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EUROPEAN STANDARD
NORME EUROPÉENNE
EUROPÄISCHE NORM

DRAFT
prEN 1990

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Will supersede EN 1990:2002

English Version

Eurocode - Basis of structural and geotechnical design

Eurocode - Bases des calculs structuraux et géotechniques

Eurocode - Grundlagen der Tragwerksplanung

This draft European Standard is submitted to CEN members for enquiry. It has been drawn up by the Technical Committee CEN/TC 250.

If this draft becomes a European Standard, CEN members are bound to comply with the CEN/CENELEC Internal Regulations which stipulate the conditions for giving this European Standard the status of a national standard without any alteration.

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EUROPEAN COMMITTEE FOR STANDARDIZATION
COMITÉ EUROPÉEN DE NORMALISATION
EUROPÄISCHES KOMITEE FÜR NORMUNG

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European foreword

This document (prEN 1990:2020) has been prepared by Technical Committee CEN/TC 250 “Structural Eurocodes”, the secretariat of which is held by BSI. CEN/TC 250 is responsible for all Structural Eurocodes and has been assigned responsibility for structural and geotechnical design matters by CEN.

This document is currently submitted to the CEN Enquiry.

This document will supersede EN 1990:2002 and its amendments and corrigenda.

The first generation of EN Eurocodes was published between 2002 and 2007. This document forms part of the second generation of the Eurocodes, which have been prepared under Mandate M/515 issued to CEN by the European Commission and the European Free Trade Association.

The Eurocodes have been drafted to be used in conjunction with relevant execution, material, product and test standards, and to identify requirements for execution, materials, products and testing that are relied upon by the Eurocodes.

The Eurocodes recognize the responsibility of each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level through the use of National Annexes.

Introduction

0.1 Introduction to the Eurocodes

The Structural Eurocodes comprise the following standards generally consisting of a number of Parts:

- EN 1990 Eurocode Basis of structural and geotechnical design
- EN 1991 Eurocode 1: Actions on structures
- EN 1992 Eurocode 2: Design of concrete structures
- EN 1993 Eurocode 3: Design of steel structures
- EN 1994 Eurocode 4: Design of composite steel and concrete structures
- EN 1995 Eurocode 5: Design of timber structures
- EN 1996 Eurocode 6: Design of masonry structures
- EN 1997 Eurocode 7: Geotechnical design
- EN 1998 Eurocode 8: Design of structures for earthquake resistance
- EN 1999 Eurocode 9: Design of aluminium structures
- <New parts>

NOTE 1 The Structural Eurocodes are referred to as the Eurocodes in this document.

The Eurocodes are intended for use by designers, clients, manufacturers, constructors, relevant authorities (in exercising their duties in accordance with national or international regulations), educators, software developers, and committees drafting standards for related product, testing and execution standards.

NOTE 2 Some aspects of design are most appropriately specified by relevant authorities or, where not specified, can be agreed on a project-specific basis between relevant parties such as designers and clients. The Eurocodes identify such aspects making explicit reference to relevant authorities and relevant parties.

0.2 Introduction to EN 1990

This document gives the principles and requirements for safety, serviceability, and durability of structures that are common to all Eurocodes parts and are to be applied when using them.

This document is addressed to all parties involved in construction activities (e.g. public authorities, clients, designers, contractors, producers, consultants, etc.).

0.3 Verbal forms used in the Eurocodes

The verb “shall" expresses a requirement strictly to be followed and from which no deviation is permitted in order to comply with the Eurocodes.

The verb “should” expresses a highly recommended choice or course of action. Subject to national regulation and/or any relevant contractual provisions, alternative approaches could be used/adopted where technically justified.

The verb “may" expresses a course of action permissible within the limits of the Eurocodes.

The verb “can" expresses possibility and capability; it is used for statements of fact and clarification of concepts.

0.4 National Annex for EN 1990

National choices can be made where explicitly allowed by this standard within notes. Therefore, the National Standard implementing EN 1990 can have a National Annex containing all Nationally Determined Parameters to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

When no national choice is made, the default value given in this standard is to be used.

When no national choice is made and no default value is given in this standard, the choice can be specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

National choice is allowed in EN 1990 through the following clauses:

— In main text through 4.2(3) Note 1, 4.3(1) Note 1 Table 4.1, 6.1.3.2(4) Notes 1 to 4, 6.1.3.2(6) Note, 7.1.5(7) Note, 8.3.3.1(5) Note, 8.3.4.2(2) Notes 1 and 2;

— In A.1 through A.1.2(1) Note 1 Table A.1.1, A.1.3(1) Note Table A.1.2, A.1.5.1(1) Table A.1.3, A.1.5.1(1) Notes 1 and 3, A.1.5.3(1) Note Table A.1.7, A.1.6(1) Note 1 Table A.1.8, Note 2 Table A.1.8 and Note 3 Table A.1.9, A.1.7.2.2(2) Note Table A.1.10, A.1.7.2.3(2) Note Table A.1.11, A.1.7.3(3) Note 1, A.1.7.3(4) Note, A.1.7.4(2) Note Table A.1.16, A.1.7.4(4) Note Table A.1.13, Table A.1.14 and Table A.1.15.

National choice is allowed in the informative annexes through the following clauses:

— In Annex B through B.4(2) Note Table B.1, B.5(1) Note Table B.2, B.6(1) Note, B.6(2) Note 2, B.7(1) Note 2 Table B.3, B.8(1) Note Table B.4;

— In Annex C through C.3.4.2(2) Note 1 Table C.3;

— In Annex D through D.4.1(1) Note;

— In Annex E through E.4(4) Note 1.

National choice is allowed in EN 1990 on the use of the following informative annexes:

— Annex B (informative) Technical management measures for design and execution;

— Annex C (informative) Reliability analysis and code calibration;

— Annex D (informative) Design assisted by testing;

— Annex E (informative) Additional guidance for enhancing the robustness of buildings and bridges;

— Annex F (informative) Rain-flow and reservoir counting methods for the determination of stress ranges due to high cycle fatigue.

The National Annex can contain, directly or by reference, non-contradictory complementary information for ease of implementation, provided it does not alter any provisions of the Eurocodes.

1 Scope

1.1 Scope of EN 1990

(1) This document establishes principles and requirements for the safety, serviceability, robustness and durability of structures, including geotechnical structures, appropriate to the consequences of failure.

(2) This document is intended to be used in conjunction with the other Eurocodes for the design of buildings and civil engineering works, including temporary structures.

(3) This document describes the basis for structural and geotechnical design and verification according to the limit state principle.

(4) Design and verification in this document are based primarily on the partial factor method.

NOTE 1 Alternative methods are given in the other Eurocodes for specific applications.

NOTE 2 The Annexes to this document also provide general guidance concerning the use of alternative methods.

(5) This document is also applicable for:

— structural appraisal of existing construction;

— developing the design of repairs, improvements and alterations;

— assessing changes of use.

(6) This document is applicable for the design of structures where materials or actions outside the scope of EN 1991 to EN 1999 are involved.

NOTE In this case, additional or amended provisions can be necessary.

1.2 Assumptions

(1) It is assumed that reasonable skill and care appropriate to the circumstances is exercised in the design, based on the knowledge and good practice generally available at the time the structure is designed.

(2) It is assumed that the design of the structure is made by appropriately qualified and experienced personnel.

(3) The design rules provided in the Eurocodes assume that:

— execution will be carried out by personnel having appropriate skill and experience;

— adequate control and supervision will be provided during design and execution of the works, whether in factories, plants, or on site;

— construction materials and products will be used as specified in the Eurocodes, in the relevant product and execution standards, and project specifications;

— the structure will be adequately maintained;

— the structure will be used in accordance with the design assumptions.

NOTE Guidance on management measures to satisfy the assumptions for design and execution is given in Annex B.

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

EN 1991 (all parts), *Eurocode 1: Actions on structures*

EN 1992 (all parts), *Eurocode 2: Design of concrete structures*

EN 1993 (all parts), *Eurocode 3: Design of steel structures*

EN 1994 (all parts), *Eurocode 4: Design of composite steel and concrete structure*

EN 1995 (all parts), *Eurocode 5: Design of timber structures*

EN 1996 (all parts), *Eurocode 6: Design of masonry structures*

EN 1997 (all parts), *Eurocode 7: Geotechnical design*

EN 1998 (all parts), *Eurocode 8: Design of structures for earthquake resistance*

EN 1999 (all parts), *Eurocode 9: Design of aluminium structures*

3 Terms, definitions and symbols

3.1 Terms and definitions

For the purposes of this document, the following terms and definitions apply.

NOTE Table 3.1 lists the terms defined hereafter in alphabetical order with reference to the number hereafter where they are defined (different table for each language).

Table 3.1 — Terms in alphabetical order with reference to reference numbers for definition

Term	Reference
accidental action	3.1.3.8
accidental design situation	3.1.2.5
accompanying action	3.1.3.24
action	3.1.3.1
basic variable	3.1.2.25
bounded action	3.1.3.13
characteristic value of a material or product property	3.1.4.1
characteristic value of an action	3.1.3.19
combination of actions	3.1.3.22
combination value of a variable action	3.1.3.25
consequence class	3.1.2.32
construction works	3.1.1.1

Term	Reference
contact non-linearity	3.1.6.8
design case	3.1.2.8
design criteria	3.1.2.1
design situation	3.1.2.2
design value of a geometrical property	3.1.5.2
design value of a material or product property	3.1.4.3
design value of an action	3.1.3.20
design service life	3.1.2.10
direct action	3.1.3.2
durability	3.1.2.30
dynamic action	3.1.3.16
effect of actions	3.1.3.4
excessive deformation	3.1.2.22
execution	3.1.1.7
fatigue action	3.1.3.7
fatigue design situation	3.1.2.7
fire design	3.1.2.9
first order theory	3.1.6.5
fixed action	3.1.3.11
free action	3.1.3.12
frequent value of a variable action	3.1.3 26
geometric non-linearity	3.1.6.4
geotechnical action	3.1.3.10
geotechnical structure	3.1.1.6
gross human error	3.1.2.33
ground	3.1.1.5
hazard	3.1.2.11
indirect action	3.1.3.3
irreversible serviceability limit state	3.1.2.17
leading action	3.1.2.22
limit state	3.1.2.14
linear behaviour	3.1.6.2
load arrangement	3.1.2.12
load case	3.1.2.13

Term	**Reference**
maintenance	3.1.2.26
material non-linearity	3.1.6.7
nominal value	3.1.2.28
nominal value of a geometrical property	3.1.5.1
non-linear behaviour	3.1.6.3
non-linearity of the limit state function	3.1.6.9
permanent action	3.1.3.5
persistent design situation	3.1.2.3
quasi-permanent value of a variable action	3.1.3.27
quasi-static action	3.1.3.17
reference period	3.1.3.21
reliability differentiation	3.1.2.24
repair	3.1.2.27
representative value of a material or product property	3.1.4.2
representative value of an action	3.1.3.18
resistance	3.1.2.20
reversible serviceability limit state	3.1.2.18
robustness	3.1.2.29
second order theory	3.1.6.6
seismic action	3.1.3.9
seismic design situation	3.1.2.6
serviceability criterion	3.1.2.19
serviceability limit state	3.1.2.16
single action	3.1.3.14
static action	3.1.3.15
strength	3.1.2.21
stress history	3.1.6.10
structural analysis	3.1.6.1
structural member	3.1.1.3
structural or geotechnical model	3.1.1.4
structural reliability	3.1.2.23
structure	3.1.1.2
sustainability	3.1.2.31
transient design situation	3.1.2.4

Term	Reference
ultimate limit state	3.1.2.15
variable action	3.1.3.6

3.1.1 Common terms used in the Eurocodes

3.1.1.1
construction works
everything that is constructed or results from construction operations

Note 1 to entry: The term covers both buildings and civil engineering works. It refers to the complete construction works comprising structural members, geotechnical elements and elements other than structural.

3.1.1.2
structure
part of the construction works that provides stability, resistance, and rigidity against various actions

Note 1 to entry: This definition includes structures that comprise one member or a combination of connected members.

3.1.1.3
structural member
physically distinguishable part of a structure, e.g. column, beam, plate, foundation

3.1.1.4
structural or geotechnical model
physical, mathematical, or numerical idealization of the structural or geotechnical system used for the purposes of analysis, design, and verification

3.1.1.5
ground
soil, rock and fill existing in place prior to the execution of construction works

[SOURCE: ISO 6707-1:2017, 3.4.2.1]

3.1.1.6
geotechnical structure
structure that includes ground or a structural member that relies on the ground for resistance

3.1.1.7
execution
all activities carried out for the physical completion of the work including procurement, the inspection and documentation thereof

Note 1 to entry: The term covers work on site; it can also signify the fabrication of parts off site and their subsequent erection on site.

3.1.2 Terms relating to design

3.1.2.1
design criteria
quantitative formulations describing the conditions to be fulfilled for each limit state

3.1.2.2
design situation
physical conditions that could occur during a certain time period for which it is to be demonstrated, with sufficient reliability, that relevant limit states are not exceeded

3.1.2.3
persistent design situation
normal condition of use or exposure of the structure

Note 1 to entry: The duration of a persistent design situation is of the same order as the design service life of the structure.

3.1.2.4
transient design situation
temporary conditions of use or exposure of the structure that are relevant during a period much shorter than the design service life of the structure

Note 1 to entry: A transient design situation refers to temporary conditions of the structure, of use, or exposure, e.g. during construction or repair or under dynamic loads.

3.1.2.5
accidental design situation
design situation in which the structure is subjected to exceptional events or exposure

Note 1 to entry: Caused by events such as fire, explosion, impact or local failure.

3.1.2.6
seismic design situation
design situation in which the structure is subjected to a seismic event

3.1.2.7
fatigue design situation
design situation in which the structure is subjected to repeated load or deformation induced stress cycles

3.1.2.8
design case
set of partial factors applied to actions or effects of actions for verification of a specific limit state

3.1.2.9
fire design
design of a structure to fulfil the required performance in case of fire

3.1.2.10
design service life
assumed period for which a structure or part of it is to be used for its intended purpose with anticipated maintenance but without major repair being necessary

3.1.2.11
hazard
unusual and severe event, e.g. an abnormal action or environmental influence, insufficient strength or stiffness, or excessive detrimental deviation from intended dimensions

3.1.2.12
load arrangement
identification of the position, magnitude, and direction of a free action

3.1.2.13
load case
compatible load arrangements, deformations and geometrical imperfections considered, where relevant, for verification of a specific limit state

3.1.2.14
limit state
state beyond which the structure no longer satisfies the relevant design criteria

3.1.2.15
ultimate limit state
ULS
state associated with collapse or other forms of structural failure

Note 1 to entry: Ultimate limit states generally correspond to the maximum load-carrying resistance of a structure or structural member.

3.1.2.16
serviceability limit state
SLS
state that corresponds to conditions beyond which specified service requirements for a structure or structural member are no longer met

3.1.2.17
irreversible serviceability limit state
serviceability limit state in which the effects of actions remain when the actions are removed

3.1.2.18
reversible serviceability limit state
serviceability limit state in which the effects of actions do not remain when the actions are removed

3.1.2.19
serviceability criterion
design criterion for a serviceability limit state

3.1.2.20
resistance
capacity of a structure, or a part of it, to withstand actions without failure

3.1.2.21
strength
mechanical property of a material indicating its ability to resist actions, usually given in units of stress

3.1.2.22
excessive deformation
deformation that exceeds limits to such an extent that the structure can be considered to have reached an ultimate limit state

3.1.2.23
structural reliability
ability of a structure or a structural member to fulfil the specified requirements during the service life for which it has been designed

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

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

3.1.2.24
reliability differentiation
measures intended for the socio-economic optimisation of the resources to be used to execute construction works, taking into account all the expected consequences of failure and the cost of the construction works

3.1.2.25
basic variable
variable representing a physical quantity that characterizes actions and environmental influences, geometrical quantities, and material properties, including ground properties

3.1.2.26
maintenance
set of activities performed during the service life of the structure so that it fulfils the requirements for reliability

Note 1 to entry: Activities to restore the structure after an accidental or seismic event are normally outside the scope of maintenance.

3.1.2.27
repair
activities, beyond the definition of maintenance, performed to preserve or to restore the function of a structure

3.1.2.28
nominal value
value fixed on a non-statistical basis; for instance, on acquired experience or on physical conditions

3.1.2.29
robustness
ability of a structure to withstand unforeseen adverse events without being damaged to an extent disproportionate to the original cause

3.1.2.30
durability
ability of a structure or structural member to satisfy, with planned maintenance, its design performance requirements over the design service life

3.1.2.31
sustainability
ability to minimize the adverse impact of the construction works on non-renewable resources in the environment, on society, and on economy during their entire life cycle

3.1.2.32
consequence class
categorization of the consequences of structural failure in terms of loss of human lives or personal injury and of economic, social, or environmental losses

3.1.2.33
gross human error
error resulting from ignorance or oversight that causes a fundamental change in the behaviour or a fundamental reduction in reliability of the structure

3.1.3 Terms relating to actions

3.1.3.1
action
F
mechanical influence on a structure, or a structural member, exerted directly or indirectly from its environment

3.1.3.2
direct action
set of forces, or loads, applied to the structure

3.1.3.3
indirect action
set of imposed deformations or accelerations caused for example, by temperature changes, moisture variation, uneven settlement or earthquakes

3.1.3.4
effect of actions
E
action-effect
resulting effect, on a structural member or on the whole structure, from the application of actions

EXAMPLE Internal forces, moments, stresses, strains, deflections, and rotations.

3.1.3.5
permanent action
G
action that is likely to act throughout the design service life and for which any variation in magnitude is either small, compared with the mean value, or monotonic; i.e. it either only increases or decreases, until it reaches a limit value

3.1.3.6
variable action
Q
action that is likely to occur during the design service life for which the variation in magnitude with time is neither negligible nor monotonic

3.1.3.7
fatigue action
Q_{fat}
action inducing repeated stress cycles

3.1.3.8
accidental action
A
action, usually of short duration but of significant magnitude, that is unlikely to occur during the design service life

Note 1 to entry: An accidental action can be expected in many cases to cause severe consequences unless appropriate measures are taken.

3.1.3.9
seismic action
A_{E}
action that arises due to earthquake

3.1.3.10
geotechnical action
action that originates from the self-weight of the ground or groundwater or is transmitted to the structure through the ground or groundwater

3.1.3.11
fixed action
action that has a fixed distribution and position over a structure or structural member such as its magnitude and direction are determined unambiguously for the whole structure or structural member

3.1.3.12
free action
action that can have various spatial distributions over the structure

3.1.3.13
bounded action
action that has a limiting value that cannot be exceeded and which is known to a sufficient accuracy

3.1.3.14
single action
action that can be assumed to be statistically independent in time and space of any other action acting on the structure

3.1.3.15
static action
action that does not cause significant acceleration of the structure or structural members

3.1.3.16
dynamic action
action that causes significant acceleration of the structure or structural members

3.1.3.17
quasi-static action
dynamic action represented by an equivalent static action in a static model

3.1.3.18
representative value of an action
F_{rep}
value of an action used for the verification of a limit state

Note 1 to entry: The representative value can be the characteristic, combination, frequent, or quasi-permanent value (or a nominal value).

3.1.3.19
characteristic value of an action
F_k
value of an action chosen, as far as it can be fixed on a statistical basis, to correspond to a prescribed probability of not being exceeded unfavourably during a specified reference period

3.1.3.20
design value of an action
F_d
value obtained by multiplying the representative value of an action by a partial factor γ_F or determined directly

3.1.3.21
reference period
period of time that is used as a basis for statistically assessing extreme realizations of variable actions and possibly for accidental actions

3.1.3.22
combination of actions
set of design values of actions used for the verification of the structural reliability for a limit state considering the simultaneous influence of different actions

3.1.3.23
leading action
principal action in a combination

3.1.3.24
accompanying action
action that accompanies the leading action in a combination

3.1.3.25
combination value of a variable action
Q_{comb}
value of an accompanying action to be used in the verification of ultimate limit states in persistent or transient design situations and irreversible serviceability limit states, chosen - in so far as it can be fixed on statistical bases - so that the probability that the effects caused by the combination will be exceeded is approximately the same as by the characteristic value of an individual action

Note 1 to entry: Q_{comb} can be expressed as a proportion ψ_0 of the characteristic value (i.e. $Q_{comb} = \psi_0 Q_k$) where $\psi_0 \leq 1$.

3.1.3.26
frequent value of a variable action
Q_{freq}
value used in the verification of ultimate limit states involving accidental actions and in the verification of some reversible serviceability limit states

Note 1 to entry: Q_{freq} can be expressed as a proportion ψ_1 of the characteristic value (i.e. $Q_{freq} = \psi_1 Q_k$), where $\psi_1 \leq 1$

3.1.3.27
quasi-permanent value of a variable action
Q_{qper}
value used in the verification of ultimate limit states involving accidental or seismic actions; in the verification of some reversible serviceability limit states and in the calculation of long-term effects

Note 1 to entry: Q_{qper} can be expressed as a proportion ψ_2 of the characteristic value (i.e. $Q_{\mathrm{qper}} = \psi_2 Q_{\mathrm{k}}$), where $\psi_2 \leq 1$.

3.1.4 Terms relating to material and product properties

3.1.4.1
characteristic value of a material or product property
X_{k}
value of a material or product property having a prescribed probability of not being attained in a hypothetical unlimited test series

Note 1 to entry: This value generally corresponds to a specified fractile of the assumed statistical distribution of the particular property of the material or product. A nominal value is used as the characteristic value in some circumstances.

3.1.4.2
representative value of a material or product property
X_{rep}
value obtained by multiplying the characteristic value of a material or product property by a conversion factor accounting for scale effects, effects of moisture and temperature, effects of ageing of materials, and any other relevant parameters

3.1.4.3
design value of a material or product property
X_{d}
value obtained by either dividing the inferior representative value of a material or product property by a partial material factor or, when it is more critical, by multiplying the superior representative value by a partial material factor

Note 1 to entry: In special circumstances, the value may be obtained by direct determination.

Note 2 to entry: See the other Eurocodes for specific rules.

3.1.5 Terms relating to geometrical property

3.1.5.1
nominal value of a geometrical property
a_{nom}
value of a geometrical property corresponding to the dimensions specified in the design

Note 1 to entry: Where appropriate, nominal values of geometrical properties can be replaced by a prescribed fractile of their statistical distribution.

3.1.5.2
design value of a geometrical property
a_{d}
value of a geometrical property that includes any deviation

Note 1 to entry: Where relevant, it can include possible deviations from nominal value.

3.1.6 Terms relating to structural and geotechnical analysis

3.1.6.1
structural analysis
procedure or algorithm for determination of effects of actions in every point of a structure

Note 1 to entry: Structural analyses are sometimes performed at more than one level using different models (e.g. global, member and local analyses).

3.1.6.2
linear behaviour
behaviour of a structure or a structural member in which the relationship between actions and their effects is directly proportional

Note 1 to entry: The principle of superposition is applicable to a structure which has a linear behaviour.

3.1.6.3
non-linear behaviour
behaviour of a structure or a structural member in which the relationship between actions and their effects is not proportional

3.1.6.4
geometric non-linearity
non-linearity caused by changes in geometry from the initial undeformed state

Note 1 to entry: Examples of geometric non-linearity include membranes, cables, flat arches, catenaries, slender columns and beams.

3.1.6.5
first order theory
relationship between actions and effects when the deformations of a structural member or the entire structure do not have significant influence on the equilibrium equation

3.1.6.6
second order theory
relationship between actions and effects when the deformations have influence on the equilibrium equation

3.1.6.7
material non-linearity
non-linearity caused by a non-linear stress-strain relationship of the material

Note 1 to entry: Examples of material non-linearity include plasticity, cracking in concrete, strain hardening, hysteresis.

3.1.6.8
contact non-linearity
non-linearity caused by changes at the contact boundary between structural parts during introduction of actions

Note 1 to entry: Examples of contact non-linearity include friction interface, interface between concrete floor slab and masonry wall, soil and footing.

3.1.6.9
non-linearity of the limit state function
non-linearity between the resistance and the variables influencing the resistance

Note 1 to entry: This is important for the application of partial factors.

3.1.6.10
stress history
stress variation during time

3.2 Symbols and abbreviations

For the purposes of this document, the following symbols apply.

NOTE The notation used is based on ISO 3898:2013.

