

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) can 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.

E.7.2 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:

$$\sum \frac{n_i}{N_i} < \frac{D_c}{g_\mathrm{d}} \tag{E.14}$$

$$N_i = N_i \left(g_\mathrm{Ff} S_i, \frac{R_\mathrm{fc}}{g_\mathrm{Mf}} \right) \tag{E.15}$$

where

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

R_fc is the characteristic value of the fatigue resistance;

g_Ff is the partial factor that deals with the uncertainties in the load level and load model;

g_Mf is the partial factor that deals with the uncertainties in the material model;

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

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, and

— the ability to effect repairs.

An example of the calibration for partial factors for fatigue design can be found in Reference [11].

Annex F
(informative)

Structural robustness

F.1 Introduction

In Clause 4, it is specified that the design of structures shall be supported by risk-based robustness assessments and/or by consideration of robustness provisions in dependence of the exposures acting on the structure, as well as the structural system and the consequences of system failure. In the present Annex, a methodical framework is outlined which supports such assessments, as well as for the identification of appropriate provisions.

First a classification of structures is presented which allows for a “rule of thumb” assessment of whether a risk-based assessment and design is required in a given situation, as well as the level of necessary efforts. Thereafter, a guideline for the selection of appropriate robustness design strategies and provisions is provided for the cases where risk-based assessments are not required and finally the theoretical framework is provided for a full risk or reliability-based robustness assessment.

This annex is based on the results of the COST Action TU0601 Robustness of Structures (see also Reference [2]) from where most of the text segments, tables, and figures have been taken and adapted.

F.2 Classification of structures according to consequences

The classification of structures is facilitated by the consequence classes shown in Table F.1.

Table F.1 — Consequence Classes (partly adapted from Eurocode EN 1991–1-7:2006)

Consequences class	Description of expected consequences	Examples of structures
Class 1	Predominantly insignificant material damages.	Low-rise buildings where only few people are present, minor wind turbines, stables, etc.
Class 2	Material damages and functionality losses of significance for owners and operators but with little or no societal impact. Damages to the qualities of the environment of an order which can be restored completely in a matter of weeks. Expected number of fatalities fewer than 5.	Smaller buildings and industrial facilities, minor bridges, major wind turbines, smaller or unmanned offshore facilities, etc.
Class 3	Material losses and functionality losses of societal significance, causing regional disruptions and delays in important societal services over several weeks. Damages to the qualities of the environment limited to the surroundings of the failure event and which can be restored in a matter of weeks. Expected number of fatalities fewer than 50.	Most residential buildings, typical bridges and tunnels, typical offshore facilities, larger and or hazardous industrial facilities

Table F.1 *(continued)*

Consequences class	Description of expected consequences	Examples of structures
Class 4	Disastrous events causing severe losses of societal services and disruptions and delays at national scale over periods in the order of months. Significant damages to the qualities of the environment contained at national scale but spreading significantly beyond the surroundings of the failure event and which can only be partly restored in a matter of months. Expected number of fatalities fewer than 500.	High-rise buildings, grandstands, major bridges and tunnels, dikes, dams, smaller offshore facilities, pipelines, refineries, chemical plants, etc.
Class 5	Catastrophic events causing losses of societal services and disruptions and delays beyond national scale over periods in the order of years. Significant damages to the qualities of the environment spreading significantly beyond national scale and which can only be partly restored in a matter of years to decades. Expected number of fatalities larger than 500.	Buildings of national significance, major containments and storages of toxic materials, major offshore facilities, major dams and dikes, etc.

The appropriate robustness measures and the appropriate method of analysis to use for a situation can depend on its consequence category, e.g. in the following manner:

Consequence Class 1: No specific consideration of robustness.

Consequence Class 2: Depending on the specific circumstances of the structure in question. Simplified analyses based on idealized load and structural performance models and/or prescriptive design/detailing rules.

Consequence Class 3: Systematic identification of scenarios leading to structural collapse. Address of strategies to deal with the identified scenarios. Analyses of structural performance can be based on simplified and idealized models only subject to justification. Prescriptive design and detailing rules can be utilized but should specifically address the identified scenarios. Reliability and risk analyses addressing direct and indirect consequences should be used as basis for simplifications and idealizations.

Consequences Class 4: Extensive study and analyses of scenarios leading to structural collapses utilizing risk screening meetings involving experts on all relevant subject matters. Detailed assessments shall be undertaken using dynamic and nonlinear structural analyses and risk analyses rigorously addressing direct and indirect consequences.

Consequence Class 5: As for Consequence Class 4 but with the addition of the involvement of an external expert/review panel for quality control.

It should be mentioned that the consequences described in Table F.1 are to be understood as the expected value of consequences and thus implicitly depend on leading exposures, structural performance, and loss reduction. Nuclear facilities are not explicitly mentioned in the table, however, they can be addressed along the lines of Consequence Class 5 structures.

F.3 Guideline for selection of appropriate design strategies and robustness provisions

F.3.1 General

In the case where a risk and reliability based approach is not required, the available design methods to identify appropriate robustness provisions can be classified as follows (see also Table F.2):

— Event control (EC): Affects the probability of occurrence of hazard H. (Note: This method is applicable only when designing for identified hazards).

— Specific load resistance (SLR): Influences the probability of local damage, L, given that H occurs, i.e. reduces the vulnerability of the structure and its members. Local damage is more generally known as direct damage.

— Alternative load paths (ALP): Influences the probability of further (i.e. “follow-up”) failure, such as collapse given local failure. Structural provisions such as ties can help to provide ALP.

— Measures that reduce the consequences of failure, especially of indirect consequences.

The measures a) and d) are indirect methods while b) and c) are direct methods for preventing disproportionate collapse. Direct design aims at explicitly ensuring collapse resistance in the design process by demonstrating that the structure meets the specified performance objectives when specified hazard scenarios occur and affect the structure. Direct design thus strongly relies on structural analysis. Indirect design, on the other hand, aims at reducing the effects of a hazard implicitly by incorporating agreed design features that help to achieve the performance objectives.

Table F.2 — Classification of design methods

Method	Reduces	Issues to address
a) Event control (EC)	Probability of occurrence and/or the intensity of an accidental event	— Monitoring, quality control, correction, and prevention
b) Specific load resistance (SLR)	Probability of local damage due to an accidental event	— Strength and stiffness — Benefits of strain hardening — Ductility versus brittle failure — Post-buckling resistance — Mechanical devices
c) Alternative load path method (ALP), including provision of ties	Probability of further damage in the case of local damage	— Multiple load path or redundancy — Progressive failure versus the zipper stopper — Second line of defence — Capacity design and the fuse element — Sacrificial and protective devices — Testing — Strength and stiffness — Continuity and ductility
d) Reduction of consequences	Consequences of follow up damage such as progressive collapse	— Segmentation — Warnings, active intervention, and rescue — Redundancy of the services of the facilities

F.3.2 Event control

Event control refers to avoiding or protecting against an incident that might lead to disproportionate failure. This approach does not increase the inherent resistance of a structure to disproportionate failure. Once the building is in use, the effectiveness of this method depends on how those operating and using it comply with the designer's specifications and recommendations regarding event control. However, there are preventive measures that can reduce the probability of a hazard materializing at a high intensity, such as the following:

— Planning of the location of the structure.

