EUROPEAN STANDARD

NORME EUROPÉENNE

EUROPÄISCHE NORM

FINAL DRAFT
FprEN 1990

October 2022

ICS 91.010.30 Will supersede EN 1990:2002, EN 1997-1:2004

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 formal vote. 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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Recipients of this draft are invited to submit, with their comments, notification of any relevant patent rights of which they are aware and to provide supporting documentation.

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

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European foreword

This document (FprEN 1990:2022) 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 Formal Vote.

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

In comparison with the previous edition, the following main changes have been made:

— extension of scope to include provisions for bearings;

— improved approach for ULS verification;

— improved provisions on robustness;

— improved provisions on fatigue verification;

— improved provisions for basis of design for geotechnical structures in alignment with EN 1997;

— inclusion of provisions for sustainability;

— improved guidance on reliability analysis and code calibration;

— improved guidance for SLS verification of buildings related to deflection limits, vibrations and foundation movements;

— improved guidance on management of structural reliability of construction works;

— inclusion of guidance on verification of vibration of footbridges due to pedestrian traffic.

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.

0 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>

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 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 FprEN 1990

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

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 FprEN 1990

National choice is allowed in this document where explicitly stated within notes. National choice includes the selection of values for Nationally Determined Parameters (NDPs).

The national standard implementing FprEN 1990 can have a National Annex containing all national choices to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

When no national choice is given, the default choice given in this document is to be used.

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

National choice is allowed in FprEN 1990 through notes to the following:

4.2(3)	4.3(1)	4.4(2)	4.7(1)
6.1.3.2(4) – 3 choices	6.1.3.2(6)	7.1.5(7)	8.3.2.1(4)
8.3.3.1(5)	8.3.3.6(1)	8.3.4.2(2) – 2 choices	A.1.3(1)
A.1.4(1)	A.1.6.1(1) – 3 choices	A.1.6.1(2) – 2 choices	A.1.6.2(1)
A.1.6.3(1)	A.1.6.3(2)	A.1.7(1) – 2 choices	A.1.8.1(1)
A.1.8.2.2(2)	A.1.8.2.3(2)	A.1.8.3(1)	A.1.8.3(3)
A.1.8.3(4)	A.1.8.4(2)	A.1.8.4(4) – 3 choices	A.2.3(1)
A.2.4(1)	A.2.7.1(1) – 3 choices	A.2.7.3.6(1)	A.2.7.4.1(1) – 2 choices
A.2.7.4.3(1)	A.2.7.4.5(1)	A.2.7.4.6(1) – 2 choices	A.2.7.5.1(1)
A.2.7.5.3(1)	A.2.7.5.4(1) – 2 choices	A.2.7.6.1(1)	A.2.7.6.4(1)
A.2.7.10(5) – 2 choices	A.2.7.10(9)	A.2.8(1) – 3 choices	A.2.9.1(1)
A.2.9.3.1(5)	A.2.9.3.3(1)	A.2.9.3.3(3)	A.2.9.3.3(4)
A.2.9.4.1(1) – 2 choices	A.2.9.4.2.1(3)	A.2.9.4.2.2(4)	A.2.9.4.2.2(5)
A.2.9.4.2.3(1)	A.2.9.4.2.3(2)	A.2.9.4.2.4(2) – 2 choices	A.2.9.4.2.4(4)
A.2.9.5(1)	A.2.10(1)	A.2.11.1(9)	A.2.11.4.5(3)
A.2.11.4.7(1)	B.2(1)	B.4(2)	B.5(1)
B.6(1)	B.6(2)	B.7(1)	B.8(1)
C.3.1(5)	C.3.4.2(3)	D.4.1(1)	E.4(4)
G.2(1)	G.3.1(6)	G.3.3.2(1)	G.3.3.2(2)
G.3.4(2)	G.3.4(3)	G.6(2)	G.7.1.2(2)
G.7.1.3(2)	G.7.3.2(2)	G.7.4.2(1)	G.7.5.1(1)
G.7.5.2(1) – 2 choices			

National choice is allowed in FprEN 1990 on the application of the following informative annexes:

Annex B	Annex C	Annex D	Annex E
Annex F	Annex H		

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 FprEN 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) The verification methods 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 assessment of existing structures;

— developing the design of repairs, improvements and alterations;

— assessing changes of use.

NOTE Additional or amended provisions can be necessary.

(6) This document is applicable for the design of structures where materials or actions outside the scope of EN 1991 (all parts) to EN 1999 (all parts) 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 in accordance with 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.

NOTE See the Bibliography for a list of other documents cited that are not normative references, including those referenced as recommendations (i.e. in 'should' clauses), permissions ('may' clauses), possibilities ('can' clauses), and in notes.

EN 1337-3, *Structural bearings - Part 3: Elastomeric bearings*

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

prEN 1991-2:2021, *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges and other civil engineering works*

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.

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, to meet the safety, serviceability and durability requirements

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 6707-1:2020, 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
elements other than structural
completion and finishing elements connected with the structure that are not classified as structural members and that has the lowest consequence of failure

Note 1 to entry: See 4.3 for the classification of consequences of failure.

EXAMPLE Roofing; surfacing and coverings; partitions and linings; kerbs; wall cladding; suspended ceilings; thermal insulation; bridge furniture, road surfacing; services fixed permanently to, or within, the structure such as equipments for lifts and moving stairways; heating, ventilating and air conditioning equipment; electrical equipment; pipes; cable trunking and conduits.

3.1.1.8
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.

3.1.1.9
quality
degree to which a set of inherent characteristics of an object fulfils requirements

Note 1 to entry: The term “quality” can be used with adjectives such as poor, good or excellent.

Note 2 to entry: “Inherent”, as opposed to “assigned”, means existing in the object.

[SOURCE: EN ISO 9000:2015, 3.6.2]

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 expected to 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.

3.1.2.5
fundamental design situation
design situation that is either a persistent or a transient design situation

3.1.2.6
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.

3.1.2.7
seismic design situation
design situation in which the structure is subjected to a seismic event

3.1.2.8
fatigue design situation
design situation where fatigue actions may cause fatigue failure

Note 1 to entry: For some materials, a distinction applies between low and high cycle fatigue. The other Eurocodes give guidance, where relevant.

3.1.2.9
verification case
classification of load cases for fundamental design situations in ultimate limit states, for which a set of partial factors is defined

3.1.2.10
fire design
design of a structure to fulfil the required performance in case of fire

3.1.2.11
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

3.1.2.12
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

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 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
performance 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
fatigue
damaging process caused by cyclic actions or actions inducing cyclic effects that may culminate in failure

3.1.2.23
excessive deformation
deformation that exceeds limits to such an extent that the structure can be considered to have reached an ultimate limit state

3.1.2.24
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 covers safety, serviceability and durability of a structure.

3.1.2.25
reliability differentiation
measures intended for the socio-economic optimization 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

3.1.2.26
basic variable
variable representing a physical quantity that characterizes actions and environmental influences, geometrical quantities, and material properties, including ground properties

3.1.2.27
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.

3.1.2.28
repair
activities, beyond the definition of maintenance, performed to preserve or to restore the function of a structure

3.1.2.29
nominal value
value fixed on a non-statistical basis; for instance, on acquired experience or on physical conditions

3.1.2.30
robustness
ability of a structure to withstand unforeseen adverse events without being damaged to an extent disproportionate to the original cause

3.1.2.31
durability
ability of a structure or structural member to satisfy, with planned maintenance, its design performance requirements over the design service life

3.1.2.32
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

3.1.2.33
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

3.1.2.34
gross human error
error resulting from ignorance or oversight that causes a change in the behaviour or a reduction in reliability of the structure that are unacceptable

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

EXAMPLE Imposed deformations or accelerations caused 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
F_{fat}
cyclic action or action inducing cyclic effects that can cause fatigue

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
climatic action
action arising from the climatic system

EXAMPLE Snow, wind, temperature variation, atmospheric icing, and humidity.

Note 1 to entry: Waves and currents are considered as an effect of climatic actions and other influences on water.

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.

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

Note 1 to entry: If a characteristic value cannot be fixed on a statistical basis, it can be replaced by a nominal value.

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
return period
average number of years in which a stated action statistically is exceeded once

3.1.3.23
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

3.1.3.24
leading action
principal action in a combination

3.1.3.25
accompanying action
action that accompanies the leading action in a combination

3.1.3.26
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. $Q_{\mathrm{comb}} = \psi_0 Q_{\mathrm{k}}$) where $\psi_0 \leq 1$.

3.1.3.27
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. $Q_{\mathrm{freq}} = \psi_1 Q_{\mathrm{k}}$), where $\psi_1 \leq 1$.

3.1.3.28
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 $\psi_2 \leq 1$.

3.1.3.29
natural frequency
frequency of free oscillation of a structure

3.1.3.30
dynamic amplification factor
ratio between the characteristic value of the effect of the dynamic action and the characteristic value of the effect of the static action

Note 1 to entry: The definition applies also for other representative values.

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: For specific rules, see the other Eurocodes.

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 within 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

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

EXAMPLE Examples of structures where geometric non-linearity can occur are membranes, cables, flat arches, catenaries, slender columns and slender 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

EXAMPLE 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

EXAMPLE 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

3.1.7 Terms relating to bridges

NOTE 1 Definitions, symbols, notations, load models and groups of loads for traffic loads are those used or defined in the relevant section of prEN 1991-2.

NOTE 2 Symbols, notations and models of actions during execution are those defined in EN 1991-1-6.

3.1.7.1
integral abutment bridge
bridge without joints in the deck that accommodates thermal expansion and contraction by movement of the abutments in and out of the backfill

3.1.7.2
support system
arrangement of supports that transfer the loads from the superstructures (e.g. bridge, overpass) to the substructures (e.g. piers, abutments) and allow movements for specified degrees of freedom

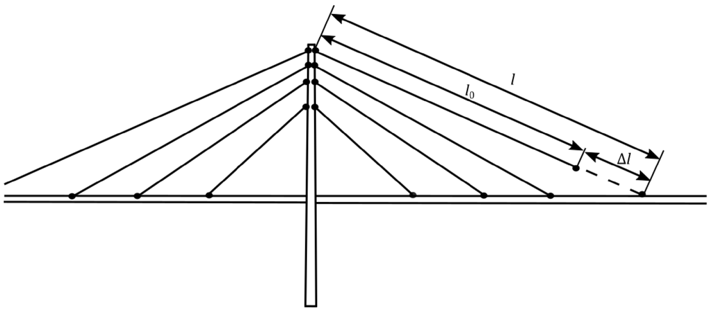
3.1.7.3
notional fixed point
point where thermal expansion, or other free or partially free strains of the superstructure do not generate global movements

3.1.7.4
expansion length
distance from notional fixed point to location in question

3.1.7.5
prestrain
prestressing force caused by controlled deformations imposed to a tension component

Note 1 to entry: Contrary to the axial force, the prestrain is independent of the loads on the structure. It changes only when the adjustment of the stay is modified. The prestrain can be applied step by step, depending on the construction process. Due to the deflection during construction, the distance between terminations in the completed structure can be different from the distance between terminations at the time of installation.

Note 2 to entry: The prestrain of a stay cable is characterized by its pre-deformation (see Figure 3.1) or the reference tension F_{ref} that is to be applied to the stay to force its ends to coincide with the position of the terminations in the reference geometry. If the stay is modelled with a linear behaviour, the reference tension is proportional to the predeformation: $F_{ref} = EA_m\varepsilon$ where A_m is the nominal metallic cross-section of the tension element. If the stay is modelled as a catenary, it is more convenient to use the reference tension. As the prestrain, the reference force does not depend on the loads applied on the structure. It can differ from the stay force at the time of stay installation.



Key
l distance between terminations in the reference geometry
l_0 length of the cable without tension and gravity
ε predeformation, $\varepsilon = \Delta l / l_0$

Figure 3.1 — Prestrain for stay cables in cable supported bridges

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

$A_{Ed,ULS}$ Design value of seismic action in an ultimate limit state

$A_{Ed,SLS}$ Design value of seismic action in a serviceability limit state

$A_{eff,i}$ Effective plan area for layer i of elastomeric bearing

A_m Nominal metallic cross-section of the tension element

$A_{w,rep}$ Representative value of accidental water action

$C_{d,SLS}$ Limiting design value of the relevant serviceability criterion

$C_{d,ULS}$ Limiting design value for ultimate limit state of excessive deformation

CC Consequence class

CL Comfort level

D_{int} Limiting value for characteristic movement at integral abutment

DL Damage limitation

E Effect of actions

$E(X)$ Mean value of X

E_d Design value of effect of actions

F Action

F_{bd} Design value of bearing force

F_d Design value of an action

F_{Ed} Design value of actions used in assessment of E_d

F_{fat} Fatigue action

$F_{fat,d}$ Design value of a fatigue action

F_k Characteristic value of an action

F_{ref} Reference tension

F_{rep} Representative value of an action

F_w^* Limiting value for accompanying wind force for road bridges

F_w^{**} Limiting value for accompanying wind force for railway bridges

F_{wk} Wind force

F_{wk}^* Limiting value for wind force

G Permanent action

G_d Design value of a permanent action

G_{de} Design shear modulus of elastomer

$G_{de,i}$ Design value of shear modulus of elastomer for layer i

$G_{d,fav}$ Design value of a permanent action that produces a favourable effect

G_{exp}	Tolerance for shear modulus
G_{k}	Characteristic value of a permanent action
$G_{\text{k},i}$	Characteristic value of a permanent action *i*
$G_{\text{k,inf}}$	Lower characteristic value of a permanent action
G_{nom}	Nominal shear modulus
$G_{\text{k,sup}}$	Upper 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
$G_{\text{wk,inf}}$	Lower characteristic value of water action
$G_{\text{w,rep}}$	Representative value of water action
$G_{\text{wk,sup}}$	Upper characteristic value of water action
HSLM	High speed load model
LM	Load model
N_{d}	Design value of the normal forces at the bearing in the applicable combination
$N_{\text{d},i}$	Design value of normal force at sliding bearing of support *i*
$N_{\text{d},j}$	Design value of normal forces of permanent actions and prestressing at sliding bearing of support *j*
NC	Near collapse
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
$P_{\text{k,inf}}$	Lower characteristic value of a prestressing force
$P_{\text{k,sup}}$	Upper characteristic value of a prestressing force
Q	Variable action
Q_{c}	Construction action
Q_{comb}	Combination value of a variable action
Q_{d}	Design value of a variable action
Q_{freq}	Frequent value of a variable action