3.2.1 Latin upper-case letters

A	Accidental action
A_{d}	Design value of an accidental action
A_{E}	Seismic action
A_{Ed}	Design value of seismic action
$A_{\mathrm{Ed,ULS}}$	Design value of seismic action in an ultimate limit state
$A_{\mathrm{Ed,SLS}}$	Design value of seismic action in a serviceability limit state
$A_{\mathrm{w,rep}}$	Representative value of accidental water action
$C_{\mathrm{d,SLS}}$	Limiting design value of the relevant serviceability criterion
$C_{\mathrm{d,ULS}}$	Limiting design value for ultimate limit state of excessive deformation
CC	Consequence class
E	Effect of actions
$E(.)$	Mean value of (.)
E_{d}	Design value of effect of actions
F	Action
F_{Ed}	Design value of actions used in assessment of E_{d}
F_{d}	Design value of an action
F_{k}	Characteristic value of an action
F_{rep}	Representative value of an action
G	Permanent action
G_{d}	Design value of a permanent action
$G_{\mathrm{d,fav}}$	Design value of a permanent action that produces a favourable effect
G_{k}	Characteristic value of a permanent action

$G_{k,i}$	Characteristic value of a permanent action *i*
$G_{k,sup}$	Upper characteristic value of a permanent action
$G_{k,inf}$	Lower characteristic value of a permanent action
G_{rep}	Representative value of a permanent action
G_{w}	Water action
G_{wk}	Characteristic value of water action
$G_{wk,sup}$	Upper characteristic value of water action
$G_{wk,inf}$	Lower characteristic value of water action
$G_{w,rep}$	Representative value of water action
H	Height of building
H_i	Storey height
L	Span
N_C	Number of cycles to failure
NDP	Nationally Determined Parameter
P	Prestressing force
P_d	Design value of a prestressing force
P_f	Failure probability level
P_k	Characteristic value of a prestressing force
$P_{k,sup}$	Upper characteristic value of a prestressing force
$P_{k,inf}$	Lower characteristic value of a prestressing force
Q	Variable action
Q_{comb}	Combination value of a variable action
Q_d	Design value of a variable action
Q_{fat}	Fatigue action
Q_{freq}	Frequent value of a 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 the accompanying variable action *j*
Q_{qper}	Quasi-permanent value of a variable action
Q_{rep}	Representative value of a variable action
Q_{wk}	Characteristic value of variation in water action
$Q_{w,comb}$	Combination value of variation in water action

$Q_{w,freq}$ Frequent value of variation in water action

$Q_{w,qper}$ Quasi-permanent value of variation in water action

$Q_{w,rep}$ Representative value of variation in water action

R Resistance

R_d Design value of the resistance

SLS Serviceability limit state

T_{life} Design service life

T_{fire} Duration of fire exposure

ULS Ultimate limit state

V Coefficient of variation, V = (standard deviation)/(mean value)

V_X Coefficient of variation of X

V_δ Estimator for the coefficient of variation of the error term δ

X Array of j basic variables X_1 ... X_j

X_d Design value of a material or product property

X_k Characteristic value of a material or product property

$X_k(n)$ Characteristic value, including statistical uncertainty for a sample of size n, with any conversion factor excluded prior to application of any correction factor.

X_m Array of mean values of the basic variables

X_N Array of nominal values of the basic variables

X_{Rd} Value of a material or product property used in the assessment of R_d

X_{rep} Representative value of a material or product property

3.2.2 Latin lower-case letters

a_d Design value of a geometrical property

a_{nom} Nominal value of a geometrical property

b Correction factor

b_i Correction factor for test specimen i

$g_{rt}(\underline{X})$ Theoretical resistance function, of the basic variables $\underline{X}$, used as the design model

k_n Characteristic fractile factor for a sample size n

m_X Mean of the variable X from n sample results

n Number of test results

r Resistance value

r_d Design value of the resistance

r_e Experimental resistance value

r_{ee} Extreme (maximum or minimum) value of the experimental resistance, i.e. value of r_e that deviates most from the mean value r_{em}

r_{ei} Experimental resistance for specimen i

r_{em} Mean value of the experimental resistance

r_k Characteristic value of the resistance

r_m Resistance value calculated using the mean values X_m of the basic variables

r_t Theoretical resistance determined from the resistance function $g_{rt}(\underline{X})$

r_{ti} Theoretical resistance determined using the measured parameters $\underline{X}$ for specimen i

s Estimated value of the standard deviation σ

i_d Design value of a geometrical imperfection

$k_{d,n}$ Design fractile factor for a sample size n

k_F Consequence factor

s_{Cd} Differential settlement

s_Δ Estimated value of σ_Δ

s_δ Estimated value of σ_δ

u Horizontal displacement of a structure or structural member

u_i Relative horizontal displacement over a storey height excluding rigid body rotation

w Vertical deflection of a structural member

w_c Precamber

w_1 Initial part of deflection under permanent loads

w_2 Long-term part of deflection under permanent loads

w_3 Instantaneous deflection due to variable actions

w_{tot} Total deflection

w_{max} Remaining total deflection taking into account precamber

3.2.3 Greek upper-case letters

Φ Cumulative distribution function of the standardised Normal distribution

Δ Logarithm of the error term δ, $\Delta_i = \ln(\delta_i)$

$\overline{\Delta}$ Estimated value for $E(\Delta)$

Δa Deviation in a geometrical property

$\Delta s_{Cd,SLS}$ Maximum differential settlement

$\Delta\sigma_C$ Fatigue resistance corresponding to N_C cycles to failure

$\Delta\sigma_{\mathrm{Cd}}$ Design value of fatigue resistant stress range

$\Delta\sigma_{i,\mathrm{d}}$ Design stress range

$\Delta\sigma_i$ i-th stress range of a stress spectrum

3.2.4 Greek lower-case letters

α_{E} FORM (First Order Reliability Method) sensitivity factor for effects of actions

α_{R} FORM (First Order Reliability Method) sensitivity factor for resistance

β Reliability index

β_{Cd} Angular distortion

$\beta_{\mathrm{Cd,SLS}}$ Maximum angular distortion

γ Partial factor

γ_{E} Partial factor applied to the effects of actions accounting for the uncertainties covered by γ_{f} and γ_{Rd}

$\gamma_{\mathrm{E,fav}}$ Partial factor applied to the favourable effects of actions

γ_{f} Partial factor for actions, which takes account of unfavourable deviations of the action values from the representative values

γ_{F} Partial factor for actions, accounting for the uncertainties covered by γ_{f} and γ_{Rd}

γ_{Ff} Partial factor for fatigue actions

γ_{G} Partial factor for a permanent action that produces unfavourable effects

$\gamma_{\mathrm{G,fav}}$ Partial factor for a permanent action that produces favourable effects

$\gamma_{\mathrm{G},i}$ Partial factor for permanent action i

$\gamma_{\mathrm{G,stb}}$ Partial factor for the favourable (stabilizing) part of a permanent action treated as a single-source

γ_{Gw} Partial factor for water actions

$\gamma_{\mathrm{Gw,stb}}$ Partial factor for the favourable (stabilizing) part of water actions

γ_{m} Partial factor for a material property accounting for unfavourable deviation of the material or product properties from their characteristic values, the random part of the conversion factor η and geometric deviations, if these are not modelled explicitly

γ_{M} Partial factor for a material property, accounting for the uncertainties covered by γ_{m} and γ_{Rd}

$\gamma_{\mathrm{M}}{}^{*}$ Corrected partial factor for resistances $\gamma_{\mathrm{M}}{}^{*} = \mathrm{r}_n/\mathrm{r}_{\mathrm{d}}$ so $\gamma_{\mathrm{M}}{}^{*} = k_{\mathrm{c}}\gamma_{\mathrm{M}}$

γ_{Mf} Partial factor for fatigue resistance

γ_{P} Partial factor for prestressing forces

γ_{Q} Partial factor for variable actions, accounting for the uncertainties covered by γ_{F}

$\gamma_{\mathrm{Q,fav}}$ Partial factor applied to variable actions that produce favourable effects

$\gamma_{\mathrm{Q},j}$	Partial factor for variable action j
γ_{Qw}	Partial factor for variation in water actions
γ_{R}	Partial factor accounting for unfavourable deviation of the material or product properties from their characteristic values, the random part of the conversion factor η, geometric deviations (if these are not modelled explicitly) and uncertainty in the resistance model
γ_{Rd}	Partial factor associated with the uncertainty of the resistance model and geometric deviations, if these are not modelled explicitly
γ_{Sd}	Partial factor associated with the uncertainty of the action and/or action effect model
δ	Error term
δ_i	Observed error term for test specimen i obtained from a comparison of the experimental resistance r_{ei} and its theoretical resistance $br_{\mathrm{t}i}$ corrected using correction factor for corresponding mean values (b_{m})
η	Conversion factor accounting for scale effects, effects of moisture and temperature, effects of ageing of materials, and any other relevant parameters
η_{d}	Design value of the possible conversion factor, so far as is not included in partial factor for resistance γ_{M}
η_{K}	Reduction factor applicable in the case of prior knowledge
ξ	Reduction factor applied to unfavourable permanent actions
ρ	Reduction factor applied to γ_{G} when deriving $\gamma_{\mathrm{G,stb}}$
σ	Standard deviation, $\sigma = \sqrt{\text{variance}}$
$\sigma_{\Delta}{}^2$	Variance of the term Δ
ψ	Combination factor applied to a characteristic variable action
ψ_0	Combination factor applied to a variable action to determine its combination value
ψ_1	Combination factor applied to a variable action to determine its frequent value
ψ_2	Combination factor applied to a variable action to determine its quasi-permanent value
$\psi_{0,j}$	Combination factor applied to variable action j to determine its combination value
$\psi_{1,j}$	Combination factor applied to variable action j to determine its frequent value
$\psi_{2,j}$	Combination factor applied to variable action j to determine its quasi-permanent value
ω_{Cd}	Tilt
ω_{Cd}	Maximum tilt

4 General rules

4.1 Basic requirements

(1) The assumptions given in this document and the other Eurocodes shall be verified.

(2) A structure shall be designed and executed in such a way that it will, during its design service life, with appropriate degrees of reliability and in an economical way:

— sustain all foreseeable and specified actions and influences that are likely to occur during its execution and use;

— meet the specified serviceability requirements for the structure or a structural member;

— meet the specified durability requirements for the structure of the structural member.

NOTE Design carried out in accordance with the Eurocodes will satisfy these requirements.

(3) In the case of fire, the structural resistance shall be adequate for the required period of time.

NOTE See also EN 1991-1-2 for general provisions related to fire design.

4.2 Structural reliability

(1) The reliability required for structures within the scope of this document shall be achieved by design in accordance with the Eurocodes.

(2) Appropriate measures should be taken to avoid gross human errors and omissions and to limit their effects on the structural reliability.

NOTE 1 This document does not make allowance for gross human errors.

NOTE 2 Guidance on appropriate measures to limit the probability of occurring of gross human errors and omissions is given in Annex B.

(3) The choice of an appropriate level of reliability for the structure should take account of the following:

— possible consequences of failure in terms of risk to life, injury, and potential economic losses, see 4.3;

— the possible cause and mode of attaining a limit state;

— public aversion to failure;

— the expense and procedures necessary to reduce the risk of failure.

NOTE 1 Minimum reliability levels can be set by the National Annex for use in a country. Further guidance is given in Annex C.

NOTE 2 Different levels of reliability are commonly adopted for limit states relating to structural failure, serviceability, and durability.

NOTE 3 Levels of reliability for structural failure and serviceability are achieved by:

— appropriate representation of the basic variables, see Clause 6;

— accuracy of the mechanical models used and interpretation of their results;

— prevention of errors in design and execution of the structure, including gross human errors, see also Annex B;

— adequate inspection and maintenance according to procedures specified in the project documentation.

4.3 Consequences of failure

(1) The consequences of failure of the structure or a structural member shall be classified into one of the five following consequence classes:

— CC4 – highest consequence;

— CC3 – higher consequence;

— CC2 – normal consequence;

— CC1 – lower consequence;

— CC0 – lowest consequence.

NOTE 1 Table 4.1 (NDP) gives the classification of consequence classes with reference to indicative qualification of consequences, unless the National Annex gives different qualifications for use in a country.

Table 4.1 (NDP) — Qualification of consequence classes

Consequence class	Indicative qualification of consequences	
	Loss of human life or personal injury[a]	Economic, social or environmental consequences[a]
CC4 – Highest	Extreme	Huge
CC3 – Higher	High	Very great
CC2 – Normal	Medium	Considerable
CC1 – Lower	Low	Small
CC0 – Lowest	Very low	Insignificant

[a] The consequence class is chosen based on the more severe of these two columns.

NOTE 2 Annex A gives examples of the classification of structures into consequence classes.

NOTE 3 The consequence class is used to determine the value of consequence factor k_{F}, see Annex A.

NOTE 4 The consequence class can be used to determine the management measures to achieve the intended structural reliability, see Annex B for further guidance.

NOTE 5 The consequence class can be used to modify the acceptable failure probability levels P_{f} or target reliability indices β, see Annex C for further guidance.

NOTE 6 The consequence class can be used in the direct assessment of the design values for ULS verifications, see Annex D for further guidance.

NOTE 7 The consequence class can be used to choose design methods for enhancing robustness, see Annex E for further guidance.

(2) Consequence classes CC1 to CC3 may be divided into upper and lower sub-classes in other Eurocodes.

NOTE 1 The provisions in Eurocodes cover design rules for structures classified as CC1 to CC3.

NOTE 2 The provisions in the Eurocodes do not entirely cover design rules needed for structures classified as CC4. For these structures, additional provisions to those given in the Eurocodes can be needed.

(3) For consequence class CC0, either the Eurocodes or alternative provisions may be used.

(4) Elements other than structural may be classified as CC0.

4.4 Robustness

(1) A structure should be designed to have an adequate level of robustness so that, during its design service life it will not be damaged by unforeseen adverse events, such as the failure or collapse of a structural member or part of a structure, to an extent disproportionate to the original cause.

NOTE 1 Progressive collapse is an example of a damage that is disproportionate to the original cause.

NOTE 2 For most structures, design in accordance with the Eurocodes provides an adequate level of robustness without the need for any additional design measures to enhance structural robustness.

(2) Design measures to enhance structural robustness should be applied when specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

NOTE Guidance on additional design measures to enhance structural robustness for buildings and bridges is given in Annex E.

4.5 Design service life

(1) The design service life T_{life} of the structure should be specified.

NOTE Values of T_{life} are given in Annex A for different categories of structures.

(2) The design service life should be used to determine the time-dependent performance of the structure.

NOTE Examples of time-dependent performance include durability, fatigue, and deformation due to consolidation of the ground.

(3) Structures or parts of structures that can be dismantled in order to be re-used should not be classified as temporary structures.

(4) A reduced design service life may be used for the verification of fatigue and durability of replaceable structures and parts, provided that the replacement is explicitly taken into account in the design.

NOTE See 4.6 for the verification of durability and 8.3.5.4 for the verification of fatigue.

4.6 Durability

(1) The structure shall be designed such that any deterioration over its design service life does not impair its intended performance, having due regard to its exposure to the environment and its anticipated level of maintenance.

(2) To achieve adequate durability, the structural design should take into account:

— the structure's intended or foreseeable use;

— any required design criteria;

— expected environmental conditions;

— composition, properties and performance of structural materials and products, both on their own and in combination with other materials;

— properties of the ground;

— the choice of structure, the shape of structural members, and structural detailing;

— the quality of workmanship and level of control on site;

— any protective measures that are implemented;

— any intended maintenance during the structure's design service life.

NOTE The other Eurocodes specify appropriate measures to increase the durability of the structure.

(3) The environmental conditions shall be identified during design so that their impact on durability can be assessed and adequate provisions can be made for protection of the materials used in the structure.

(4) The degree of any deterioration affecting the capacity of a structure or a structural elment may be estimated using calculation, experimental investigation, experience from earlier constructions, or a combination of these methods.

4.7 Sustainability

(1) The structure should be designed to limit its adverse impact on non-renewable environmental resources, on society, and on economy during its entire life cycle, as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

NOTE The adverse impact of a structure on its environment, on society, and on economy can be minimized by appropriate choice of construction process, environmentally compatible building materials, including their manufacture, design solutions, durability, and recyclability.

4.8 Quality management

(1) Appropriate quality management measures should be implemented to provide a structure that corresponds to the design requirements and assumptions.

(2) The following quality management measures should be implemented:

— organizational procedures in design, execution, use, and maintenance;

— controls at the stages of design, detailing, execution, use, and maintenance.

NOTE See Annex B and the other Eurocodes for guidance on appropriate quality management measures.

5 Principles of limit state design

5.1 General

(1) A distinction shall be made between ultimate and serviceability limit states.

(2) Verification of a particular limit state may be omitted if the verification of another limit state demonstrates that the former will not be exceeded.

(3) Limit states shall be verified for all relevant design situations.

(4) Limit states that involve the time-dependent performance of the structure should be verified taking into account its design service life.

NOTE See 4.5(1).

5.2 Design situations

(1) Design situations shall be selected appropriately for the conditions under which the structure has to meet its requirements.

(2) Design situations shall be sufficiently severe and varied so that they encompass all conditions that can reasonably be foreseen to occur during execution and use of the structure.

(3) Design situations should be classified according to Table 5.1.

NOTE Information on specific design situations within each of these classes is given in the other Eurocodes.

Table 5.1 — Classification of design situations

Design situation	Conditions	Examples
Persistent	Normal use and exposure	During everyday use
Transient	Temporary use and exposure during a period much shorter than the design service life of the structure	During execution, repair or temporary environmental influence
Accidental	Exceptional conditions or exposure	During flooding, fire, explosion, or impact; or local failure
Seismic	Exceptional conditions during a seismic event	During an earthquake
Fatigue	Conditions caused by repeated load or deformation induced stress cycles	Owing to traffic loads on a bridge, wind induced vibration of chimneys, or machinery-induced vibration

5.3 Ultimate limit states (ULS)

(1) Limit states that concern the safety of the structure to prevent:

— human losses or injury to people;

— unacceptable economic or environmental losses;

shall be classified as ultimate limit states (ULS).

(2) States prior to structural collapse may be treated as ultimate limit states.

NOTE For example when the structural response is ductile and collapse is difficult to define, it can be convenient to treat a state prior to collapse as the ultimate limit state.

(3) The following ultimate limit states shall be verified, as relevant:

— failure of the structure or the ground, or any part of them including supports and foundations, by rupture, excessive deformation, transformation into a mechanism, or buckling;

— loss of static equilibrium of the structure or any part of it;

— failure of the ground by hydraulic heave, internal erosion, or piping caused by excessive hydraulic gradients;

— failure caused by fatigue;

— failure caused by vibration;

— failure caused by other time-dependent effects.

NOTE 1 Details of ultimate limit states caused by fatigue are given in the other Eurocodes.

NOTE 2 Details of ultimate limit states caused by hydraulic gradients are given in EN 1997.

NOTE 3 Loss of static equilibrium includes uplift by water pressure (buoyancy) or other vertical actions.

(4) When verifying loss of static equilibrium, variations in the magnitude or spatial distribution of permanent actions from a single-source should be considered.

NOTE The term 'single-source' is explained in 6.1.1.

5.4 Serviceability limit states (SLS)

(1) Limit states that concern:

— the functioning of the structure or structural members under normal use;

— the comfort of people;

— the appearance of the construction works;

shall be classified as serviceability limit states (SLS).

NOTE The term 'appearance' here is concerned with criteria such as large deflections or extensive cracking, rather than aesthetics.

(2) A distinction shall be made between reversible and irreversible serviceability limit states.

(3) The verification of serviceability limit states should be based on criteria concerning the following:

— deformations that adversely affect the appearance, the comfort of users, or the functioning of the structure (including the functioning of machines or services);

— deformations that cause damage to finishes or elements other than structural;

— vibrations that cause discomfort to people or limit the functional effectiveness of the structure;

— damage that is likely to adversely affect the appearance, durability, or functioning of the structure.

NOTE 1 Elements 'other than structural' refers to partition walls, false ceilings, etc.

NOTE 2 Additional provisions related to serviceability criteria are given in the other Eurocodes.

(4) Serviceability requirements should be specified individually for each project.

NOTE Serviceability criteria for some serviceability limit states are given in Annex A.

5.5 Structural models and load models

(1) Limit states shall be verified using appropriate structural models and load models.

(2) The structural models and load models that are used to verify limit states shall be based on design values for:

— actions;

— material and product properties;

— geometrical properties.

(3) All relevant design situations shall be identified.

(4) The structure shall be verified for all critical load cases in each relevant design situation.

(5) Design values for the basic variables given in (2) should be obtained using the partial factor method, given in Clause 8.

(6) As an alternative to (5), design based on probabilistic methods may be used when specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

NOTE Further guidance on probabilistic methods is given in Annex C.

6 Basic variables

6.1 Actions and environmental influences

6.1.1 Classification of actions

(1) Actions F shall be classified by their variation in time as follows:

— permanent (G); or

— variable (Q); or

— accidental (A); or

— seismic (A_E).

(2) Climatic actions, such as wind and snow actions, may be classified as either variable or accidental, depending on site location.

NOTE See EN 1991.

(3) Actions may also be classified by their:

— origin, as direct or indirect; or

— spatial variation, as fixed or free; or

— nature and/or the structural response, as static or dynamic.

(4) Actions that, owing to physical reasons, induce effects that are strongly correlated with one another, even when they originate in, or act on, different parts of the structure, or originate from different materials, may be treated as an action arising from a single source.

NOTE 1 This rule is commonly known as the 'single-source principle'.

NOTE 2 The single-source principle typically applies to the self-weight of the structure or the ground and of components made of different materials as well as for water pressures acting on both sides of a structure with flow passing around or underneath.

(5) Climatic actions from wind, snow, temperature or water that act on different parts of a structure may be considered to come from a single-source.

6.1.2 Representative values of actions

6.1.2.1 General

(1) The principal representative value of an action F_{rep} should be its characteristic value F_{k}.

NOTE Representative values are not defined for accidental and seismic actions, see Clause 8 for the definition of design values.

(2) For variable actions (see 6.1.2.3) and actions inducing fatigue (see 6.1.3.3), other representative values may be chosen, depending on the limit state being verified.

(3) The characteristic value of an action shall be chosen according to the methods given in EN 1991, EN 1997 and EN 1998.

NOTE The characteristic value of an action F_{k} can be:

— a mean value; or

— an upper or lower value; or

— a nominal value.

6.1.2.2 Permanent actions

(1) The representative value G_{rep} of a permanent action G shall be taken as its characteristic value G_{k}.

(2) Provided its coefficient of variation is small, a permanent action G should be represented by a single characteristic value G_{k}.

(3) For most structural members, the coefficient of variation of G may be considered small if:

— it is not greater than 5 %, when verifying limit states involving overturning or uplift; or

— it is not greater than 10 %, otherwise.

NOTE See EN 1997 for the assessment of the coefficient of variation of permanent actions from the ground.

(4) If a single characteristic value of G_{k} is used, then its value may be taken as the mean value of G.

(5) If the self-weight of the structure or structural member is represented by a single characteristic value, it may be calculated from the product of the nominal dimensions of the structure or structural member and its nominal density.

NOTE Values of nominal densities for various materials are given in EN 1991-1-1.

(6) If the uncertainty in G is not small, or if the structure is sensitive to variations in its value or spatial distribution, then the permanent action G should be represented by upper and lower characteristic values $G_{\mathrm{k,sup}}$ and $G_{\mathrm{k,inf}}$ respectively.

NOTE Permanent actions are usually assumed to be normally distributed.

(7) For materials other than the ground, the upper (or "superior") characteristic value $G_{k,sup}$ should be selected as the 95 % fractile and the lower (or "inferior") characteristic value $G_{k,inf}$ as the 5 % fractile of the statistical distribution of G.

NOTE 1 For the ground, see EN 1997 for the specification of $G_{k,sup}$ and $G_{k,inf}$.

NOTE 2 See 6.1.3.2 for the specification of permanent water actions.

6.1.2.3 Variable actions

(1) The representative value Q_{rep} of a variable action Q shall be taken as one of the following, depending on the limit state being verified:

— its characteristic value Q_k ; or

— its combination value Q_{comb} ; or

— its frequent value Q_{freq} ; or

— its quasi-permanent value Q_{qper}.

(2) The characteristic value of a variable action Q_k shall correspond to one of the following:

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

— a lower value with an intended probability of being exceeded during a specific reference period; or

— when the statistical distribution of Q is not known, a nominal value.

NOTE 1 Upper and lower values and nominal values are given in the various Parts of EN 1991.

NOTE 2 The characteristic value of a variable climatic action is based upon a 2 % probability that its time-varying part is exceeded during a one-year reference period. This is equivalent to a mean return period of 50 years.

NOTE 3 All coefficients or models, to derive characteristic values of variable actions are chosen so that the annual probability of exceedance of the calculated characteristic value does not exceed the annual probability of exceedance of the time-varying part of the variable action.

(3) The combination, frequent and quasi-permanent values should be determined by multiplying the characteristic values Q_k by combination factors ψ_0, ψ_1 and ψ_2 as given in Formulae (6.1) to (6.3):

$$Q_{comb} = \psi_0 Q_k \tag{6.1}$$

$$Q_{freq} = \psi_1 Q_k \tag{6.2}$$

$$Q_{qper} = \psi_2 Q_k \tag{6.3}$$

NOTE 1 The values of ψ_0, ψ_1, and ψ_2 are given in Annex A.

NOTE 2 The characteristic value is used in the verification of ultimate limit states, see 8.3.

NOTE 3 The combination value is used in the verification of ultimate limit states and irreversible serviceability limit states.

NOTE 4 The frequent value is used in the verification of ultimate limit states involving accidental actions and in the verification of reversible serviceability limit states. For buildings, the frequent value is chosen so that the the time it is exceeded is 0,01 of the reference period. For road traffic loads on bridges, the frequent values are assessed on the basis of a return period of one week.

NOTE 5 The quasi-permanent value is used in the verification of ultimate limit states involving accidental or seismic actions; in the verification of reversible serviceability limit states; and in the calculation of long-term effects. For loads on building floors, the quasi-permanent value is chosen so that the the time it is exceeded is 0,50 of the reference period. It can alternatively be determined as the value averaged over a chosen period of time.

(4) For some specific types of actions, particularly water actions, representative values may be defined directly, without the use of combination factors.

NOTE See 6.1.3.2 for the specification of variable water actions.