— Provision of stand-off perimeter.

— Provision for surveillance systems such as alarm and security.

— Prohibiting the storage of explosives.

— Placing protective measures around the structure to prevent impacts.

— Gas detectors and automatic cut-off devices for gas.

— Control or limiting of fire ignition sources.

— Limiting fire loads.

— Fire suppression systems.

— Installation of smoke detectors and alarms.

— Use of Structural Health and Monitoring Systems.

— Quality control during construction, maintenance, and repair activities.

The preventive safety measures can lead to a reduction of the probability of a hazard $P(H)$ occurring at a high intensity and to a reduction of the associated return period.

F.3.3 Specific Load Resistance (SLR)

In this method, sufficient strength to resist failure from accidents or misuse is provided to structural members in certain regions of a building to allow them to resist accidental loads. For this, it is necessary to classify members according to their importance to the survival of the structure and identify the so-called key elements. Their failure is expected to cause further damage that violates the performance objectives because, in their absence, the structure as a whole is unable to develop sufficiently strong alternative load paths. Examples of potential key elements could be columns and load-bearing walls of a building, a pier of a continuous bridge, or a cable in a cable-supported structure.

It is necessary to bear in mind that ensuring higher safety against initial damage requires more than the use of higher design loads or recourse to protective measures. An initial damage can also be caused by occurrences such as corrosion or fire — events that are more effectively counteracted by 'Event Control' measures such as corrosion protection, regular inspection, fire protection, and fire-fighting systems than by increasing design loads.

The SLR method is suitable and cost-effective for structures with a limited number of identifiable key elements. However, it needs to be applied to structures with a larger number of key elements when other methods are not easy or practical to implement, e.g. where the structural system lacks alternative load paths and event and consequence controls are difficult, if not impossible, to implement.

F.3.4 Alternative Load Paths (ALP)

Alternative load path (ALP) is a direct method to enhance the robustness of a structure. In this approach, typically by sequential removal of structural members, alternatives for a load to be transferred from a point of application to a point of resistance are being assessed. This enables redistribution of forces originally carried by failed components (provided the new paths are sufficiently resistant) in order to prevent a failure from spreading. Alternative load paths can also form through load-transfer mechanisms, for example via:

— Inversion of flexural load transfer (from hogging to sagging above a failing column).

— Transition from flexural to tensile load transfer (catenary action).

— Transition from plane to spatial load transfer (in one-way slabs turning into two-way slabs).

This direct method requires the designer to prove that a structure is capable of fulfilling its performance objectives by bridging over one or more failed (or notionally removed) structural elements, with a potential additional damage level lower than a specified limit. The method can be applied in both hazard-specific and non-hazard-specific situations because the notional damage to be considered in the application of the alternative- paths method is non-threat-specific. When using the alternative-path method in a threat-specific manner, the initial damage that could result from an accidental event is first determined from a preliminary analysis. In many standards for building design ties are recommended via prescriptive design rules such that catenary action can be generated and ductility ensured in a building structure. It appears that the aim of most of the codified measures is to provide, in one way or another, alternative load paths.

Basically, the ALP strategy considers the situation that one or more structural elements (beams, columns, walls) have been damaged, by whatever event, to such an extent that their normal load bearing capacity has vanished completely.

The probability that some element is removed by some cause depends on the type of structure. In the case of a non-hazard-specific design, the ALP strategy starts with the assumption of reasonable scenarios of initial damage. The structure is then designed such that the spread of this local initial damage remains limited.

The generation of alternative load paths, in detail locations or structures where they are not explicitly considered, is helped by the provision of minimum levels of strength, continuity, and ductility to a structure. For example, the following are good practices that can enhance robustness of a building:

— Good plan lay-out.

— Integrated tie system.

— Returns on walls.

— Redundancy.

— Ductile Detailing.

— Fire resistance of structural members.

However, sometimes it is good to incorporate segmentation into large structures, without tying all its parts together, so that any failure can be stopped from progressing beyond a segment.

F.3.5 Consequence reducing measures

The implementation of consequence reducing measures aims at reducing the direct and indirect consequences of failure and thus the total risk. Such measures can be for example:

— Structural and architectural.

— Electro-Mechanical (equipment).

— Organizational, including emergency planning.

— Self-rescue and rescue by others.

— Backup facilities.

Important structural and architectural measures are, for example, the possible segmentation, or compartmentation, of the structure and the provision of effective escape/evacuation routes.

Electro-mechanical equipment measures include, for example, automatic sprinkler systems, warning systems for evacuation, control centres including video-monitoring systems, etc. Organizational measures are, for example, a clear emergency management system and safety consciousness of all staff and users of a structure. Self-rescue, and rescue by other means, can be made more effective by having regular trial exercises such as flood and fire drills.

The consequences of the loss of an important structure can be mitigated by having alternative facilities that can be used for the same purpose within a short time. In the case of banks, this can involve the duplication and storage of records elsewhere so that a different branch could take over the function of the one that has become dysfunctional.

F.4 Risk-based approach to robustness assessments

F.4.1 General

Structural systems risk assessments benefit from representation of the systems which facilitate that decision alternatives aiming to improve the systems performances can be identified and ranked in accordance with their risk-reducing effects. In the following, such a framework is introduced and its application for robustness assessments of structures is outlined.

F.4.2 Risk assessment for structural systems

In formal risk assessments carried out for the purpose of decision-making, a scenario approach can be used as defined by the three steps given below.

— Step 1: The modelling of the hazards (exposures) H_i.

— Step 2: The assessment of the direct damage D_j.

— Step 3: The assessment of follow-up structural behaviour S_k and corresponding total consequences $C(S_\mathrm{k})$.

Given the relevant (conditional) probabilities, the risk related to a structural system can then be written as:

$$R = \sum_i \sum_j C_{\mathrm{dir},ij} P\left(D_j \middle| H_i\right) P\left(H_i\right) + \sum_\mathrm{k} \sum_i \sum_j C_{\mathrm{ind},ij\mathrm{k}} P\left(S_\mathrm{k} \middle| D_j \cap H_i\right) P\left(D_j \middle| H_i\right) P\left(H_i\right) \quad \text{(F.1)}$$

where

$C_{\mathrm{dir},ij}$ is the expected consequence (cost) of damage (local failure) D_j due to exposure H_i;

$C_{\mathrm{ind},ij\mathrm{k}}$ is the expected consequence (cost) of damages (follow-up) S_k given local damage D_j due to exposure EX_i:

$P\left(H_i\right)$ is the is the probability of exposure H_i;

$P\left(D_j \middle| H_i\right)$ is the probability of damage D_j given exposure H_i;

$P\left(S_\mathrm{k} \middle| D_j \cap H_i\right)$ is the probability of comprehensive damages S_k given local damage D_j due to exposure H_i.