Q_{fwk}	Characteristic value of the concentrated load (wheel load) on a footbridge
Q_{k}	Characteristic value of a variable action
$Q_{\text{k},j}$	Characteristic value of the accompanying variable action j
$Q_{\text{k},1}$	Characteristic value of the leading variable action 1
Q_{Lk}	Horizontal force resulting from acceleration and braking
Q_{qper}	Quasi-permanent value of a variable action
Q_{rep}	Representative value of a variable action
$Q_{\text{Sn,k}}$	Characteristic value of snow load
$Q_{\text{w,comb}}$	Combination value of variation in water action
$Q_{\text{w,freq}}$	Frequent value of variation in water action
$Q_{\text{w,qper}}$	Quasi-permanent value of variation in water action
$Q_{\text{w,rep}}$	Representative value of variation in water action
Q_{wk}	Characteristic value of variation in water action
R	Resistance
R_{d}	Design value of the resistance
R_{fat}	Representative value of the fatigue resistance
$R_{\text{fat,d}}$	Design value of the fatigue resistance
SD	Significant damage
SLS	Serviceability limit state
T_{fi}	Duration of fire exposure
T_{k}	Thermal action
T_{lf}	Design service life
T_0	Initial temperature
TS	Tandem system
UDL	Uniformly distributed load
ULS	Ultimate limit state
V	Coefficient of variation, V = (standard deviation)/(mean value)
V_{T}	Train speed
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
$X_{\mathrm{k}}(n)$	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_{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_{bt}	Limiting value for bridge deck acceleration for ballasted track
a_{d}	Design value of a geometrical property
a_{df}	Limiting value for bridge deck acceleration for ballastless track
$a_{\mathrm{lim,v}}$	Limiting value for acceleration in vertical direction
$a_{\mathrm{lim,h}}$	Limiting value for acceleration in horizontal direction
a_{nom}	Nominal value of a geometrical property
b	Correction factor
b_i	Correction factor for test specimen i
b_{v}	Limiting value of vertical acceleration
b_{v}'	Maximum permissible vertical acceleration
$d_{\mathrm{d,con}}$	Design contraction movement of a bearing
$d_{\mathrm{d,exp}}$	Design expansion movement of a bearing
$d_{\mathrm{execution}}$	Movements of bearings during execution before the structure becomes restrained
d_{Gd}	Design movement due to permanent effects
d_{Pd}	Design movement due to prestressing
$d_{\mathrm{Qd,con}}$	Design contraction movement due to variable actions
$d_{\mathrm{Qd,exp}}$	Design expansion movement due to variable actions
$d_{\mathrm{Set},i}$	Difference in settlement of an individual foundations or part of a foundation
f_{h0}	Limitation value for fundamental natural frequency of lateral vibration of a span
$\mathrm{g}_{\mathrm{rt}}(\underline{X})$	Theoretical resistance function, of the basic variables $\underline{X}$, used as the design model
h	Height of building
h_i	Storey height

i_{d}	Design value of a geometrical imperfection
$k_{\mathrm{d},n}$	Design fractile factor for a sample size n
k_{dyn}	Dynamic factor
k_{F}	Consequence factor
k_n	Characteristic fractile factor for a sample size n
k_{temp}	Temperature factor
k_{φ}	Factor for reducing geometric imperfections
l	Span
m_X	Mean value of the variable X from n sample results
n	Number of a quantity
$q_{1\mathrm{k}}$	Action due to aerodynamic effects
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}
$r_{\mathrm{e},i}$	Experimental resistance for specimen i
r_{em}	Mean value of the experimental resistance
r_{k}	Characteristic value of the resistance
r_{m}	Mean value of the resistance calculated using the mean values X_{m} of the basic variables
r_{t}	Theoretical resistance determined from the resistance function $\mathrm{g_{rt}}\left(\underline{X}\right)$
$r_{\mathrm{t},i}$	Theoretical resistance determined using the measured parameters $\underline{X}$ for specimen i
r_{tr}	Change of radius of a track across a deck
s	Estimated value of the standard deviation σ
s_{Cd}	Differential settlement
s_{g}	Track gauge
s_{Δ}	Estimated value of σ_{Δ}
s_{δ}	Estimated value of σ_{δ}
t	Maximum twist
$t_{\mathrm{el;eff},i}$	Effective thickness of elastomer layer i
t_{T}	Limiting value for total track twist

u	Horizontal displacement of a structure or structural member
$u_{d,i}$	Design value of relative displacement of elastomer layer i
u_i	Relative horizontal displacement over a storey height excluding rigid body rotation
x_i	the i-th measurement of the variable X
w	Vertical deflection of a structural member
w_c	Precamber
w_{max}	Remaining total deflection taking into account precamber
w_{tot}	Total deflection
w_1	Initial part of deflection under permanent loads
w_2	Long-term part of deflection under permanent loads including quasi-permanent loads
w_3	Instantaneous deflection due to variable actions excluding quasi-permanent loads

3.2.3 Greek upper-case letters

Φ	Cumulative distribution function of the standardised Normal distribution
Φ_{dyn}	Dynamic factor
Δ	Logarithm of the error term δ, $\Delta_i = \ln(\delta_i)$
$\bar{\Delta}$	Estimated value for $E(\Delta)$
Δa	Deviation in a geometrical property
Δd	Geometric uncertainty
$\Delta d_{Set,i}$	Uncertainty attached to the assessment of the settlement of a foundation or part of a foundation
$\Delta s_{Cd,SLS}$	Maximum differential settlement
$\Delta T_{d,con}$	Characteristic value of maximum fall of a uniform temperature component resulting in contraction
$\Delta T_{d,exp}$	Characteristic value of maximum rise of a uniform temperature component resulting in expansion
$\Delta T_{N,con}$	Characteristic value of maximum fall of a uniform temperature component resulting in contraction
$\Delta T_{N,exp}$	Characteristic value of maximum rise of a uniform temperature component resulting in expansion
ΔT_0	Range of initial bridge temperature
$\Delta \delta_h$	Maximum differential transverse deflection

3.2.4 Greek lower-case letters

α	Adjustment factor

α_{Qi}	Adjustment factors of some load models on lanes i (i = 1, 2...)
α_n	Factor depending on the type of bearing and number of bearings
α_{qi}	Adjustment factors of some load models on lanes i (i = 1, 2...)
α_{qr}	Adjustment factor of load models on the remaining area
$\alpha_{Q_{i,\max,T}}$	FORM (First Order Reliability Method) sensitivity factor of Q_i dominating (i = 1, 2)
$\alpha_{Q_{i,\tau_1}}$	FORM (First Order Reliability Method) sensitivity factor of Q_i not dominating (i = 1, 2)
α_Y	Sensitivity factor indicating the importance of Y in the reliability estimation
β	Reliability index
β_{Cd}	Angular distortion
$\beta_{Cd,SLS}$	Maximum angular distortion
β_Q	Adjustment factor of load model 2
γ	Partial factor
γ_E	Partial factor applied to the effects of actions accounting for the uncertainties covered by γ_f and γ_{Sd}
$\gamma_{E,fav}$	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_{Sd}
γ_{Ff}	Partial factor for fatigue actions
γ_G	Partial factor for a permanent action that produces unfavourable effects
$\gamma_{G,fav}$	Partial factor for a permanent action that produces favourable effects
$\gamma_{G,i}$	Partial factor for permanent action i
$\gamma_{G,set}$	Partial factor for the effects of settlement
$\gamma_{G,set,fav}$	Partial factor for the effects of settlement that produces favourable effects
$\gamma_{G,stb}$	Partial factor for the favourable (stabilizing) part of a permanent action treated as a single-source
γ_{Gw}	Partial factor for water actions
$\gamma_{Gw,stb}$	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}

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}
$\gamma_{Q,fav}$	Partial factor applied to variable actions that produce favourable effects
$\gamma_{Q,j}$	Partial factor for variable action j
$\gamma_{Q,red}$	Reduced partial factor applied to unfavourable variable actions in geotechnical design
$\gamma_{Q,T}$	Partial factor for thermal actions
γ_{Qw}	Partial factor for variation in water actions
γ_{R}	Partial factor for the resistance 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 for the resistance 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 $r_{e,i}$ and its theoretical resistance $br_{t,i}$ corrected using correction factor for corresponding mean values (b_{m})
δ_{v}	Vertical deflection
δ_{h}	Transverse deflection
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
θ	End rotation of structure
μ_{d}	Design value of the bearing friction coefficient
$\mu_{d,fav}$	Favourable coefficient of friction
μ_{max}	Maximum coefficient of friction
ξ	Reduction factor applied to unfavourable permanent actions
ρ	Reduction factor applied to γ_{G} when deriving $\gamma_{G,stb}$
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_{j}	Reduction coefficient taking into account the probability that the footfall frequency (jogging) approaches the critical range of natural frequencies under consideration
ψ_{W}	Reduction coefficient taking into account the probability that the footfall frequency (walking) approaches the critical range of natural frequencies under consideration
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

4 General rules

4.1 Basic requirements

(1) The assumptions given in 1.2 and in the other Eurocodes shall be met.

(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 reasonably foreseeable actions and influences that can occur during its execution and use, as specified for the structure;

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

— meet the specified durability requirements for the structure or a structural member.

NOTE Design carried out in accordance with the Eurocodes is assumed to satisfy these requirements.

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

NOTE For general provisions related to fire design, see also EN 1991-1-2.

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 (all parts): EN 1991, EN 1992, EN 1993, EN 1994, EN 1995, EN 1996, EN 1997, EN 1998 and EN 1999.

(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;

NOTE 1 Examples of modes of attaining a limit state are failure modes with or without warnings, e.g. ductile or brittle failure.

— public aversion to failure;

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

NOTE 2 Reliability levels can be set by the National Annex. Further guidance is given in Annex C.

NOTE 3 Different levels of reliability are commonly adopted for limit states relating to structural failure, serviceability, and durability.

NOTE 4 Levels of reliability for structural failure, serviceability and durability 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 for further guidance;

— 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 – high consequence;

— CC2 – normal consequence;

— CC1 – low 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.

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 – High	High	Very great
CC2 – Normal	Medium	Considerable
CC1 – Low	Low	Small
CC0 – Lowest	Very low	Insignificant
[a] The consequence class is chosen based on the more severe of these two columns.		

NOTE 2 The provisions in Eurocodes cover design rules for structures classified as CC1 to CC3.

NOTE 3 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.

NOTE 4 Annex A gives examples of the classification of structures into consequence classes.

NOTE 5 The consequence class is used to determine the value of consequence factor k_{F}, see Annex A.

NOTE 6 The consequence class can be used to determine the management measures to achieve the intended structural reliability, see Annex B for further guidance.

NOTE 7 The consequence class can be used to modify the failure probability levels P_{f} or reliability indices β, see Annex C for further guidance.

NOTE 8 The consequence class can be used in the direct assessment of the design values for ULS verifications, see Annex D for further guidance.

NOTE 9 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, provided that consistency when specifying the consequence classes is ensured for the design of the structures as a whole.

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

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 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 is assumed to provide 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, as agreed for a specific project by the relevant parties.

NOTE 1 Guidance on additional design measures to enhance structural robustness for buildings and bridges is given in Annex E.

NOTE 2 Further guidance can be given in the National Annex.

4.5 Design service life

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

NOTE Values of T_{lf} 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 For the verification of durability, see 4.6. For the verification of fatigue, see 8.3.5.4.

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 inspection and 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 as to enable assessment of their impact on durability and to allow adequate provisions for protection of the materials used in the structure to be made.

(4) The degree of any deterioration affecting the resistance of a structure or a structural member 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, as agreed for a specific project by the relevant parties.

NOTE 1 The adverse impact of a structure on its environment, on society, and on economy can be minimized by for example appropriate choice of construction process and environmentally compatible building materials, including their manufacture, design solutions, durability, recyclability, and reusability.

NOTE 2 Supplementary requirements to account for sustainability in the design can be given in the National Annex.

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 for design, execution, use, and maintenance;

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

NOTE For guidance on appropriate quality management measures, see Annex B and the other Eurocodes.

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 particular limit state 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 all 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, extreme sea water level, fire, explosion, or impact; or local failure
Seismic	Exceptional conditions during a seismic event	During an earthquake
Fatigue	Conditions caused by fatigue actions	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.

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 the relevant part of EN 1997.

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

NOTE 4 Details of ultimate limit states for seismic design situations are given in EN 1998 (all parts).

(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(3) and 8.3.3.1.

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;

  NOTE 1 The functioning of the structure includes the functioning of machines and 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 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, geotechnical models and loading models

(1) Limit states shall be verified using appropriate structural models, geotechnical models and loading models.

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

— actions;

— material and product properties;

— geometrical properties.

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

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

(5) As an alternative to (4), design based on probabilistic methods may be used when specified by the relevant authority or, where not specified, as 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}).

NOTE 1 Climatic actions, such as wind and snow actions, can be classified as either variable or accidental, depending on site location. See EN 1991 (all parts).

NOTE 2 Actions arising from elements other than structural can be classified as either permanent or variable. See EN 1991-1-1.

NOTE 3 For fatigue actions, see 6.1.3.3.

(2) 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.

(3) 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, including self-weight from different materials, as well as for water pressures acting on opposite sides of a structure with flow passing around or underneath.

(4) Climatic actions 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 1 Representative values are not defined for accidental and seismic actions, nor for bearing forces.

NOTE 2 For the definition of design values, see Clause 8.

(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 (all parts), the relevant part of EN 1997 and EN 1998 (all parts).

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 loss of static equilibrium or uplift; or

— it is not greater than 10 %, otherwise.

NOTE For the assessment of the coefficient of variation of permanent actions from the ground, see the relevant part of EN 1997.

(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 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 $G_{k,sup}$ and $G_{k,inf}$ respectively.

NOTE Permanent actions are usually assumed to be normally distributed.

(6) 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 the relevant part of EN 1997 for the specification of $G_{k,sup}$ and $G_{k,inf}$.

NOTE 2 For the specification of permanent water actions, see 6.1.3.2.