6.1.3 Specific types of action

6.1.3.1 Prestressing

(1) Prestressing forces P that are caused by the controlled application of forces or deformations to a structure should be classified as permanent actions.

NOTE Prestressing forces acting on a structure can arise from prestressing tendons, imposed deformations at supports, etc.

(2) A prestressing force P should be represented by its upper or lower characteristic value, $P_{\mathrm{k,sup}}$ or $\mathrm{P}_{\mathrm{k,inf}}$ respectively.

(3) For ultimate limit states, if allowed by the other Eurocodes, a prestressing force P may be represented by a single characteristic value P_{k}.

6.1.3.2 Water actions

(1) Actions that arise from water should be classified as permanent, variable, or accidental.

NOTE This paragraph 6.1.3.2 is not applicable to water actions induced by currents and waves and actions in hydraulic structures with fast-flowing water. For actions induced by currents and waves, see Annex A, A.6 and EN 1991-1-8.

(2) When a water action is classified as permanent, as defined in 6.1.2.2, its representative value $G_{\mathrm{w,rep}}$ should be selected as either:

— a single characteristic value G_{wk} taken as the mean value of G_{w}; or

— the more onerous of its characteristic upper and lower values $G_{\mathrm{wk,sup}}$ or $G_{\mathrm{wk,inf}}$; or

— a nominal value.

NOTE Further information can be found in Annex A, EN 1991 and EN 1997.

(3) When a water action is classified as variable, it should be represented by two components:

— a permanent component $G_{\mathrm{w,rep}}$ taken as the mean of G_{w};

— a variable component $Q_{\mathrm{w,rep}}$ equal to the representative value of the variation in water action.

(4) The representative value $Q_{\mathrm{w,rep}}$ should be selected in line with the definitions of characteristic, frequent and quasi-permanent values in 6.1.2.3(1).

NOTE 1 The value of Q_{wk} is based on an annual probability of exceedance of 2 % (corresponding to a return period of 50 years), unless the National Annex gives a different value for use in a country.

NOTE 2 The value of $Q_{w,comb}$ is based on an annual probability of exceedance of 5 % (corresponding to a return period of 20 years), unless the National Annex gives a different value for use in a country.

NOTE 3 The value of $Q_{w,freq}$ is chosen so that the time it is exceeded is 0,01 of the reference period, unless the National Annex gives a different value for use in a country.

NOTE 4 The value of $Q_{w,qper}$ is chosen so that the time it is exceeded is 0,50 of the reference period, unless the National Annex gives a different value for use in a country.

NOTE 5 For seismic verifications involving water actions, see EN 1998-5.

(5) The values of Q_{wk} and $Q_{w,comb}$ may alternatively be determined taking account of any physical limitations that affect the actions.

NOTE Physical limitation can include, for example, the top of a retaining wall or the presence of drains.

(6) When a water action is classified as accidental, it should be represented by a single value $A_{w,rep}$ equal to the representative value of the water action.

NOTE The value of $A_{w,rep}$ is based on an annual probability of exceedance of 0,1 % (corresponding to a return period of 1000 years), unless the National Annex gives a different value for use in a country.

6.1.3.3 Fatigue actions

(1) Models for fatigue actions shall be defined for the action effect spectrum and the expected number of cycles during the design service life.

NOTE The action effect spectrum depends on the type of structure.

(2) When relevant, the stress variation during time (stress history) should take account of any interaction between the action and the structure.

(3) The fatigue load models in EN 1991 may be used for fatigue verifications.

NOTE The fatigue load models in EN 1991 include the dynamic action effect, either:

— implicitly in the equivalent and frequent fatigue load values; or

— explicitly by applying dynamic enhancement amplification factors to fatigue loads.

(4) For structures outside the field of application of the models given in EN 1991, fatigue actions should be defined from the evaluation of measurements or ad hoc studies devoted to determine the expected action effect spectra.

(5) Models for fatigue actions may be based on the evaluation of structural response to load fluctuations, expressed in terms of frequent or equivalent fatigue load models.

NOTE 1 The frequent fatigue load models are intended to assess unlimited fatigue life. These models are defined only for materials for which a a constant amplitude fatigue limit is given in the relevant Eurocodes

NOTE 2 The equivalent fatigue load models are intended to produce the same fatigue damage induced by the action spectra, considering relevant S-N curves.

(6) Stress ranges due to high-cycle fatigue should be derived by an appropriate cycle counting method.

(7) Stress ranges due to high-cycle fatigue may be derived by either the rain-flow or reservoir counting method.

NOTE 1 Rain-flow and reservoir counting methods are equivalent.

NOTE 2 See Annex F.

(8) When fatigue damage depends on the mean stress of the cycle, the assessment procedure should be consistent with this dependency.

6.1.3.4 Geotechnical actions

(1) Geotechnical actions shall be assessed in accordance with EN 1997-1.

6.1.4 Environmental influences

(1) The environmental influences that could affect the durability of the structure should be considered in the choice of structural materials, their specification, the structural concept, and detailed design.

NOTE The other Eurocodes give relevant measures for considering environmental influences.

6.2 Material and product properties

(1) Properties of materials and products should be represented by characteristic values.

NOTE Materials in this sub-clause include the ground.

(2) Unless otherwise stated in the Eurocodes, when the verification of a limit state is sensitive to the variability of a material property, its characteristic value should be defined as:

— the 5 % fractile value where a low value of material or product property is unfavourable; or

— the 95 % fractile value where a high value of material or product property is unfavourable.

NOTE See EN 1997 for the specification of characteristic values of ground properties.

(3) When the verification of a limit state is insensitive to the variability of a material property, its characteristic value should be defined as the mean value, unless otherwise stated in the other Eurocodes.

(4) Material properties should be determined from standard tests performed under specified conditions.

(5) A conversion factor shall be applied when it is necessary to convert test results into values that represent the behaviour of the material or product in the structure or ground.

NOTE See Annex D and the other Eurocodes for values of the conversion factor, if needed.

(6) When insufficient statistical data is available to establish the characteristic value of a material or product property, the characteristic value may be taken as a nominal value.

(7) When material or product properties are not specified in the Eurocodes, their values should be chosen to achieve a level of structural reliability no less than that specified in the Eurocodes.

NOTE See Annex C for guidance on structural reliability.

6.3 Geometrical properties

(1) Unless the design of the structure is sensitive to deviations of a geometrical property, that property should be represented by its nominal value.

(2) If the design of the structure is sensitive to deviations of a geometrical property, corresponding geometrical imperfections defined in the other Eurocodes should be taken into account.

(3) When there is sufficient data, the characteristic value of a geometrical property may be determined from its statistical distribution and used instead of a nominal value.

(4) Connected parts that are made from different materials shall be physically compatible.

(5) For geotechnical design, geometrical properties that affect the mechanical behaviour of the ground should be considered when determining ground properties, as specified in EN 1997.

NOTE For example, the spacing and orientation of discontinuities are taken into account when selecting the characteristic material properties of rock.

7 Structural analysis and design assisted by testing

7.1 Structural modelling

7.1.1 General

(1) Calculations shall be carried out using appropriate structural models involving relevant variables.

NOTE Such calculations can be used to model potential failure modes, predict ultimate capacity, or model deformations (provided the results can be verified with satisfactory accuracy).

(2) Structural models shall be based on established engineering theory and practice.

7.1.2 Static actions

(1) The modelling of static actions shall be based on an appropriate choice of the force-deformation relationships of the members and their connections and between members and the ground.

(2) Effects of displacements and deformations shall be taken into account in ultimate limit state verifications if they result in a significant increase of the effects of actions.

NOTE Particular methods for dealing with effects of deformations are given in the other Eurocodes.

(3) Indirect actions should be introduced into the analysis as follows:

— in linear elastic analysis, directly or as equivalent forces (using appropriate modular ratios where relevant); or

— in non-linear analysis, directly as imposed deformations.

7.1.3 Dynamic actions

(1) When time-dependent actions cause significant acceleration of the structure, dynamic analysis of the system should be performed.

NOTE 1 Guidance on the need for dynamic analysis is given in EN 1991.

NOTE 2 For seismic actions, see EN 1998.

(2) Dynamic actions may be expressed by time histories or content in the frequency domain.

(3) Where relevant, action effects may be defined by a modal analysis.

(4) For structures that have regular geometry, stiffness, and mass distribution, an explicit modal analysis may be replaced by an analysis with equivalent static actions provided.

NOTE Examples of relevant actions are wind induced vibrations or seismic actions.

(5) When it is appropriate to consider a dynamic action as quasi-static, its dynamic part may be considered either by including it in the quasi-static value or by applying an equivalent dynamic amplification factor to the static action.

NOTE 1 Quasi-static actions are defined in EN 1991.

NOTE 2 For limitations to verifications by the partial factor method, see 8.2.

(6) When dynamic action causes vibrations of magnitude or frequency that could exceed serviceability requirements, a specific serviceability limit state verification should be carried out.

NOTE Guidance for assessing these limits is given in Annex A and the other Eurocodes.

7.1.4 Actions inducing fatigue

(1) The parameters needed for fatigue verification should be consistent with the S-N curve used.

(2) Stress history should be determined in terms of nominal stresses, hot spot stresses or effective notch stresses at locations of stress concentrations.

7.1.5 Fire design

(1) Structural fire design analysis shall be based on design fire scenarios and shall consider models for the temperature evolution within the structure as well as models for the mechanical behaviour of the structure exposed to fire.

NOTE See EN 1991-1-2 for guidance on selecting design fire scenarios.

(2) The behaviour of a structure exposed to fire shall be assessed by taking into account the accompanying actions and either:

— nominal fire exposure; or

— physically-based fire exposure.

(3) The required performance of a structure exposed to fire should be verified by either:

— global analysis; or

— analysis of parts of the structure; or

— member analysis by means of one or more of the following design methods:

  — tabulated design data;

  — simplified design methods;

  — advanced design methods.

(4) As an alternative to design by calculation, fire resistance assessment may be based on the results of fire tests or on fire tests in combination with calculations.

(5) In case of design by calculation, the behaviour of the structure exposed to fire shall be assessed according to relevant clauses in the other Eurocodes concerning thermal and mechanical models for analysis.

(6) The mechanical models of the structure exposed to fire should take account of both temperature-dependant mechanical properties and non-linear behaviour, where relevant.

(7) With physically-based fire exposure, the behaviour of the structure should be assessed for the duration of the fire exposure T_{fire}.

NOTE The value of T_{fire} is the full duration of the fire (including the cooling phase) unless the National Annex or national regulations give a different value for use in a country.

7.2 Analysis

7.2.1 Linear Analysis

(1) Linear analysis may be used when the relation between stresses and strains is linear and deformations do not influence the equilibrium of the structure.

NOTE The law of superposition is valid for linear analysis.

(2) Linear analysis may also be used as a simplification of real behaviour of the structure, as specified in the other Eurocodes.

7.2.2 Non-linear analysis

(1) Non-linear analysis should be used when the behaviour of the structure or members has a significant influence on forces in and deformations of the structure.

NOTE The law of superposition is not valid for non-linear analysis.

(2) Non-linear analysis should take into account the relevant type of non-linearity.

NOTE Non-linearity can occur in loading, material behaviour, and geometry.

(3) Numerical models that describe material properties and their interaction should capture all significant and relevant aspects of mechanical behaviour for the specific problem being considered.

(4) Non-linear numerical models should be validated by tests, to establish whether the numerical model correctly reproduces the necessary physical phenomena.

NOTE Depending on the physical phenomena being investigated, validation tests can include basic material tests, physical reference tests, and mesh sensitivity tests.

(5) A sensitivity study should be carried out when a non-linear limit state function is used or, when no explicit limit state function is given, to determine the most sensitive input parameter and how to apply the partial factors given in the Eurocodes.

7.3 Design assisted by testing

(1) Physical or numerical testing may be used to determine parameters for use in design.

(2) Testing may be used to determine the performance of a structure or structural member as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

NOTE Testing is carried out, for example, in the following circumstances:

— if adequate calculation models are not available;

— to confirm by control checks assumptions made in the design;

— to define S-N curves;

— to determine shape factors for snow load;

— to determine pressure or force coefficients for wind actions;

— if a large number of similar components are to be used.

(3) Design assisted by test results shall achieve the level of reliability required for the relevant design situation, taking into account the statistical uncertainty due to a limited number of test results and the uncertainty of the model.

NOTE Further information about the application of probabilistic methods is given in Annex C. Further information about design assisted by testing and related statistical uncertainty is given in Annex D.

(4) The partial factors (including those for model uncertainties) derived from testing for use in design, should provide a level of reliability that is consistent with that set out in 4.2.

(5) The test procedure that is used to determine S-N curves should specify:

— if nominal or peak stresses are to be measured at locations of stress concentrations;

— the method to be used to evaluate the nominal or peak stresses.

8 Verification by the partial factor method

8.1 General

(1) When using the partial factor method, it shall be verified that no relevant limit state is exceeded in any applicable design situation.

NOTE This Clause provides general requirements, while Annex A provides specific application rules for different types of structures.

(2) Calculation models shall be based on design values of actions, geometrical properties, and material properties or on design values of effects of actions and resistances.

NOTE See 8.3 for ultimate limit state verifications and 8.4 for serviceability limit state verifications.

(3) Design values may be determined directly provided the resulting level of reliability is no less than that required by this document.

NOTE Guidance on the direct determination of design values is given in Annex D and in the other Eurocodes.

8.2 Limitations

(1) The design rules given in Clause 8 should be used for structures subject to static loading.

(2) The design rules given in this document may be used where dynamic effects are represented by quasi-static loading using dynamic amplification factors, as specified in the other Eurocodes.

NOTE Wind and traffic loads are examples of dynamic loads that are commonly represented by quasi-static loading.

(3) Additional rules that are given in other Eurocodes should be used for design situations that require:

— non-linear analysis; or

— explicit consideration of dynamic loading; or

— consideration of fatigue.

8.3 Verification of ultimate limit states (ULS)

8.3.1 General

(1) When checking ultimate limit states, the inequality given by Formula (8.1) shall be verified:

$$E_{\mathrm{d}} \leq R_{\mathrm{d}} \tag{8.1}$$

where

E_{d} is the design value of the effect of actions, defined in 8.3.2;

R_{d} is the design value of the corresponding resistance, defined in 8.3.5.

NOTE Verification of ULS can be carried out both at member level and at system level.

(2) When checking ultimate limit states caused by excessive deformation, the inequality given by Formula (8.2) shall be verified:

$$E_{\mathrm{d}} \leq C_{\mathrm{d,ULS}} \tag{8.2}$$

where, in addition to the symbols defined for Formula (8.1)

$C_{\mathrm{d,ULS}}$ is the limiting design value for ultimate limit state of the excessive deformation that is considered to cause an ultimate limit state.

NOTE 1 In Formula (8.2), E_{d} is a displacement or strain, rather than a force or stress.

NOTE 2 In ductile materials, in particular, an ultimate limit state of excessive deformation can occur before rupture of the material.

NOTE 3 See the other Eurocodes for guidance on the selection of $C_{\mathrm{d,ULS}}$.

8.3.2 Design values of the effects of actions

8.3.2.1 General

(1) The design value of the effect of actions E_{d} for a specific combination of actions should be calculated from Formula (8.3):

$$E_{\mathrm{d}} = \gamma_{\mathrm{Sd}} E\left\{\Sigma(\gamma_{\mathrm{f}} \psi F_{\mathrm{k}}); a_{\mathrm{d}}; X_{\mathrm{Rd}}\right\} \tag{8.3}$$

where

γ_{Sd} is a partial factor associated with the uncertainties of the actions and/or in modelling the effects of actions;

$E\{\ldots\}$ denotes the combined effect of the enclosed variables;

$\Sigma(\ldots)$ denotes the combination of actions;

γ_{f} is a partial factor that takes account of unfavourable deviation of an action from its representative value;

ψ is a combination factor either equal to 1,0 for permanent actions or as defined in 6.1.2.3 for variable actions;

F_k is the characteristic value of an action;

a_d denotes design values of geometrical properties, defined in 8.3.7;

X_{Rd} denotes the values of material properties used in the assessment of R_d, see 8.3.6.

NOTE 1 The term X_{Rd} appears in Formula (8.3) because, in general, effects of actions depend on material properties; for example, in design situations involving earth pressure.

NOTE 2 X_{Rd} can be a design value (X_d) or a representative value (X_{rep}) depending on the method used to determine R_d.

(2) For simplicity, the partial factors γ_f and γ_{Sd} given in Formula (8.3) may be combined and then applied as a single partial factor on actions ($\gamma_F = \gamma_f \times \gamma_{Sd}$) or on effects of actions ($\gamma_E = \gamma_f \times \gamma_{Sd}$).

NOTE Although the formulations of γ_F and γ_E are identical, because of the simplifications made, the values of γ_F and γ_E are not necessarily the same.

(3) Partial factors for actions (γ_F) should be used for the design of:

— linear structural systems;

— non-linear structural systems in which an increase in action causes a disproportionally larger increase in the effects of actions;

— certain types of geotechnical structure, as specified in EN 1997.

NOTE A simplified version of Formula (8.3) with partial factors applied to actions is given in 8.3.2.2.

(4) Partial factors for effects of actions (γ_E) should be used for the design of:

— non-linear structural systems involving a single predominant action in which an increase in action causes a disproportionally smaller increase in its effect;

— certain types of geotechnical structure, as specified in EN 1997.

NOTE A simplified version of Formula (8.3) with partial factors applied to effects of actions is given in 8.3.2.3.

8.3.2.2 Factors on actions

(1) When applying factors to actions, the design value of the effect of actions E_d should be calculated from Formula (8.4):

$$E_d = E\left\{\Sigma F_d ; a_d ; X_{Rd}\right\} = E\left\{\Sigma\left(\gamma_F \psi F_k\right) ; a_d ; X_{Rd}\right\} \quad (8.4)$$

where, in addition to the symbols defined for Formula (8.3)

F_d denotes the design values of actions, defined in 8.3.3;

γ_F is defined in 8.3.2.1(2).

NOTE See 8.3.4 for the details of combinations of actions when applying factors to actions.

8.3.2.3 Factors on effects of actions

(1) When applying factors to the effects of actions, their design value E_{d} should be calculated from Formula (8.5):

$$E_{\mathrm{d}} = \gamma_{\mathrm{E}} E\left\{\Sigma F_{\mathrm{rep}}; a_{\mathrm{d}}; X_{\mathrm{rep}}\right\} = \gamma_{\mathrm{E}} E\left\{\Sigma\left(\psi F_{\mathrm{k}}\right); a_{\mathrm{d}}; X_{\mathrm{rep}}\right\} \quad (8.5)$$

where, in addition to the symbols defined for Formula (8.3)

F_{rep} denotes the representative values of actions, defined in 6.1.2;

X_{rep} denotes representative values of material properties, defined in 8.3.5.3;

γ_{E} is defined in 8.3.2.1(2).

(2) γ_{E} may be taken as the highest of the applicable factors on actions (γ_{F}).

NOTE Further guidance can be found in other Eurocodes.

8.3.2.4 High-cycle fatigue

(1) Fatigue actions should be represented according to 6.1.3.3.

(2) The fatigue actions should be supplemented by additional specifications concerning fatigue strength curves, cycle counting methods and damage calculation formulae to be used in the assessment.

NOTE 1 The additional specifications are normally needed to specify how fatigue actions have been determined.

NOTE 2 For the assessment of low-cycle fatigue and for consideration of material specific effects, see the relevant Eurocodes.

(3) Spectra of effects of actions and number of cycles should be derived from stress histories in accordance with 6.1.3.3(2), considering the relevant influence surfaces and fatigue actions at locations that give the largest design value of the fatigue damage.

8.3.3 Design values of actions

8.3.3.1 Permanent actions

(1) The design value of a permanent action (G_{d}) that produces an unfavourable effect should be calculated from Formula (8.6):

$$G_{\mathrm{d}} = \gamma_{\mathrm{G}} \times G_{\mathrm{k}} \quad (8.6)$$

where

γ_{G} is the partial factor for permanent actions specified in Annex A;

G_{k} is the characteristic value of the permanent action.

NOTE The value of G_{k} in Formula (8.6) can be a mean value or an upper value ($G_{\mathrm{k,sup}}$). See 6.1.2.2 for further guidance.

(2) The design value of a permanent action that produces a favourable effect ($G_{\mathrm{d,fav}}$) should be calculated from Formula (8.7):

$$G_{\mathrm{d,fav}} = \gamma_{\mathrm{G,fav}} \times G_{\mathrm{k}} \tag{8.7}$$

where, in addition to the symbols defined for Formula (8.6)

$\gamma_{\mathrm{G,fav}}$ is a partial factor specified in Annex A.

NOTE The value of G_{k} in Formula (8.7) can be a mean value or a lower value ($G_{\mathrm{k,inf}}$). See 6.1.2.2 for further guidance.

(3) Permanent actions that have both unfavourable and favourable effects may be considered as coming from a single-source, see 6.1.1, provided the design is not sensitive to spatial variation of those permanent actions.

NOTE For example, all actions originating from the self-weight of the structure, or the self-weight of different materials in the ground, are commonly considered to come from a single-source.

(4) Except as specified in (5) below, permanent actions from a single-source may be multiplied by a single partial factor, using Formula (8.6) if the resulting action-effect is unfavourable or Formula (8.7) if the resulting effect is favourable.

NOTE For example, the self-weight of the structure or ground can produce simultaneously both unfavourable and favourable effects. For simplicity and economy of design, the self-weight can be considered as coming from a single source and therefore treated as a single action for design purposes.

(5) When verifying limit states involving overturning or uplift, if a permanent action that arises from a single-source (see 6.1.1) has both favourable and unfavourable effects, the design value of the unfavourable (destabilizing) part should be calculated from Formula (8.6) and the design value of the favourable (stabilizing) part should be calculated from Formula (8.7), replacing $\gamma_{\mathrm{G,fav}}$ with the partial factor $\gamma_{\mathrm{G,stb}}$ given by Formula (8.8):

$$\gamma_{\mathrm{G,stb}} = \gamma_{\mathrm{G}} \times \rho \tag{8.8}$$

where

ρ is a reduction factor.

NOTE The value of ρ is 0,85 unless the National Annex gives a different value for use in a country.

8.3.3.2 Prestressing

(1) The design value of prestressing force (P_{d}) that produces an unfavourable effect should be calculated from Formula (8.9):

$$P_{\mathrm{d}} = \gamma_{\mathrm{P}} \times P_{\mathrm{k}} \tag{8.9}$$

where

γ_{P} is the partial factor for the prestressing force specified in Annex A or in the relevant Eurocodes;

P_{k} is the characteristic value of the prestressing force, see 6.1.3.1.

8.3.3.3 Variable actions

(1) The design value of a variable action (Q_{d}) that has an unfavourable effect should be calculated from Formula (8.10):

$$Q_{\mathrm{d}} = \gamma_{\mathrm{Q}} \times Q_{\mathrm{rep}} \tag{8.10}$$

where

γ_{Q} is a partial factor for variable actions specified in Annex A;

Q_{rep} is the representative value of the variable action defined in 6.1.2.3.

8.3.3.4 Accidental actions

(1) The design value of an accidental action (A_{d}) should be specified directly.

NOTE 1 See EN 1991, and in particular EN 1991-1-7, for the specification of accidental actions.

NOTE 2 See 6.1.3.2 for the specification of accidental water actions.

8.3.3.5 Seismic actions

(1) The design value of a seismic action (A_{Ed}) shall be determined according to EN 1998.

8.3.3.6 Fatigue actions

(1) Design stress spectra should be derived from appropriate time histories considering design stress ranges $\Delta\sigma_{i,\mathrm{d}}$ given by Formula (8.11):

$$\Delta\sigma_{i,\mathrm{d}} = \gamma_{\mathrm{Ff}} \Delta\sigma_i \tag{8.11}$$

where

$\Delta\sigma_i$ is the i-th stress range of the stress spectrum;

γ_{Ff} is a partial factor for fatigue actions.

NOTE The value of γ_{Ff} is 1,0 unless the National Annex gives a different value for use in a country.

8.3.3.7 Partial factors

(1) The values of partial factors for actions should be chosen as follows:

— for general application and for buildings, from A.1;

— for bridges, from A.2[1)];

— for towers, masts and chimneys, from A.3[1)];

— for silos and tanks, from A.4[1)];

— for structures supporting cranes and other machineries, from A.5[1)];

— for marine coastal structures, from A.6[1)].

1) The Annex A subclauses A.2, A.3, A.4, A.5 and A.6 will be published in subsequent amendments.

8.3.4 Combination of actions

8.3.4.1 General

(1) For each critical load case, the design values of the effects of actions E_d shall be determined by combining the values of actions that are considered to occur simultaneously.

(2) Each combination of actions should include, in addition to any permanent actions and accompanying variable actions, either:

— a leading variable action; or

— an accidental action; or

— a seismic action;

according to the specifications given below.

(3) Actions that cannot occur simultaneously should not be considered together in combination.

NOTE Physical reasons, for example, maximum high air temperature occurring simultaneously with snow loads, can prevent some actions from occurring simultaneously.

(4) Imposed deformations should be taken into account when present.

NOTE For further guidance, see 7.1.2(2) and the other Eurocodes.

(5) Combinations of actions for ultimate limit states should be calculated from:

— for fundamental (persistent and transient) design situations, 8.3.4.2;

— for accidental design situations, 8.3.4.3;

— for seismic design situations, 8.3.4.4;

— for fatigue design situations, 8.3.4.5.