The metric of the consequences C could be either a monetary unit (often per time unit) or the expected number of casualties when only life safety is the concern. The latter is usually used to assess the societal risk or individual risk, where relevant. As mentioned previously, the relevant consequences depend on the boundary of the system considered in the risk analysis.

The risk assessment of a given system can be facilitated by considering the generic representation illustrated in Figure F.1. The exposure to hazards is represented by different exposure events that can act on the constituents of the considered system.

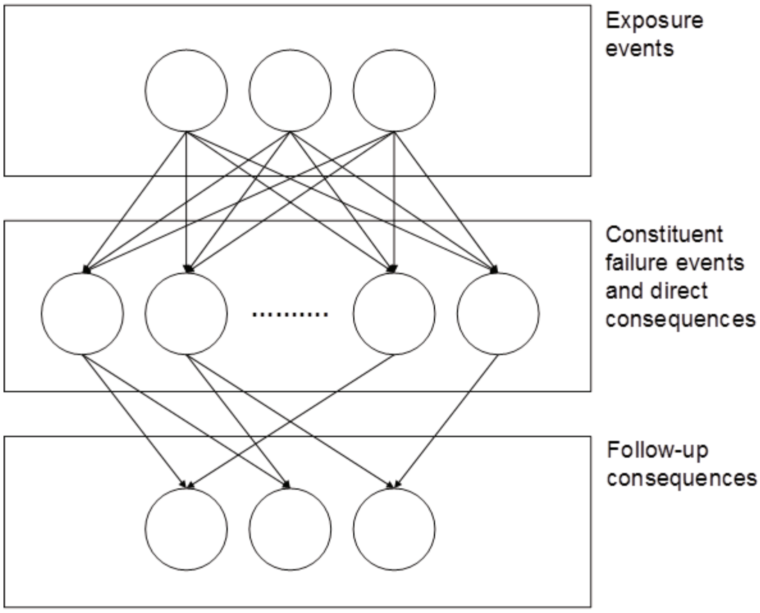


Figure F.1 — Generic system representation in risk assessments (follow-up consequences are known also as “indirect” consequences)

The constituents of a system can be considered as its first defence against a hazard. The damage to the system, caused by failures of the constituents, is considered as associated with “direct consequences”. Direct consequences can comprise different attributes of a system, such as monetary losses, loss of lives, and damage to the environment or even just changes to the characteristics of the constituents (see Table F.4). Depending on the combination of events of constituent failure and the corresponding consequences, follow-up (or “indirect”) consequences can occur. If the structure is robust, these follow up consequences will be small. The opposite is true when a structure is not robust.

F.4.3 Representation of hazards

The hazards which should be considered strongly depend of the situation of the structure. Three different categories can be identified:

— The first category is the type of hazards that are more or less given rise to by nature or general human activities. Natural hazard are those such as strong winds and earthquake. (Unintentional) man-made hazards include explosions. However, the difference between them is hardly relevant for structural design.

— The second category includes the type of (man-made) actions that are deliberate, such as vandalism and malicious attacks. To some extent, it might not help to make a structure stronger to resist them because it could generate more (re)action on the loading side. This type of hazard has become more important after the events of 11 September 2001.

— The third category includes errors and negligence. There is a direct link of this type of hazards with quality control and supervision. These hazards are best controlled by good supervision and quality control during all stages of the life of structure and by including general robustness measures suitable to deal with unidentified actions.

As a guideline, the hazards indicated in Table F.4 should be considered as potential candidates.

Table F.4 — Overview of relevant hazards for risk assessments and structural safety

Hazard	Category
Internal gas explosion	1
Internal dust explosion	1
Internal bomb explosion	2
External bomb explosion	2
Internal fire	1
External fire	1
Impact by vehicle	1
Impact of aircraft, ships	1
Earthquakes	1
Landslide	1
Mining subsidence	1
Tornado and Typhoons/Hurricanes/ Cyclones	1
Avalanche	1
Rock fall	1
High groundwater	1
Flood	1
Storm surge	1
Volcanic eruption	1
Environmental attack	1
Tsunami	1
Vandalism	2
Public disorder effects	2
Design or assessment error	3
Material error	3
Construction error	3
User error	3
Lack of maintenance (deterioration)	3
Errors in communication	3

Note In some situations, it is possible for one hazard to be followed by another, resulting in much more serious consequences to a structural system. Some examples of such situation are the following:

— gas explosion and/or fire following an earthquake;

— tsunami following an earthquake;

— fire following either a gas explosion or bomb blast;

— fire following a tornado or other wind storm;

— component deterioration, following damage from an accidental action.

F.4.4 Direct and indirect consequences

The consequences can be expressed by, for example, the sum of monetary losses associated with the constituent failures and the physical changes of the system as a whole caused by the combined effect of constituent failures.

In the risk assessment of systems, a major role is played by the follow-up consequences, and the modelling of these should be given great emphasis. It should be noted that any constituent in a system can be modelled as a system itself. A system could be a road network with constituents being, for example, bridges. A bridge, in turn, could also be a system with constituent structural members. Depending on the level of detail in the risk assessment, i.e. the system definition, the exposure, constituents, and consequences will be different.

Depending on the situation, the consequences listed in Table F.5 might be considered.

Table F.5 — Types of consequences of an undesirable event

Consequence Type	Consequence
Health	Fatalities
	Injuries
	By damaging vital facilities (e.g. hospitals), spread of diseases
	Delayed long-term health effects
	Psychological
Economic/Property	Damage to the building/structure
	Damage to surrounding properties
	Damage to contents
Business Continuity	Loss of income
	Loss of customers
	Inability to provide vital services and/or activities
	Costs of detours and delays
	Costs to the economy of a region
Environmental	Reversible environmental damage
	Irreversible environment damage
Social and Political	Loss of reputation
	Increase of public fears
	Loss of political support/enforcement of stringent new measures
	“Blight”/long-term evacuation

As an example of the evolution of consequences associated with a scenario leading to structural collapse, consider a building subject to an explosion at one of the upper storeys, as shown in Figure F.2. Here, the direct consequences are defined by the change in behaviour and/or damage suffered by the (directly) exposed components (see ‘Step 2’ of the figure). Based on the level of this response, the behaviour of the other parts of the structure can result in follow-up or indirect consequences (see ‘Step 3’ of the figure).