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. For other actions than climatic actions another fractile and/or return period can be more appropriate.

NOTE 3 For actions from waves and currents on coastal structures, see Clause A.6.

NOTE 4 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):

$$Q_{\mathrm{comb}} = \psi_0 Q_{\mathrm{k}} \tag{6.1}$$

$$Q_{\mathrm{freq}} = \psi_1 Q_{\mathrm{k}} \tag{6.2}$$

$$Q_{\mathrm{qper}} = \psi_2 Q_{\mathrm{k}} \tag{6.3}$$

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

NOTE 2 The characteristic value and combination value are used in the verification of ultimate limit states and irreversible serviceability limit states.

NOTE 3 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 fraction of time it is exceeded is 1 %. For road traffic loads on bridges, the frequent values are assessed on the basis of a return period of one week.

NOTE 4 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. The quasi-permanent value is chosen so that the fraction of time it is exceeded is 50 %. 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 For the specification of variable water actions, see 6.1.3.2.

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 (prestressing) or deformations (prestrain) to a structure or to a tension component should be classified as permanent actions.

NOTE Prestressing forces acting on a structure or on a tension component can arise from prestressing tendons, imposed deformations at supports, prestrain applied to tension components, etc.

(2) A prestressing force P should be represented by its upper or lower characteristic value, $P_{\mathrm{k,sup}}$ or $P_{\mathrm{k,inf}}$ 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 6.1.3.2 is not applicable to water actions induced by currents and waves and to actions for hydraulic structures with fast-flowing water, e.g. in rivers. For actions for coastal structures induced by currents and waves, see Annex A 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 (all parts) and the relevant part of 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 a 2 % probability that it is exceeded during a one-year reference period, unless the National Annex gives a different value.

NOTE 2 The value of $Q_{\mathrm{w,freq}}$ is chosen so that the fraction of time it is exceeded is 1 %, unless the National Annex gives a different value.

NOTE 3 The value of $Q_{\mathrm{w,qper}}$ is chosen so that the fraction of time it is exceeded is 50 %, unless the National Annex gives a different value.

NOTE 4 For seismic verifications involving water actions, see EN 1998-5.

(5) The values of Q_{wk} and $Q_{\mathrm{w,comb}}$ 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 $A_{\mathrm{w,rep}}$ equal to the representative value of the water action.

NOTE The value of $A_{\mathrm{w,rep}}$ is based on a 0,1 % probability that it is exceeded during a one-year reference period, unless the National Annex gives a different value.

6.1.3.3 Fatigue actions

(1) Models for fatigue actions shall be defined considering the magnitude and the number of repetitions of the actions during the design service life.

NOTE The action repetitions relevant for fatigue depend on the type of structure and the type of material.

(2) When relevant, interaction between the action and the structure shall be taken into account, either in the action or in the effect of action.

(3) The models for fatigue actions in EN 1991 (all parts) may be used for the description of fatigue actions.

NOTE The models for fatigue actions in EN 1991 (all parts) include the dynamic action effect, either:

— implicitly for those models for which this is specified in EN 1991 (all parts); or

— explicitly by applying dynamic amplification factors to fatigue actions.

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

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

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

NOTE For further guidance, see Annex F.

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

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 For the specification of characteristic values of ground properties, see the relevant part of EN 1997.

(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 should 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, or when a material property is sensitive to the duration of the effects of actions.

NOTE 1 For values of the conversion factor, see the other Eurocodes.

NOTE 2 Further guidance is given in Annex D.

(6) When available statistical data is considered insufficient 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, or when nominal values are selected, their values should be chosen to achieve a level of structural reliability no less than that specified in the Eurocodes.

NOTE For guidance on structural reliability, see Annex C.

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) The effect of degradation on geometrical properties (e.g. due to corrosion) should be taken into account.

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

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 and geotechnical models involving relevant variables and relevant boundary conditions.

NOTE 1 Such calculations can be used to model potential failure modes, predict ultimate capacity, or model deformations and vibrations (provided the results can be verified with satisfactory accuracy).

NOTE 2 For geotechnical analysis, see EN 1997-1.

(2) Structural and geotechnical models shall be based on well-founded engineering theory and practice.

(3) Structural models should be validated to establish whether the model reproduces the physical phenomena to be investigated with an acceptable level of accuracy.

NOTE Depending on the physical phenomena being investigated, validation tests can include basic material tests, physical reference tests and/or comparisons with other structural models that have been already validated.

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, a dynamic structural analysis of the system should be performed.

NOTE 1 Guidance on the need for dynamic structural analysis is given in EN 1991 (all parts).

NOTE 2 For seismic actions, see EN 1998 (all parts).

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

(3) Where relevant, effects of actions may be determined 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 effects of actions are wind induced vibrations or seismic action effects.

(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 of the action or by applying an equivalent dynamic amplification factor to the quasi-static action.

NOTE 1 Quasi-static actions are defined in EN 1991 (all parts).

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

7.1.4 Actions inducing fatigue

(1) The structural models used with fatigue actions should allow for fatigue action effects to be determined taking into account relevant material specific aspects.

NOTE Relevant material specific aspects are defined by the other Eurocodes and can be for example:

— mean stress effects;

— load sequence effects;

— stress state.

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 For guidance on selecting design fire scenarios, see EN 1991-1-2.

(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-dependent 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_{fi}.

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

7.2 Structural 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, in accordance with 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 (contact non-linearity), material behaviour (material non-linearity), and geometry (geometric non-linearity) or non-linearity of the limit state function.

(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, to establish whether the numerical model correctly reproduces the necessary physical phenomena.

NOTE Depending on the physical phenomena being investigated, validation 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 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, as agreed for a specific project by the relevant parties.

NOTE Physical 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 determine the fatigue resistance;

— 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 physical testing and related statistical uncertainty is given in Annex D. See also the other Eurocode parts.

(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.

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 For verification of ultimate limit states, see 8.3. For verification of serviceability limit states, see 8.4.

(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, in accordance with 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, together with the rules given in Clause 8 as appropriate, 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:

$$E_d \leq R_d \tag{8.1}$$

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:

$$E_d \leq C_{d,ULS} \tag{8.2}$$

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

$C_{d,ULS}$ 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.

(3) The design values of effects of actions and strength parameters to be used in verification of Formula (8.2) shall be obtained such that the required reliability level for ultimate limit states is obtained.

NOTE For guidance on the selection of $C_{d,ULS}$, see the other Eurocodes.

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):

$$E_d = \gamma_{Sd} E\left\{\Sigma\left(\gamma_f \psi F_k\right); a_d; X_{Rd}\right\} \tag{8.3}$$

where

γ_{Sd} is a partial factor associated with the uncertainty of the action and/or action effect model;

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

$\Sigma(...)$ 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 ($\gamma_F = \gamma_f \cdot \gamma_{Sd}$) or on effects of actions ($\gamma_E = \gamma_f \cdot \gamma_{Sd}$).

NOTE Although the formulations of γ_F and γ_E are identical, because of the simplifications made and the use in (3) and (4), 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 and non-linear structural systems;

— certain types of geotechnical structure, in accordance with the relevant part of 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:

— certain types of geotechnical structure, in accordance with the relevant part of EN 1997;

— ropes, cables and membrane structures, where the application of partial factors on the effects of actions is more adverse than the application of partial factors on actions.

NOTE 1 A simplified version of Formula (8.3) with partial factors applied to effects of actions is given in 8.3.2.3.

NOTE 2 The National Annex can define additional cases where partial factors are applied to effects of actions (γ_E).

8.3.2.2 Partial factors on actions

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

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

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 For the details of combinations of actions when applying partial factors to actions, see 8.3.4.

8.3.2.3 Partial factors on effects of actions

(1) When applying partial factors to the effects of actions, their design value E_d should be calculated from Formula (8.5):

$$E_d = \gamma_E E\left\{\Sigma F_{rep}; a_d; X_{rep}\right\} = \gamma_E E\left\{\Sigma\left(\psi F_k\right); a_d; X_{rep}\right\} \quad (8.5)$$

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 partial factors on actions (γ_F).

NOTE Further guidance can be found in the other Eurocodes.

(3) In geotechnical design, γ_E may be taken as γ_G provided that an additional reduced partial factor $\gamma_{Q,red}$ is applied to unfavourable variable actions.

(4) In geotechnical design, γ_E may be applied separately to unfavourable and favourable components of effects of actions in cases when the single source principle is not applied.

8.3.2.4 Fatigue

(1) Unless otherwise stated by the other Eurocodes, linear response may be assumed for determining the effects of fatigue actions.

(2) The fatigue actions should be supplemented by additional specifications concerning fatigue resistance, 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 consideration of material specific effects, see the relevant Eurocodes.

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

NOTE Effects of actions can be expressed as a spectrum, a histogram, or a table, or any other kind of representation expressing the effects of actions and the number of repetitions.

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):

$$G_d = \gamma_G \times G_k \quad (8.6)$$

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 ($G_{k,sup}$). See 6.1.2.2 for further guidance.

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

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

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

$\gamma_{G,fav}$ 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 ($G_{k,inf}$). 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.

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), 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.

EXAMPLE The self-weight of the structure or ground can produce simultaneously both unfavourable and favourable effects. For simplicity, 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 loss of static equilibrium 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 that part of the permanent action that causes an unfavourable (destabilizing) effect should be calculated from Formula (8.6), and the design value of that part of the permanent action that causes a favourable (stabilizing) effect should be calculated from Formula (8.7), replacing $\gamma_{G,fav}$ with the partial factor $\gamma_{G,stb}$ given by Formula (8.8):

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

where

ρ is a reduction factor.

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

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):

$$P_d = \gamma_P \times P_k \tag{8.9}$$

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):

$$Q_d = \gamma_Q \times Q_{rep} \tag{8.10}$$

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 For the specification of accidental actions, see EN 1991 (all parts), and in particular EN 1991-1-7.

NOTE 2 For the specification of accidental water actions, see 6.1.3.2.

8.3.3.5 Seismic actions

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

8.3.3.6 Fatigue actions

(1) The design value of a fatigue action $F_{fat,d}$ should be calculated from Formula (8.11):

$$F_{fat,d} = \gamma_{Ff} F_{fat} \tag{8.11}$$

where

F_{fat} is the fatigue action;

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

NOTE 1 The value of γ_{Ff} depends on the models for fatigue actions and on the models for the effects of the fatigue actions.

NOTE 2 The National Annex gives the partial factors for models for fatigue actions. The value of γ_{Ff} is 1,0 when the models for fatigue actions are safesided such that uncertainties of the action effects up to 10 % are covered. Otherwise, the value is 1,1.

8.3.3.7 Bearing actions

(1) The design value of a bearing action shall be determined according to Annex G.

8.3.3.8 Partial factors

(1) The values of partial factors for actions should be chosen in accordance with Annex A.

NOTE 1 For general application and for buildings, see Clause A.1. For bridges, see Clause A.2. For towers, masts and chimneys, see Clause A.3[1)]. For silos and tanks, see Clause A.4[1)]. For structures supporting cranes, see Clause A.5[1)]. For coastal structures, see Clause A.6[1)]

NOTE 2 For partial factors for fatigue actions, see 8.3.3.6.

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 and the other Eurocodes.

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

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

— accidental design situations, see 8.3.4.3;

— seismic design situations, see 8.3.4.4;

— fatigue design situations, see 8.3.4.5.

NOTE For application rules, see Annex A.

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

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

— the design value of the leading variable action;

— the design combination values of accompanying variable actions.

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

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

— Formula (8.12); or

— the most adverse of the two expressions in Formula (8.13); or

— the most adverse of the two expressions in Formula (8.14).

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

or

$$\sum F_d = \begin{cases} \sum_i \gamma_{G,i} G_{k,i} + \gamma_{Q,1} \psi_{0,1} Q_{k,1} + \sum_{j>1} \gamma_{Q,j} \psi_{0,j} Q_{k,j} + \left(\gamma_P P_k\right) \\ \sum_i \xi_i \gamma_{G,i} G_{k,i} + \gamma_{Q,1} Q_{k,1} + \sum_{j>1} \gamma_{Q,j} \psi_{0,j} Q_{k,j} + \left(\gamma_P P_k\right) \end{cases} \tag{8.13}$$

or

$$\sum F_d = \begin{cases} \sum_i \gamma_{G,i} G_{k,i} + \left(\gamma_P P_k\right) \\ \sum_i \xi_i \gamma_{G,i} G_{k,i} + \gamma_{Q,1} Q_{k,1} + \sum_{j>1} \gamma_{Q,j} \psi_{0,j} Q_{k,j} + \left(\gamma_P P_k\right) \end{cases} \tag{8.14}$$

where

F_d	represents the design value of an action;
Σ	denotes the combination of the enclosed variables;
$\gamma_{G,i}$	is the partial factor for permanent action *i*;
$G_{k,i}$	is the characteristic value of permanent action *i*;
$\gamma_{Q,1}$	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);
$Q_{k,1}$	is the characteristic value of the leading variable action 1;
$Q_{k,j}$	is the characteristic value of an accompanying variable action *j*;
$\psi_{0,j}$	is the combination factor for the variable action *j;*
$\gamma_{Q,j}$	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.

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

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; or

— refer to a situation after an accidental event $A_d = 0$.

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

$$\sum F_d = \sum_i G_{k,i} + A_d + \left(\psi_{1,1} \text{ or } \psi_{2,1}\right) Q_{k,1} + \sum_{j>1} \psi_{2,j} Q_{k,j} + \left(P_k\right) \quad (8.15)$$

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

A_d is the design value of the accidental action, see 8.3.3.4;

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

$\psi_{2,1}$ is the combination factor applied to the leading variable action 1 to determine its quasi-permanent 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):

$$\sum F_d = \sum_i G_{k,i} + A_{\mathrm{Ed,ULS}} + \sum_j \psi_{2,j} Q_{k,j} + \left(P_k\right) \quad (8.16)$$

where, in addition 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 (all parts).

8.3.4.5 Combination of fatigue actions with other actions

(1) Unless otherwise stated in the other Eurocodes, the fatigue action does not need to be combined with other actions.

(2) If the other Eurocodes explicitly require a structural analysis under the fatigue action in combination with other actions, this combination should be calculated by Formula (8.17):

$$\sum F_d = \sum_i G_{k,i} + \sum_j \psi_{2,j} Q_{k,j} + \left(P_k\right) + F_{\mathrm{fat,d}} \quad (8.17)$$

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

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

$F_{\mathrm{fat,d}}$ is the design value of the fatigue action, see 8.3.3.6.