NOTE See Annex A for application rules.

8.3.4.2 Combination of actions for fundamental (persistent and transient) design situations

(1) The actions considered for persistent and transient design situations should include:

— the design value of the leading variable action;

— the design combination values of accompanying variable actions.

(2) When applying factors to actions, combinations of actions ΣF_d for persistent and transient design situations should be calculated by one of the following:

— Formulae (8.12); or

— the most adverse combination given by Formulae (8.13a) and (8.13b); or

— the most adverse combination given by Formulae (8.14a) and (8.14b).

$$\sum F_d = \sum_i \gamma_{G,i} G_{k,i} + \gamma_{Q,1} Q_{k,1} + \sum_{j>1} \gamma_{Q,j} \psi_{0,j} Q_{k,j} + \left(\gamma_P P_k\right) \tag{8.12}$$

or

$$\sum F_{\mathrm{d}} = \begin{cases} \sum_i \gamma_{\mathrm{G},i}\, G_{\mathrm{k},i} + \gamma_{\mathrm{Q},1}\psi_{0,1} Q_{\mathrm{k},1} + \sum_{j>1} \gamma_{\mathrm{Q},j}\, \psi_{0,j}\, Q_{\mathrm{k},j} + \left(\gamma_{\mathrm{P}} P_{\mathrm{k}}\right) & (8.13\mathrm{a}) \\ \sum_i \xi_i\, \gamma_{\mathrm{G},i}\, G_{\mathrm{k},i} + \gamma_{\mathrm{Q},1} Q_{\mathrm{k},1} + \sum_{j>1} \gamma_{\mathrm{Q},j}\, \psi_{0,j} Q_{\mathrm{k},j} + \left(\gamma_{\mathrm{P}} P_{\mathrm{k}}\right) & (8.13\mathrm{b}) \end{cases}$$

or

$$\sum F_{\mathrm{d}} = \begin{cases} \sum_i \gamma_{\mathrm{G},i}\, G_{\mathrm{k},i} + \left(\gamma_{\mathrm{P}} P_{\mathrm{k}}\right) & (8.14\mathrm{a}) \\ \sum_i \xi_i\, \gamma_{\mathrm{G},i}\, G_{\mathrm{k},i} + \gamma_{\mathrm{Q},1} Q_{\mathrm{k},1} + \sum_{j>1} \gamma_{\mathrm{Q},j}\, \psi_{0,j} Q_{\mathrm{k},j} + \left(\gamma_{\mathrm{P}} P_{\mathrm{k}}\right) & (8.14\mathrm{b}) \end{cases}$$

where

F_{d} represents the design value of an action;

Σ denotes the combination of the enclosed variables;

$\gamma_{\mathrm{G},i}$ is the partial factor for permanent action *i*;

$G_{\mathrm{k},i}$ is the characteristic value of permanent action *i*;

$\gamma_{\mathrm{Q},1}$ is the partial factor for the leading variable action 1;

$\psi_{0,1}$ is the combination factor for the leading variable action 1 (if applied);

$Q_{\mathrm{k},1}$ is the characteristic value of the leading variable action 1;

$\mathrm{Q}_{\mathrm{k,j}}$ is the characteristic value of an accompanying variable action *j*;

$\psi_{0,\mathrm{j}}$ is the combination factor for the variable action *j;*

$\gamma_{\mathrm{Q,j}}$ is the partial factor for the variable action *j*;

P_{k} is the characteristic value of a prestressing force

γ_{P} is the partial factor for the prestressing forces;

ξ is a reduction factor applied to the unfavourable permanent actions only.

NOTE 1 The formula to be used is Formula (8.12) unless the National Annex gives a different choice for use in a country.

NOTE 2 The value of ξ = 0,85 unless the National Annex gives a different value for use in a country.

NOTE 3 Possible simplifications for non-linear analysis are given in 8.3.2.

8.3.4.3 Combination of actions for accidental design situations

(1) Combinations of actions for accidental design situations should either:

— involve an explicit accidental action A_{d} (including fire); or

— refer to a situation after an accidental event $\mathrm{A}_{\mathrm{d}} = 0$.

(2) The combination of actions for accidental design situations should be calculated by Formula (8.15):

$$\sum F_{\mathrm{d}} = \sum_{i} G_{\mathrm{k},i} + A_{\mathrm{d}} + \left(\psi_{1,1} \text{ or } \psi_{2,1}\right) Q_{\mathrm{k},1} + \sum_{j>1} \psi_{2,j} Q_{\mathrm{k},j} + \left(P_{\mathrm{k}}\right) \tag{8.15}$$

where, in addition to the symbols defined for Formulae (8.12) to (8.14)

A_{d} is the design value of the accidental action, defined in 8.3.3.4;

$\psi_{1,1}$ is the combination factor applied to the leading variable action 1 to determine its quasi-permanent value;

$\psi_{2,1}$ is the combination factor applied to the leading variable action 1 to determine its frequent value;

$\psi_{2,j}$ is the combination factor applied to an accompanying variable action *j* to determine its quasi-permanent value.

NOTE 1 For non-linear analysis, see 7.2.2.

NOTE 2 The choice between $\psi_{1,1}$ or $\psi_{2,1}$ depends on the relevant accidental design situation: impact, fire, or survival after an accidental event or situation. Guidance is given in Annex A and the other Eurocodes.

8.3.4.4 Combination of actions for seismic design situations

(1) The combination of actions for seismic design situations should be calculated by Formula (8.16):

$$\sum F_{\mathrm{d}} = \sum_{i} G_{\mathrm{k},i} + A_{\mathrm{Ed,ULS}} + \sum_{j} \psi_{2,j} Q_{\mathrm{k},j} + \left(P_{\mathrm{k}}\right) \tag{8.16}$$

where, in additional to the symbols defined for Formulae (8.12) to (8.14)

$A_{\mathrm{Ed,ULS}}$ is the design value of the seismic action in an ultimate limit state;

$\psi_{2,j}$ is the combination factor applied to an accompanying variable action *j* to determine its quasi-permanent value.

NOTE This combination of actions covers ultimate limit states defined in EN 1998.

8.3.4.5 Combination of fatigue actions with other actions

(1) In fatigue design situations, values for fatigue actions shall be taken from the pertinent action spectrum and the expected number of cycles during the design service life.

NOTE See 6.1.3.3 for further information about action spectra.

(2) Unless otherwise specified in the relevant Eurocodes, stress history may be evaluated considering only fatigue actions.

(3) When either the absolute value of the load affects the stress range or when the relevant Eurocodes require the influence of mean stress of the cycles on fatigue damage to be considered, the stress history should be calculated by Formula (8.17):

$$\sum F_{\mathrm{d}} = \sum_{i} G_{\mathrm{k},i} + \sum_{j} \psi_{2,j} Q_{\mathrm{k},j} + \left(P_{\mathrm{k}}\right) + Q_{\mathrm{fat}} \tag{8.17}$$

where, in addition to the symbols defined for Formula (8.16)

Q_{fat} is the fatigue action.

NOTE Fatigue loads include, for example, traffic loads, as defined in EN 1991, or other cyclic loads.

8.3.5 Design values of resistance

8.3.5.1 General

(1) The design value of resistance R_{d} for a specific design situation should be calculated from Formula (8.18):

$$R_{\mathrm{d}} = \frac{1}{\gamma_{\mathrm{Rd}}} R\left\{\frac{\eta X_{\mathrm{k}}}{\gamma_{\mathrm{m}}}; a_{\mathrm{d}}; \Sigma F_{\mathrm{Ed}}\right\} \tag{8.18}$$

where

γ_{Rd} is a partial factor associated with the uncertainty of the resistance model, and for geometric deviations, if these are not modelled explicitly;

$R\{...\}$ denotes the output of the resistance model;

η is a conversion factor accounting for scale effects, effects of moisture and temperature, effects of ageing of materials, and any other relevant parameters, see 6.2(5);

X_{k} represents the characteristic values of material or product properties, see 6.2(2);

γ_{m} is a partial factor for a material property accounting for:

— unfavourable deviation of the material or product properties from their characteristic values;

— the random part of the conversion factor η;

a_{d} denotes the design values of geometrical property, defined in 8.3.7;

F_{Ed} denotes design values of actions used in the assessment of E_{d}, see 8.3.2.

NOTE 1 The term F_{Ed} appears in Formula (8.18) because, in some cases, design resistance depends on actions, for example, resistance due to friction.

NOTE 2 F_{Ed} can be a design value (F_{d}) or a representative value (F_{rep}) depending on the method used to determine E_{d}.

(2) For simplicity, the partial factors γ_{m} and γ_{Rd} given in Formula (8.18) may be combined into a single partial factor for material property ($\gamma_{\mathrm{M}} = \gamma_{\mathrm{m}} \times \gamma_{\mathrm{Rd}}$) or into a single partial factor for resistance ($\gamma_{\mathrm{R}} = \gamma_{\mathrm{m}} \times \gamma_{\mathrm{Rd}}$).

NOTE Although the formulations of γ_{M} and γ_{R} are identical, because of the simplifications made, the values of γ_{M} and γ_{R} are not necessarily the same.

(3) Partial factors for material properties γ_{M} should be used for the design of:

— certain types of structure, as specified in the other Eurocodes;

— certain types of geotechnical structure, as specified in EN 1997.

NOTE A simplified version of Formula (8.18) with factors applied to material properties is given in 8.3.5.2.

(4) Partial factors for resistance γ_R should be used for the design of:

— certain types of structure, as specified in the other Eurocodes;

— certain types of geotechnical structure, as specified in EN 1997.

NOTE A simplified version of Formula (8.18) with factors applied to resistance is given in 8.3.5.3.

8.3.5.2 Factors on material properties (the “material factor approach”, MFA)

(1) When applying factors to material properties, the design value of resistance R_d should be calculated from Formula (8.19):

$$R_d = R\left\{X_d ; a_d ; \Sigma F_{Ed}\right\} = R\left\{\frac{\eta X_k}{\gamma_M} ; a_d ; \Sigma F_{Ed}\right\} \tag{8.19}$$

where, further to symbols already defined for Formula (8.18)

X_d denotes the design values of material properties, defined in 8.3.6;

γ_M is defined in 8.3.5.1(2).

(2) The values of the partial factors for material properties γ_M that are used in the verification of ultimate limit states should be taken from the other Eurocodes.

8.3.5.3 Resistance factor approach (the "resistance factor approach", RFA)

(1) When applying partial factors to resistance, its design value R_d should be calculated from Formula (8.20):

$$R_d = \frac{R\left\{X_{rep} ; a_d ; \Sigma F_{Ed}\right\}}{\gamma_R} = \frac{R\left\{\eta X_k ; a_d ; \Sigma F_{Ed}\right\}}{\gamma_R} \tag{8.20}$$

where, further to symbols already defined for Formula (8.18)

X_{rep} denotes the representative values of material properties, defined as ηX_k;

γ_R is defined in 8.3.5.1(4).

(2) For structures or structural members that are analysed by non-linear methods and comprise more than one material, the design resistance R_d may be calculated from Formula (8.21):

$$R_d = \frac{1}{\gamma_{R,1}} R\left\{\eta_1 X_{k,1} ; \frac{\eta_i X_{k,i}}{\gamma_{m,i} / \gamma_{m,1}} ; a_d ; F_{Ed}\right\} \tag{8.21}$$

where, further to symbols already defined for Formula (8.18)

1 denotes factors applied to material 1;

i denotes factors applied to material i.

NOTE In some cases, the design resistance can be expressed by applying partial factors directly to individual resistances. See the other Eurocodes for further guidance.

(3) The values of the partial factors on resistance γ_R that are used in the verification of ultimate limit states should be taken from the other Eurocodes.

(4) Alternatively, in geotechnical design, the design value of resistance may be determined using prescriptive measures, as specified in EN 1997.

8.3.5.4 Fatigue

(1) Using S-N curves given in the relevant Eurocodes, the design value of fatigue resistant stress range $\Delta\sigma_{Cd}$ should be calculated from Formula (8.22):

$$\Delta\sigma_{C,d} = \frac{\Delta\sigma_C}{\gamma_{Mf}} \tag{8.22}$$

where

$\Delta\sigma_C$ is the fatigue resistance corresponding to N_C cycles to failure;

γ_{Mf} is the partial factor for fatigue resistance.

NOTE 1 Values for γ_{Mf} are given in the relevant Eurocodes.

NOTE 2 The partial factor for fatigue resistance accounts for the consequence of fatigue failure and the ease of inspection and repair of fatigue-sensitive members.

8.3.6 Design values of material properties

(1) The design value of a material property X_d should be calculated from Formula (8.23):

$$X_d = \frac{X_{rep}}{\gamma_M} = \frac{\eta X_k}{\gamma_M} \tag{8.23}$$

where, in addition to the symbols defined for Formulae (8.19) and (8.20)

X_{rep} is the representative value of material or product property.

NOTE Values for η and γ_M are given in the other Eurocodes.

(2) Provided that the level of reliability is no less than that implied by the use of Formula (8.23), the design value of a material property may be determined directly from:

— empirical or theoretical relations with measured physical properties;

— physical and chemical composition;

— from previous experience;

— in geotechnical design, prescriptive measures;

— in geotechnical design, the most unfavourable value that the parameter could practically adopt;

— values given in European Standards or other documents that are specified in the other Eurocodes;

— reliability analysis, see Annex C; or

— results of tests, see Annex D.

NOTE Guidance on the assessment of design values of ground properties is given in EN 1997. Permission to use specific prescriptive measures is given in EN 1997.

(3) When material or product properties are established directly, the more adverse of the upper and lower design values should be used in the verification of the limit state.

8.3.7 Design values of geometrical properties

(1) When the design of the structure is sensitive to deviations in a geometrical property the design value of that parameter a_{d} should be calculated from Formula (8.24):

$$a_{\mathrm{d}} = a_{\mathrm{nom}} \pm \Delta a \tag{8.24}$$

where

a_{nom} is the nominal value of the geometrical property;

Δa is the deviation in the geometrical property that takes account of:

- unfavourable deviations from the nominal value;
- the cumulative effect of a simultaneous occurrence of several geometrical deviations.

NOTE 1 Examples of deviations in geometrical properties include inaccuracy in the positioning of loads, location of supports, and dimensions of structural members.

NOTE 2 Effects of deviations in geometrical properties can be important when second order effects are significant.

NOTE 3 The deviation of a geometrical property that is within tolerance is assumed to be catered for by the partial factors γ_{F}, γ_{M}, γ_{E}, and γ_{R}.

NOTE 4 Tolerances are defined in the other Eurocodes or in the execution standards they refer to.

(2) When the design of the structure is not significantly sensitive to deviations in a geometrical property, the design value of parameter ad may be calculated from Formula (8.25):

$$a_d = a_{nom} \tag{8.25}$$

(3) The design value of a geometrical imperfection i_{d} may be calculated from Formula (8.26):

$$i_{\mathrm{d}} = \Delta a \tag{8.26}$$

NOTE The value of Δa can be given in the other Eurocodes.

8.4 Verification of serviceability limit states (SLS)

8.4.1 General

(1) When checking serviceability limit states, the inequality given by Formula (8.27) shall be verified:

$$E_{\mathrm{d}} \le C_{\mathrm{d,SLS}} \tag{8.27}$$

where

E_{d} is the design value of the effects of actions specified in the serviceability criterion, determined on the basis of the relevant combination;

$C_{\mathrm{d,SLS}}$ is the limiting design value of the relevant serviceability criterion.

8.4.2 Design values of the effects of actions

(1) The design value of the effects of actions E_{d} for a specific combination of actions should be calculated by Formula (8.28):

$$E_{\mathrm{d}} = E\left\{F_{\mathrm{d}}; a_{\mathrm{d}}; X_{\mathrm{d}}\right\} \tag{8.28}$$

where

$E\{...\}$ denotes the combined effect of the enclosed variables;

F_{d} represents the design values of actions, see 8.3.2.1, where, the value of γ_{F} = 1,0;

a_{d} represents the design values of geometrical properties, see 8.3.7;

X_{d} represents the design values of material properties, see 8.3.6.

NOTE 1 The term X_{d} appears in Formula (8.28) because, in general, effects of actions depend on material properties, e.g. stiffness.

NOTE 2 The term a_{d} appears in Formula (8.28) because effects of actions typically depend on the dimensions of the structure.

8.4.3 Combinations of actions

8.4.3.1 General

(1) The combinations of actions to be taken into account in the relevant design situations should be appropriate for the serviceability requirements and performance criteria being verified.

(2) For each critical load case, the design values of the effects of actions E_{d} shall be determined by combining the values of actions that are considered to occur simultaneously.

(3) Combinations of actions ΣF_{d} for serviceability limit states should be calculated from:

— for the characteristic combinations, 8.4.3.2;

— for the frequent combinations, 8.4.3.3;

— for quasi-permanent combinations, 8.4.3.4.

NOTE See Annex A for application rules.

(4) Each combination of actions should include a leading variable action and any accompanying variable actions.

(5) Imposed deformations should be taken into account where relevant.

NOTE For further guidance, see 7.1.2(3) and the other Eurocodes.

8.4.3.2 Characteristic combination of actions

(1) For the characteristic combination, Formula (8.29) should be used:

$$\sum F_{\mathrm{d}} = \sum_{i} G_{\mathrm{k},i} + Q_{\mathrm{k},1} + \sum_{j>1} \psi_{0,j} Q_{\mathrm{k},j} + \left(P_{\mathrm{k}}\right) \tag{8.29}$$

where the symbols are as defined for Formulae (8.12) to (8.14).

NOTE Irreversible serviceability limit states are generally assessed using this combination of actions.

8.4.3.3 Frequent combination of actions

(1) For the frequent combination, Formula (8.30) should be used:

$$\sum F_{\mathrm{d}} = \sum_{i} G_{\mathrm{k},i} + \psi_{1,1} Q_{\mathrm{k},1} + \sum_{j>1} \psi_{2,j} Q_{\mathrm{k},j} + \left(P_{\mathrm{k}}\right) \tag{8.30}$$

where, in addition to the symbols defined for Formula (8.29)

$\psi_{1,1}$ is the combination factor applied to the leading variable action 1 to determine its frequent value;

$\psi_{2,j}$ is the combination factor applied to the accompanying variable action j to determine its quasi-permanent value.

NOTE 1 Reversible serviceability limit states are generally assessed using this combination of actions.

NOTE 2 The other Eurocodes can specify when this combination of actions is to be used.

8.4.3.4 Quasi-permanent combination of actions

(1) For the quasi-permanent combination, Formula (8.31) should be used:

$$\sum F_{\mathrm{d}} = \sum_{i} G_{\mathrm{k},i} + \sum_{j} \psi_{2,j} Q_{\mathrm{k},j} + \left(P_{\mathrm{k}}\right) \tag{8.31}$$

where symbols are as defined for Formula (8.30).

NOTE 1 Long-term effects and the appearance of the structure are generally assessed using this combination of actions.

NOTE 2 The other Eurocodes can specify when this combination of actions is to be used.

8.4.3.5 Combination of actions in seismic design situations

(1) For seismic design situations, Formula (8.32) should be used:

$$\sum F_{\mathrm{d}} = \sum_{i} G_{\mathrm{k},i} + A_{\mathrm{Ed,SLS}} + \sum_{j} \psi_{2,j} Q_{\mathrm{k},j} + \left(P_{\mathrm{k}}\right) \tag{8.32}$$

where, in addition to the symbols defined for Formula (8.29)

$A_{\mathrm{Ed,SLS}}$ is the design value of the seismic action in a serviceability limit state, defined in EN 1998.

NOTE Depending on the magnitude of $A_{\mathrm{Ed,SLS}}$, this combination of actions covers both the damage limitation (DL) and fully operational (OP) serviceability limit states defined in EN 1998.

8.4.4 Design criteria

(1) The deformations to be taken into account in relation to serviceability requirements should be either:

— as given in Annex A for different types of construction works; or

— as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

NOTE 1 For other specific serviceability criteria such as crack width, stress or strain limitation, or slip resistance, see the other Eurocodes.

NOTE 2 Serviceability criteria for seismic design are given in EN 1998.

8.4.5 Design values of geometrical properties

(1) Design values of geometrical properties for serviceability limit states should be chosen in accordance with 8.3.7, except as specified below.

(2) The deviation Δa may be taken as zero in the verification of serviceability limit states, unless the other Eurocodes specify differently.

Annex A
(normative)

Application rules

A.1 General application and application for buildings

A.1.1 Field of application

(1) This Annex subclause A.1 should be used for the design of buildings, other structures not covered by subclauses A.2 to A.6 and of geotechnical structures not covered by subclauses A.2 to A.6.

NOTE A.1 provides the specific application of the general rules in Clauses 1 to 8 for these structures.

(2) When a structure falls into the field of application of different parts of Annex A, these parts should be applied in conjunction, as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

A.1.2 Consequence classes

(1) Buildings and geotechnical structures should be classified into consequence classes, according to the consequences of their failure as described in 4.3.

NOTE 1 Examples of buildings in different consequence classes are given in Table A.1.1 (NDP) unless the National Annex gives different examples for use in a country.

NOTE 2 Examples of geotechnical structures in different consequence classes are given in EN 1997-1.

NOTE 3 Examples of other structures in different consequence classes are given in other parts of Annex A.

Table A.1.1 (NDP) — Examples of buildings in different consequence classes

Consequence class	Description of consequence	Examples
CC3	Higher	Buildings where people assemble, e.g. grandstands, concert halls
CC2	Normal	Buildings where people normally enter, e.g. residential and office buildings
CC1	Lower	Buildings where people do not normally enter, e.g. agricultural buildings, storage buildings

A.1.3 Design service life

(1) The design service life T_{life} of a building or geotechnical structure, as described in 4.5, should be specified.

NOTE The value of T_{life} is given in Table A.1.2 (NDP) for different categories of buildings unless the National Annex gives different values or categories for use in a country.

Table A.1.2 (NDP) — Design service life categories for buildings

Category of buildings	**Design service life, T_{life}** years
Monumental building structures	100
Building structures not covered by another category	50
Agricultural, industrial, and similar structures Replaceable structural parts	25
Temporary structures[a, b]	≤ 10

[a] For structures or parts of structures that can be dismantled in order to be re-used, see 4.5(3).

[b] For specific temporary structural members, such as anchors, $T_{life} \leq 2$ years can be considered.

A.1.4 Actions

(1) The actions, as described in Clause 6, to be included in the design of structures shall be those defined by EN 1991, EN 1997, and EN 1998.

A.1.5 Combinations of actions

A.1.5.1 Ultimate limit states (ULS)

(1) Combination of actions for ultimate limit states with factors on actions should be chosen depending on the design situation, according to:

— Table A.1.3, when using Formula (8.12); or

— Table A.1.4, when using Formula (8.13); or

— Table A.1.5, when using Formula (8.14).

NOTE 1 The formula to be used is Formula (8.12) unless the National Annex gives a different choice for use in a country, see 8.3.4.2(2).

NOTE 2 As defined in 8.3.2.2, factors on actions are used, and Formula (8.4) applies, for the design of:

— structural linear systems;

— non-linear structural systems, in which an increase in action causes a disproportionally larger increase in the effects of actions;

— certain types of geotechnical structure, as specified in EN 1997.

NOTE 3 The value of ξ in Tables A.1.4 and A.1.5 is 0,85 unless the National Annex gives a different value for use in a country, see 8.3.4.2(2) Note 2.

NOTE 4 The characteristic value of prestressing P_{k} can be an upper, lower, or mean value, as specified in the other Eurocodes.

(2) If design values of actions for persistent and transient design situations are chosen according to Table A.1.4 or Table A.1.5, then both combinations (a and b) shall be verified.

Table A.1.3 — Combinations of actions for ultimate limit states when using Formula (8.12)

Design situation	**Fundamental (persistent/ transient)**[a]	**Accidental**[b]	**Seismic**[c]	**Fatigue**[d]
General formula for effects of actions	(8.4)			
Formula for combination of actions	(8.12)	(8.15)	(8.16)	(8.17)
Permanent ($G_{\mathrm{d},i}$)	$\gamma_{\mathrm{G},i}G_{\mathrm{k},i}$	$G_{\mathrm{k},i}$	$G_{\mathrm{k},i}$	$G_{\mathrm{k},i}$
Leading variable ($Q_{\mathrm{d},1}$)	$\gamma_{\mathrm{Q},1}Q_{\mathrm{k},1}$	$\psi_{1,1}Q_{\mathrm{k},1}$ or $\psi_{2,1}Q_{\mathrm{k},1}$	$\psi_{2,j}Q_{\mathrm{k},j}$	$\psi_{2,j}Q_{\mathrm{k},j}$
Accompanying variable ($Q_{\mathrm{d},j}$)	$\gamma_{\mathrm{Q},j}\psi_{0,j}Q_{\mathrm{k},j}$	$\psi_{2,j}Q_{\mathrm{k},j}$		
Prestressing (P_{d})	$\gamma_{\mathrm{P}}P_{\mathrm{k}}$	P_{k}	P_{k}	P_{k}
Accidental (A_{d})	-	A_{d}	-	-
Seismic (A_{Ed})	-	-	$A_{\mathrm{Ed,ULS}}$	-
Fatigue (Q_{fat})	-	-	-	Q_{fat}

[a] For persistent and transient design situations, when $\gamma_{\mathrm{Q,j}}\psi_{0,\mathrm{j}} \approx 1$ the design value of the accompanying variable action can be approximated by its characteristic value.