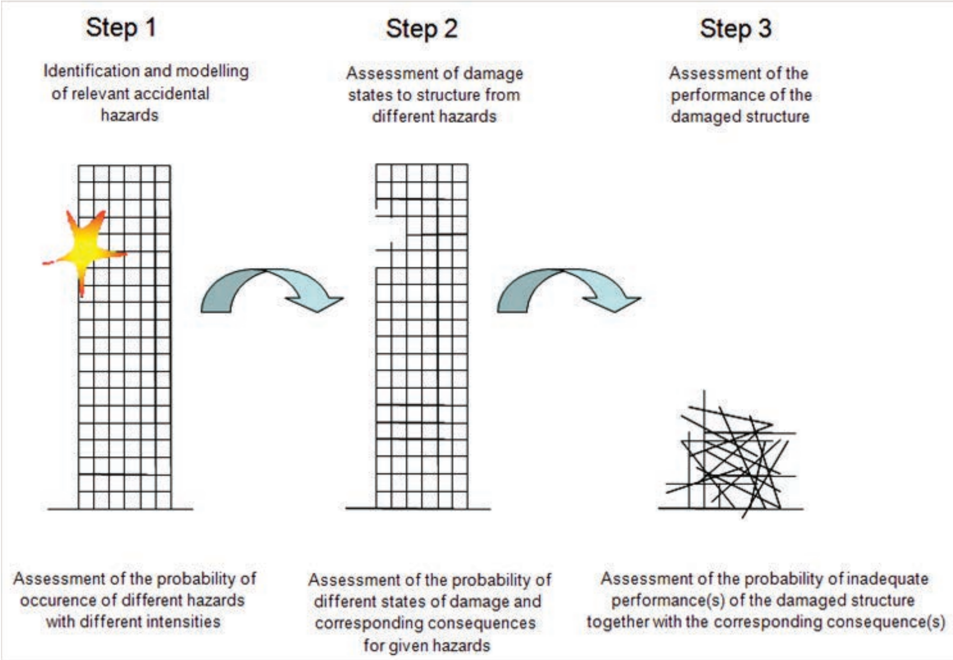


Figure F.2 — Steps in a consequence analysis

It is important to notice that the definition of the boundaries of the systems which is considered plays an important role for the consequences and thus the total risk. The boundaries can be defined such that only the consequences associated with the structure itself are considered. In this case, the direct consequences are the marginal consequences associated with damages of the constituents of the structure and its contents. The indirect consequences would be associated with cascading constituent failures and ultimately structural collapse leading to loss of the entire structure. However, in many cases especially for structures belonging to Consequence Class 3 and upwards, it is important to include consequences beyond effects to the considered structure alone. This is especially important when considering consequences in terms of damages to the qualities of the environment as well as business and reputation related losses.

F.4.5 Further characteristics of risks and structural robustness

While the risk from failure can be determined as given in Formula (F.1), a non-dimensional index of robustness facilitates the comparison of different decision alternatives to improve robustness.

A risk-based robustness index (I_{rob}) can be defined as:

$$I_{\text{rob}} = \frac{R_{\text{Dir}}}{R_{\text{Dir}} + R_{\text{Ind}}} \tag{F.2}$$

where

R_{Dir} and R_{Ind} are the direct and indirect risks associated with the first and the second term in Formula (F.1).

The index takes values between zero and one, with larger values indicating larger robustness.

Considering that the optimal decision is the one minimizing the total risk, and which can be attained by reducing the first or the second term of Formula (F.1), this robustness index is seen to be not always fully consistent with a full-risk analysis in situations where decisions can be taken freely to affect both direct and indirect consequences. However, in practice, codes and regulations largely fix the direct consequences and robustness improvements consequently mainly address the indirect consequences. The robustness index can thus be considered as a helpful indicator of robustness based on risk analysis principles. It is to be noted that since the direct risks typically are related to code-based limit states, they can generally be estimated with higher accuracy than the indirect risks.

The index accounts not only for the characteristics of the structural performance but also for the performance of the system after damage and all relevant consequences. Furthermore, all measures (decision alternatives), which can be implemented either to improve the structural performance in regard to robustness or to decrease the vulnerability (increasing component reliability), are explicitly accounted for. It should be noted that the index of robustness is conditional in the same principal manner as is the structural reliability. The robustness index is to be understood as being conditional on the level of reliability of the individual components/failure modes of the system (the level of direct consequences), as well as the ratio between direct and indirect consequences. The reliability of structural components and failure modes is conditional on the probabilistic modelling and the best practice technology and design practices.

The risks conditional to the occurrence of hazard decommissioning events and the corresponding conditional index of robustness provides decision support for the cases where the hazard events are not well understood and quantifiable in probabilistic terms. In such cases, the robustness of structures can be addressed by assuming that a hazard takes place and leads to a certain predefined level of damage characterized by the damage state vector $\boldsymbol{D}=(D_1,D_2,..D_n)^T$ and where n is the number of defined damage states for the constituents of the structural systems. The corresponding conditional risk can then be evaluated as:

$$R\big|_{\boldsymbol{D}}=\sum_j C_{\mathrm{dir},j}P\left(D_j\right)+\sum_{\mathrm{k}}\sum_j C_{\mathrm{ind,k}}P\left(S_{\mathrm{k}}\middle|D_j\right)P\left(D_j\right) \tag{F.3}$$

where the considered damage states are imposed to the system by setting their occurrence probabilities equal to 1.

F.4.6 Alternative measures relating to robustness

Alternatively to the risk-based robustness index, a reliability-based redundancy index (RI) (Frangopol and Curley [1987]) can be defined through:

$$RI=\frac{P_{\mathrm{f(damaged)}}-P_{\mathrm{f(intact)}}}{P_{\mathrm{f(intact)}}} \tag{F.4}$$

where

$P_{\mathrm{f(damaged)}}$ is the probability of failure for a damaged structural system;

$P_{\mathrm{f(intact)}}$ is the probability of failure of an intact structural system.

This redundancy index provides a measure on the redundancy of a structural system. The index takes values between zero and infinity, with smaller values indicating larger robustness.

Finally, a simple and practical measure of structural redundancy used in the offshore industry is based on the so-called Residual Influence Factor (RIF) value (ISO 19902).

A Reserve Strength Ratio (RSR) is defined as:

$$\mathrm{RSR}=\frac{R_{\mathrm{c}}}{S_{\mathrm{c}}} \tag{F.5}$$

where

R_{c} denotes characteristic value of the base shear capacity of an offshore platform (typically a steel jacket);

S_{c} is the design load corresponding to ultimate limit state.

In order to measure the effect of full damage (or loss of functionality) of structural member “i” on the structural capacity, the so-called ‘RIF’ value (sometimes referred to as the Damaged Strength Ratio) is defined by:

$$\mathrm{RIF}_i = \frac{\mathrm{RSR}_{\mathrm{fail},i}}{\mathrm{RSR}_{\mathrm{intact}}} \tag{F.6}$$

where

$\mathrm{RSR}_{\mathrm{intact}}$ is the RSR value of the intact structure;

$\mathrm{RSR}_{\mathrm{fail},i}$ is the RSR value of the structure where member “i ” has either failed or has been removed.