NOTE 1 Other Eurocodes can define additional verifications based on combination of actions in Clause 8. Such additional verifications can, for example, impose a limitation on the maximum stress or stress range.

NOTE 2 This applies if the absolute value of the effect of the action or if the mean effect of the action are of importance for the fatigue verification.

(3) Variable actions of the same type as the fatigue action should not be considered in the part $\sum_{j}\psi_{2,j}Q_{\mathrm{k},j}$.

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;

$R\{\ldots\}$ 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 properties, 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 ($\gamma_M = \gamma_m \cdot \gamma_{Rd}$) or into a single partial factor for resistance ($\gamma_R = \gamma_m \cdot \gamma_{Rd}$).

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, in accordance with the other Eurocodes;

— certain types of geotechnical structure, in accordance with the relevant part of EN 1997.

NOTE A simplified version of Formula (8.18) with partial 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, in accordance with the other Eurocodes;

— certain types of geotechnical structure, in accordance with the relevant part of 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 Partial factors on material properties (the “material factor approach”, MFA)

(1) When applying partial 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 Partial factors applied to resistance (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(2).

(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):

$$R_{\mathrm{d}} = \frac{1}{\gamma_{\mathrm{R},1}} R\left\{\eta_1 X_{\mathrm{k},1}; \frac{\eta_i X_{\mathrm{k},i}}{\gamma_{\mathrm{m},i} / \gamma_{\mathrm{m},1}}; a_{\mathrm{d}}; \Sigma F_{\mathrm{Ed}}\right\} \tag{8.21}$$

where, further to symbols already defined for Formula (8.20)

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, in accordance with the relevant part of EN 1997.

8.3.5.4 Fatigue

(1) The design value of fatigue resistance $R_{\mathrm{fat,d}}$ should be calculated from Formula (8.22):

$$R_{\mathrm{fat,d}} = \frac{R_{\mathrm{fat}}}{\gamma_{\mathrm{Mf}}} \tag{8.22}$$

where

R_{fat} is the representative value of the fatigue resistance;

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):

$$X_{\mathrm{d}} = \frac{X_{\mathrm{rep}}}{\gamma_{\mathrm{M}}} = \frac{\eta X_{\mathrm{k}}}{\gamma_{\mathrm{M}}} \tag{8.23}$$

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

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 for further guidance; or

— results of tests, see Annex D for further guidance.

NOTE Guidance on the assessment of design values of ground properties is given in the relevant part of EN 1997. Permission to use specific prescriptive measures is given in the relevant part of 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 a_{d} may be calculated from Formula (8.25):

$$a_{\mathrm{d}} = a_{\mathrm{nom}} \tag{8.25}$$

(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:

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

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

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

F_{d} represents the design values of actions, see 8.3.3, where, the value of γ_{F} = 1,0;

a_{d} represents the design values of geometrical properties, see 8.4.6;

X_{d} represents the design values of material properties, see 8.4.5.

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, see 8.4.3.2;

— for the frequent combinations, see 8.4.3.3;

— for quasi-permanent combinations, see 8.4.3.4.

NOTE 1 For application rules, see Annex A.

NOTE 2 The other Eurocodes can specify when these combinations of actions are to be used.

(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 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 Formulae (8.12) to (8.14)

$\psi_{1,1}$ 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 Reversible serviceability limit states are generally assessed using this combination of actions.

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

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 Formulae (8.12) to (8.14)

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

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 (all parts).

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, as 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 (all parts).

8.4.5 Design values of material properties

(1) Design values of material properties for serviceability limit states should be chosen in accordance with 8.3.6.

8.4.6 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 in (2).

(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 Use of this annex

(1) This subclause A.1 contains additional provisions to the general rules in Clauses 1 to 8 for the structures specified in A.1.2.

A.1.2 Scope and field of application

(1) This subclause A.1 applies to the verification by the partial factor method for the design of buildings and associated geotechnical structures.

(2) This subclause A.1 applies to the verification by the partial factor method for the design of geotechnical structures not covered by A.2 to A.6.

(3) This subclause A.1 may also be applied to the verification by the partial factor method for the design of structures not covered by A.2 to A.6.

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

(4) When a structure falls into the field of application of different parts of Annex A, these parts should be applied in conjunction.

(5) As an alternative to (4), the application of the different parts of Annex A may be agreed for a specific project by the relevant parties.

NOTE In agreeing the application of the different parts for a specific project it is important for the relevant parties not to conflict with what is specified by the relevant authority.

A.1.3 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.

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.

NOTE 4 Parts of the structure can be classified to a different consequence class than the building if the consequences of their failure are different.

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

Consequence class	Description of consequence	Examples
CC4[a]	Highest	Nuclear power plant, dams
CC3	High	Buildings or parts of buildings where a very large number of people could be affected by failure, e.g. grandstands, concert halls, high-rise buildings
CC2	Normal	Buildings or parts of buildings not covered by CC1 or CC3
CC1	Low	Buildings or part of buildings where very few people could be affected by failure, e.g. agricultural buildings, storage buildings
CC0[a]	Lowest	Elements other than structural, see 3.1.1.7.

[a] For provisions concerning CC0 and CC4, see 4.3.

A.1.4 Design service life

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

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

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

Category of buildings	Design service life T_{lf} years
Monumental building structures	100
Building structures not covered by another category	50
Agricultural, 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, shorter design service lives can apply, see the other Eurocodes.

[c] For design service life of geotechnical structures, see the relevant part of EN 1997.

A.1.5 Actions

(1) The actions, as described in 6.1, to be included in the design of structures shall be those defined by EN 1991 (all parts), the relevant part of EN 1997, and EN 1998 (all parts).

A.1.6 Combinations of actions

A.1.6.1 Ultimate limit states (ULS)

(1) Combination of actions for ultimate limit states with partial 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, see 8.3.4.2(2).

NOTE 2 As defined in 8.3.2.1, partial factors on actions are used, and Formula (8.4) applies, for the design of:

— linear and non-linear structural systems;

— certain types of geotechnical structure, in accordance with the relevant part of 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, see 8.3.4.2(2), Note 2.

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

NOTE 5 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.

(2) If design values of actions for persistent and transient (fundamental) design situations are chosen according to Table A.1.4 or Table A.1.5, then the most adverse of the two expressions in the relevant formula for combination of actions shall be verified.

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

Design situation	Persistent and transient (fundamental)	Accidental	Seismic[a]	Fatigue[b]
General formula for effects of actions	(8.4)			
Formula for combination of actions	(8.12)	(8.15)	(8.16)	(8.17)
Permanent ($G_{d,i}$)	$\gamma_{G,i}G_{k,i}$	$G_{k,i}$	$G_{k,i}$	$G_{k,i}$
Leading variable ($Q_{d,1}$)	$\gamma_{Q,1}Q_{k,1}$	$\psi_{1,1}Q_{k,1}$ or $\psi_{2,1}Q_{k,1}$	$\psi_{2,j}Q_{k,j}$	c
Accompanying variable ($Q_{d,j}$)	$\gamma_{Q,j}\psi_{0,j}Q_{k,j}$	$\psi_{2,j}Q_{k,j}$		
Prestressing (P_d)	$\gamma_P P_k$	P_k	P_k	P_k
Accidental (A_d)	-	A_d	-	-
Seismic (A_{Ed})	-	-	$A_{Ed,ULS}$	-
Fatigue (F_{fat})	-	-	-	$\gamma_{Ff}\,F_{fat}$

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

[b] For conditions of use, see 8.3.4.5.

[c] The action type of which F_{fat} is considered should not be taken into account as variable in the combination.

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

Design situation	Persistent and transient (fundamental)		Accidental	Seismic	Fatigue
General formula for effects of actions	(8.4)				
Formula for combination of actions	The upper part of (8.13)	The lower part of (8.13)	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})	-	-			
Fatigue (F_{fat})	-	-			

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

Design situation	**Persistent and transient (fundamental)**		**Accidental**	**Seismic**	**Fatigue**
General formula for effects of actions	(8.4)				
Formula for combination of actions	The upper part of (8.14)	The lower part of (8.14)	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})	-	-			
Fatigue (F_{fat})					

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

NOTE As defined in 8.3.2.1, partial factors on effects of actions are used, and Formula (8.5) applies, for the design of:

— certain types of geotechnical structure, in accordance with the relevant part of EN 1997;

— ropes, cables and membrane structures, where the application of partial factors on the effects of actions is more adverse than the application of partial factors on actions.

A.1.6.2 Serviceability limit states (SLS)

(1) Combinations of actions for serviceability limit states, for which 8.4.3 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 (all parts).

A.1.6.3 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.

(2) Foundation movements should be classified as a permanent action, G_{set}, and included in combinations of actions for ultimate and serviceability limit state verifications of the structure.

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):	0,7	0,5	0,3
Category A: domestic, residential areas	0,7	0,5	0,3
Category B: office areas	0,7	0,7	0,6
Category C: congregation areas	0,7	0,7	0,6
Category D: shopping areas	1,0	0,9	0,8
Category E: storage areas			
Category F: traffic area,	0,7	0,7	0,6
vehicle weight ≤ 30 kN	0,7	0,5	0,3
Category G: traffic area,	0,7	0	0
30 kN < vehicle weight ≤ 160 kN			
Category H: roofs accessible for normal maintenance and repair only			
Construction loads (see EN 1991-1-6)	1,0	-	-
Snow loads on buildings (see EN 1991-1-3):	0,7	0,5	0,2
— Finland, Iceland, Norway, Sweden;			
— remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.;	0,7	0,5	0,2
— remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,5	0,2	0
Wind actions 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
Water actions[a] (see 6.1.3.2)	-	-	-
Waves and currents (see EN 1991-1-8)			

[a] The combination value for water actions can be based on a 10 % probability that it is exceeded during a one-year reference period.

A.1.7 Partial factors for ultimate limit states (ULS)

(1) Ultimate limit states may 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 factors γ_F and γ_E are given in Table A.1.8 (NDP) for persistent and transient (fundamental) design situations, unless the National Annex gives different values.

NOTE 2 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.

NOTE 3 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 the lower parts of Formulae (8.13) and (8.14), 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 verification case VC1 in both structural and geotechnical design (see also (7)).

(5) When variations in the magnitude or spatial variation of permanent actions from a single source (see 6.1.1(4)) are significant, ultimate limit states that involve loss of static equilibrium and/or strength of elements contributing to the equilibrium, should be verified using partial factors for verification cases VC2(a) and VC2(b), using whichever gives the less favourable design outcome.

(6) Verification of verification case VC2(b) may be omitted when it is obvious that verification using verification case VC2(a) governs the design outcome.

(7) Ultimate limit states that involve failure of ground should be verified using partial factors for verification cases VC1, VC2, VC3 and VC4, in accordance with the relevant part of EN 1997.

NOTE The relevant part of EN 1997 gives guidance on which Verification Cases to use for different geotechnical structures.

Table A.1.8 (NDP) — Partial factors on actions and effects for verification cases VC1 to VC4 for persistent and transient (fundamental) design situations

Action or effect				Partial factors γ_F and γ_E for verification cases				
Type	**Group**	**Symbol**	**Resulting effect**	**Structural resistance**[a]	**Static equilibrium and uplift**[b]		**Geotechnical design**	
Verification case				**VC1**[a]	**VC2(a)**[b]	**VC2(b)**[b]	**VC3**[c]	**VC4**[d]
Permanent action (G_k)	All[f]	γ_G	unfavourable /destabilizing	$1{,}35k_F$	$1{,}35k_F$	1,0	1,0	G_k is not factored
	Water[l]	γ_{Gw}		$1{,}2k_F$	$1{,}2k_F$	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	
Prestressing (P_k)		γ_P[k]						
Variable action (Q_k)	All[f]	γ_Q	unfavourable	$1{,}5k_F$	$1{,}5k_F$	$1{,}5k_F$	1,3	$\gamma_{Q,red}$[j]
	Water[l]	γ_{Qw}		$1{,}35k_F$	$1{,}35k_F$	$1{,}35k_F$	1,15	1,0
	All	$\gamma_{Q,fav}$	favourable	0				
Effects of actions (E)		γ_E	unfavourable	γ_E is not applied				$1{,}35k_F$
		$\gamma_{E,fav}$	favourable					1,0

a Verification case VC1 is used both for structural and geotechnical design. Formula (8.4) is used for VC1.

b Verification case VC2 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 VC2(a) or VC2(b), whichever gives the less favourable outcome. See 8.3.3.1(5). Formula (8.4) is used for VC2.

c Verification case VC3 is typically used for the design of slopes and embankments, spread foundations, and gravity retaining structures. See the relevant part of EN 1997 for details. Formula (8.4) is used for VC3.

d Verification case VC4 is typically used for the design of transversally loaded piles and embedded retaining walls and (in some countries) gravity retaining structures. See EN 1997 (all parts) for details. Formula (8.5) is used for VC4.

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 actions.

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 $\gamma_{Q,red} = \gamma_{Q,1}/\gamma_{G,1}$ where $\gamma_{Q,1}$ = corresponding value of γ_Q from VC1 and $\gamma_{G,1}$ = corresponding value of γ_G from VC1.

k For the definition of γ_P where γ_P is materially dependent, see other relevant Eurocodes.

l For water actions induced by waves and currents, see Clause A.6.

Table A.1.9 (NDP) — Consequence factors for buildings and geotechnical structures

Consequence class (CC)[a]	Description of consequences	Consequence factor k_F
CC3	High	1,1
CC2	Normal	1,0
CC1	Low	0,9

[a] The provisions in Eurocodes cover design rules for structures classified as CC1 to CC3, see 4.3.

A.1.8 Serviceability criteria for buildings

A.1.8.1 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 Limiting values can be defined in the National Annex.

NOTE 3 Design values of serviceability criteria for non-industrial buildings, expressed independently of structural materials, are defined in A.1.8.2 for deformations. Industrial buildings include storage buildings.

NOTE 4 Design values of serviceability criteria for geotechnical structures are given in A.1.8.4.

(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.1.8.2 Vertical and horizontal deformations

A.1.8.2.1 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).