[b] In accidental design situations, the choice between ψ_1 and ψ_2 depends on details of the design situation, e.g. impact, fire, or survival after an accidental event or situation. Further guidance is given in the other Eurocodes and in the National Annex.

[c] Depending on the magnitude of $\mathrm{A_{Ed,ULS}}$, the seismic combination of actions covers both the near collapse (NC) and significant damage (SD) ultimate limit states defined in EN 1998.

[d] See 8.3.4.5 for conditions of use.

Table A.1.4 — Combinations of actions for ultimate limit states when using Formulae (8.13)

Design situation	**Fundamental (persistent/transient)**		**Accidental**	**Seismic**	**Fatigue**
General formula for effects of actions	(8.4)				
Formula for combination of actions	(8.13a)	(8.13b)	use values given in Table A.1.3		
Permanent ($G_{d,i}$)	$\gamma_{G,i}G_{k,i}$	$\xi\gamma_{G,i}G_{k,i}$			
Leading variable ($Q_{d,1}$)	$\gamma_{Q,j}\psi_{0,j}Q_{k,j}$	$\gamma_{Q,1}Q_{k,1}$			
Accompanying variable ($Q_{d,j}$)		$\gamma_{Q,j}\psi_{0,j}Q_{k,j}$			
Prestressing (P_d)	$\gamma_P P_k$	$\gamma_P P_k$			
Accidental (A_d)	-	-			
Seismic (A_{Ed})	-	-			

Table A.1.5 — Combinations of actions for ultimate limit states when using Formula (8.14)

Design situation	**Fundamental (persistent/transient)**		**Accidental**	**Seismic**	**Fatigue**
General formula for effects of actions	(8.4)				
Formula for combination of actions	(8.14a)	(8.14b)	use values given in Table A.1.3		
Permanent ($G_{d,i}$)	$\gamma_{G,i}G_{k,i}$	$\xi\gamma_{G,i}G_{k,i}$			
Leading variable ($Q_{d,1}$)	-	$\gamma_{Q,1}Q_{k,1}$			
Accompanying variable ($Q_{d,j}$)		$\gamma_{Q,j}\psi_{0,j}Q_{k,j}$			
Prestressing (P_d)	$\gamma_P P_k$	$\gamma_P P_k$			
Accidental (A_d)	-	-			
Seismic (A_{Ed})	-	-			

(3) Combination of actions for ultimate limit states with factors on effects of actions should be chosen according to 8.3.2.3.

NOTE As defined in 8.3.2.3, factors on effects actions are used, and Formula (8.5) applies, for the design of:

— non-linear structural systems involving a single predominant action in which an increase in action causes a disproportionally smaller increase in its effect;

— certain types of geotechnical structure, as specified in EN 1997.

A.1.5.2 Serviceability limit states (SLS)

(1) Combinations of actions for serviceability limit states, for which 8.4.2 and the general Formula (8.28) apply, should be chosen according to Table A.1.6, depending on the combinations of actions being considered.

Table A.1.6 — Combinations of actions for serviceability limit states

Combinations	Characteristic	Frequent	Quasi-permanent	Seismic[b]
General formula effects of actions	(8.28)			
Formula for combination of actions	(8.29)	(8.30)	(8.31)	(8.32)
Permanent ($G_{d,i}$)	$G_{k,i}$	$G_{k,i}$	$G_{k,i}$	$G_{k,i}$
Leading variable ($Q_{d,1}$)	$Q_{k,1}$	$\psi_{1,1}Q_{k,1}$	$\psi_{2,j}Q_{k,j}$	$\psi_{2,j}Q_{k,j}$
Accompanying variable ($Q_{d,j}$)	$\psi_{0,j}Q_{k,j}$	$\psi_{2,j}Q_{k,j}$		
Prestressing (P_d)[a]	P_k	P_k	P_k	P_k
Seismic (A_{Ed})	-	-	-	$A_{Ed,SLS}$

[a] The characteristic value of prestressing P_k can be an upper, lower, or mean value. Guidance is given in the other Eurocodes.

[b] Depending on the magnitude of $A_{Ed,SLS}$, the seismic combination of actions covers both the damage limitation (DL) and fully operational (OP) serviceability limit states defined in EN 1998.

A.1.5.3 Combination factors

(1) Combinations of actions may be calculated using the combination factors ψ, as defined in 6.1.2.3(3).

NOTE Values of the combination factors ψ are as given in Table A.1.7 (NDP) unless the National Annex gives different values for use in a country.

Table A.1.7 (NDP) — Combination factors for buildings

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings (see EN 1991-1-1): Category A: domestic, residential areas Category B: office areas Category C: congregation areas Category D: shopping areas Category E: storage areas	0,7 0,7 0,7 0,7 1,0	0,5 0,5 0,7 0,7 0,9	0,3 0,3 0,6 0,6 0,8
Category F: traffic area, vehicle weight ≤ 30 kN Category G: traffic area, 30 kN < vehicle weight ≤ 160 kN Category H: roofs accessible for normal maintenance and repair only (see EN 1991-1-1)	0,7 0,7 0,7	0,7 0,5 0	0,6 0,3 0
Construction loads (see EN 1991-1-6)	0,6 to <u>1,0</u>	--	0,2
Snow loads on buildings (see EN 1991-1-3)[a] — Finland, Iceland, Norway, Sweden — Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l. — Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,7 0,7 0,5	0,5 0,5 0,2	0,2 0,2 0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
Icing (see EN 1991-1-9)	0,5	0,2	0
Standing water (see the other Eurocodes)	-	-	-
Waves and currents (see EN 1991-1-8)			
NOTE Where ranges are given, the recommended value is underlined.			
[a] For countries not mentioned, see the National Annex or relevant local guidance.			

A.1.6 Partial factors for ultimate limit states (ULS)

(1) Ultimate limit states should be verified using partial factors γ_F applied to actions or γ_E applied to effects of actions, as defined in 8.3.

NOTE 1 Values of the partial factor γ_F are given in Table A.1.8 (NDP) for persistent and transient design situations unless the National Annex gives different values for use in a country.

NOTE 2 Values of the partial factors γ_E are given in Table A.1.8 (NDP) for persistent and transient design situations for relevant geotechnical design cases, unless the National Annex gives different values for use in a country.

NOTE 3 Values of consequence factor k_F for different consequence classes in Table A.1.8 (NDP) are given in Table A.1.9 (NDP), unless the National Annex gives different values for use in a country.

NOTE 4 For fatigue, see 8.3.3.6.

(2) The value of the partial factors γ_F when applied to unfavourable actions or actions effects shall not be less than 1,0.

(3) When using Formulae (8.13b) and (8.14b), the value of $\xi\gamma_G$ shall not be less than 1,0.

(4) Ultimate limit states that involve structural resistance should be verified using partial factors for Design Case 1.

(5) When variations in the magnitude or spatial variation of permanent actions from the same source are significant, ultimate limit states that involve loss of static equilibrium should be verified using partial factors for Design Cases 2(a) and 2(b), using the whichever gives the less favourable design outcome.

(6) Verification of Design Case 2(b) may be omitted when it is obvious that verification using Design Case 2(a) governs the design outcome.

(7) Ultimate limit states that involve failure of ground should be verified using partial factors for Design Cases 1, 2, 3 and 4, as specified in EN 1997.

NOTE EN 1997 gives guidance on which Design Cases to use for different geotechnical structures.

Table A.1.8 (NDP) — Partial factors on actions and effects for fundamental (persistent and transient) design situations

Action or effect				Partial factors γ_F and γ_E for Design Cases 1 to 4				
Type	Group	Symbol	Resulting effect	Structural resistance	Static equilibrium and uplift		Geotechnical design	
Design case				DC1[a]	DC2(a)[b]	DC2(b)[b]	DC3[c]	DC4[d]
Formula				(8.4)	(8.4)		(8.4)	(8.5)
Permanent action (G_k)	All[f]	γ_G	unfavourable /destabilizing	$1{,}35 k_F$	$1{,}35 k_F$	1,0	1,0	G_k is not factored
	Water	γ_{Gw}		$1{,}2 k_F$	$1{,}2 k_F$	1,0	1,0	
	All[f]	$\gamma_{G,stb}$	stabilizing[g]	not used	1,15 [e]	1,0	not used	
	Water[l]	$\gamma_{Gw,stb}$			1,0 [e]	1,0		
	All	$\gamma_{G,fav}$	favourable[h]	1,0	1,0	1,0	1,0	
Prestress-ing (P_k)		γ_P [k]						
Variable action (Q_k)	All[f]	γ_Q	unfavourable	$1{,}5 k_F$	$1{,}5 k_F$	$1{,}5 k_F$	1,3	$\dfrac{\gamma_{Q,1}}{\gamma_{G,1}}$ [j]
	Water[l]	γ_{Qw}		$1{,}35 k_F$	$1{,}35 k_F$	$1{,}35 k_F$	1,15	1,0
	All	$\gamma_{Q,fav}$	favourable	0				
Effects of actions (E)		γ_E	unfavourable	effects are not factored				$1{,}35 k_F$
		$\gamma_{E,fav}$	favourable					1,0

[a] Design Case 1 (DC1) is used both for structural and geotechnical design.

[b] Design Case 2 (DC2) is used for the combined verification of strength and static equilibrium, when the structure is sensitive to variations in permanent action arising from a single-source. Values of γ_F are taken from columns (a) or (b), whichever gives the less favourable outcome.

[c] Design Case 3 (DC3) is typically used for the design of slopes and embankments, spread foundations, and gravity retaining structures. See EN 1997 for details.

[d] Design Case 4 (DC4) is typically used for the design of transversally loaded piles and embedded retaining walls and (in some countries) gravity retaining structures. See EN 1997 for details.

[e] The values of $\gamma_{G,stb}$ = 1,15 and 1,0 are based on $\gamma_{G,inf}$ = 1,35 ρ and 1,2 ρ with ρ = 0,85.

[f] Applied to all actions except water pressures.

[g] Applied to the stabilizing part of an action originating from a single source.

[h] Applied to actions whose entire effect is favourable and independent of the unfavourable action.

[j] $\gamma_{Q,1}$ = corresponding value of γ_Q from DC1 and $\gamma_{G,1}$ = corresponding value of γ_G from DC1.

[k] See other relevant Eurocodes for the definition of γ_P where γ_P is materially dependent.

[l] For water actions induced by waves and currents, see subclause A.6.

Table A.1.9 (NDP) — Consequence factors for buildings

Consequence class (CC)	Description of consequences	Consequence factor k_F
CC3	Higher	1,1
CC2	Normal	1,0
CC1	Lower	0,9

A.1.7 Serviceability criteria

A.1.7.1 General

(1) Serviceability criteria should be specified for each building project in accordance with 5.4.

NOTE 1 Serviceability criteria for buildings can include, for example, floor deflection and stiffness; differential settlements; storey sway or/and building sway; roof deflection and stiffness; vibration frequency and amplitude/acceleration; and concrete crack width.

NOTE 2 Design values of serviceability criteria for non-industrial buildings, expressed independently of structural materials, are defined in A.1.7.2 for deformations.

NOTE 3 Design values of serviceability criteria for geotechnical structures are given in A.1.7.4.

(2) Depending on specific characteristics of the structural system and its material, other limiting values may be specified and agreed by the relevant parties involved in the design.

A.1.7.2 Vertical and horizontal deformations

A.1.7.2.1 General

(1) Vertical and horizontal deformations should be calculated, when necessary, using appropriate combinations of actions, as specified in Table A.1.6, accounting for the serviceability requirements given in 5.4(1).

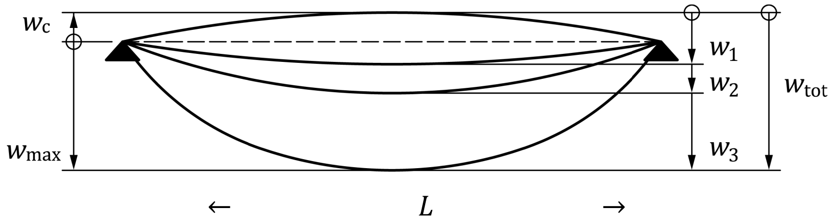
NOTE Guidance on the calculation of deformations is given in the other Eurocodes.

(2) The deformations obtained using a combination of actions do not include the effects of execution tolerances and these should be considered additionally, if significant.

(3) The distinction between reversible and irreversible limit states should be considered.

A.1.7.2.2 Vertical deflections

(1) Vertical deflections should be calculated using the parameters shown in Figure A.1.1.



Key

w_c precamber in the unloaded structural member

w_1 initial part of the deflection under permanent loads of the relevant combination of actions according to Formulae (8.29) to (8.32)

w_2 long-term part of the deflection under permanent loads including quasi-permanent loads

w_3 instantaneous deflection due to variable actions excluding the quasi-permanent loads.

w_{tot} total deflection as the sum of w_1, w_2, w_3

w_{max} remaining total deflection taking into account the precamber

L span

Figure A.1.1 — Vertical deflections

NOTE Numerical values of w_1, w_2 and w_3 are to be calculated taking into account the material behaviour of the structural member (e.g. cracking and creep in concrete).

(2) Maximum values of vertical deflections may be specified by the relevant authorities or, where not specified, agreed for a specific project by relevant parties.

NOTE Suggested values of maximum permitted vertical deflections are given in Table A.1.10 (NDP) unless the National Annex gives different values.

(3) Where the functioning of, or potential damage to, the structure or to elements other than structural is being considered, the verification of deflection should take account of effects from permanent and variable actions that occur after execution.

NOTE Elements other than structural, that this can apply to, include partition walls, claddings, and finishes.

(4) Long term deformations due to shrinkage, relaxation or creep should be considered, where relevant, and calculated by using the effects of the permanent actions and quasi-permanent values of the variable actions.

(5) The limiting values of vertical deflection w_{tot} specified in this clause should only be applied to structures and structural members. If partition walls prone to cracking are used, appropriate detailing should be adopted or more severe limiting design values of deflection defined.

Table A.1.10 (NDP) — Suggested maximum vertical deflections for non-industrial buildings

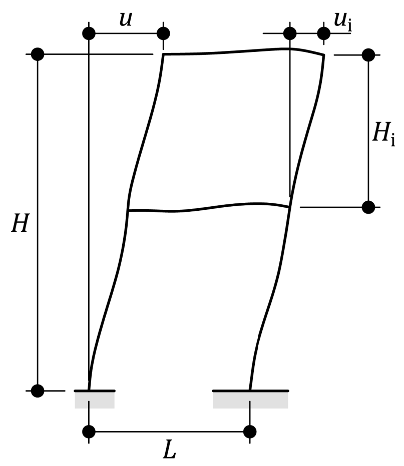
Serviceability criteria	**Limiting damage to elements other than structural**[a]	**Comfort of users**	**Appearance**
Combination of actions to be considered	**Characteristic, Formula (8.29)**	**Frequent, Formula (8.30)**	**Quasi-permanent, Formula (8.31)**
Not accessible roof	Roofing rigid roofing: $w_2+w_3 \leq L/250$ resilient roofing: $w_2+w_3 \leq L/125$ Ceiling plastered ceiling: $w_2+w_3 \leq L/350$ false ceiling: $w_2+w_3 \leq L/250$	$w_2+w_3 \leq L/300$	$w_1+w_2-w_c \leq L/250$
Floor, accessible roof	Internal partition walls not reinforced: — partitions of brittle material or non-flexible: $w_2+w_3 \leq$ L/500 — partitions of non-brittle materials: $w_{max} \leq$ L/400 reinforced walls: $w_2+w_3 \leq L/350$ removable walls: $w_2+w_3 \leq L/250$ Flooring: — tiles rigidly fixed: $w_2+w_3 \leq L/500$ — small tiles[b] or deflection not fully transmitted: $w_2+w_3 \leq L/350$ — resilient flooring: $w_2+w_3 \leq L/250$ Ceiling plastered ceiling: $w_2+w_3 \leq L/350$ false ceiling: $w_2+w_3 \leq L/250$	$w_2+w_3 \leq L/300$	$w_1+w_2-w_c \leq L/250$
Structural frames	Windows: — no loose joints (no clearance between glass and frame): $w_2+w_3 \leq L/1000$ — with loose joints: $w_2+w_3 \leq L/350$		

[a] L = span (or, for cantilever, twice the length); w_1, w_2, w_3, w_{max} are defined in Figure A.1.1.

[b] Small tiles: sides less than 10 cm.

A.1.7.2.3 Horizontal displacements

(1) Horizontal displacements should be calculated using the parameters shown in Figure A.1.2.



Key

u Overall horizontal displacement over the building height H

u_i Relative horizontal displacement over a storey height H_i

Figure A.1.2 — Definition of horizontal displacements

(2) Maximum values of horizontal displacements may be specified by the relevant authorities or, where not specified, agreed for a specific project by relevant parties.

NOTE Suggested values of maximum permitted horizontal displacements are given in Table A.1.11 (NDP) unless the National Annex gives different values.

Table A.1.11 (NDP) — Suggested maximum permitted horizontal displacements for non-industrial buildings

Serviceability criteria[a]	**No damage to elements other than structural**	**Comfort of users**	**Appearance**
Combination of actions to be considered	**Characteristic Formula (8.29)**	**Frequent Formula (8.30)**	**Quasi-permanent Formula (8.31)**
Overall horizontal displacement u	Single-storey buildings: $u \leq H/400$ Multi-storey buildings: $u \leq H/500$	$u \leq H/250$	
Horizontal displacement u_i over a storey height	Brittle partition walls: $u_i \leq H_i/500$ $u_i \leq 6\text{mm}$ No brittle partition walls: $u_i \leq H_i/200$	$u_i \leq H_i/250$	$u_i \leq H_i/250$
[a] H = height of building; H_i = storey height; u_i and u are defined in Figure A.1.2.			

A.1.7.3 Vibrations

(1) To achieve satisfactory vibration behaviour of buildings and their structural members under serviceability conditions, the following aspects, amongst others, should be considered:

— the comfort of the user;

— the functioning of the structure or its structural members, e.g. resulting from cracks in partitions, damage to cladding, sensitivity of building contents to vibrations;

— other aspects specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

(2) For the serviceability limit states of a structure or a structural member not to be exceeded when subjected to vibrations, the natural frequency of vibrations of the structure or structural member should be kept above appropriate values.

(3) Minimum values of natural frequencies of a structure or a structural member should be as agreed for a specific project by relevant parties depending upon the function of the building, the structural materials and the source of the vibration.

NOTE 1 Suggested minimum values of natural frequencies of a structure or a structural member can be set in the National Annex for use in a country.

NOTE 2 SLS due to vibrations are to be considered, e.g. for hospitals, laboratories, gymnasia and sport halls, dance rooms, concert halls, and for floors, stircases and balconies in general.

NOTE 3 Further guidance is given in the other Eurocodes.

(4) If a natural vibration frequency of the structure is lower than the appropriate value, a more refined analysis of the dynamic response of the structure, including the consideration of damping, should be performed.

NOTE Limiting values for the dynamic response of the structure can be given in the National Annex for use in a country. For further guidance, see EN 1991-1-1, EN 1991-1-4, and ISO 10137.

(5) The sources of vibration should be as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

NOTE Possible sources of vibration include walking, synchronised movements of people, machinery, ground borne vibrations from traffic, and wind actions.

A.1.7.4 Limiting foundation movements

(1) The design criterion for the serviceability limit state $C_{\mathrm{d,SLS}}$ for foundation movement beneath a building shall be selected during the design of the supported structure.

(2) The sensitivity of a structure to foundation movement should be classified according to Table A.1.12, separately for different modes of foundation movement.

NOTE Examples of buildings in different structural sensitivity classes are given in Table A.1.16 (NDP) unless the National Annex gives different examples.

Table A.1.12 —Classification of structural sensitivity to foundation movement

Structural sensitivity class	Description of sensitivity
SSC5	Highest
SSC4	Higher
SSC3	Normal
SSC2	Lower
SSC1	Lowest

(3) The value of $C_{d,SLS}$ may be specified by the relevant authorities or, where not specified, agreed for a specific project by relevant parties.

(4) When the value of $C_{d,SLS}$ is not otherwise specified, $C_{d,SLS}$ may be chosen according to the structure's sensitivity to foundation movement.

NOTE Suggested values of $C_{d,SLS}$ are given in Tables A.1.13 (NDP), A.1.14 (NDP), and A.1.15 (NDP) unless the National Annex gives different values.

Table A.1.13 (NDP) — Suggested maximum permitted differential settlement of foundations for different structural sensitivity classes

Structural sensitivity class	Description of sensitivity	Maximum differential settlement[a] $\Delta s_{Cd,SLS}$
SSC5	Highest	10 mm
SSC4	Higher	15 mm
SSC3	Normal	30 mm
SSC2	Lower	60 mm
SSC1	Lowest	100 mm
[a] See EN 1997-1 for the definition of differential settlement of foundations.		

Table A.1.14 (NDP) — Suggested maximum permitted angular distortion of foundations for different structural sensitivity classes

Structural sensitivity class	Description of sensitivity	Maximum angular distortion[a] $\beta_{\mathrm{Cd,SLS}}$
SSC5	Highest	0,05 %
SSC4	Higher	0,075 %
SSC3	Normal	0,15 %
SSC2	Lower	0,3 %
SSC1	Lowest	0,5 %
[a] See EN 1997-1 for the definition of angular distortion of foundations.		

Table A.1.15 (NDP) — Suggested maximum permitted tilt of foundations for different structural sensitivity classes

Structural sensitivity class	Description of sensitivity	Maximum tilt[a] $\omega_{\mathrm{Cd,SLS}}$
SSC5	Highest	0,1 %
SSC4	Higher	0,2 %
SSC3	Normal	0,3 %
SSC2	Lower	0,4 %
SSC1	Lowest	0,5 %
[a] See EN 1997-1 for the definition of foundation tilt.		

Table A.1.16 (NDP) — Examples of buildings in different structural sensitivity classes

<table>
<tr><th>Design criteria for SLS, $C_{\mathrm{d,SLS}}$</th><th>Type or use of structure</th><th>Structural sensitivity class</th></tr>
<tr><td>Differential settlement, s_{Cd}</td><td>Utility connections</td><td>SSC1</td></tr>
<tr><td rowspan="2">Angular distortion, β_{Cd}</td><td>Framed buildings and reinforced load-bearing walls</td><td>SSC3</td></tr>
<tr><td>Floors, slabs</td><td>SSC1</td></tr>
<tr><td rowspan="2">Tilt, ω_{Cd}</td><td>Towers, tall buildings (visual), height H < 24 m
Towers, tall buildings (visual), 24 m ≤ H < 60 m
Towers, tall buildings (visual), 60 m ≤ H < 100 m
Towers, tall buildings (visual), 100 m ≤ H</td><td>SSC2
SSC3
SSC4
SSC5</td></tr>
<tr><td>Lift and escalator operation</td><td>SSC5</td></tr>
</table>

A.2 Application for bridges[2)]

A.3 Application for towers, masts and chimneys[2)]

A.4 Application for silos and tanks[2)]

A.5 Application for structures supporting cranes[2)]

A.6 Application for marine coastal structures[2)]

2) Annex A subclauses A.2, A.3, A.4 and A.6 will be included in a subsequent amendment.

Annex B
(informative)

Technical management measures for design and execution

B.1 Use of this informative Annex

(1) This informative Annex provides supplementary guidance to that given in 4.2, 4.3 and 4.8 for technical management measures covering design and execution to meet the assumptions given in 1.2.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

B.2 Scope and field of application

(1) This informative Annex provides a framework for technical management measures for:

— design quality;

— design checking;

— execution quality;

— inspection during execution;

so that the intended level of structural reliability of a structure (or part of structure) designed and executed according to the Eurocodes is achieved and the assumptions given in 1.2 are satisfied.

NOTE The implementation of this informative Annex depends on the legal system in force in each country. This Annex is provided as guidance to the writers of National Annexes that can enable a consistent approach to this subject.

B.3 Choice of technical management measures

(1) The technical management measures should be chosen relevant to the selected consequence classes (see 4.3).

NOTE See B.8 for the selection of appropriate technical management measures.

B.4 Design quality

(1) The term 'quality', as used within the design and execution process for structures, deals with the use of adequate technical knowledge and its correct application to achieve the required mechanical resistance, stability, serviceability, and durability of a structure.

(2) The personnel responsible for the design of a structure should have appropriate qualifications and experience, depending on the consequences of failure of the structure and the complexity of its design.

NOTE Minimum appropriate qualifications and experience of personnel designing structures can be defined in the National Annex. Design qualification and experience levels (DQLs) presented in Table B.1 (NDP) can be used as a framework to define minimum requirements for qualification and experience of personnel.

(3) Additional project-specific requirements for qualification and experience of the personnel responsible for the design of a structure may be as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

Table B.1 (NDP) — Design qualification and experience levels (DQL)

DQL	Design qualification and experience of personnel	Required level
DQL3	Have at least the same level of design qualification and experience to that required to perform complex design	To be defined nationally
DQL2	Have at least the same level of design qualification and experience to that required to perform advanced design	To be defined nationally
DQL1	Have the required level of design qualification and experience to perform simple design	To be defined nationally

B.5 Design checking

(1) The design should be checked to reduce the risk of human errors that might have arisen during the design process.