The RIF takes values between zero and one, with larger values indicating larger redundancy.

Annex G
(informative)

Optimization and criterion on life safety

G.1 Introduction

To ensure that the economic optimal and feasible decisions are also coherent with the societal preferences for health and life safety, 4.4.2.1 specifies under which premises such decisions can be considered acceptable. The principle invoked for this purpose is the marginal lifesaving cost principle (MLSC) which specifies that decisions with possible effects of life safety are acceptable if the costs associated with possible means of saving one additional life are in balance with the societal willingness to pay for saving one statistical individual. The MLSC principle can adequately be implemented in practice using the Life Quality Index (LQI).

In the present annex selected important aspects relating to the practical use of 4.2.2 and 4.4.2.1 are outlined, namely the following:

— How do the principles relate to present best practices;

— How to calculate the expected value of the benefits as basis for optimization;

— How to calculate the limiting value of the lifesaving costs based on the LQI;

— How to apply the principles in practice.

G.2 Relationship between the MLSC principle and present best practice

Presently, regulatory criteria concerning life safety risk acceptance are typically specified in terms of the As Low As Reasonably Possible/Practicable (ALARP) format. The idea is that risks depending on their magnitude can be evaluated to be negligible, tolerable, or non-acceptable, as shown in Figure G.1 where the ALARP format is illustrated.

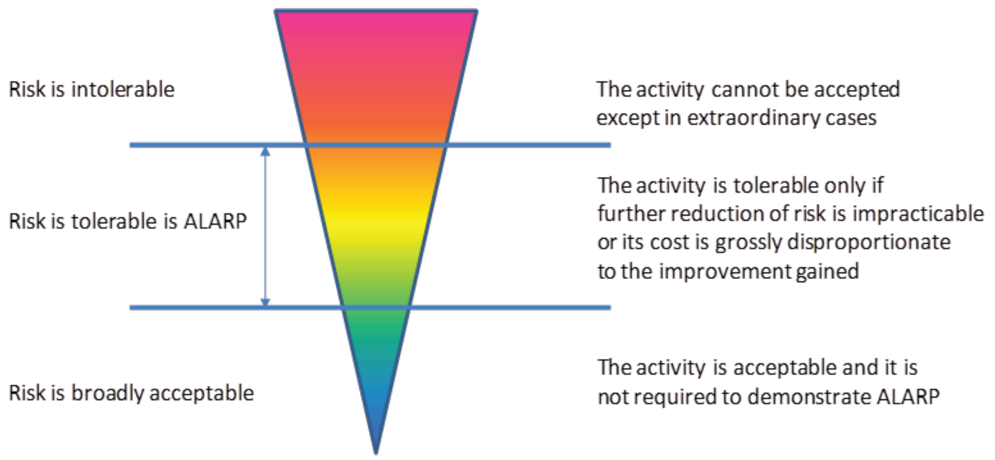


Figure G.1 — Illustration of the typical ALARP principle in life safety risk regulation

In Table G.1, the risk criteria as applied within the ALARP framework for European nations are given for purposes of illustration. It is seen that there is some variability on the selection of risk criteria among nation states.

In addition to the ALARP, the so-called precautionary principle deserves mentioning. This principle which has been formulated in the context of European risk regulation is now applied in a large part of the world. The controversial principle can be interpreted as a measure to safeguard society against (technical) developments having disproportionally large consequences for society. The precautionary principle finds application in relation to the assessment of activities for which there are no experience and for which there is suspicion that they might have unacceptable adverse consequences. For such situations, it is required that more knowledge should be collected to ensure that such activities can be sufficiently controlled and managed using known best practice technologies.

It can be observed that the general ALARP framework is very close to the marginal lifesaving cost principle. However, to ensure this, certain specific interpretation of the ALARP format is necessary.

Table G.1 — Overview of risk criteria in terms of annual fatality rates from the EU

Annual fatality rate	UK	Netherlands	Hungary	Czech republic
10^{-4}	Intolerable limit for members of the public			
10^{-5}	ALARP region	Limit for existing installations (ALARP)	Upper limit	Limit for existing installations. Risk reduction shall be carried out.
3×10^{-6}	Land Use Planning (LUP) criteria		Lower limit	Limit for new installations
10^{-6}	Broadly acceptable	Limit for new installations and general limit after 2010 (ALARP)		

The implementation of the marginal lifesaving costs principle in the ALARP format can be formulated as:

a) Decisions and activities associated with life safety risks should be assessed with regard to the subsequently stated principles unless it can be documented that the risks are regulated by other prescriptive standards and regulations or the risks can be documented to be negligible.

b) Risk analyses shall be performed in compliance with best practices such as outlined in Risk Assessment in Engineering — Principles, System Representation & Risk Criteria (JCSS 2008) and relevant ISO Standards.

c) Activities shall be assessed with regard to their societal acceptance in compliance with the marginal lifesaving costs principle. It should be demonstrated that life safety risks are reduced by means of best practice technical, organizational, and procedural measures to a level at which further life safety risk reductions would exceed the limiting lifesaving costs (as described in G.4). Measures for risk reduction shall as a principle be sought from the best practice, such as expressed in codes and standards unless it can be justified that such are inappropriate or inefficient in a given situation.

d) An activity which is found acceptable should be assessed in regard to the corresponding absolute level of life safety risk. If the absolute level of life safety risk exceeds a threshold value, it should be evaluated specifically whether best practice has been followed in all steps of the assessment. Furthermore, it should be assessed whether the case considered is appropriately treated by best practice procedures or whether a further assessment following the precautionary principle should be undertaken. If this is not the case, the activity is considered acceptable.

Although the marginal lifesaving costs principle does not necessitate or even allow for specifying absolute values of the acceptable life safety risks, its practical implementation can require such boundaries. First of all, to ensure that a full quantitative risk assessment is not required for situations for which risks are known to be small and where life safety risks are adequately managed by specific, often prescriptive, regulations, codes and standards. Secondly, to ensure that, if absolute risks are indeed high compared to risks in general, the assessment in itself is performed in accordance to the state of the art and that the activity does not fall into the category of activities which should be assessed under the precautionary principle. Finally, the risk levels which correspond to the necessary investments into life safety can be utilized as a basis for the calibration of codes for the design of structures as explained in G.5.

G.3 How to calculate the expected value of the benefits as basis for optimization

G.3.1 General

Concerning the calculation of the expected value of benefits, it is necessary to differentiate between two situations, namely the situation where 1) a private corporation or owner optimizes a decision concerning the design of a structure and 2) public authorities or code committees are optimizing the design of structures or calibrating optimal design codes (as described in Annex E).