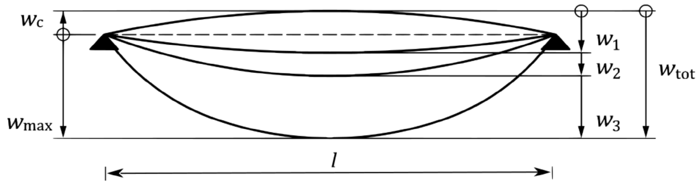
NOTE Guidance on the calculation of deformations is given in the other Eurocodes.

(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.1.8.2.2 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 the quasi-permanent part of variable actions

w_3 instantaneous deflection due to variable actions excluding their quasi-permanent parts.

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 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.

NOTE Suggested values of maximum vertical deflections for non-industrial buildings are given in Table A.1.10 (NDP), unless the National Annex gives different values.

(3) Other maximum values should be as specified by the relevant authority or, where not specified, may be agreed for a specific project by the relevant parties.

(4) 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.

(5) 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.

(6) 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

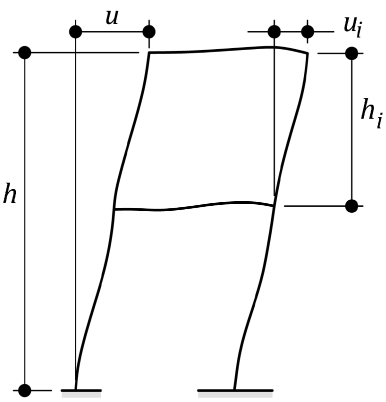
Serviceability criteria	**Limiting damage to elements other than structural**[a]	**Comfort of users**	**Appearance**
Combination of actions to be considered	**Characteristic combination** **Formula (8.29)**	**Frequent combination Formula (8.30)**	**Quasi-permanent combination Formula (8.31)**
Not accessible roof	<u>Roofing</u> rigid roofing: w_2+w_3 ≤ l/250 resilient roofing: w_2+w_3 ≤ l/125 <u>Ceiling</u> plastered ceiling: w_2+w_3 ≤ l/350 false ceiling: w_2+w_3 ≤ l/250	w_2+w_3 ≤ l/300	w_1+w_2-w_c ≤ l/250
Floor, accessible roof	<u>Internal partition walls</u> not reinforced: — partitions of brittle material or non-flexible: w_2+w_3 ≤ l/500 — partitions of non-brittle materials: w_{max} ≤ l/400 reinforced walls: w_2+w_3 ≤ l/350 removable walls: w_2+w_3 ≤ l/250 <u>Flooring:</u> — tiles rigidly fixed: w_2+w_3 ≤ l/500 — small tiles[b] or deflection not fully transmitted: w_2+w_3 ≤ l/350 — resilient flooring: w_2+w_3 ≤ l/250 <u>Ceiling:</u> plastered ceiling: w_2+w_3 ≤ l/350 false ceiling: w_2+w_3 ≤ l/250	w_2+w_3 ≤ l/300	w_1+w_2-w_c ≤ l/250
Structural frames	<u>Windows:</u> — no loose joints (no clearance between glass and frame): w_2+w_3 ≤ l/1000 — with loose joints: w_2+w_3 ≤ 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.1.8.2.3 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.

NOTE Suggested maximum horizontal displacements for non-industrial buildings are given in Table A.1.11 (NDP), unless the National Annex gives different values.

(3) Other maximum values should be as specified by the relevant authority or, where not specified, may be agreed for a specific project by the relevant parties.

Table A.1.11 (NDP) — Suggested maximum 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 combination Formula (8.29)**	**Frequent combination Formula (8.30)**	**Quasi-permanent combination Formula (8.31)**
Overall horizontal displacement u	Single-storey buildings: $u \leq h/400$ Multi-storey buildings: $u \leq h/500$	$u \leq h/250$	
Relative horizontal displacement u_i over a storey height h_i	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.1.8.3 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.

NOTE Other aspects to consider to achieve satisfactory vibration behaviour can be specified by the National Annex.

(2) Other aspects than in (1) should be as specified by the relevant authority or, where not specified, may be agreed for a specific project by the relevant parties.

(3) For the serviceability limit states of a structure or a structural member not to be exceeded when subjected to vibrations, the accelerations of vibrations of the structure or structural member should be kept below appropriate acceleration limits relevant for user comfort and functionality.

NOTE 1 Relevant structures are for example hospitals, laboratories, gymnasia and sport halls, dance rooms, concert halls, and for floors, staircases and balconies in general.

NOTE 2 For specific types of structures or structural members having typical mass and damping properties, the acceleration limits can be assumed met when the natural frequency of vibrations is kept above appropriate values. The limits and values can be set in the National Annex.

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 further guidance, see EN 1991-1-1, EN 1991-1-4, and ISO 10137.

(5) The sources of vibration should be as specified.

NOTE 1 Possible sources of vibration include walking, synchronised movements of people, machinery, ground borne vibrations from traffic, wind actions, pile driving, and placing of sheet piling walls.

NOTE 2 When specifying the sources of vibration for a specific project it is important for the relevant parties not to conflict with what is specified by the relevant authority.

A.1.8.4 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 1 Examples of buildings in different structural sensitivity classes are given in Table A.1.16 (NDP), unless the National Annex gives different examples.

NOTE 2 For definition of modes of foundation movement, see EN 1997-1.

(3) The classification should consider the ground conditions within the zone of influence of the structure.

Table A.1.12 —Classification of structural sensitivity to foundation movement

Structural sensitivity class	Description of sensitivity
SSC5	Highest
SSC4	High
SSC3	Normal
SSC2	Low
SSC1	Lowest

(4) The value of $C_{d,SLS}$ may be chosen according to the structure's sensitivity to foundation movement.

NOTE Suggested values of $C_{d,SLS}$ are given in Table A.1.13 (NDP), Table A.1.14 (NDP) and Table A.1.15 (NDP), unless the National Annex gives different values.

(5) Alternative values of $C_{d,SLS}$ should be as specified by the relevant authority or, where not specified, may be agreed for a specific project by the relevant parties.

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	High	15 mm
SSC3	Normal	30 mm
SSC2	Low	60 mm
SSC1	Lowest	100 mm
[a] For the definition of differential settlement of foundations, see EN 1997-1.		

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] $\beta_{Cd,SLS}$
SSC5	Highest	0,05 %
SSC4	High	0,075 %
SSC3	Normal	0,15 %
SSC2	Low	0,3 %
SSC1	Lowest	0,5 %
[a] For the definition of angular distortion of foundations, see EN 1997-1.		

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] $\omega_{Cd,SLS}$
SSC5	Highest	0,1 %
SSC4	High	0,2 %
SSC3	Normal	0,3 %
SSC2	Low	0,4 %
SSC1	Lowest	0,5 %
[a] For the definition of foundation tilt, see EN 1997-1.		

Table A.1.16 (NDP) — Examples of buildings in different structural sensitivity classes

Design criteria for SLS $C_{d,SLS}$	Type or use of structure	Structural sensitivity class
Differential settlement, s_{Cd}	Utility connections	SSC1
Angular distortion, β_{Cd}	Framed buildings and reinforced load-bearing walls	SSC3
	Floors, slabs	SSC1
Tilt, ω_{Cd}	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	SSC2 SSC3 SSC4 SSC5
	Lift and escalator operation	SSC5

A.2 Application for bridges

A.2.1 Use of this annex

(1) This Clause A.2 contains additional provisions to the general rules in Clauses 1 to 8 for the structures specified in A.2.2.

A.2.2 Scope and field of application

(1) This Clause A.2 applies to the design of road bridges, footbridges and railway bridges.

NOTE 1 This Clause A.2 provides the specific application of the general rules in Clauses 1 to 8 for these structures.

NOTE 2 Guidance on additional design measures to enhance structural robustness for bridges is given in Annex E.

(2) This Clause A.2 should be used to determine combinations of actions for road bridges, footbridges and railway bridges, and also to determine combinations of actions for accidental design situations and transient design situations including execution.

NOTE Most of the combination rules defined in A.2.7.3 to A.2.7.10 are simplifications intended to avoid needlessly complicated calculations.

(3) This Clause A.2 may be used as the basis to determine combinations of actions for other civil engineering structures carrying traffic actions, as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

(4) 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.

(5) The rules given in this Clause A.2 may need to be supplemented as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties for the following:

— bridges for which the actions are not covered by prEN 1991-2 (for example bridges under an airport runway, bridges carrying water, etc.);

— bridges subjected to loads outside the scope of EN 1991 (all parts) (e.g. ice pressure, flooding, mud slides, nature/animal overbridges);

— bridges carrying both road and rail traffic;

— moveable bridges;

— roofed bridges;

— military and other demountable emergency bridges; and

— assessment of existing bridges.

A.2.3 Consequence classes

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

NOTE 1 Examples of bridges in different consequence classes are given in Table A.2.1 (NDP), unless the National Annex gives different examples.

NOTE 2 Parts of the structure can be classified to a different consequence class than the bridge if the consequences of their failure are different.

Table A.2.1 (NDP) — Examples of bridges in different consequence classes

Consequence class [a]	Description of consequence	Examples
CC4[b]	Highest	
CC3b	High (upper class)	Where an increased level of reliability is required, when specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties
CC3a	High (lower class)	Railway bridges on main railway lines, bridges over main railway lines, bridges over and under major roads
CC2	Normal	Bridges not in other consequence classes
CC1	Low	Short span bridges on local roads with little traffic (provided they do not span over main railway lines or major roads)
CC0[b]	Lowest	Elements other than structural, see 3.1.1.7.

[a] CC3b corresponds to an increased level of reliability compared to CC3a.

[b] For provisions concerning CC0 and CC4, see 4.3.

(2) Foundations and geotechnical structures that support a bridge should be classified with the same or higher consequence class as the bridge.

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

A.2.4 Design service life

(1) The design service life T_{lf} of a bridge, as described in 4.5, should be specified.

NOTE 1 The value of T_{lf} is given in Table A.2.2 (NDP) for different categories of structures, unless the National Annex gives different values or categories.

NOTE 2 For the use of the design service life, see 4.5(2).

Table A.2.2 (NDP) — Design service life categories for bridges

Category of structures	Design service life, T_{lf} years
Bridges, other civil engineering structures supporting road or railway traffic[a]	100[b]
Bridges where the main structural members have reduced protection[a]	50[b]
Replaceable structural parts other than tension components	25
Temporary structures[c]	≤ 10

[a] See the other Eurocodes for durability requirements to protect structural members to achieve the full design service life or reduced protection that achieves a lower design service life, .

[b] A different value of design service life may be used where specified by the relevant authority, or where not specified, agreed for a specific project by the relevant parties. A lower value of design service life can be relevant, for example, for bridges in a low consequence class where the economic consequences of replacement after a shorter design service life are agreed to be acceptable by the relevant authority or relevant parties.

[c] See 4.5(3) for classification of temporary structures, which excludes structures that can be dismantled and reused.

(2) Structural parts, and elements other than structural, that cannot be designed to achieve the design service life of the bridge should be replaceable.

NOTE For design requirements for replaceable structural parts, see 4.5. Examples of replaceable structural parts and elements other than structural include:

— bearings (or parts of bearings);

— expansion joints;

— drainage devices;

— guardrails, parapets;

— asphalt layer and other surface protection;

— wind shields;

— noise barriers.

(3) Where a bridge includes structural parts, and elements other than structural, that are replaceable, the possibility of their safe replacement should be verified as a transient design situation.

A.2.5 Durability

(1) All structural parts that rely on a design assumption of inspection or maintenance in order to satisfy their durability requirements over the design service life, shall be designed to permit inspection and maintenance.

NOTE 1 Regarding durability requirements, see 4.6.

NOTE 2 Maintenance activities can include: renewal of protective coatings, renewal of replaceable structural parts or elements other than structural, and cleaning.

(2) Where inspection or maintenance of a structural part is not possible, the structural part shall be designed to achieve adequate durability over the design service life without inspection or maintenance.

NOTE See the other Eurocodes for measures to achieve adequate durability over the design service life without inspection or maintenance, which can include: provision of sacrificial material; protection of the part; use of materials with enhanced durability; control of the environment surrounding the part.

A.2.6 Actions

(1) The actions, as described in 6.1, to be included in the design of bridges and associated geotechnical structures shall be those defined by EN 1991 (all parts), EN 1997 (all parts), and EN 1998 (all parts) as relevant.

(2) Effects resulting from foundation movements should be classified as a permanent action, G_{set}, and included in combinations of actions for ultimate and serviceability limit state verifications of the structure.

NOTE For limiting values of foundation movements at serviceability limit state including total and differential settlement, see A.2.9.5.

(3) Other actions, not defined in EN 1991 (all parts), EN 1997 (all parts), or EN 1998 (all parts), should be taken into account in the design of bridges and associated geotechnical structures as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

NOTE For combinations involving actions outside the scope of the Eurocodes, see A.2.7.3.6.

(4) Values of ice force on bridge elements (from ice on rivers, lakes, etc.) should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

A.2.7 Combinations of actions

A.2.7.1 Ultimate limit states (ULS)

(1) Combinations of actions for ultimate limit states with partial factors on actions (see 8.3.2.2) should be chosen depending on the design situation, according to:

— Table A.2.3, when using Formula (8.12); or

— Table A.2.4, when using Formula (8.13); or

— Table A.2.5, when using Formula (8.14).

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

NOTE 2 See 8.3.2.1(3) to determine whether applying partial factors on actions is relevant.

NOTE 3 The choice of ψ_1 or ψ_2 for accidental design situations can be set by the National Annex, depending on the details of the design situation; see 8.3.4.3. For combination rules for accidental design situations, see also A.2.7.8.

NOTE 4 For seismic design situations, see EN 1998 (all parts).

NOTE 5 For cases where the characteristic value of permanent actions G_{k} is represented by upper and lower characteristic values $G_{\mathrm{k,sup}}$ and $G_{\mathrm{k,inf}}$, see 6.1.2.2.

NOTE 6 The characteristic value of prestressing P_{k} can be an upper, lower, or mean value, as specified in the other Eurocodes.

NOTE 7 The value of ξ in Table A.2.4 and Table A.2.5 is 0,85, unless the National Annex gives a different value, see 8.3.4.2(2), Note 2.