NOTE Minimum requirements for design checking can be set in the National Annex. Design check levels (DCLs) presented in Table B.2 (NDP) can be used as a framework to define minimum requirements for design checking.

(2) Additional project-specific requirements for design checking may be specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

Table B.2 (NDP) — Design check levels (DCL)

DCL	Design checking	Required level
DCL3	Extended independent checking[a]	To be defined nationally
DCL2	Normal independent checking[a]	To be defined nationally
DCL1	Self-checking	To be defined nationally
[a] The term may be defined nationally.		

(3) Self-checking shall be performed for all designs.

(4) Design checking should cover:

— loads, models for calculation of loads and design situations;

— structural models, calculation of load effects and design verification;

— adequate knowledge of ground conditions and the design parameters;

— where appropriate, separate calculations as alternatives to reviewing the design calculations;

— consistency of calculations, drawings, detailing and the execution specification.

(5) Measures for design checking should concentrate on those parts of a structure where failure would have the most serious consequences with respect to structural resistance, durability, and function.

B.6 Execution quality

(1) Execution and execution quality assurance for structures and structural products shall be in accordance with the relevant European standards on execution.

NOTE Additional requirements for execution quality assurance can be defined in the National Annex.

(2) When no relevant European standard on execution exists, execution quality assurance measures should be in place.

NOTE 1 These measures can include a management system that defines roles and responsibilities.

NOTE 2 Requirements for execution management systems where no relevant European execution standard exists can be defined in the National Annex.

NOTE 3 For the assuptions of the Eurocodes relevant to execution, see 1.2(3).

B.7 Inspection during execution

(1) Inspection during execution should be undertaken to check the compliance of the execution with the design and the execution specification, and to reduce human errors during execution.

NOTE 1 The term "execution specification" covers e.g. calculations, drawings, descriptions of the works, choice of the products, execution classes, tolerance classes, etc.

NOTE 2 Minimum requirements for the level of inspection during execution can be set in the National Annex. Inspection levels (ILs) presented in Table B.3 (NDP) can be used as a framework to define minimum requirements for inspection during execution.

(2) Additional project-specific requirements for levels of inspection may be specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

Table B.3 (NDP) — Inspection levels (IL)

IL	Inspection	Required level
IL3	Extended independent inspection[a]	To be defined nationally
IL2	Normal independent inspection[a]	To be defined nationally
IL1	Self inspection	To be defined nationally
[a] The term may be defined nationally.		

(3) Self-inspection shall be carried out for all execution.

(4) Inspection should ensure that:

— the execution specifications are available during manufacturing and execution;

— the execution is performed according to the execution specification;

— the personnel have the skills and training required for the work;

— inspection is properly documented;

— materials and construction products used are as specified.

(5) Measures for inspection should concentrate on those parts of a structure where failure would have the most serious consequences with respect to structural resistance, durability, and function.

B.8 Technical management measures

(1) Where used, the DQL, DCL, and IL should be chosen according to the consequences of failure.

NOTE The minimum DQL, DCL, and IL are given in Table B.4 (NDP) unless the National Annex gives different minima.

(2) For geotechnical structures, where used the DQL, DCL, and IL should be chosen according to the consequences of failure and the complexity of the ground, as specified in EN 1997-1.

Table B.4 (NDP) — Minimum design quality level, design check level, execution class and inspection level for different consequence classes

Consequence class	**Minimum design quality level (DQL)**	**Minimum design check level (DCL)**	**Minimum execution class (EXC)**	**Minimum inspection level (IL)**
CC3	DQL3	DCL3	See relevant execution standards[a]	IL3
CC2	DQL2	DCL2		IL2
CC1	DQL1	DCL1		IL1

[a] Relevant execution standards might not be available for all materials, see B.6(2).

Annex C
(informative)

Reliability analysis and code calibration

C.1 Use of this informative Annex

(1) This informative Annex provides guidance on reliability-based methods of analysis and on the calibration of partial factors.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

C.2 Scope and field of application

(1) Subclause C.3 provides the basis for the reliability verification formats that can be used within the Eurocodes. Information and theoretical background and the reliability-based calibration of the partial factor method described in Clause 5 and Annex A are given. This Annex also provides the background to Annex D, and is relevant to the contents of Annex B.

(2) Subclause C.3 also provides information on:

— structural reliability assessment;

— application of reliability-based methods to determine design values of partial factors in design formulae by calibration.

NOTE The majority of structures can be designed according to the Eurocodes without applying the methods presented in this Annex. These methods can, however, be useful for design situations that are not well covered and for possible extensions of the standard.

(3) Subclause C.4 sets out principles for reliability-based code-calibration and corresponding guidance for possible extensions and developments of the partial factor design method.

NOTE Calibration and verification of partial factors is done by national competent authorities.

C.3 Basis for reliability analysis and partial factor design

C.3.1 Overview of reliability verification approaches

(1) In order to verify whether a structure complies with reliability requirements for all design and assessment situations, one of the following approaches, with corresponding criteria, shall be chosen:

— Semi-probabilistic, in which the structure fulfils a set of inequalities using specified design values of the basic variables; or

— Reliability-based, in which the structure fulfils a set of reliability requirements; or

— Risk-informed, in which the sum of all costs (building, maintenance, etc.) and economic risks (with respect to failure or malfunctioning) is minimized while ensuring that aspects of human safety are consistent with the preferences of the society.

NOTE The choice between these approaches is made according to (3) to (6).

(2) In order to assess structural performance, structural responses should be divided into two domains: desirable and undesirable states. The boundary between these domains is called the limit state. 'Failure' is defined as entering the undesirable state (see 5.2).

NOTE According to this definition, 'failure' can also refer to limit states, where structural capacity is not involved, e.g. a serviceability limit state.

(3) Except where stated otherwise in the Eurocodes, the semi-probabilistic approach via a partial factor design format should be applied in all design situations.

(4) The reliability-based approach may be applied to design situations where uncertainties in the representation of loads, load effects, material resistances, and system-effects mean that the reliability-based approach gives a significantly better representation of reality than the partial factor design format.

NOTE Design situations that are not covered by the partial factor design format can include:

— situations where relevant loads or hazard scenarios are not covered by EN 1991;

— the use of building materials or combination of different materials outside the usual application domain, e.g. new materials, behaviour at very high temperatures;

— ground conditions, such as rock, which are strongly affected by discontinuities and other geometrical phenomena.

(5) Conditions for the use of reliability-based methods may be specified by the relevant authority or, where not specified, agreed for the specific project by the relevant parties.

(6) The reliability-based approach should also be used for the calibration of partial factors in the semi-probabilistic approach, see subclause C.4.

NOTE Calibration of partial factors is performed by National Standards Bodies, not designers.

(7) The use of the risk-informed approach may apply to design situations where both the uncertainties and the consequences are outside common ranges.

NOTE 1 Design situations where the uncertainties and the consequences are outside common ranges include, for example, those associated with accidents and those which clearly deviate from situations generally covered by the Eurocodes.

NOTE 2 No further guidance on the risk-informed approach is given in this Annex. Relevant guidance can be found in ISO 2394:2015.

NOTE 3 Table C.1 presents an overview of the various methods for the verification of the adequate reliable performance of structures.

(8) Risk-informed and reliability-based approaches shall only be employed if uncertainties are represented consistently based on unbiased assumptions.

Table C.1 — Overview of methods for the verification of adequate reliable performance of structures together with typical application areas

	Applied when
Semi-probabilistic approach Safety format prescribing the design equations and the analysis procedures to be used	Default method in the Eurocodes, i.e. to be used for usual design situations
Reliability-based design and assessment **Reliability requirements to fulfil**	Unusual design situations in regard to uncertainties Code calibration
Risk-informed decision making Decisions are taken with due consideration of the total risks (e.g. loss of lives, injuries,	Exceptional design situations in regard to uncertainties and consequences. Derivation of reliability requirements.

C.3.2 Uncertainty representation and modelling

C.3.2.1 General

(1) The axioms of probability theory shall apply. Uncertainties shall be represented through probabilistic models consisting of random variables, stochastic processes, and/or random fields.

(2) All uncertainties that are important for the verification of adequate structural performance should be considered.

(3) The possibility and impact of gross human errors should be minimized by appropriate quality control.

NOTE Gross human errors are not considered in the uncertainty modelling presented here. See Annex B for their possible treatement.

C.3.2.2 Semi-probabilistic approach

NOTE In the semi-probabilistic approach, uncertainties are considered implicitly by the specification of characteristic values of random variables and partial safety factors that are applied to those variables.

C.3.2.3 Reliability-based and risk-informed approaches

NOTE Reliability-based and risk-informed approaches allow a more detailed representation of uncertainties, see C.3.3.

(1) Uncertainty modelling may include, where relevant, the representation of temporal and spatial dependency among the considered uncertainties and events.

NOTE 1 The importance of different uncertainties can be revealed by sensitivity analysis.

NOTE 2 The basic variables introduced in Clause 6 allow for the representation of several types of uncertainty, for instance: inherent natural variability, statistical uncertainty, measurement uncertainty, uncertainty related to the precision of new information, and model uncertainty.

(2) The quantification of uncertainties and their probabilistic representation should incorporate both subjective information and available evidence. Bayesian probability theory, which provides a consistent framework for the treatment of different types of information, should be used.

(3) The description of uncertain quantities by probabilistic models should correspond to well-defined sets of populations.

NOTE The results of the application of risk-informed or reliability-based approaches are only valid for the same sets.

C.3.3 Reliability-based design

C.3.3.1 General

(1) Following the reliability-based approach, decisions with respect to the design of structures shall take basis in reliability assessments, which ensure that the structure meets defined reliability requirements.

NOTE Reliability assessment involves the estimation of the probability of adverse events. Adverse events are events that include consequences and are conventionally termed 'failure events'.

(2) Failure events should be represented by limit states.

(3) Where analytical models for the representation of adverse events or failure events are available, the limit state function g() may be represented by Formula (C.1):

$$\mathrm{g}(\boldsymbol{X}(t))=0 \tag{C.1}$$

The time-variant basic variables $\boldsymbol{X}(t)$ may be represented by Formula (C.2):

$$\boldsymbol{X}(t)=X_1(t),X_2(t),... \tag{C.2}$$

The domain of adverse (failure), events $\Omega(\boldsymbol{x}(t))$ is given by Formula (C.3):

$$\Omega(\boldsymbol{x}(t))=\left\{\mathrm{g}(\boldsymbol{x}(t))<0\right\} \tag{C.3}$$

(4) When failure events are represented by numerical models such as finite element models, surrogate models as response surfaces $\mathrm{r}(\boldsymbol{x}(t))$ may be used for analytical representation $\mathrm{g}(\boldsymbol{x}(t)) \approx \mathrm{r}(\boldsymbol{x}(t))$.

C.3.3.2 Reliability estimation

C.3.3.2.1 General

(1) The calculation of the probability of failure should account for all available knowledge, and the uncertainty representation shall follow the provisions in C.3.4.

(2) The specific type of reliability analysis that should be used depends on the failure event being analysed, as specified in C.3.3.2.2 to C.3.3.2.3.

C.3.3.2.2 Time-invariant reliability analysis

(1) Time-invariant reliability analysis may be used to model a single failure mode that does not depend on time (or spatial characteristics).

(2) Time-invariant reliability analysis may also be used for problems that can be transformed such that they do not depend on time.

NOTE For example, by use of the time-integrated approach considering extreme values.

(3) In time-invariant reliability analysis, the probability of failure occurrence should be calculated as given in Formula (C.4):

$$P_\mathrm{f} = \int_{\Omega(\mathbf{x})} \mathrm{f}_{\boldsymbol{X}}(\boldsymbol{x})d\boldsymbol{x} \tag{C.4}$$

where

$\boldsymbol{x}$ is the vector of basic random variables;

$\Omega(\boldsymbol{x})=\{g(\boldsymbol{x})<0\}$ is the failure domain defined with limit state function;

$g(\boldsymbol{x})$ representing the considered failure mode;

$f_{\boldsymbol{X}}(\boldsymbol{x})$ is the joint probability density function of x.

NOTE In a time-invariant reliability analysis, time-variable loads can be represented by the probability distributions of their yearly extreme values. Correspondingly, the calculated probability of failure refers to a one-year reference period $P_{\mathrm{f}} = P_{\mathrm{f,1y}}$.

(4) Structural reliability methods may be used for the computation of the failure probability, according to Formula (C.4).

(5) Depending on the problem, one of the following methods should be selected:

— First/Second Order Reliability Method (FORM/SORM); or

— simulation techniques, e.g. Monte Carlo simulation, importance sampling, asymptotic sampling, subset simulation, and adaptive sampling; or

— numerical integration.

(6) The annual reliability index β_a and the annual probability of failure $P_{\mathrm{f},a}$ may be used as standard metrics to express structural reliability.

NOTE 1 Independent from the reference period, the functional relationship between the failure probability and the reliability index is given in Formula (C.5):

$$P_{\mathrm{f}} = \Phi(-\beta) \tag{C.5}$$

where

$\Phi(.)$ is the standard normal cumulative probability distribution function.

NOTE 2 Numerical values of β for indicative values of P_{f} are given in Table C.2.

Table C.2 — Relation between P_{f} and β

P_{f}	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}
$\beta = -\Phi^{-1}(P_{\mathrm{f}})$	1,28	2,33	3,09	3,72	4,26	4,75	5,20

C.3.3.2.3 Time-variant reliability analysis

(1) A time-variant reliability analysis should be used when a single failure mechanism is being analyzed and its occurrence probability does depend on the point in time.

C.3.4 Reliability requirements

C.3.4.1 General

(1) Reliability requirements shall be as prescribed by the relevant national authority.

NOTE 1 In the partial factor method, reliability requirements are implicitly satisfied through the use of partial factors specified in the Eurocodes.

NOTE 2 The following clauses are addressed to the relevant national authorities to assist them in defining the reliability requirements.

(2) Reliability requirements can be established by a risk assessment.

(3) Reliability requirements can be formulated in terms of minimum reliability requirements and/or service life cost optimal target reliability requirements:

— the minimum reliability requirements depend on the societal capacity and preferences to invest into life safety (minimum reliability requirements are normally compulsory);

— Target reliability requirements depend on the expected failure consequences and on all costs associated with the design, operation, inspection, maintenance and renewal of structures over the time period for which they are needed.

(4) Reliability requirements shall be fulfilled for all relevant failure events including single member failure, partial structural failure, and full structural-system collapse.

NOTE 1 The specified reliability requirements relevant for ultimate and serviceability limit state design do not account for human errors. Therefore, failure probabilities are not directly related to the observed failure rates, which are highly influenced by failures involving some effects of human errors.

NOTE 2 Requirements to minimize, detect and mitigate human errors are given in Annex B.

(5) Explicit reliability requirements may be used to:

— establish criteria for the reliability-based design and assessment;

— support design assisted by testing;

— facilitate the calibration of partial safety factor design formats.

NOTE For the first two cases, the requirements are relevant to the designer. For the last case, the requirements are relevant to the relevant national authorities, see subclause C.4.

C.3.4.2 Criterion for reliability-based design and assessment

(1) If the design situation can be directly related to a similar reference design situation that is covered by the partial safety factor design format, it should be demonstrated that for a relevant type of structures the same reliability level as the reference design is obtained.

NOTE This relative comparison should be made based on similar probabilistic models.

(2) When it is stated in the Eurocodes that a design and assessment situation is not covered by the partial safety factor design format, the reference period and the associated target reliability values should be defined.

NOTE 1 The target values of reliability index β for the 1-year (β_1) and 50-year (β_{50}) reference periods for fundamental and fatigue design situations in ULS for structures included in the scope of Annexes A.1 and A.2 are given in Table C.3 (NDP), unless the National Annex gives different values for use in a country.

NOTE 2 The partial factors given in subclauses A.1 and A.2 are expected to lead to a structure with a reliability index β_{50} greater than the values given in Table C.3 for a 50-year reference period.

Table C.3 (NDP) — Target values for reliability index β for different consequence classes (for fundamental and fatigue design situations in ULS) relevant to structures in the scope of Annex subclauses A.1 and A.2

Consequence class	1-year reference period β_1	50-year reference period	
		β_{50}	$P_{f,50}$
CC3	5,2	4,3	$\sim 10^{-5}$
CC2	4,7	3,8	$\sim 10^{-4}$
CC1	4,2	3,3	$\sim 10^{-3}$

NOTE 3 The values given in Table C.3 for β_1, corresponding to the β_{50} values, are based on the assumptions that failure events in each year of the 50-year reference period are independent events and that no deterioration is considered.

These assumptions lead to a theoretical upper bound for β_1 values, which is approached, for instance, in the case for wind dominated load combinations in structures with low variability in resistance.

Lower values of β_1 would correspond to cases where the failure events in each year of the 50-year reference period are partially correlated, due to the constant presence of variables that do not change with time, like strength (if deterioration is not accounted for) and self-weight.

(3) When referring to the 1-year reliability index β_1, the target should be met for every year of the required (or chosen) design (or remaining) working life of the structure.

C.3.4.3 Reliability requirements for reliability-based code calibration

(1) For the purpose of code calibration of partial safety factors and other reliability elements in semi-probabilistic safety formats, the reliability requirement should be defined as a target value for reliability levels, taken as representative averages over the considered design situations.

NOTE Code calibration is performed by national competent authorities.

(2) If the partial factor design format that is being calibrated can be related to an existing partial factor design format for which the safety level is considered satisfactory, the corresponding average reliability level should be used as a target value for calibration.

(3) If the partial factor design format being calibrated cannot be related to an existing design format, the reliability targets specified in C.3.4.2(2) should be used.

C.3.4.4 Reliability requirements for design assisted by testing

(1) The reliability level to be used for the determination of design values based on data from tests or observations based on D.7.2 should correspond to a one-year reference period and should be chosen as specified in C.3.4.2(2).

C.4 Approach for calibration of design values

C.4.1 Reliability requirements for reliability-based code calibration

(1) For the purpose of code calibration of partial safety factors and other reliability elements in semi-probabilistic safety formats, the reliability requirement should be defined as a target value.

(2) When the reliability of a representative set of comparable structures designed according to existing codes is considered satisfactory, target value of the reliability level for structures similar to those included in this set may be derived from the reliability assessment of this set.

NOTE Comparable structures means that they are made with the same material and have similar destinations, similar structural schemes and design dominated by the same actions.

(3) As the numerical values of the reliability depend on the structure layout, on the material and on the limit state equation as well as on the assumed statistical properties of the relevant variables, code calibration of partial safety factors should be performed according the same assumptions as adopted in the definition of target values.

NOTE Probabilistic models for loads and resistances are given in the technical report.

(4) The reliability targets specified according to C.3.4.2(2) may be used.

C.4.2 Partial factor design format and code parameters

(1) The principle form of the design formula (Formula (8.1)) may be rearranged as given in Formula (C.6):

$$R_d - E_d \geq 0 \tag{C.6}$$

where the design values for the load bearing capacity R_d and the effect of actions E_d are obtained from Formulae (C.7) and (C.8):

$$R_\mathrm{d} = R_\mathrm{d}(\boldsymbol{X}_\mathrm{d}\,;\,\boldsymbol{a}_\mathrm{d}\,;\,\boldsymbol{\theta}_\mathrm{d}\,;\boldsymbol{F}_\mathrm{d,R}) \tag{C.7}$$

$$E_\mathrm{d} = E_\mathrm{d}(\boldsymbol{F}_\mathrm{d}\,;\,\boldsymbol{a}_\mathrm{d}\,;\,\boldsymbol{\theta}_\mathrm{d}\,;\boldsymbol{X}_\mathrm{d}) \tag{C.8}$$

where

$\boldsymbol{F}_\mathrm{d}$ are vectors of design values of actions independent on the material properties;

$\boldsymbol{F}_\mathrm{d,R}$ are vectors of design values of actions depending on the material properties, when relevant;

$\boldsymbol{X}_\mathrm{d}$ are vectors of design values of material properties independent on the combination of actions;

$\boldsymbol{a}_\mathrm{d}$ is a vector of design values of geometrical properties;

$\boldsymbol{\theta}_\mathrm{d}$ is a vector of design values of model uncertainties.

NOTE Formulae (C.6) to (C.8) are applicable for time-invariant reliability problems represented by linear limit state functions with independent variables, see 8.3. For some particular design situations (e.g. fatigue, geotechnics) a more general formulation may be necessary to express a limit state.

C.4.3 Partial factors

(1) Partial safety factors for actions, γ_F, and for resistance, γ_M, should include model uncertainties.

(2) The design value of a basic variable related to loads (F_d) may be defined as the multiplication of the characteristic value F_k by a corresponding partial safety factor γ_F (i.e. $F_\mathrm{d} = \gamma_\mathrm{F} F_\mathrm{k}$).

(3) The design value of a basic variable related to resistance (X_d) may be defined as the division of the characteristic value X_k by a corresponding partial safety factor γ_M (i.e. $X_\mathrm{d} = X_\mathrm{k} / \gamma_\mathrm{M}$).

NOTE More specific formats for the determination of design values for actions and resistances are given in Clause 8.

(4) When model uncertainties are considered separately, partial safety factors may be derived from Formula (C.9) or Formula (C.10):

$$\gamma_{\mathrm{F}} = \gamma_{\mathrm{Sd}}\gamma_{\mathrm{f}} \tag{C.9}$$

$$\gamma_{\mathrm{M}} = \gamma_{\mathrm{Rd}}\gamma_{\mathrm{m}} \tag{C.10}$$

where

γ_{f} covers uncertainty in representative values of actions;

γ_{Sd} covers the model uncertainty in actions and action effects;

γ_{m} covers uncertainty in basic variables describing the resistance;

γ_{Rd} covers the model uncertainty in structural resistance, also accounting, when relevant, the bias in resistance model, see Annex D.

(5) The characteristic value Y_{k} may be taken as a specified p-fractile value from the statistical distribution F_Y chosen to represent the basic variable, as given in Formula (C.11):

$$Y_{\mathrm{k}} = F_Y^{-1}(p) \tag{C.11}$$

where

F_Y is the cumulative probability distribution function of the basic variable Y.

NOTE Typical values for p are:

— resistance related variables: p = 0,05;

— permanent actions: p = 0,5;

— time-variable actions: p = 0,98, referring to the distribution of the yearly extreme values.

(6) The partial safety factors for the various actions and materials characteristics entering the design formulae should be determined by calibration to satisfy Formula (C.6) and be consistent with the reliability requirements.

C.4.4 Basis for calibration of design values

C.4.4.1 General

(1) The design value method, see C.4.4.2, may be used to determine partial safety factors.

(2) For the calibration of design formats that cover a multitude of design situations the guidelines in C.4.5 may be followed.

C.4.4.2 The design value method

(1) For simple cases, a direct correspondence between the design value and the reliability requirements may be established by the so-called 'design value method' in Formula (C.12):

$$Y_{\mathrm{d}} = F_Y^{-1}\left(\Phi\left(-\alpha_Y \beta\right)\right) \tag{C.12}$$

where

Y_{d} is the design value;

F_Y is the cumulative probability distribution function of the basic variable Y;

α_Y with $|\alpha_Y| \leq 1$ is a sensitivity factor indicating the importance of Y in the reliability estimation;

$\beta = \beta^{\mathrm{tgt}}$ is the target value for the 50-year reliability index according to the reliability requirement in C.3.4.

(2) Design values Y_{d} and characteristic values Y_{k} for some common distributions may be determined according to Formulae (C.13) to (C.20).

Normal distribution:

— Characteristic value:

$$Y_{\mathrm{k}} = \mu_Y + \Phi^{-1}(p)\sigma_Y = \mu_Y(1 + \Phi^{-1}(p)V_Y) \tag{C.13}$$

— Design value:

$$Y_{\mathrm{d}} = \mu_Y - \alpha_Y \beta \sigma_Y = \mu_Y(1 - \alpha_Y \beta V_Y) \tag{C.14}$$

Log-normal distribution:

— Characteristic value:

$$Y_{\mathrm{k}} = \mu_Y e^{(-\frac{1}{2}\ln(1+V_Y^2) + \Phi^{-1}(p)\sqrt{\ln(1+V_Y^2)})} \tag{C.15}$$

$$Y_{\mathrm{k}} \simeq \mu_Y e^{(\Phi^{-1}(p)V_Y)} \quad \text{for } V_Y < 0{,}2 \tag{C.16}$$

— Design value:

$$Y_{\mathrm{d}} = \mu_Y e^{(-\frac{1}{2}\ln(1+V_Y^2) - \alpha_Y \beta \sqrt{\ln(1+V_Y^2)})} \tag{C.17}$$

$$Y_{\mathrm{d}} \simeq \mu_Y e^{(-\alpha_Y \beta V_Y)} \quad \text{for } V_Y < 0{,}2 \tag{C.18}$$

Gumbel distribution:

— Characteristic value:

$$Y_{\mathrm{k}} = \mu_Y \left(1 - V_Y \frac{\sqrt{6}}{\pi}(0{,}5772 + \ln(-\ln(p)))\right) \tag{C.19}$$

— Design value:

$$Y_d = \mu_Y \left(1 - V_Y \frac{\sqrt{6}}{\pi} (0{,}5772 + \ln(-\ln(\Phi(-\alpha_Y \beta))) \right) \tag{C.20}$$

where

μ_Y denotes the mean value and V_Y the coefficient of variation of *Y;*

β is the target value for the reliability index specifying the reliability requirement;

α_Y is a sensitivity factor indicating the importance of in the reliability estimation;

p is the distributions fractile that defines the characteristic value.