G.3.2 Case of private ownership

In case of private ownership, the issue on what to include in the calculation of benefits and how to calculate these is left completely to the free market mechanisms and possible incitements imposed on this from the public administrations. The possible incomes arising from a structure depend strongly on the specific situation at hand, hereunder the scheme for the financing of the project and the applied contractual framework. On the cost side, it is suggested to include the following cost items:

— Design and quality assurance costs.

— Construction costs.

— Inspection and maintenance costs.

— Down-time costs.

— Repair costs.

— Failure/reconstruction costs.

— Compensation costs (life and limb, damages to the environment, as well as other inconveniences and damages for third parties) or alternatively the costs for insurance.

— Loss of reputation costs.

— Decommissioning costs.

It is suggested to include the above mentioned cost items in a life-cycle based assessment of risks whereby the expected value of the costs are calculated taking into consideration the probability that the costs occur during the considered life-cycle and discounted appropriately to their net present value.

G.3.3 Case of public ownership

In cases where the ownership is public, e.g. the common situation in standardization work, it is here recommended that the benefits are calculated according to the best available knowledge concerning the societal benefit arising from the structures under consideration, whether these concern specific projects or projects more generally specified through design codes and related regulations.

In this case, all the cost items listed in the foregoing should be included in the calculation of the life cycle benefits with an appropriate consideration of the probabilities associated with their occurrence. For this purpose, risk and reliability assessments should be utilized. The life cycle should be assessed with due consideration of the characteristics of the considered structural design. In case of specific singular projects, it can be reasonable to consider a life cycle in accordance with the projected service life of the considered specific structure. However, in general and in the case of design codes in particular, it is recommended that the life cycle does not focus on the service life of the individual structures but rather the duration of the demand the structures are aiming to fulfil. This would in practice correspond to service lives in excess of 100 years.

To ensure socio-economic sustainability, in the calculation of the net present value of life cycle costs, the discounting should take basis in a carefully assessed long-term economic growth rate; presently for most mature economies around 1 % to 3 % per annum. For many developing economies, the corresponding rate is presently around 5 % to 8 % per annum.

G.4 How to calculate the limiting value of the lifesaving costs based on the LQI

G.4.1 General

The Life Quality Index (LQI) was proposed some 15 years ago.[11] The philosophy behind the LQI is that the preference of a society in regard to investments into health and life safety improvements can be described in terms of the life expectancy at birth, the Gross Domestic Product (GDP) per capita and the ratio between working time and leisure time. Whereas the LQI was originally proposed on the basis of socio-economic theoretical considerations, its validity has been empirically justified subsequently. The LQI facilitates the assessment of decisions with regard to their conformity with societal preferences in regard to life safety investments. Based on the LQI, it is possible to derive the so-called limiting value of lifesaving costs, i.e. the necessary and affordable expenditures which should be invested into saving one additional life. Alternative approaches from the field of economics such as contingent life value evaluations have been found to yield results similar to those of the LQI.

The marginal lifesaving cost as assessed through the LQI depends on the economic capacity and preferences to invest into life safety for a given society, as well as on the effect of the life-saving measures over the population. This poses the following questions:

— How do societal economic capacity and preferences influence the marginal lifesaving costs?

— How to account for the effect of a life saving measure over the population?

To answer these questions, the basic methodology behind the LQI is summarized in the following (see Reference [11]).

G.4.2 Assessment of the marginal lifesaving costs based on the LQI

The LQI can be expressed in the following principal form:

$$L(g,e) = g^q e \tag{G.1}$$

where

g is the GDP per capita;

e is the life expectancy;

q is the measure of the trade-off between the resources available for consumption and the value of the time of healthy life.

q depends on the fraction of life w allocated for economic activity (working time to free time ration, typically around 0,18 to 0,2) and furthermore accounts for the fact that a part of the GDP is realized through work and the other part through returns of investments. e and w are assessed at national scales. The constant q is assessed as:

$$q = \frac{w}{(1-w)\beta} \tag{G.2}$$

where

β is the Cobb-Douglas elasticity constant, here set equal to 0,7.

The Societal Willingness To Pay (SWTP) and the Societal Value of a Statistical Life (SVSL) can be assessed as:

$$\mathrm{SWTP} = \frac{g}{q}\frac{de_{\mathrm{d}}}{e_{\mathrm{d}}} \approx \frac{g}{q} C_x dm = G_x dm \tag{G.3}$$

$$\mathrm{SVSL} = \frac{g}{q} e_{\mathrm{d}} \tag{G.4}$$

where e_{d} is the age averaged discounted life expectancy, C_x is a demographical constant for a specific x mortality regime, and G_x is the SWTP for a unitary change in mortality m.

In the evaluation of e_{d}, the age averaging is introduced to account for the effect that different strategies of life safety improvements might affect different age groups differently. Discounting is introduced to account for the effect that the lives saved are saved in the future.

The SWTP corresponds to the amount of money which should be invested into saving one additional life for a given life risk reduction activity. The SVSL corresponds to the amount which should be compensated for each fatality. The SVSL as provided here can be considered a value for guidance. Different legal systems in different nations in practice lead to different compensation practices.

When implementing life risk reductions as a part of the design strategy in the context of individual structures as well as in the context of calibration of design codes, the effect of risk reductions can, as mentioned, be different for different age groups of persons within society. This in turn affects the efficiency of the life risk reduction seen from a societal perspective. To include this in the assessment, the age averaged discounted life expectancy can be calculated to different principal forms of the life risk reduction of a design strategy as measured over the population. These are referred to as mortality reduction schemes.

Two different mortality reduction regimes are considered here for the calculation of both the SWTP and the SVSL, however, others can be formulated and used as found appropriate. The first is the π regime which applies to the case where the design strategy with respect to life risk reduction is associated with a proportional change in mortality over the age distribution. The Δ regime is the case in which the change in mortality associated with the design strategy for life risk reduction is uniformly distributed over all ages. The Δ regime is appropriate for most cases related to structural safety as risk reduction measures typically has the same effect for all individuals independent of their age. Details on how to calculate G_{Π} and G_{Δ} on the basis of information available from cohort life tables are provided in Reference [5].

In Table G.2, results corresponding to 2 %, 3 %, and 4 % annual discounting rates are provided for a selection of different nations (see also Reference [5]). The corresponding values for the SVSL are also provided. In the calculations, it has been assumed that the entire GDP is available for life risk reduction.