(2) If design values of actions for persistent and transient (fundamental) design situations are chosen according to Table A.2.4 or Table A.2.5, then the most adverse of the two expressions in the relevant formula for combination of actions shall be verified.

Table A.2.3 — Combinations of actions for ultimate limit states when using Formula (8.12)

Design situation	Persistent and transient (fundamental)	Accidental[a]	Seismic[b]	Fatigue[c]
General formula for effects of actions	(8.4)			
Formula for combination of actions	(8.12)	(8.15)	(8.16)	(8.17)
Permanent ($G_{d,i}$)	$\gamma_{G,i} G_{k,i}$	$G_{k,i}$	$G_{k,i}$	$G_{k,i}$
Leading variable ($Q_{d,1}$)	$\gamma_{Q,1} Q_{k,1}$	$\psi_{1,1} Q_{k,1}$ or $\psi_{2,1} Q_{k,1}$	$\psi_{2,j} Q_{k,j}$	$\psi_{2,j} Q_{k,j}$
Accompanying variable ($Q_{d,j}$)	$\gamma_{Q,j} \psi_{0,j} Q_{k,j}$	$\psi_{2,j} Q_{k,j}$		
Prestressing (P_d)	$\gamma_P P_k$	P_k	P_k	P_k
Accidental (A_d)	-	A_d	-	-
Seismic (A_{Ed})	-	-	$A_{Ed,ULS}$	-
Fatigue (F_{fat})	-	-	-	$\gamma_{Ff} F_{fat}$

[a] For combinations of actions for accidental design situations, see A.2.7.8.

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

[c] For conditions of use for fatigue, see 8.3.4.5.

Table A.2.4 — Combinations of actions for ultimate limit states when using Formula (8.13)

Design situation	Persistent and transient (fundamental)		Accidental	Seismic	Fatigue
General formula for effects of actions	(8.4)				
Formula for combination of actions	The upper part of (8.13)	The lower part of (8.13b)	use values given in Table A.2.3		
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})	-	-			
Fatigue (F_{fat})	-	-			

Table A.2.5 — Combinations of actions for ultimate limit states when using Formula (8.14)

<table>
<tr><th>Design situation</th><th colspan="2">Persistent and transient (fundamental)</th><th>Accidental</th><th>Seismic</th><th>Fatigue</th></tr>
<tr><td>General formula for effects of actions</td><td colspan="5">(8.4)</td></tr>
<tr><td>Formula for combination of actions</td><td>The upper part of (8.14)</td><td>The lower part of (8.14)</td><td colspan="3" rowspan="8">use values given in Table A.2.3</td></tr>
<tr><td>Permanent ($G_{d,i}$)</td><td>$\gamma_{G,i} G_{k,i}$</td><td>$\xi \gamma_{G,i} G_{k,i}$</td></tr>
<tr><td>Leading variable ($Q_{d,1}$)</td><td rowspan="2">-</td><td>$\gamma_{Q,1} Q_{k,1}$</td></tr>
<tr><td>Accompanying variable ($Q_{d,j}$)</td><td>$\gamma_{Q,j} \psi_{0,j} Q_{k,j}$</td></tr>
<tr><td>Prestressing (P_d)</td><td>$\gamma_P P_k$</td><td>$\gamma_P P_k$</td></tr>
<tr><td>Accidental (A_d)</td><td>-</td><td>-</td></tr>
<tr><td>Seismic (A_{Ed})</td><td>-</td><td>-</td></tr>
<tr><td>Fatigue (F_{fat})</td><td>-</td><td>-</td></tr>
</table>

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

NOTE See 8.3.2.1(4) to determine whether to apply partial factors on effects of actions.

A.2.7.2 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.2.6, depending on the combinations of actions being considered.

NOTE The characteristic value of prestressing P_k can be an upper, lower, or mean value, as specified in the other Eurocodes.

Table A.2.6 — Combinations of actions for serviceability limit states

<table>
<tr><th>Combinations</th><th>Characteristic</th><th>Frequent</th><th>Quasi-permanent</th><th>Seismic[a]</th></tr>
<tr><td>General formula for effects of actions</td><td colspan="4">(8.28)</td></tr>
<tr><td>Formula for combination of actions</td><td>(8.29)</td><td>(8.30)</td><td>(8.31)</td><td>(8.32)</td></tr>
<tr><td>Permanent ($G_{d,j}$)</td><td>$G_{k,i}$</td><td>$G_{k,i}$</td><td>$G_{k,i}$</td><td>$G_{k,i}$</td></tr>
<tr><td>Leading variable ($Q_{d,1}$)</td><td>$Q_{k,1}$</td><td>$\psi_{1,1} Q_{k,1}$</td><td rowspan="2">$\psi_{2,j} Q_{k,j}$</td><td rowspan="2">$\psi_{2,j} Q_{k,j}$</td></tr>
<tr><td>Accompanying variable ($Q_{d,i}$)</td><td>$\psi_{0,j} Q_{k,j}$</td><td>$\psi_{2,j} Q_{k,j}$</td></tr>
<tr><td>Prestressing (P_d)</td><td>P_k</td><td>P_k</td><td>P_k</td><td>P_k</td></tr>
<tr><td>Seismic (A_{Ed})</td><td>-</td><td>-</td><td>-</td><td>$A_{Ed,SLS}$</td></tr>
<tr><td colspan="5">[a] 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 (all parts).</td></tr>
</table>

A.2.7.3 General combination rules

A.2.7.3.1 Specific design situations

(1) For specific design situations (e.g. calculation of bridge camber for aesthetics and drainage consideration, etc.) the requirements for the combinations of actions to be used should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

A.2.7.3.2 Group of traffic actions as single variable action

(1) For any combination of traffic actions with other variable actions specified in other parts of EN 1991, any group of traffic loads, as defined in prEN 1991-2, shall be taken into account as one variable action.

A.2.7.3.3 Failure of the ground

(1) Combinations of actions should be identified for verifying limit states involving failure of the ground, in accordance with EN 1997 (all parts).

NOTE Limit states involving failure of the ground can involve material remote from the structure or temporary conditions, e.g. stability of a slope supporting a bridge pier, buoyancy or heave failure of the bottom of an excavation for a bridge foundation.

A.2.7.3.4 Water

(1) Where relevant, representative values of water forces and applicable combinations of actions should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

NOTE 1 For bridge elements subject to actions from flowing water (e.g. a bridge pier in a river), it can be relevant to determine the representative values of water force by direct assessment, in cases where using single values of ψ_0, ψ_1, and ψ_2 does not allow for the differences between sites for water flow, speed and height. For the specification of water actions, see 6.1.3.2.

NOTE 2 For water actions induced by maritime currents and waves, and applicable combinations of actions, see subclause A.6 and EN 1991-1-8.

(2) For combinations of actions involving flowing water, the general and local scour depths should be assessed and the effect of scour taken into account in relevant verifications, as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

NOTE The magnitude of scour can be reduced with appropriate scour erosion protection measures.

A.2.7.3.5 Atmospheric icing

(1) Where it is necessary to consider combinations of atmospheric icing and wind, the combinations should be in accordance with subclause A.3.

NOTE 1 For actions induced by atmospheric icing, see EN 1991-1-9.

NOTE 2 Atmospheric ice can form on tension components, e.g. on cable-supported bridges. Measures can be necessary to prevent falling ice from striking traffic or pedestrians.

A.2.7.3.6 Actions outside the scope of Eurocodes

(1) Combinations involving actions which are outside the scope of the Eurocodes should be defined, in accordance with 1.1(6).

NOTE 1 The National Annex can define combinations involving actions that are outside the scope of Eurocodes.

NOTE 2 Actions that are outside the scope of the Eurocodes can include, for example, those due to mining subsidence, particular wind effects, water, floating debris, flooding, mud slides, avalanches, fire (affecting bridges) and ice pressure.

A.2.7.4 Combination rules for road bridges

A.2.7.4.1 General

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

NOTE 1 Values of the combination factor ψ for road bridges are given in the Table A.2.7 (NDP), unless the National Annex gives different values.

NOTE 2 Combination rules for special vehicles with normal traffic (covered by LM1 and LM2) and with other variable actions can be set by the National Annex. See also prEN 1991-2:2021, Annex A.

Table A.2.7 (NDP) — Combination factors for road bridges

Action	Symbol		ψ_0	ψ_1	ψ_2
Traffic loads[e, g] (see prEN 1991-2:2021, Table 6.5 and Table 6.6)	gr1a (LM1+footway and cycle-track loads) [m, n]	TS	0,75	0,75	0
		UDL	0,40	0,40	0 or 0,2[o]
		Footway+cycle-track loads[l]	0,40	0,40	0
	gr1b (single axle)[d, n]		0	0,75	0
	gr2 (horizontal forces)		0	0	0
	gr3 (pedestrian loads)		0	0,4	0
	gr4 (LM4 – crowd loading)		0	-	0
	gr5 (LM3 – special vehicles)[c]		0	-	0
Wind forces[a, f, g, h]	F_{Wk} Persistent design situations Execution[b]		0,6 0,8	0,2 -	0 0
	F_{W}^{*}		1,0	-	-
Thermal actions[h]	T_{k}		0,6[k]	0,6	0,5
Snow loads	$Q_{\mathrm{Sn,k}}$ Persistent design situations[i] Execution[b]		0 0,8	0 -	0 -
Water actions			j	j	j
Construction actions[b]	Q_{c}		1,0	-	1,0

a For combinations with atmospheric icing, see A.2.7.3.5.

b For combination rules for execution, see A.2.7.9.

c For combination rules for special vehicles, see A.2.7.4.1(1), Note 2.

d For combinations involving the single axle load, see A.2.7.4.2.

e For combinations of traffic load on bridge deck and behind abutment, see A.2.7.4.3.

f For limiting values of wind accompanying traffic, see A.2.7.4.4.

g For combinations of wind and traffic, see A.2.7.4.4.

h For combinations of wind and thermal actions, see A.2.7.4.5.

i For combinations of snow and traffic, see A.2.7.4.6.

j For combinations with water actions and waves and currents, see A.2.7.3.4.

k The other Eurocodes give criteria where thermal actions can be neglected for ultimate limit states.

l The combination value of the footway and cycle-track load, mentioned in prEN 1991-2:2021, Table 6.5 is a "reduced" value. ψ_0, ψ_1 and ψ_2 factors are applicable to this reduced value.

m The values for gr1a are also applied to the load model on abutments and walls adjacent to bridges given in prEN 1991-2:2021, 6.9, with the values of ψ_0, ψ_1 and ψ_2 for the TS applied to the concentrated load, and the values of ψ_0, ψ_1 and ψ_2 for the UDL applied to the uniform load.

n The values of ψ_0, ψ_1 and ψ_2 for gr1a and gr1b given for road traffic correspond to adjustment factors α_{Qi}, α_{qi}, α_{qr} and β_{Q} equal to 1. Those relating to UDL correspond to common traffic scenarios, in which a rare accumulation of lorries can occur. Other values may be envisaged for other classes of routes, or of expected traffic, or for accidental design situations, related to the choice of the corresponding α factors. For example, a value of ψ_2 other than zero may be envisaged for the UDL system of LM1 only, for bridges supporting severe continuous traffic. See also EN 1998 (all parts).

o The value of ψ_2 for the UDL is taken as 0 except for seismic design situations for bridges in CC3 (representing severe traffic conditions). See also EN 1998 (all parts).

A.2.7.4.2 Combinations with concentrated loads for traffic

(1) The following concentrated loads for verifying local effects should not be combined with any other variable actions that are not due to traffic:

— single axle load model 2 and associated group of loads gr1b; and

— the characteristic value of the concentrated load on a footbridge Q_{fwk}.

A.2.7.4.3 Combination of traffic load on bridge deck and behind abutment

(1) When relevant, the traffic load model for the abutment (see prEN 1991-2:2021, 6.9) should be combined with groups of traffic loads on bridge deck with characteristic values.

NOTE Additional application rules can be set by the National Annex.

A.2.7.4.4 Combinations of wind and traffic

(1) When the wind action acts simultaneously with traffic actions, a limiting value F_{W}^{*} may be defined for the accompanying wind force $\psi_0 F_{\text{Wk}}$ acting in combination with load model 1 or with the associated group of loads gr1a.

NOTE A limiting value for wind action accompanying road traffic actions is given in EN 1991-1-4.

(2) Wind actions should not be combined with:

— braking and acceleration forces or the centrifugal forces or the associated group of loads gr2;

— loads on footways and cycle tracks or with the associated group of loads gr3;

— crowd loading (load model 4) or the associated group of loads gr4.

A.2.7.4.5 Combinations of wind and thermal actions

(1) Wind actions and thermal actions should be taken into account simultaneously unless otherwise specified for local climatic conditions.

NOTE Combination rules for wind and thermal actions, applicable for local climatic conditions, can be set by the National Annex.

A.2.7.4.6 Combinations of snow and traffic

(1) Except in the case of covered bridges, snow loads should not be combined with:

— load models 1 and 2 or with the associated groups of loads gr1a and gr1b unless otherwise specified for particular geographical areas;

— braking and acceleration forces or the centrifugal forces or the associated group of loads gr2;

— loads on footways and cycle tracks or with the associated group of loads gr3;

— crowd loading (load model 4) or the associated group of loads gr4.

NOTE 1 Geographical areas where snow loads are combined with groups of loads gr1a and gr1b in combinations of actions can be set by the National Annex.

NOTE 2 Combinations of actions for snow and traffic for covered bridges can be set by the National Annex.

A.2.7.5 Combination rules for footbridges

A.2.7.5.1 General

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

NOTE 1 Values of the combination factor ψ for footbridges are given in the Table A.2.8 (NDP), unless the National Annex gives different values.

NOTE 2 Footbridges includes bridges intended mainly to carry cycle traffic loads.