NOTE 1 α_Y is determined by reliability analysis. As simplification, the following typical values can be used as an approximation, provided that Formula (C.21) is satisfied:

$$0{,}16 < \sigma_E / \sigma_R < 7{,}6 \tag{C.21}$$

where

— if ***Y*** represents a strength related variable*:* α_Y = 0,8;

— if ***Y*** represents a leading load related variable: α_Y = -0,7;

— If ***Y*** is dominating the reliability problem: α_Y = 1 (resistance); α_Y = -1 (load).

NOTE 2 Self-weight is usually represented by a Normal distribution; Resistance variables are often represented by a Lognormal distribution; the extreme values per reference period of time-variable actions are represented by the Gumbel distribution.

(3) When the action model contains several basic variables, Formula (C.10) should be used for the leading variable only. For the accompanying actions the design values can be defined by Formula (C.22):

$$P(E > E_d) = \Phi\left(-0{,}4\text{x}0{,}7\text{x}\beta\right) = \Phi\left(-0{,}28\beta\right) \tag{C.22}$$

(4) The target value for the reliability index and the extreme value distribution used to represent time-variable actions are defined based on the same reference period, i.e. with $\beta = \beta^{tgt}$ the reference period is 50 years, with $\beta = \beta^{tgt}_{qy}$ the reference period is one year.

C.4.4.3 Code optimization

(1) The reliability elements, including partial factors γ and combination factors Ψ, should be calibrated in such a way that the target reliability index β^{tgt}, chosen according to C.3.4.2, is best achieved.

NOTE 1 The calibration procedure involves the following steps:

— selecting of a set of comparable reference structures;

— selecting and specifying a set of reliability elements, e.g. partial factors, Ψ factors;

— designing the structures according to the selected set of reliability elements;

— calculating the reliability indices β_i for the designed structures;

— calculating the difference: $D=\sum_i w_i\left(\beta_i-\beta^{tgt}\right)^2$ where w_i is the weight factor i;

— repeating steps to minimize D.

NOTE 2 A more detailed procedure how to provide this optimization is described in several sources, e.g. in ISO 2394.

C.4.5 Combination of variable actions

(1) The combination factor ψ_0 that account for the combination of two variable actions Q_1 and Q_2 can be estimated as given in Formulae (C.23) and (C.24):

— Q_1 dominating:

$$\psi_{0,2}=\frac{F^{-1}_{Q_{2,\max,\tau_1}}\left\{\Phi(\alpha_{Q_{2,\max,\tau_1}}\beta_{\mathrm{t}})\right\}}{F^{-1}_{Q_{2,\max,T}}\left\{\Phi(\alpha_{Q_{2,\max,T}}\beta_{\mathrm{t}})\right\}} \quad with \quad F_{Q_{2,\max,\tau_1}}(q)=F^{(\tau_1/T)}_{Q_{2,\max,T}}(q) \tag{C.23}$$

— Q_2 dominating:

$$\psi_{0,1}=\frac{F^{-1}_{Q_{1,\tau_1}}\left\{\Phi(\alpha_{Q_{1,\tau_1}}\beta_{\mathrm{t}})\right\}}{F^{-1}_{Q_{1,\max,T}}\left\{\Phi(\alpha_{Q_{1,\max,T}}\beta_{\mathrm{t}})\right\}} \quad with \quad F_{Q_{1,\tau_1}}(q)=F^{(\tau_1/T)}_{Q_{1,\max,T}}(q) \tag{C.24}$$

where

$F_{Q_{i,\max,\mathrm{T}}}$	is the extreme value distribution function for Q_i, where i = 1, 2 for reference period T;
τ_i	is the basic period, where the load intensity is assumed to be constant (i = 1; 2);
$F_{Q_{2,\max,\tau_1}}(\mathrm{q})$	is the extreme value distribution function for Q_2 for reference period τ_1;
$F_{Q_{1,\tau_1}}(\mathrm{q})$	is the distribution of an arbitrary realisation of Q_1;
Φ	is the cumulative standard normal distribution function;
$\alpha_{Q_{i,\max,T}}$	is the FORM sensitivity factor of Q_i dominating (i = 1, 2);
$\alpha_{Q_{i,\tau_1}}$	is the FORM sensitivity factor of Q_i not dominating (i = 1, 2);
β_{t}	is the target reliability for the reference period T.

NOTE 1 In the context of load combination, the reference period is usually T = 1 year.

NOTE 2 The given expressions imply the following assumptions:

— the two actions to be combined are independent of each other;

— the basic period τ_1 or τ_2 for each action is constant;

— τ_1 is the greater basic period and τ_1 / τ_2 is an integer;

— the action values within respective basic periods are constant;

— the intensities of an action within basic periods are uncorrelated;

— the two actions belong to ergodic processes.

NOTE 3 The values follow from a reliability analysis and are very case specic. In the absence of any other information, $\alpha_{Q_{i,\max,T}}$ = 0,7 and $\alpha_{Q_{i,\tau_1}}$ = 0,24 can be used for rough estimation.

(2) Combination factors ψ_0 can also be determined by reliability-based calibration.

Annex D
(informative)

Design assisted by testing

D.1 Use of this informative Annex

(1) This informative Annex provides additional guidance to that given in 7.3, on design assisted by testing.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

D.2 Scope and field of application

(1) This Annex gives guidance on the procedure to be followed to directly determine by testing either the resistance-side parameters of the design equation or structures that lead to a reliability level not less than that required by this document.

NOTE 1 Tests can be carried out to determine the representative or design values of actions, relations between actions and actions effects and, directly, the reliability of structures. Statistical considerations and analyses necessary for these, which can depend on the type of action, i.e. waves, currents, wind and friction/drag forces, etc., can be found in the relevant EN 1991 parts or in specialist literature.

NOTE 2 This Annex covers statistical uncertainties involved in converting test measurements to representative or design values. Where direct measurement of the parameter of interest is either not possible or not carried out, models are used to convert the measurements. Uncertainty in these models, called 'model uncertainty', is considered and can be incorporated in the coefficient of variation. For details, specialist literature can be consulted.

NOTE 3 The methods given in this Annex do not cover testing of geotechnical materials for which EN 1997 can be consulted.

(2) The application of reliability requirements for design assisted by testing should be in accordance with the legal system in force in each country.

(3) This Annex provides guidance on the application of 7.3(3), 8.1(3), and 8.3.6(2).

NOTE This Annex is not intended to replace acceptance rules given in harmonised European product specifications, other product specifications or execution standards.

D.3 Types of tests

(1) A distinction should be made between the following types of tests:

a) tests to establish directly the ultimate resistance or serviceability properties of structures or structural members for given loading conditions. Such tests can be performed, for example, for fatigue loads or impact loads;

b) tests to obtain specific material properties using specified testing procedures; for instance, ground property testing, either in situ or in the laboratory, or the testing of new materials;

c) tests to reduce uncertainties in parameters in action or action effect models; for instance, by wind tunnel testing, or in tests to identify actions from waves or currents;

d) tests to reduce uncertainties in parameters used in resistance models; for instance, by testing structural members or assemblies of structural members (e.g. roof or floor structures);

e) control tests to check the identity or quality of delivered products or the consistency of production characteristics; for instance, testing of cables for bridges, or concrete cube testing;

f) tests carried out during execution in order to obtain information needed for part of the execution; for instance, testing of pile resistance, testing of cable forces during execution;

g) control tests to check the behaviour of an actual structure or of structural members after completion, e.g. to find the elastic deflection, vibrational frequencies or damping.

(2) For test types (a), (b), (c), (d), the design values to be used should wherever practicable be derived from the test results by applying acceptable conventional statistical techniques. See D.4 to D.7.

NOTE Special techniques might be needed in order to evaluate type (c) test results.

(3) Test types (e), (f), (g) may be used as acceptance tests when no test results are available at the time of design.

(4) In this situation, design values should be conservative estimates that are confirmed via the acceptance criteria of tests (e), (f), (g) at a later stage.

D.4 Planning of tests

D.4.1 General

(1) Prior to the carrying out of tests, a test plan should be agreed with the testing organisation and other relevant parties, if any.

NOTE Other relevant parties, if any, can be described in the National Annex for use in a country.

(2) The contents of the test plan should cover:

— objectives and scope of tests;

— influencing parameters and potential failure modes;

— specification of test specimens and sampling;

— testing arrangement;

— loading specifications;

— details of measurements;

— method of evaluation;

— method of reporting of test results;

— the standard or commonly accepted procedure for the particular test.

D.4.2 Objectives and scope

(1) The objective of the tests should be clearly stated and should include aspects mentioned in D.4.1. In addition, the parameters to be varied during the test and the intended range of validity of the test results should be described. Limitations of the test and required conversions, e.g. scaling effects, should be specified.

D.4.3 Influencing parameters and potential failure modes

(1) All properties and circumstances that can influence the results of the tests performed to check the theoretical predictions should be taken into account, including:

— geometrical properties and their variability;

— geometrical imperfections;

— material properties;

— parameters influenced by fabrication and execution procedures;

— ambient environmental conditions;

— scale effects;

— relevant, any sequencing related to fabrication and testing.

(2) The expected modes of failure and/or calculation models, together with the corresponding variables should be described. If more than one failure mode might be critical, then the test plan should be preceded by pilot tests intended to identify the critical one.

NOTE Attention needs to be given to the fact that a structural member can possess several different failure modes.

D.4.4 Specification of test specimens and sampling

(1) Test specimens should be specified, or obtained by sampling, in such a way as to represent the conditions of the real structure.

NOTE Factors to take into account include:

— dimensions and tolerances;

— material and fabrication of prototypes;

— number of test specimens;

— sampling procedures;

— restraints.

(2) The objective of the sampling procedure should be to obtain a statistically representative sample.

(3) Any differences between the test specimens and the product population that could influence the test results should be noted and considered.

D.4.5 Testing arrangement

(1) The test equipment should be relevant for the type of tests and the expected range of measurements. Special attention should be given to measures to obtain sufficient strength and stiffness of the loading and supporting rigs, and clearance for deflections, etc.

D.4.6 Loading specifications

(1) The loading and environmental conditions to be specified for the test should include:

— loading points;

— loading history;

— restraints;

— temperatures;

— relative humidity;

— loading by deformation or force control, etc.

(2) Load sequencing should be selected to represent the anticipated use of the structural member, under both normal and severe conditions of use. Interactions between the structural response and the apparatus used to apply the load should be taken into account where relevant.

(3) Where structural behaviour depends upon the effects of one or more actions that do not be vary systematically, then those actions or effects should be specified by their representative values.

D.4.7 Details of measurements

(1) Prior to the testing, all relevant properties to be measured for each individual test specimen should be listed.

(2) Additionally, a list should be made of:

— measurement-locations,

— of procedures for recording results, including if relevant:

  — time histories of displacements;

  — velocities;

  — accelerations;

  — strains;

  — forces and pressures;

  — frequency of measurement;

  — accuracy of measurements;

  — appropriate measuring devices.

(3) Measurement devices should be calibrated prior to tests, be sufficiently sensitive to the data being acquired and provide sufficient accuracy.

(4) The data acquisition system should be able to record all data at the required frequency.

D.4.8 Method of evaluation

NOTE For specific guidance, see D.5 to D.8.

D.4.9 Method of reporting test results

(1) Any standards on which the tests are based should be reported.

(2) Where agreed with relevant parties, sufficient data that could enable an independent assessment should be provided within the report.

(3) The results presented should demonstrate the achievement of the objectives of the tests.

D.5 Derivation of characteristic or design values

(1) The derivation from tests of the design values for a material property, a model parameter or a resistance should be carried out according to one of the following methods:

— Method A: by assessing a characteristic value, which is then divided by a partial factor and possibly multiplied, if necessary, by an explicit conversion factor, see D.7.2 and D.8.2; or

— Method B: by direct determination of the design value, implicitly or explicitly accounting for the conversion of results and the total reliability required, see D.7.3 and D.8.3.

NOTE In general, method a) is preferred provided the value of the partial factor is determined from the normal design procedure, see (3).

(2) The derivation of a characteristic value using Method A should take into account:

— the scatter of test data;

— statistical uncertainty associated with the number of tests;

— prior statistical knowledge.

(3) The partial factor to be applied to a characteristic value should be taken from the appropriate Eurocode provided there is sufficient similarity between the tests and the usual field of application of the partial factor as used in numerical verifications.

(4) If the response of the structure or structural member or the resistance of the material depends on influences not sufficiently covered by the tests such as:

— time and duration effects;

— scale and size effects;

— different environmental, loading and boundary conditions;

— resistance effects;

then the calculation model should take such influences into account, as appropriate.

(5) Where method B is used, the following should be taken into account when determining design values:

— the relevant limit states;

— the required level of reliability;

— compatibility with the assumptions relevant to the actions side;

— where appropriate, the required design service life;

— prior knowledge from similar situations.

NOTE Further information can be found in D.6, D.7 and D.8.

D.6 General principles for statistical evaluations

(1) When evaluating test results, the behaviour of test specimens and failure modes should be compared with theoretical predictions and previous similar experience.

(2) If significant deviations from a prediction occur, an explanation should be sought and additional testing under different conditions, or modification of the theoretical model, should be considered.

(3) The evaluation of test results should be based on statistical methods, with the use of available (statistical) information about the type of distribution to be used and its associated parameters.

(4) The methods given in this Annex may be used when the following conditions are satisfied:

— the statistical data (including prior information) are taken from identified populations which are sufficiently homogeneous;

— a sufficient number of observations is available.

NOTE At the level of interpretation of tests results, three main categories can be distinguished:

— Where one test only, or very few tests, is/are performed, no classical statistical interpretation is possible. Only the use of extensive prior information, associated with hypotheses about the relative degrees of importance of this information and of the test results, makes it possible to present a statistical interpretation using Bayesian procedures, see ISO 12491;

— If a larger series of tests is performed to evaluate a parameter, a classical statistical interpretation might be possible. The commoner cases are treated, as examples, in D.6. This interpretation still needs to use some prior information about the parameter; however, the amount of information required is normally less than for the previous situation;

— When a series of tests is carried out in order to calibrate a model, as described in terms of a function, and one or more associated parameters, a classical statistical interpretation is possible.

(5) The result of a test evaluation should be considered valid only for the specifications and load characteristics considered in the tests.

(6) If the results are to be extrapolated to cover other design parameters and loading, additional information from previous tests or on a theoretical basis should be used.

D.7 Statistical determination of a single property

D.7.1 General

(1) The working Formulae given in this subclause D.7 should be used to derive characteristic or design values from test types (a) and (b) of D.3(1) for a single property using evaluation methods A and B of D.5(1).

(2) The single property X may represent either:

— a resistance of a product; or

— a property contributing to the resistance of a product.

(3) For method A, the procedures given in D.7.2 and D.7.3 may be applied directly to determine characteristic or design values or the values of partial factors.

NOTE 1 The tables and Formulae in D.7.2 and D.7.3 are based on the following assumptions:

— all variables follow either a Normal or a Log-Normal distribution;

— there is no prior knowledge about the mean value;

— for the case "V_X unknown", there is no prior knowledge about the coefficient of variation;

— for the case "V_X known", there is full knowledge of the coefficient of variation.

NOTE 2 Adopting a log-normal distribution for certain variables has the advantage that no negative values can occur as, for example, for geometrical and resistance variables.

NOTE 3 In practice, it is often preferable to use the case "V_X known" together with a conservative upper estimate of V_X, rather than to apply the rules given for the case "V_X unknown". Moreover V_X, when unknown, is assumed to be not smaller than 0,10.

(4) For Method B, the design value of the resistance should also include:

— the effects of other properties;

— the model uncertainty;

— other effects, such as due to scaling, volume, etc.

D.7.2 Assessment via the characteristic value

(1) The design value X_d of a property X should be determined as given in Formula (D.1):

$$X_d = \eta_d \frac{X_{k(n)}}{\gamma_M} = \frac{\eta_d}{\gamma_M} m_X \{1 - k_n V_X\} \tag{D.1}$$

where

η_d is the design value of the conversion factor.

NOTE The assessment of the relevant conversion factor is strongly dependent on the type of test and the type of material.

(2) The value of k_n may be taken from Table D.1.

(3) When using Table D.1, one of two cases should be considered:

— Case 1: The row "*VX* known" should be used if the coefficient of variation VX, or a realistic upper bound of it, is known from prior knowledge;

NOTE Prior knowledge might come from the evaluation of previous tests in comparable situations. Engineering judgment can be used to determine what can be considered as 'comparable', see D.7.1(3).

— Case 2: The row "V_X unknown" should be used if the coefficient of variation V_X is not known from prior knowledge and so needs to be estimated from the sample as given in Formulae (D.2) and (D.3):

$$s_x^2 = \frac{1}{n-1}\Sigma\left(x_i - m_x\right)^2 \quad \text{(D.2)}$$

$$V_X = s_X / m_X \quad \text{(D.3)}$$

(4) The partial factor γ_m should be selected according to the field of application of the test results.

Table D.1 — Values of k_n for the 5 % characteristic value

n	1	2	3	4	5	6	8	10	20	30	∞
V_X known	2,31	2,01	1,89	1,83	1,80	1,77	1,74	1,72	1,68	1,67	1,64
V_X unknown	-	-	3,37	2,63	2,33	2,18	2,00	1,92	1,76	1,73	1,64

NOTE 1 Table D.1 is based on the Normal distribution.

NOTE 2 With a log-normal distribution Formula (D.1) becomes:

$$X_d = \frac{\eta_d}{\gamma_m}\exp\left[m_y - k_n s_y\right]$$

where

$$m_y = \frac{1}{n}\Sigma\ln(x_i)$$

If V_X is known from prior knowledge: $s_y = \sqrt{\ln(V_X^2 + 1)} \approx V_X$.

If V_X is unknown from prior knowledge: $s_y = \sqrt{\frac{1}{n-1}\Sigma(\ln x_i - m_y)^2}$.

D.7.3 Direct assessment of the design value for ULS verifications

(1) The design value X_d for a basic variable *X* should be determined as given in Formula (D.4):

$$X_d = \eta_d m_X\left\{1 - k_{d,n}V_X\right\} \quad \text{(D.4)}$$

In this case, η_d should cover all uncertainties not covered by the tests.

(2) $k_{d,n}$ for consequence class 1 and 2 structures should be obtained from Table D.2.

Table D.2 — Values of $k_{d,n}$ for the ULS design

n	1	2	3	4	5	6	8	10	20	30	∞
V_X known	4,36	3,77	3,56	3,44	3,37	3,33	3,27	3,23	3,16	3,13	3,04
V_X unknown	-	-	-	11,40	7,85	6,36	5,07	4,51	3,64	3,44	3,04

NOTE 1 This table is based on the assumption that the design value corresponds to a product $\alpha_R\beta$ = 0,8×3,8 = 3,04 (see Annex C) and that *X* is Normally distributed. This gives a probability of observing a lower value of about 0,1 %.

NOTE 2 With a Log-Normal distribution, Formula (D.4) becomes (D.5):

$$X_{\mathrm{d}} = \eta_{\mathrm{d}} \exp\left\lfloor m_y - k_{\mathrm{d},n} s_y \right\rfloor \tag{D.5}$$

D.8 Statistical determination of resistance models

D.8.1 General

NOTE This subclause is mainly intended to define procedures (methods) for calibrating resistance models and for deriving design values from tests of type d), see D.3(1).

(1) Prior information, knowledge or assumptions, may be used when calibrating resistance models.

(2) Design models for the derivation of resistance functions should be based on the observation of actual behaviour in tests and on theoretical considerations. The validity of a developed model should be then checked by means of a statistical interpretation of all available test data. If necessary, the design model should be adjusted until a sufficient correlation is achieved between the theoretical values and the test data.

(3) Deviation in the predictions obtained by using the design model should also be determined from the tests. This deviation should be combined with the deviations of the other variables in the resistance function in order to obtain an overall indication of deviation. The other variables to consider should include:

— deviation in material strength and stiffness;

— deviation in geometrical properties.

(4) The characteristic resistance should be determined by taking account of the deviations of all the variables.

NOTE In D.4(1) two different methods are distinguished. These methods are given in D.8.2 and D.8.3 respectively. Additionally, some possible simplifications are given in D.8.4. These methods are presented as a number of discrete steps and some assumptions regarding the test population are made and explained.

D.8.2 Standard evaluation procedure for Method A

D.8.2.1 General

(1) The standard evaluation procedure (Method A of D.5(1) may be used provided the following assumptions are satisfied:

— the resistance function is a function of a number of independent variables *X*;

— a sufficient number of test results is available;

— all relevant geometrical and material properties are measured;

— there is no correlation (statistical dependence) between the variables in the resistance function;

— all variables follow either a Normal or a Log-Normal distribution.

NOTE Adopting a Log-Normal distribution for a variable has the advantage that no negative values can occur.

(2) Steps 1 to 7 given in D.8.2.2 should be followed when using Method A.

D.8.2.2 Standard procedure

D.8.2.2.1 Step 1 - Develop a design model

(1) A design model, represented by the resistance function in Formula (D.6):

$$r_t = g_{rt}(\underline{X}) \tag{D.6}$$

should be developed for the theoretical resistance r_t of the member or structural detail considered.

(2) The resistance function should cover all relevant basic variables X that affect the resistance at the relevant limit state.

(3) All basic parameters should be measured for each test specimen i and should be available for use in the evaluation.

D.8.2.2.2 Step 2 - Compare experimental and theoretical values

(1) To form the basis of the comparison with the experimental values r_{ei} from the tests, the actual measured properties should be substituted into the resistance function and the theoretical values r_{ti} obtained for each test i.

(2) The points representing pairs of corresponding values (r_{ti}, r_{ei}) should be plotted on a diagram, as indicated in Figure D.1.

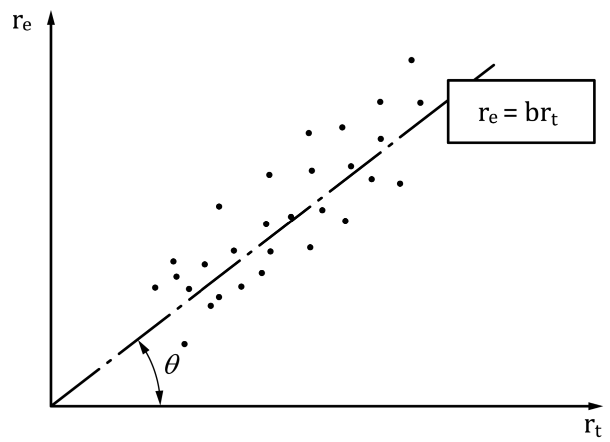


Figure D.1 — r_e - r_t diagram

(3) If all of the points lie on the line $\theta = \pi/4$ then the resistance function may be considered as exact and complete.

(4) Where the points show scatter, as could happen in practice, the causes of any systematic deviation from that line should be investigated to check whether it indicates errors in the test procedures or in the resistance function.

D.8.2.2.3 Step 3 - Estimate the mean value correction factor *b*

(1) The probabilistic model of the resistance r should be represented in the format given in Formula (D.7) and the "Least Squares" best-fit to the slope b is given by Formula (D.8):

$$r = b r_t \delta \tag{D.7}$$

$$b = \frac{\sum r_{\mathrm{e}} r_{\mathrm{t}}}{\sum r_{\mathrm{t}}^{2}} \tag{D.8}$$

where

b is the “Least Squares” best-fit to the slope;

δ is the error term.

(2) When the resistance function is linear, the mean value of the theoretical resistance function, calculated using the mean values X_{m} of the basic variables, may be obtained from Formula (D.9):

$$r_{\mathrm{m}} = b\, r_{\mathrm{t}}(\underline{X}_{m})\delta = b\, g_{\mathrm{rt}}(\underline{X}_{m})\delta \tag{D.9}$$

D.8.2.2.4 Step 4 - Estimate the coefficient of variation of the errors

(1) The error term δ_i for each experimental value $r_{\mathrm{e}i}$ should be determined from Formula (D.10):

$$\delta_i = \frac{r_{\mathrm{e}i}}{b r_{\mathrm{t}i}} \tag{D.10}$$

(2) From the values of δ_i an estimated value for V_δ should be obtained from Formula (D.11):

$$\Delta_i = \ln(\delta_i) \tag{D.11}$$

(3) The estimated value $\overline{\Delta}$ for $E(\Delta)$ should be obtained from Formula (D.12):

$$\overline{\Delta} = \frac{1}{n} \sum_{i=1}^{n} \Delta_i \tag{D.12}$$

(4) The estimated value $s_\Delta{}^2$ for $\sigma_\Delta{}^2$ should be obtained from Formula (D.13):

$$s_{\Delta}^{2} = \frac{1}{n-1} \sum_{i=1}^{n} \left(\Delta_i - \overline{\Delta}\right)^2 \tag{D.13}$$

(5) Formula (D.14) may be used as the coefficient of variation V_δ of the δ_{i} error terms:

$$V_{\delta} = \sqrt{\exp(s_{\Delta}^{2}) - 1} \tag{D.14}$$

D.8.2.2.5 Step 5 - Analyse compatibility

(1) The compatibility of the test population with the assumptions made in the resistance function should be assessed.

(2) If the scatter of the ($r_{\mathrm{e}i}$, $r_{\mathrm{t}i}$) values is too high to give economical design resistance functions, this scatter should be reduced in one of the following ways:

— by correcting the design model to take into account parameters which had previously been ignored;

— by modifying b and V_δ by dividing the total test population into appropriate sub-sets for which the influence of such additional parameters may be considered to be constant.