Table G.2 — Marginal lifesaving costs (all numbers in thousands ppp $US) for a selection of different nations, different life risk reduction schemes, and different annual discounting rates[5]

Country	2008 GDP per capita	SWTP — G_{Π}			SWTP — G_{Δ}			SVSL		
		2 %	3 %	4 %	2 %	3 %	4 %	2 %	3 %	4 %
Australia	35 624	3061	2614	2279	4840	4298	3843	6551	5261	4356
Brazil	9517	548	470	399	804	712	634	1074	864	724
Canada	36 102	2821	2369	2038	4062	3636	3236	5494	4412	3679
China	5515	252	213	184	353	314	279	482	397	335
Colombia	8125	390	330	285	475	424	380	443	362	304
Dem. Rep. of Congo	290	11	9	8	16	14	12	20	17	14
Denmark	34 005	2334	2007	1704	3842	3431	3064	3127	2549	2113
France	30 595	1969	1677	1459	2935	2601	2307	2233	1820	1523
Germany	33 668	2090	1785	1544	3219	2849	2527	2625	2158	1824

Table G.2 *(continued)*

Country	2008 GDP per capita	SWTP — G_Π			SWTP — G_Δ			SVSL		
		2 %	3 %	4 %	2 %	3 %	4 %	2 %	3 %	4 %
Hong Kong	40 599	1864	1592	1384	2875	2561	2300	2243	1805	1511
India	2721	128	110	96	175	156	139	172	140	118
Japan	31 464	1435	1227	1045	2286	2036	1812	1702	1404	1178
Mali	1043	40	34	29	54	48	43	70	56	47
Mozambique	774	33	28	25	40	36	32	51	42	35
Netherlands	38 048	2329	1989	1700	3812	3385	3016	2967	2406	2016
Norway	49 416	2794	2380	2038	3937	3500	3129	3531	2839	2348
Poland	16 418	1006	846	729	1369	1218	1080	1221	989	819
Singapore	45 553	2114	1799	1554	2735	2448	2191	2771	2267	1893
Sweden	33 769	2249	1891	1630	2710	2406	2137	2561	2113	1743
Switzerland	37 788	2943	2517	2134	4206	3727	3332	3464	2792	2332
United Kingdom	34 204	2600	2178	1873	4105	3665	3270	3127	2505	2117
United States	42 809	2488	2100	1822	3187	2833	2542	4293	3508	2953

Based on the numbers provided in Table G.2, the conformity of decisions concerning structures can be assessed with respect to the societal preferences for investments into life safety in general terms. To this end, full risk assessments are generally necessary as described in Risk Assessment in Engineering — Principles, System Representation & Risk Criteria (JCSS 2008). Examples on how to assess decisions in both the case where decision alternatives can be of discrete and continuous character can be found on http://www.jcss.byg.dtu.dk/. In the following however, it is outlined how the marginal lifesaving costs can be readily applied in a simpler manner which will be sufficient for most common structures.

G.5 How to utilize the principles in practice

In summary, the structural design process can be seen as an optimization of (the expected value of) benefits or minimization of costs subject to constraints. The constraint ensures sufficient and affordable investments into life safety (see Figure G.2).

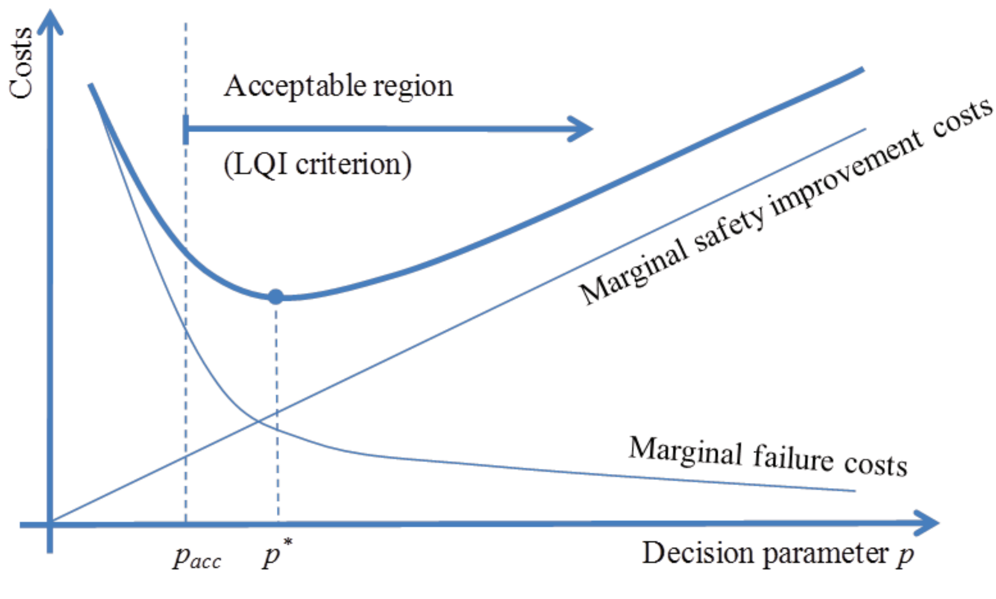


Figure G.2 — LQI acceptance criterion as a boundary condition for monetary optimization

In the foregoing, the emphasis has been on constraints concerning life safety risks, however, other constraints such as risk to qualities of the environment, use of natural resources, emissions, probable maximum losses etc., can be considered in this optimization process as mentioned in Clause 4.

In principle, for structures which do not pose risks to life, the decision basis for design and assessment can be taken from a perspective of pure economic optimization. In practice, however, such situations are very rare if at all existing. All structures impose some life safety risks during some part of their service life.

When the life cycle optimization is performed in economic terms, the corresponding risks should take into account the possible loss of lives through compensation costs, i.e. through the SVSL. When the constraint on necessary investments into life safety is considered, the SWTP should be addressed, i.e. it should be shown that the implemented design strategy implies marginal costs of saving one additional life in the order of magnitude of the SWTP. Marginal lifesaving costs larger than the SWTP are in principle not affordable but can be aimed at if required by monetary optimization. Marginal costs (safety costs in Figure G.2) smaller than the SWTP are not acceptable.

As outlined in G.2, the marginal lifesaving principle can be applied directly within the ALARP framework. For individual design decisions considering specific structures and projects, this step is straightforward. However, it is of crucial importance that the ALARP verification is performed according to a controlled scheme to avoid speculation and inconsistencies.

To avoid speculation from the side of private organizations, it is important to ensure that the costs associated with lifesaving activities which are being assessed are assessed marginally and excluding potential dis-benefits for the involved private organization. If dis-benefits are included into the costs associated with life saving measures, then these should be limited to societal costs. Furthermore, in the assessment of the lifesaving costs, the time reference applied for the calculation of the net present costs should be in the order of 100 years — including the longest surviving persons represented in the cohort life tables. This is required to account for the fact that regulatory requirements such as the present cannot be seen in isolation for one particular project or structure but rather for a portfolio of projects and structures.

Based on the marginal lifesaving costs as provided in Table G.2, it is possible to perform parameter studies to assess the life safety risk which corresponds to design decisions conforming to the marginal lifesaving costs principle. Such parameter studies facilitate the use of the marginal lifesaving costs principle for the identification of life safety related reliability acceptance criteria in design codes. The parameters studies can appropriately be performed such that the costs of improving life safety are varied relative to the fixed costs of design and construction. Moreover, the studies can be performed for different situations concerning the consequences, the uncertainty, and variability, as well as other characteristics associated with the loads and resistances of the considered structures.