Table A.2.8 (NDP) — Combination factors for footbridges

Action	Symbol	ψ_0	ψ_1	ψ_2
Traffic loads[a]	gr1	0,40	0,40	0
(see prEN 1991-2:2021,	gr2	0,40	0,40	0
Table 7.1)	gr3[b]	0,40	0,40	0
Wind forces[c, h]	F_{Wk}	0,3	0,2	0
Thermal actions[c]	T_{k}	0,6[g]	0,6	0,5
Snow loads	$Q_{\mathrm{Sn,k}}$ Persistent design situations[d] Execution[f]	0 0,8	0 -	0 -
Water actions		e	e	e
Construction actions[f]	Q_{c}	1,0	-	1,0

[a] For scope of traffic actions on footbridges, see A.2.7.5.1(1), Note 2.
[b] For combinations involving the concentrated load Q_{fwk}, see A.2.7.5.2.
[c] For combinations of wind and thermal actions, see A.2.7.5.3.
[d] For combinations of snow and traffic, see A.2.7.5.4.
[e] For combinations with water actions and waves and currents, see A.2.7.3.4.
[f] For combination rules for execution, see A.2.7.9.
[g] The other Eurocodes give criteria where thermal actions can be neglected for ultimate limit states.
[h] For combinations with atmospheric icing, see A.2.7.3.5.

A.2.7.5.2 Combinations with concentrated loads for traffic

(1) The characteristic value of the concentrated load on a footbridge Q_{fwk} and the associated group of loads gr3 should not be combined with any other variable actions that are not due to traffic.

A.2.7.5.3 Combinations of wind and thermal actions

(1) Wind actions and thermal actions should be taken into account simultaneously unless otherwise specified for local climatic conditions.

NOTE Combination rules for wind and thermal actions, applicable for local climatic conditions, can be set by the National Annex.

A.2.7.5.4 Combinations of snow and traffic

(1) Except in the case of covered bridges, snow loads should not be combined with groups of loads gr1, gr2 and gr3 for footbridges unless otherwise specified for particular geographical areas and certain types of footbridges.

NOTE 1 Geographical areas, and certain types of footbridges, where snow loads are combined with groups of loads gr1, gr2 and gr3 in combinations of actions can be set by the National Annex.

NOTE 2 Combinations of actions for snow and traffic for covered footbridges can be set by the National Annex.

A.2.7.6 Combination rules for railway bridges

A.2.7.6.1 General

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

NOTE Values of the combination factor ψ for railway bridges are given in the Table A.2.9 (NDP), unless the National Annex gives different values.

Table A.2.9 (NDP) — Combination factors for railway bridges

Action[p]			ψ_0	ψ_1	ψ_2[a]
Individual components of traffic actions[d, e, g]	LM 71[o]		0,80	[b]	0 or 0,3[q]
	SW/0[o]		0,80	[b]	0
	SW/2[h]		0	1,00	0
	Unloaded train[g]		1,00	–	–
	HSLM		1,00	1,00	0
	Traction and braking[o] Centrifugal forces[o] Interaction forces due to deformation under vertical traffic loads		Individual components of traffic actions in design situations where the traffic loads are considered as a single (multi-directional) leading action and not as groups of loads should use the same values of ψ factors as those adopted for the associated vertical loads		
	Nosing forces		1,00	0,80	0
	Non-public footpaths loads		0,80	0,50	0
	Real trains		1,00	1,00	0
	Horizontal earth pressure due to traffic load surcharge		0,80	[b]	0
	Aerodynamic effects		0,80	0,50	0
Main traffic actions [c, g, h]	gr11 (LM71 + SW/0)	Max. vertical 1 with max. longitudinal	0,80	0,80	0
	gr12 (LM71 + SW/0)	Max. vertical 2 with max. transverse			
	gr13 (braking/traction)	Max. longitudinal			
	gr14 (centrifugal/nosing)	Max. transverse			
	gr15 (unloaded train)	Lateral stability with “unloaded train”			
	gr16 (SW/2)	SW/2 with max. longitudinal			
	gr17 (SW/2)	SW/2 with max. transverse			

Action[p]			ψ_0	ψ_1	ψ_2[a]
(groups of loads)	gr21 (LM71 + SW/0)	Max. vertical 1 with max. longitudinal	0,80	0,70	0
	gr22 (LM71 + SW/0)	Max. vertical 2 with max transverse			
	gr23 (braking/traction)	Max. longitudinal			
	gr24 (centrifugal/nosing)	Max. transverse			
	gr26 (SW/2)	SW/2 with max. longitudinal			
	gr27 (SW2)	SW/2 with max. transverse			
	gr31 (LM71 + SW/0)	Additional load cases	0,80	0,60	0
Other operating actions	Aerodynamic effects [i]		0,80	0,50	0
	General maintenance loading for non-public footpaths		0,80	0,50	0
Wind forces[f, g, h, i]	F_{Wk}		0,75	0,50	0
	F_W^{**}		1,00	0	0
Thermal actions	T_k		0,60[m, n]	0,60	0,50
Snow loads	$Q_{Sn,k}$ Persistent design situations[k] Execution[l]		0 0,8	0 -	0 -
Water actions			j	j	j
Construction actions[l]	Q_c		1,0	-	1,0

[a] If deformation is being considered for persistent and transient design situations, ψ_2 should be taken equal to 1,0 for rail traffic actions.

[b] The value of ψ_1 for LM71 and SW/0 when applied as individual components is:
- 0,8 if 1 track only is loaded;
- 0,7 if 2 tracks are simultaneously loaded;
- 0,6 if 3 or more tracks are simultaneously loaded.

[c] For definition of groups of loads, see A.2.7.6.2(1).

[d] For individual components of traffic actions, see A.2.7.6.2(2).

[e] For combinations of individual components, see A.2.7.6.2(3).

[f] For limiting values of wind accompanying traffic, see A.2.7.6.3(1).

[g] For combinations of wind and traffic, see A.2.7.6.3(2).

[h] For combinations of wind and traffic, see A.2.7.6.3(3).

[i] For combinations of wind and aerodynamic effects, see A.2.7.6.3(4) and A.2.7.6.3(5).

[j] For combinations with water actions and waves and currents, see A.2.7.3.4.

[k] For combinations of snow and traffic, see A.2.7.6.4.

[l] For combination rules for execution, see A.2.7.9.

[m] For combinations for verifying track-bridge interaction, see prEN 1991-2:2021, 8.5.4.

[n] The other Eurocodes give criteria where thermal actions can be neglected for ultimate limit states.

[o] Minimum coexistent favourable vertical load with individual components of rail traffic actions (e.g. centrifugal, traction or braking) is 0,5 × LM71 or 0,5 x SW/0. See prEN 1991-2:2021, Table 8.15.

[p] For combinations with atmospheric icing, see A.2.7.3.5.

[q] The value of ψ_2 for the LM71 is taken as 0 except for seismic design situations for bridges in CC3 (representing severe traffic conditions). See also EN 1998 (all parts).

A.2.7.6.2 Combinations involving traffic actions

(1) Where groups of loads are used for railway bridges, the groups of loads defined in prEN 1991-2:2021, 8.8.2, Table 8.15 should be used. A unique ψ value should be applied to one group of loads, and taken as equal to the ψ value applicable to the leading component of the group.

(2) Where groups of loads are not used for rail traffic loading, rail traffic loading should be considered as a single multi-directional variable action with individual components of rail traffic actions to be taken as the maximum unfavourable and minimum favourable values as appropriate.

(3) Where relevant, combinations of individual traffic actions (including individual components) should be taken into account for railway bridges.

NOTE Examples of cases where individual traffic actions can be relevant are: for the design of bearings; for the assessment of maximum transverse and minimum vertical traffic loading; bearing restraints; maximum overturning effects on abutments (especially for continuous bridges); etc.

A.2.7.6.3 Combinations of wind and traffic

(1) When the wind action acts simultaneously with traffic actions, a limiting value $F_W{}^{**}$ may be defined for the accompanying wind force $\psi_0 F_{Wk}$ acting in combination with traffic actions.

NOTE A limiting value for wind action accompanying railway traffic actions is given in EN 1991-1-4.

(2) The combinations of actions to be taken into account when traffic actions and wind actions act simultaneously should include:

— vertical rail traffic actions including dynamic factor, horizontal rail traffic actions and wind forces with each action being considered as the leading action of the combination of actions one at a time;

— vertical rail traffic actions (excluding dynamic factor) and transverse rail traffic actions, both from the “unloaded train” defined in prEN 1991-2:2021, 8.3.4, 8.5 and Table 8.15, with wind forces, for checking stability.

(3) Wind action should not be combined with:

— groups of loads gr11, gr21, gr13 and gr23 (max. longitudinal);

— load model SW/2 and associated groups of loads gr16, gr17, gr26, gr27.

(4) Actions due to aerodynamic effects of rail traffic (see prEN 1991-2:2021, 8.6) and wind actions should be combined together. Each action should be considered individually as a leading variable action.

(5) If a structural member is not directly exposed to wind, the action q_{1k} due to aerodynamic effects should be determined for train speeds enhanced by the speed of the wind.

NOTE This can apply when wind acts in the line of the train but in the opposite direction.

A.2.7.6.4 Combinations involving snow

(1) Snow loads should not be taken into account in any combination for persistent design situations nor for any transient design situation after the completion of the bridge unless otherwise specified for particular geographical areas and certain types of railway bridges by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

NOTE Geographical areas, and certain types of railway bridges, where snow loads are taken into account in combinations of actions can be set by the National Annex.

A.2.7.6.5 Combinations for seismic design situations

(1) For seismic design situations, only one track should be loaded and load model SW/2 may be neglected.

(2) Requirements for seismic design situations should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

A.2.7.7 Combination rules for bridges carrying both road and railway traffic

(1) For bridges carrying both road and railway traffic, combination rules should be as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties including applicable combinations for fatigue verifications.

NOTE 1 In general, it can be appropriate to use the relevant ψ_0 values, etc., for rail traffic and road traffic treated as separate variable actions and each taken as leading in turn.

NOTE 2 For special projects and longer span bridges such as cable supported structures, it can be necessary to undertake a more detailed study about applicable combinations of road and rail traffic.

(2) Combinations for fatigue verifications should take into account the stress cycles due to the road and rail traffic acting simultaneously.

A.2.7.8 Combinations of actions for accidental design situations

(1) Where an action for an accidental design situation needs to be taken into account, no other accidental action or wind action or snow load should be taken into account in the same combination.

(2) For an accidental design situation concerning impact from traffic (road, rail or waterway traffic), the loads due to the traffic on the bridge should be taken into account in the combinations as accompanying actions with their frequent value using ψ_1, unless otherwise specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

NOTE For actions due to impact from traffic under the bridge, see EN 1991-1-7. For actions due to impact from traffic on the bridge deck, see prEN 1991-2.

(3) Structural parts of a bridge to which parapets or guardrails are connected should be designed such that plastic deformations of the parapets or guardrails under the accidental design situation can occur without damaging the structure.

NOTE For accidental actions on structural members due to collision forces on vehicle restraint systems and on pedestrian parapets, see prEN 1991-2:2021, 6.7.3.3 and 6.8.

(4) For railway bridges, for an accidental design situation concerning actions caused by a derailed train on the bridge, rail traffic actions on the other tracks should be taken into account as accompanying actions in the combinations with their combination value using ψ_0, unless otherwise specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

NOTE Actions for accidental design situations due to impact from rail traffic running on the bridge including derailment actions are specified in prEN 1991-2:2021, 8.7.

(5) Accidental design situations involving ship collisions against bridges should be identified and specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

NOTE For ship impact, see EN 1991-1-7.

(6) Additional combinations of actions for other accidental design situations (e.g. combination of road or rail traffic actions with avalanche, flood or scour effects) should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

(7) Where, in special cases, one or several variable actions need to be considered simultaneously with the accidental action, their representative values should be as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

NOTE An example of a special case is the accidental fall of a prefabricated element during execution, see A.2.7.9(7).

A.2.7.9 Combination rules for execution

(1) During execution the relevant design situations shall be taken into account.

NOTE For actions during execution, see EN 1991-1-6.

(2) The relevant design situations shall be taken into account where a bridge is brought into use in stages.

(3) Where relevant, particular construction actions should be taken into account simultaneously in the appropriate combinations of actions.

NOTE Where construction actions cannot occur simultaneously due to the implementation of control measures they need not be taken into account in the relevant combinations of actions simultaneously.

(4) Snow loads and wind actions should not be considered simultaneously with loads arising from construction activity (i.e. loads due to working personnel) unless specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

NOTE 1 For a specific project it can be relevant to agree the requirements for snow loads and wind actions to be taken into account simultaneously with other construction actions (e.g. actions due to heavy equipment or cranes) during some transient design situations.

NOTE 2 The characteristic values of snow loads and wind actions during execution are defined in EN 1991-1-6.

(5) Where relevant, thermal and water actions should be considered simultaneously with construction actions. Where relevant the various parameters governing water actions and components of thermal actions should be taken into account when identifying appropriate combinations with construction actions.

(6) Where a counterweight is used to provide stability during execution, the variability of its characteristics may be taken into account by one or both of the following rules, as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties:

— by treating the counterweight as a favourable effect using the partial factor $\gamma_{G,fav}$, where the self-weight is not well defined (e.g. containers);

— by considering a variation of its project-defined position specified proportionately to the dimensions of the bridge, where the magnitude of the counterweight is well defined.

NOTE For example, the variation of the counterweight position for bridges during launching is often taken equal to ± 1 m.

(7) For execution phases during which there is a possibility of loss of static equilibrium in an accidental design situation, construction actions should all be applied as accompanying variable actions with their quasi-permanent value and no other variable actions should be applied.

NOTE As an example, in the case of bridges built by the cantilevered method, some construction actions can be considered as simultaneous with the action corresponding to the accidental fall of a prefabricated element.

A.2.7.10 Combination rules for integral abutment bridges

(1) Effects of restraint of integral superstructure subjected to imposed displacements shall be considered.

NOTE 1 The restrained effects from the imposed displacements depend on the configuration of the structure. For example, restrained effects can be mobilized in the following cases: structures having monolithic connections between sub- and superstructure, lateral soil-structure-interaction, track-bridge-interaction, prestressing, etc.

NOTE 2 Imposed displacements can result from longitudinal actions such as traffic traction and braking.

NOTE 3 Expansion or contraction of the superstructure can result from temperature, shrinkage or creep. Material related guidance is given in the other Eurocodes.

(2) Effects resulting from thermal actions may be released or neglected in ultimate limit state depending on the deformation capacity and conditions related to the verification case.

NOTE 1 The other Eurocodes give criteria when effects can be released or neglected.

NOTE 2 Generally, the effects resulting from enhanced earth pressure cannot be significantly released.