(3) To determine which parameters have most influence on the scatter, the test results may be split into subsets with respect to these parameters.

NOTE The purpose is to improve the resistance function per subset by analysing each subset using the standard procedure. The disadvantage of splitting the test results into subsets is that the number of test results in each subset can become very small.

(4) When determining the fractile factors k_n, see step 7, the k_n value for the subsets may be determined on the basis of the total number of the tests in the original series.

NOTE Attention is drawn to the fact that the frequency distribution for resistance can be better described by a bi-modal or a multi-modal function. Special approximation techniques can be used to transform these functions into a uni-modal distribution.

D.8.2.2.6 Step 6 - Determine the coefficients of variation V_{X_i} of the basic variables

(1) If it can be shown that the test population is fully representative of reality, then the coefficients of variation V_{X_i} of the basic variables in the resistance function may be determined from the test data.

(2) Where this is not the case, the coefficients of variation V_{X_i} should be determined on the basis of some prior knowledge.

NOTE Usually the test population is not fully representative of the reality, requiring the use of prior knowledge, where available.

D.8.2.2.7 Step 7 - Determine the characteristic value r_k of the resistance

(1) If the resistance function for j basic variables is a product function of the form $r = b\, r_t\delta = b\, \{X_1 \times X_2 \dots X_j\}\delta$ the mean value $E(r)$ may be obtained from Formula (D.15):

$$E(r) = b\left\{E(X_1) \times E(X_2) \dots E\left(X_j\right)\right\} \quad b\, g_{\mathrm{rt}}(X_{\mathrm{m}}) \tag{D.15}$$

and the coefficient of variation V_r may be obtained from the product function in Formula (D.16):

$$V_r^2 = (V_\delta^2 + 1)\left[\prod_{i=1}^{j} (V_{X_i}^2 + 1)\right] - 1 \tag{D.16}$$

(2) Alternatively, for small values of V_δ^2 and $V_{X_i}^2$ the following approximation in Formula (D.17) for V_r may be used:

$$V_r^2 = V_\delta^2 + V_{rt}^2 \tag{D.17}$$

with $V_{X_i}^2$ as given in Formula (D.18):

$$V_{rt}^2 = \sum_{i=1}^{j} V_{X_i}^2 \tag{D.18}$$

(3) If the resistance function is a more complex function of the form $r = b r_t\delta = b g_{\mathrm{rt}}(X_1, \dots, X_j)\delta$ the mean value $E(r)$ may be obtained from Formula (D.19):

$$E(r) \quad b g_{\mathrm{rt}}\left(E(X_1), \dots, E\left(X_j\right)\right) = b\, g_{\mathrm{rt}}(X_{\mathrm{m}}) \tag{D.19}$$

and the coefficient of variation V_{rt} may be obtained from Formula (D.20):

$$V_{\mathrm{rt}}^2 = \frac{VAR\left[g_{\mathrm{rt}}(\underline{X})\right]}{g_{rt}^2(\underline{X}_{\mathrm{m}})} \cong \frac{1}{g_{rt}^2(\underline{X}_{\mathrm{m}})} \times \sum_{i=1}^{j}\left(\frac{\partial g_{\mathrm{rt}}}{\partial X_i}\sigma_i\right)^2 \tag{D.20}$$

(4) If the number of tests is limited (say $n < 100$) an allowance should be made in the distribution of Δ for statistical uncertainties. Then, the distribution should be considered as a central t-distribution with the parameters $\overline{\Delta}$, V_Δ and n.

(5) In the case of (4) above, the characteristic resistance r_{k} should be obtained from Formulae (D.21) to (D.26):

$$r_{\mathrm{k}} = b\,g_{\mathrm{rt}}\left(X_{\mathrm{m}}\right)\exp\left(-\mathrm{k}_{\infty}\alpha_{\mathrm{rt}}Q_{\mathrm{rt}} - \mathrm{k}_n\alpha_\Delta \mathrm{Q}_\Delta - 0{,}5\mathrm{Q}^2\right) \tag{D.21}$$

with

$$Q_{\mathrm{rt}} = \sigma_{\ln(r_{\mathrm{t}})} = \sqrt{\ln(V_{\mathrm{rt}}^2 + 1)} \tag{D.22}$$

$$Q_\delta = \sigma_{\ln(\delta)} = \sqrt{\ln(V_\delta^2 + 1)} \tag{D.23}$$

$$Q = \sigma_{\ln(r)} = \sqrt{\ln(V_{\mathrm{r}}^2 + 1)} \tag{D.24}$$

$$\alpha_{rt} = \frac{Q_{rt}}{Q} \tag{D.25}$$

$$\alpha_\delta = \frac{Q_\delta}{Q} \tag{D.26}$$

where

k_n is the characteristic fractile factor from Table D.1 for the case V_X unknown;

k_∞ is the value of k_n for $n \to \infty$, $k_\infty = 1{,}64$;

α_{rt} is the weighting factor for Q_{rt};

α_δ is the weighting factor for Q_δ.

NOTE The value of V_δ is to be estimated from the test sample under consideration.

(6) If a large number of tests ($n \geq 100$) is available, the characteristic resistance r_{k} may be obtained from Formula (D.27):

$$r_{\mathrm{k}} = b\,g_{\mathrm{rt}}\left(X_{\mathrm{m}}\right)\exp\left(-k_\infty Q - 0{,}5Q^2\right) \tag{D.27}$$

D.8.3 Standard evaluation procedure for Method B

(1) When using Method B of D.5(1), the procedure given in D.8.2 should be followed, except that in Step 7 the characteristic fractile factor k_n should be replaced by the design fractile factor $k_{\mathrm{d},n} = \alpha_{\mathrm{R}}\beta = 0{,}8 \times 3{,}8 = 3{,}04$.

NOTE The above values are those commonly used to obtain the design value r_{d} of the resistance, see Annex C.

(2) For the case of a limited number of tests the design value r_d should be obtained from Formula (D.28):

$$r_d = b g_{rt}(X_m) \exp\left(-k_{d,\infty}\alpha_{rt}Q_{rt} - k_{d,n}\alpha_\delta Q_\delta - 0{,}5\,Q^2\right) \tag{D.28}$$

where

$k_{d,n}$ is the design fractile factor from Table D.2 for the case "V_X unknown";

$k_{d,\infty}$ is the value of $k_{d,n}$ for $n \to \infty$, $k_{d,\infty} = 3{,}04$.

NOTE The value of V_δ is estimated from the test sample under consideration.

(3) For the case of a large number of tests the design value r_d may be obtained from Formula (D.29):

$$r_d = b g_{rt}(X_m) \exp\left(-k_{d,\infty}Q - 0{,}5Q^2\right) \tag{D.29}$$

D.8.4 Use of additional prior knowledge

(1) If the validity of the resistance function r_t and an upper bound (conservative estimate) for the coefficient of variation V_r are already known from a significant number of previous tests, the following simplified procedure may be adopted when further tests are carried out.

(2) If only one further test is carried out, the characteristic value r_k may be determined from the result r_e of this test by applying Formula (D.30):

$$r_k = \eta_k r_e \tag{D.30}$$

where

η_k is a reduction factor applicable in the case of prior knowledge that may be obtained from Formula (D.31):

$$\eta_k = 0{,}9 \exp\left(-2{,}31V_r - 0{,}5V_r^2\right) \tag{D.31}$$

where

V_r is the maximum coefficient of variation observed in previous tests.

(3) If two or three further tests are carried out, the characteristic value r_k may be determined from the mean value r_{em} of the test results by applying Formula (D.32):

$$r_k = \eta_k r_{em} \tag{D.32}$$

where

η_k is a reduction factor applicable in the case of prior knowledge that may be obtained from Formula (D.33):

$$\eta_k = \exp\left(-2{,}0\,V_r - 0{,}5V_r^2\right) \tag{D.33}$$

where

V_r is the maximum coefficient of variation observed in previous tests provided that each extreme (maximum or minimum) value r_{ee} satisfies the condition in Formula (D.34):

$$|r_{ee} - r_{em}| \leq 0{,}10\ r_{em} \tag{D.34}$$

(4) The values of the coefficient of variation V_r given in Table D.3 may be assumed for the types of failure specified in the relevant Eurocode, leading to the listed values of η_k according to Formulae (D.31) and (D.33).

Table D.3 — Reduction factor η_k

Coefficient of variation V_r	**Reduction factor η_k**	
	For 1 test	**For 2 or 3 tests**
0,05	0,80	0,90
0,11	0,70	0,80
0,17	0,60	0,70

Annex E
(informative)

Additional guidance for enhancing the robustness of buildings and bridges

E.1 Use of this informative Annex

(1) This informative Annex provides additional guidance to that given in 4.4, for enhancing the robustness of buildings and bridges.

NOTE 1 National choice on the application of this informative annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

NOTE 2 Although the guidance contained in this Annex is for buildings and bridges, its principles can be applied to other types of structures.

E.2 Scope and field of application

(1) The aim of enhancing the robustness in accordance with this Annex is either to prevent disproportionate consequences as a result of hazardous events such as the failure or collapse of a structural member or part of a structure, or to provide some additional structural resistance to reduce the likelihood and extent of such an event.

(2) Design for identified accidental actions should be undertaken in accordance with EN 1991 and other relevant Eurocodes.

NOTE 1 An identified accidental action is one that is possible to occur during the design service life of the structure and against which a structure is explicitly designed. Such an action is considered known, either in terms of its relevant statistics or a specified design value.

NOTE 2 The distinction between designing for robustness in accordance with this Annex and designing for identified accidental actions in accordance with EN 1991, which also contributes to the structural performance in terms of robustness (see Note 3), is shown in Table E.1.

NOTE 3 In the case of design for identified accidental actions in accordance with EN 1991 a target level of reliability is expected to be achieved. Whilst the strategies and methods given in this Annex enhance structural performance regarding robustness, they are not generally associated with a target level of reliability as in structural member design against identified actions and could involve consideration of a conditional reliability.

Table E.1 — Strategies for designing for identified accidental actions and for enhanced robustness

Design for accidental actions (EN 1991) Explicit design of the structure (e.g. against explosion, impact)		**Design for enhanced robustness (EN 1990)** Strategies based on limiting the extent of damage		
Design structure to resist the action[a]	Prevent or reduce the action e.g. protective measures, control of events	Alternative load paths either providing adequate deformation capacity and ductility or applying prescriptive design rules	Key elements i.e. designing selected members to resist notional action(s)	Segmentation i.e. separation into parts
[a] Structural design against identified accidental actions can incorporate specifically designed members, which fail partially or fully, provided their failure does not lead to further structural collapse as agreed with the authorities (for strategies and methods to limit the extent of damages, see E.3 and E.4).				

E.3 Design strategies

(1) Strategies for designing structures for robustness may be selected from the following (see Table E.1):

a) Creation of alternative load paths:

— by providing sufficient ductility, deformation capacity and redundancy to the structure; and/or

— applying prescriptive design rules, such as for tying;

b) Key members: Provision of increased resistance in selected structural members;

c) Segmentation: Separation of the structure into distinct parts by means of one or more weaker structural members so that each part is able to collapse independently without affecting the safety of the other parts.

(2) Strategies for designing for robustness are not mutually exclusive and may be used singly or in combination.

NOTE 1 Enhanced redundancy is suitable for preventing vertically propagating collapse while segmentation is suitable for preventing horizontally propagating collapse. In a vertically propagating collapse, the failure of a member or part of a structure would give rise to further collapse that propagates above or above and below it. In horizontally propagating collapse, the failure of a member or part of a structure would give rise to further collapse that propagates in a lateral direction.

NOTE 2 Vertical segmentation into parts can be a suitable strategy for structures with a large footprint.

E.4 Design methods

(1) Robustness verifications arising from the methods given in this Annex should be considered as accidental design situations, unless specified otherwise.

(2) The design method for providing enhanced robustness may be selected based on the consequence class (CC) of the structure, see Table E.2.

Table E.2 — Indicative design methods for enhancing robustness

Consequence class	Design methods
CC3	When specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties, satisfy the requirements for CC2 appropriately adapted and in addition consider: a) potential initial failure events; b) propagation of failure; c) resulting consequences; d) risks, where appropriate.
CC2	When specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties, either: a) For buildings: use of prescriptive design rules for horizontal ties to provide integrity and ductility; or b) For buildings: use of prescriptive design rules for horizontal and vertical ties to provide integrity, ductility and alternative load paths; or c) Design of particular structural members as 'Key members'; and/or d) Segmentation.
CC1	No design methods to provide enhanced robustness need be applied.

NOTE 1 Prescriptive design rules are given in other Eurocodes.

NOTE 2 Whether only horizontal ties are sufficient or both horizontal and vertical ties are required depends on the building being considered and the potential consequences of its failure.

NOTE 3 Values of actions for designing horizontal and vertical ties can be found in EN 1991-1-7 and other Eurocodes.

NOTE 4 Design methods given for higher consequence class structures can be used with lower consequence class structures. For some special structures such use of higher level methods may be specified as required in the relevant material Eurocodes.

(3) As an alternative to the application of prescriptive design rules to provide alternative load paths, one or more of the other design strategies given in E.3(1) may be used via relevant design methods.

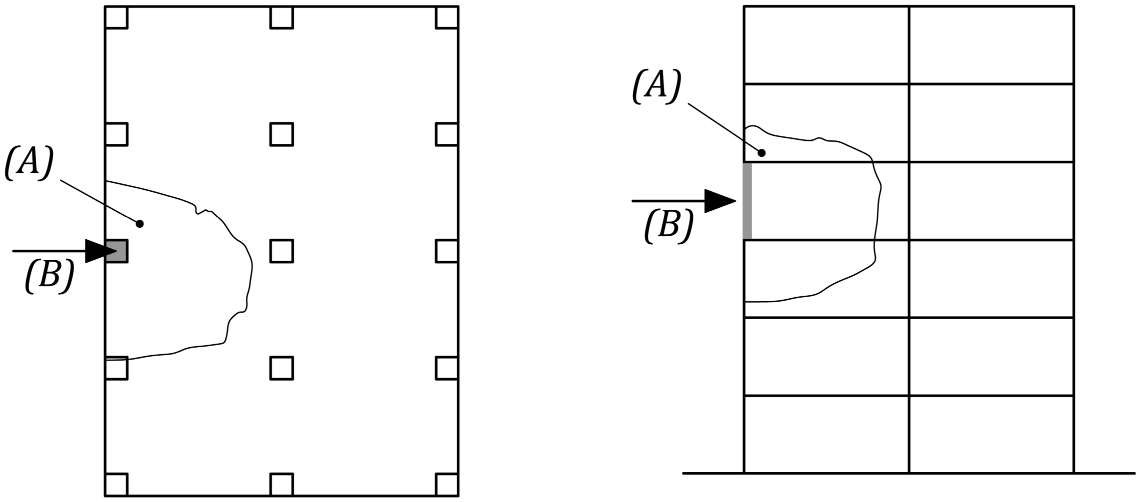
NOTE 1 Reasons for using alternatives to prescriptive designs rules include situations where they are unsuitable for a particular structure or where alternative approaches are more economically advantageous.

NOTE 2 Prescriptive design rules given in EN 1991-1-7 and other Eurocodes are generally applicable for buildings with a regular form. Special consideration is necessary for structures with an irregular form, for example, those with beams or columns that are not aligned with each other, or with mega columns or transfer beams.

(4) When the key member design strategy is used, key members should be identified as those whose absence would result in damage that is greater than a tolerable limit.

NOTE 1 Tolerable damage limits can be specified in the National Annex for use in a country.

NOTE 2 An example of an acceptable limit of damage in case of the absence of a column in a frame structure is shown in Figure E.1.



a) Plan b) Section

Key

(A) is 15 % of the floor area, or 100 m², whichever is smaller, in each of two adjacent storeys

(B) is the column notionally removed

Figure E.1 — Example of a tolerable limit to structural damage (A) on the removal of a load-bearing member of frame building

(5) The design of key members may either use a minimum notional action, applied as an accidental action, or increased partial factors in persistent or transient design situations, as specified by the relevant authorities or, where not specified, agreed for a specific project by the relevant parties.

(6) Where a notional action is used for the design of key members it should be applied in all physically possible principal directions, one at a time (see also EN 1991-1-7).

(7) Where increased partial factors are used for the design of key members account should be taken of the possibility that actions could be applied in a different direction from those upon which the member design is based.

Annex F
(informative)

Rain-flow and reservoir counting methods for the determination of stress ranges due to high-cycle fatigue

F.1Use of this informative Annex

(1) This informative Annex provides guidance on rain-flow and reservoir counting methods for the determination of stress ranges due to high-cycle fatigue.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

F.2Scope and field of application

(1) This informative annex applies to structures subjected to high-cycle fatigue.

F.3Rain-flow counting method

(1) The rain-flow counting method consists in the following steps:

a) Consider the stress history in Figure F.1; cut it at its absolute maximum and move the left part so obtained at the end of the diagram, in such a way that the modified diagram is bounded by two absolute maxima, see Figure F.2.

b) Sort the relative maxima of the stress history in descending order (1, 2,…, 6) and the relative minima in ascending order (1’, 2’, …,6’).

c) Assume that the gravity is parallel to the *t* axis and consider the modified stress history as a guide for water drops falling from peaks and valleys.

d) Release drops from peaks in descending order (1, 2, ..6) and from valleys in increasing order of the stress history itself: the path of each drop on a dry part of the guide identifies a semicycle and its extension on the σ(t) axis ordinates stress range of the semycicle; i.e. each drop path ends when the drop itself falls down or when it encounters a wet part (see Figure F.3)); (at the end of the process the whole guide is wet and cycles are obtained coupling semicyles).

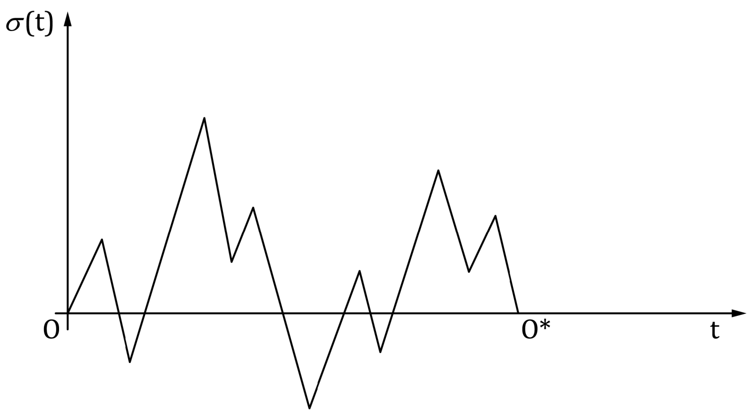


Figure F.1 — Rainflow method - Stress history

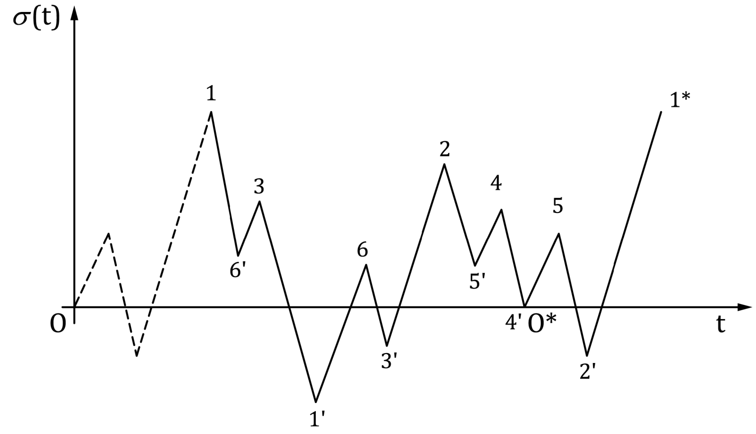


Figure F.2 — Rainflow method - Modified stress history

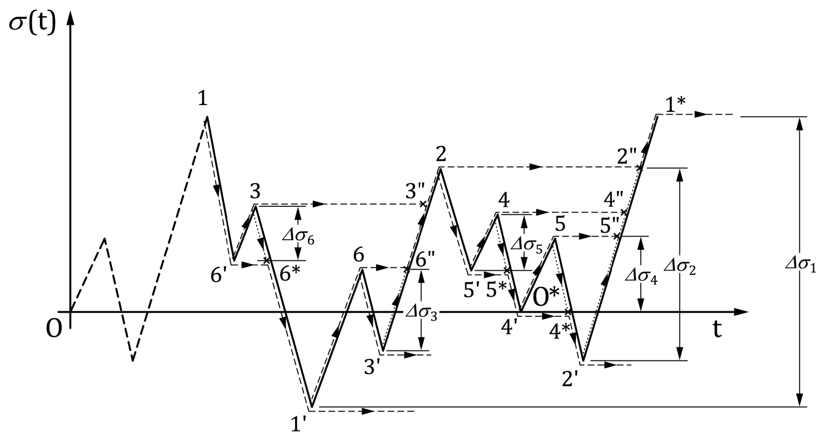


Figure F.3 — Example of rainflow method

F.4Reservoir counting method

(1) The reservoir counting method consists in the followingsteps:

a) Consider the stress history in Figure F.4; cut it at its absolute maximum and move the left part so obtained at the end of the diagram, in such a way that the modified diagram is bounded by two absolute maxima, see Figure F.5.

b) Sort the relative minima of the stress history in ascending order (1’, 2’, …,6’).

c) Assume that the modified stress history is the bottom of a water reservoir.

d) Void the reservoir from the valleys in ascending order (1’, 2’, …, 6’) till the reservoir is empty (see c)); (each discharging operation corresponds to one cycle and the height of the water discharged is the stress range). See Figure F.6.

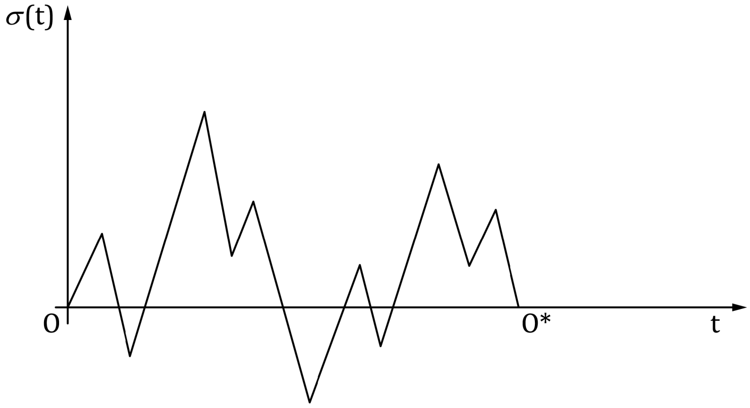


Figure F.4 — Reservoir method - Stress history

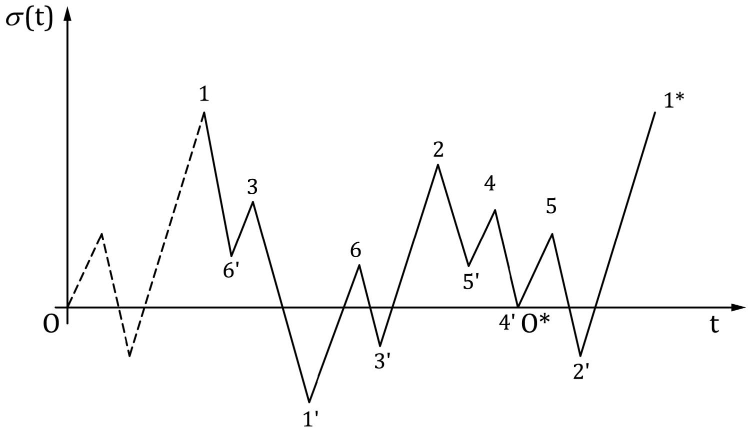


Figure F.5 — Reservoir method - Modified stress history

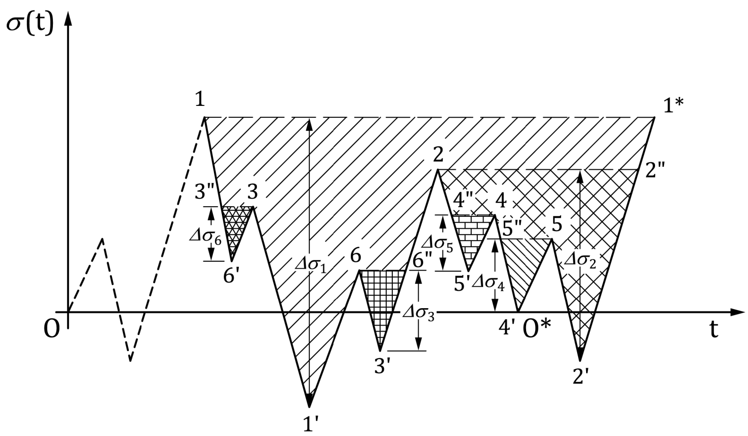


Figure F.6 — Example of reservoir method

Annex G
(normative)

Basis of design for bearings[3)]

3) Annex G will be included in a subsequent amendment.

Annex H
(informative)

Verifications concerning vibration of footbridges due to pedestrian traffic[4)]

4) Annex H will be included in a subsequent amendment.

Bibliography

ISO 2394, *General principles on reliability for structures*

ISO 3898:2013, *Bases for design of structures - Names and symbols of physical quantities and generic quantities*

ISO 10137, *Bases for design of structures - Serviceability of buildings and walkways against vibrations*

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