G.5.1 Maximum acceptable target failure probabilities

The constrained optimization problem can be addressed idealized in the case the failure event can be represented through a limit state function[7] of the form:

$$g = R - S \tag{G.5}$$

where

R and S are random variables representing the resistance and the load, respectively.

By defining a decision parameter p as the ratio between the expected value of the resistance and the expected value of the load as:

$$p = \frac{E[S]}{E[R]} \tag{G.6}$$

the so-called LQI acceptance criterion can be written as:

$$-\frac{dP_f(p)}{dp} \leq \frac{C_1(\gamma_S + \omega)}{g/q \cdot C_x \cdot N_F} = \frac{C_1(\gamma_S + \omega)}{G_x \cdot N_F} = K_1 \tag{G.7}$$

where

C_1 are the marginal costs associated with a considered safety measure;

γ_S is the interest rate of relevance for decision making on behalf of society (i.e. the moderately selected rate of economic growth);

ω is the annual rate of obsolescence;

N_F is the expected number of fatalities given structural failure.

The other terms are defined as given in G.4.2.

The numerator on the right-hand side of the inequality, $C_1(\gamma_S + \omega)$, indicates how much the yearly safety cost rise with a unit increase in the global safety factor p (γ_S can be understood as financing cost and ω as the cost of rebuilding the structure in case of obsolescence). For the denominator, the human consequences of structural failure N_F have been transformed into monetary units by multiplying with the SWTP to save one additional life. LQI target reliabilities can now be derived as a function of the constant K_1 for given probabilistic models of R and S.

Assuming that both the resistance and the load are log-normal distributed and that the coefficients of variation V for both R and S are in the range 0,1 ″ V ″ 0,3, the following target annual failure probabilities (Table G.3) can be found as a function of the constant K_1 in Formula (G.7).

Table G.3 — Tentative minimum target reliabilities related to one year reference period and ultimate limit states, based on the LQI acceptance criterion

Relative life-saving costs	Range for K_1 constant	LQI target reliability
Large	$10^{-3} - 10^{-2}$	$\beta = 3{,}1\left(P_f \approx 10^{-3}\right)$
Medium	$10^{-4} - 10^{-3}$	$\beta = 3{,}7\left(P_f \approx 10^{-4}\right)$
Small	$10^{-5} - 10^{-4}$	$\beta = 4{,}2\left(P_f \approx 10^{-5}\right)$

The target probability of failure can be increased by a factor 5 for higher coefficients of variation of the basic random variables. For low variability, on the other hand, it should be reduced by a factor 2.

G.5.2 Target reliabilities based on economic optimization

The Probabilistic Model Code issued by the Joint Committee of Structural Safety (JCSS) provides tentative target reliabilities for different structural classes, based on economic optimization (see Table G.4). The target reliabilities are given as a function of the costs of the risk reduction measure and the consequences in case of failure, both defined relative to the initial construction costs of the structure at hand. The target reliabilities relate to structural failure events and can thus be used for failure events ranging from component failures, over partial to full structural collapse by adjustment of the failure consequences. The optimal reliability of structural systems can be related to the safety costs and failure consequences generically[15] as provided in Table G.4.

Table G.4 — Tentative target reliabilities related to one year reference period and ultimate limit states, based on monetary optimization[8]

Relative cost of safety measure	Consequences of failure (classes from Table F1)		
	Class 2	Class 3	Class 4
Large	$\beta = 3{,}1 \left(P_f \approx 10^{-3}\right)$	$\beta = 3{,}3 \left(P_f \approx 5 \times 10^{-4}\right)$	$\beta = 3{,}7 \left(P_f \approx 10^{-4}\right)$
Medium	$\beta = 3{,}7 \left(P_f \approx 10^{-4}\right)$	$\beta = 4{,}2 \left(P_f \approx 10^{-5}\right)$	$\beta = 4{,}4 \left(P_f \approx 5 \times 10^{-6}\right)$
Small	$\beta = 4{,}2 \left(P_f \approx 10^{-5}\right)$	$\beta = 4{,}4 \left(P_f \approx 5 \times 10^{-6}\right)$	$\beta = 4{,}7 \left(P_f \approx 10^{-6}\right)$

The target reliabilities given in Table G.4 should be seen as indicative for the support of economic optimization and might not be acceptable for what concerns life safety risks. This should always be checked individually as explained in the foregoing.

As the relative costs of risk reduction underlying the target reliabilities of Table G.3 and Table G.4 are differently defined, it is shown in the following how for specific structural classes the K_1 constant in Formula (G.7) can be calculated as a function of the ratio between the costs of risk reduction and the construction costs in the same manner as the reliabilities given in Table G.4.

Table G.5 shows K_1 values calculated based on Formula (G.7) for different relative costs of risk-reduction measure (C_1/C_0) and failure consequences (N_F) for the example of two types of structures with different absolute values for the construction costs C_0. Table G.5 (a) refers to a typical Swiss office building with construction costs C_0 = 2'000 CHF/m^2; the expected number of fatalities N_F are estimated per m^2 floor area of the building. Table G.5 (b) gives K_1 values for a structure (e.g. a bridge) with total construction costs of C_0 = 10 Mio.CHF; the number of fatalities N_F here has to be assessed for the whole structure. Further assumptions made during the calculation of the values for both types of structures are a yearly obsolescence rate ω of 2 % and a societal interest rate γ_s of 3 %. The SWTP per life saved, $G_x = \frac{g}{q} C_x$, is set to 5,1 Mio.CHF.

Table G.5 — K_1 values for two types of Swiss structures with different construction costs C_0 as a function of the relative costs of safety measure and the human consequences in case of failure

(a) Office building, C_0 = 2'000 CHF/m^2			
N_F/m^2	C_1/C_0 – relative cost of safety		
	0,001 (small)	0,01 (normal)	0,1 (large)
0,0001	2E-04	2E-03	2E-02
0,001	2E-05	2E-04	2E-03
0,01	2E-06	2E-05	2E-04
0,1	2E-07	2E-06	2E-05
(b) Bridge, C_0 = 10 Mio.CHF			
N_F	C_1/C_0 – relative cost of safety		
	0,001 (small)	0,01 (normal)	0,1 (large)
0,1	1E-03	1E-02	1E-01
1	1E-04	1E-03	1E-02

Table G.5 *(continued)*

10	1E-05	1E-04	1E-03
100	1E-06	1E-05	1E-04

The K_1 values contain information on the safety costs, on the failure consequences, and on the SWTP for life safety. Now based on the K_1 values in Table G.5, the corresponding minimum target reliabilities can be found from Table G.4. It is seen that for the case of the bridge for which the expected value of the number of fatalities N_F is equal to 100 and the relative costs of safety measure is 0,01, the K_1 factor is equal to 10^{-5}. From Table G.3, it is seen that the corresponding minimum annual target reliability index β is equal to 4,2.

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