(3) The interaction of earth pressures on integral bridge abutments should be considered, including the effect of cyclic movements on the earth pressures and the change of earth pressure over time and with displacement of the abutment.

NOTE Guidance on the inertial effects of the earth behind abutments in seismic design situations is given in EN 1998-5.

(4) Soil properties should be determined in accordance with EN 1997 (all parts) such that most adverse effects to the structure are achieved.

NOTE 1 The soil-structure interaction can be favourable or unfavourable depending on load, structural system and structural element. For example, in case of brake load, interaction is favourable for the superstructure but unfavourable for the end screen, whereas temperature increase is unfavourable for superstructure and the end screen.

NOTE 2 The difference of component stiffness can influence the restrained effects. For example, the restrained effects can increase due to the difference of material properties, construction process or construction phases between the bridge ends.

(5) The characteristic movement at an integral abutment should not exceed value D_{int}.

NOTE 1 The value of D_{int} is ± 30 mm, unless the National Annex gives a different value.

NOTE 2 The movement due to characteristic thermal actions is the characteristic thermal movement, unless the National Annex gives a different approach.

NOTE 3 Cyclic movements of the abutment cause progressive movement of the backfill behind the abutment which can lead to settlement over time and increased earth pressures that lead to higher forces on the structure. These effects increase with the movement range.

(6) Measures to control or mitigate settlement behind integral abutments should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

NOTE For example, run-on slabs can be provided.

(7) The effect of repeated expansion and contraction caused by thermal cycles of integral bridges may be represented as an enhanced earth pressure applied to the abutment.

NOTE A progressive year-on-year increase in soil pressure occurs due to imposed cyclic contraction and expansion which causes realignment of granular soil particles.

(8) A range of earth pressures should be applied to the abutment, taking account of the development of enhanced earth pressures over repeated cycles of movement and the directions of movement of the abutment (expansion into the backfill or contraction away from the backfill).

(9) The most adverse combinations of enhanced earth pressure, surcharge loading and longitudinal loads should be considered.

NOTE The detailed combination rules for how traffic surcharge loading and brake load are considered in conjunction with enhanced earth pressures can be set by the National Annex.

(10) Where earth pressure behind an abutment wall is used to resist variable longitudinal loads, the earth pressure should be established based on the movement of the abutment and including the case early in the life of the bridge before strain ratcheting of the backfill has occurred.

NOTE Methods of estimating the movement required to mobilize a given proportion of passive resistance are given in EN 1997-3.

A.2.8 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 factors γ_F are given in Table A.2.10 (NDP) for persistent and transient design situations, unless the National Annex gives different values.

NOTE 2 Values of the partial factors γ_E are given in Table A.2.10 (NDP) for persistent and transient design situations for relevant geotechnical verification cases, unless the National Annex gives different values.

NOTE 3 Values of k_F for different consequence classes in Table A.2.10 (NDP) are given in Table A.2.11 (NDP), unless the National Annex gives different values.

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 verification case VC1.

(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 verification cases VC2(a) and VC2(b) using whichever gives the less favourable design outcome.

(6) Verification of verification case VC2(b) may be omitted when it is obvious that verification using verification case VC2(a) governs the design outcome.

(7) Ultimate limit states that involve failure of ground should be verified using partial factors for verification cases VC1, VC2, VC3 and VC4, in accordance with EN 1997-1.

NOTE EN 1997 (all parts) gives guidance on which verification cases to use for different geotechnical structures.

(8) Partial factors for actions which are outside the scope of EN 1991 (all parts) should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

(9) Where a relevant value of γ_P for prestressing forces is not defined in the other Eurocodes, then γ_P should be as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties; see 6.1.3.1.

(10) A partial factor $\gamma_{G,Set}$ may be applied to the effects of settlement (and other types of foundation movements) induced in the structure by $C_{d,SLS}$, the limiting serviceability criteria for foundation movements, defined in A.2.9.5.

(11) In cases where the design value of water action is not obtained by applying a partial factor to the representative water action, then the design value of water action should be derived as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties, in accordance with this document and, where relevant, EN 1997-1.

NOTE EN 1997 (all parts) permits direct assessment or applying a deviation to the representative level, e.g. in cases where the water level is subject to a physical limit.

(12) During execution, if the construction process is adequately controlled to ensure that the required safety level is maintained, the partial factor for variable construction actions used in verifications of static equilibrium may be reduced to a value as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

NOTE For example, a value γ_Q = 1,35 can be selected in place of the value γ_Q = 1,5 used for other variable actions.

Table A.2.10 (NDP) — Partial factors on actions and effects for verification cases VC1 – VC4 for persistent and transient (fundamental) design situations for bridges and associated geotechnical structures

Action or effect [n]				Partial factors $\gamma_{\mathbf{F}}$ and $\gamma_{\mathbf{E}}$ for verification cases				
Type	**Group**	**Symbol**	**Resulting effect**	**Structural resistance**	**Combined structural resistance and static equilibrium / uplift**		**Geotechnical design**	
Verification case				**VC1**[a]	**VC2(a)**[a]	**VC2(b)**[a]	**VC3**[a]	**VC4**[a]
Perma-nent action (G_{k})	All[c]	γ_{G}	unfavourable /destabilizing	$1{,}35k_{\mathrm{F}}$	$1{,}35k_{\mathrm{F}}$	1,0	1,0	G_{k} is not factored
	Water[m]	γ_{Gw}		$1{,}2k_{\mathrm{F}}$	$1{,}2k_{\mathrm{F}}$	1,0	1,0	
	Settlement[o]	$\gamma_{\mathrm{G,set}}$		$1{,}2k_{\mathrm{F}}$[h]	$1{,}2k_{\mathrm{F}}$[h]	1,0	1,0	
	All[c]	$\gamma_{\mathrm{G,stb}}$	stabilizing[d]	not used	1,25[b]	1,0	not used	
	Water[m]	$\gamma_{\mathrm{Gw,stb}}$			1,0[b]	1,0		
	All[c]	$\gamma_{\mathrm{G,fav}}$	favourable[e]	1,0	1,0	1,0	1,0	
	Settlement[o]	$\gamma_{\mathrm{G,set,fav}}$		0	0	0	0	0
Prestress-ing (P_{k})		γ_{P}[g]						
Variable action (Q_{k})	Road / pedestrian traffic		unfavourable	$1{,}35k_{\mathrm{F}}$	$1{,}35k_{\mathrm{F}}$	$1{,}35k_{\mathrm{F}}$	1,15	$\gamma_{\mathrm{Q,red}}$[f]
	Rail traffic (except as below)[i]			$1{,}45k_{\mathrm{F}}$	$1{,}45k_{\mathrm{F}}$	$1{,}45k_{\mathrm{F}}$	1,25	
	SW/2, gr16, gr17[j]			$1{,}2k_{\mathrm{F}}$	$1{,}2k_{\mathrm{F}}$	$1{,}2k_{\mathrm{F}}$	1,0	
	Other[k]	γ_{Q}		$1{,}5k_{\mathrm{F}}$	$1{,}5k_{\mathrm{F}}$	$1{,}5k_{\mathrm{F}}$	1,3	
	Variable water[l]	γ_{Qw}		$1{,}35k_{\mathrm{F}}$	$1{,}35k_{\mathrm{F}}$	$1{,}35k_{\mathrm{F}}$	1,15	1,0
	All	$\gamma_{\mathrm{Q,fav}}$	favourable	0				
Effects of actions (E)		γ_{E}	unfavourable	γ_{E} is not applied				$1{,}35k_{\mathrm{F}}$
		$\gamma_{\mathrm{E,fav}}$	favourable					1,0

[a] For verifications using verification cases VC1 to VC4, see A.2.7(4) to A.2.7(7).

[b] The value of $\gamma_{\mathrm{G,stb}}$ = 1,25 is based on 1,35 x ρ with ρ = 0,925. The value of $\gamma_{\mathrm{Gw,stb}}$ = 1,0 is based on 1,2 x ρ with ρ = 0,85, see 8.3.3.1.

[c] Applied to all permanent actions except water actions and settlement, including: self-weight of structural members and elements other than structural, ballast, soil, ground water and free water, removable loads, etc.

[d] Applied to the stabilizing component of an action originating from a single source.

[e] Applied to actions whose entire effect is favourable and independent of the unfavourable action.
[f] $\gamma_{Q,red} = \gamma_{Q,1}/\ \gamma_{G,1}$ where $\gamma_{Q,1}$ = corresponding value of γ_Q from VC1 and $\gamma_{G,1}$ = corresponding value of γ_G from VC1.
[g] See other relevant Eurocodes for the definition of γ_P where γ_P is materially dependent.
[h] Applies in the case of a linear elastic analysis. The partial factor for settlement is increased to 1,35k_F in the case of non-linear analysis. For design situations where actions due to uneven settlements have favourable effects, these actions are not taken into account and the partial factor is taken as 0.
[i] Applied to rail traffic actions for groups of loads gr11 to gr31 (except gr16, gr17, gr26 and gr27). Applied to load models LM71, SW/0 and HSLM and real trains, when considered as individual leading traffic actions. For rail traffic actions for groups of loads 26 and 27, applied to individual components of traffic actions associated with load models LM71, SW/0 and HSLM and real trains.
[j] Applied to rail traffic actions for groups of loads gr16 and gr17 and SW/2. For rail traffic actions for groups of loads gr26 and gr27, applied to individual components of traffic actions associated with SW/2.
[k] Applied to other variable actions (except water) including: traffic actions not identified above, variable horizontal earth pressure from soil, ground water, free water and ballast, traffic load surcharge earth pressure, traffic aerodynamic actions, wind and thermal actions, construction actions, etc.
[l] For variable water actions, see A.2.7.3.4
[m] Value applies only where the design value of permanent water action is obtained by applying a partial factor to the representative value. Direct determination and deviation to levels are also permitted. See A.2.8(11).
[n] The other Eurocodes give criteria where thermal actions, actions due to foundation movements or creep and shrinkage can be relieved or neglected for ultimate limit states.
[o] See A.2.8(10).

Table A.2.11 (NDP) — Consequence factors for bridges and associated geotechnical structures

Consequence class (CC)	Description of consequences	Consequence factor k_F
CC3b	High (upper class)	1,1
CC3a	High (lower class)	1,0
CC2	Normal	1,0
CC1	Low	0,9
[a] The provisions in Eurocodes cover design rules for structures classified as CC1 to CC3, see 4.3.		

A.2.9 Serviceability criteria

A.2.9.1 General

(1) Serviceability criteria, supplementing A.2.9, may be defined for bridges.

NOTE Minimum requirements for additional serviceability limit states for bridges can be defined in the National Annex.

(2) Additional serviceability criteria should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties, in accordance with 5.4.

(3) Where specified, serviceability limit states during execution should be defined in accordance with this document and the other relevant Eurocodes.

(4) Specified clearance gauges should be maintained without encroachment by any part of the structure for deformations under the characteristic load combination.

(5) Wind-induced vibrations of structures and structural elements shall be verified in accordance with EN 1991-1-4.

A.2.9.2 Serviceability criteria for road bridges

(1) Requirements and criteria should be as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties for road bridges concerning deformations and vibrations, where relevant.

NOTE 1 The frequent combination of actions can be used for the assessment of deformation.

NOTE 2 Vibrations of road bridges can have various origins, in particular traffic actions and wind actions. For vibrations due to wind actions, see EN 1991-1-4. For vibrations due to traffic actions, comfort criteria can be considered. When considering fatigue it can be relevant to take into account vibrations of the road bridge.

NOTE 3 It can be relevant to define limits on deformations where excessive deformations could:

— endanger traffic by excessive transverse slope when the surface is iced;

— affect the dynamic load on the bridge by impact from wheels;

— affect the dynamic behaviour causing discomfort to users;

— lead to cracks in asphaltic surfacings;

— adversely affect the drainage of water from the bridge deck.

A.2.9.3 Serviceability criteria for footbridges

A.2.9.3.1 Approach for the assessment of vibrations due to pedestrian traffic

(1) Human-induced vibration shall be assessed to ensure that vibrations due to pedestrian traffic are acceptable.

(2) Assessment of human-induced vibrations may be omitted if it can be demonstrated by long term experience from comparable bridges with similar pedestrian traffic that the requirements are satisfied, as specified by the relevant authorities or, where not specified, agreed for a specific project by the relevant parties.

(3) Vibrations due to pedestrian traffic should be assessed if any relevant natural frequency of the bridge is situated within the critical natural frequency limits defined in section A.2.8.3.3.

NOTE For certain structures with natural frequencies related to torsional or combined modes (lateral-torsional) other critical frequency limits or assessment criteria can be relevant.

(4) For cable supported structures, stress ribbon structures and other exceptional structures, the vibrations due to pedestrian traffic should be assessed even if all natural frequencies are outside the critical natural frequency limits defined in A.2.9.3.3, unless otherwise specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

(5) The approach for the assessment of vibrations due to pedestrian traffic should be specified.

NOTE 1 The approach for the assessment of vibrations due to pedestrian traffic can be set by the National Annex.

NOTE 2 Annex H gives guidance on acceptance limits that can be used to assess vibrations due to pedestrian traffic.

NOTE 3 prEN 1991-2:2021, Annex G gives guidance on dynamic analysis for vibrations due to pedestrian traffic.

A.2.9.3.2 Pedestrian comfort criteria

(1) Relevant design situations including appropriate comfort levels and traffic classes should be as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

NOTE 1 The perception of vibrations is subjective and therefore different for each pedestrian. Acceptable limits for vibration therefore depend on the location and type of user of the footbridge.

NOTE 2 Annex H gives guidance on relevant design situations, comfort levels and traffic classes.

NOTE 3 Mass gatherings (for example sports events, demonstrations, exceptional events) are not covered by the guidance on dynamic load models and comfort levels given in Annex H and, where relevant, can be agreed separately either as a serviceability limit state or as an ultimate limit state design situations.

(2) Dynamic actions of cyclists may be considered negligible when compared to the actions caused by pedestrians and joggers.

(3) Intentional excitation should be considered as an accidental design situation at ULS.

NOTE Structures develop an increase in damping associated to an increase in vibration amplitude, and people lose concentration and power to excite the bridge over a longer time period necessary for affecting the fatigue strength of the construction material. Intentional excitation is stopped when the amplitude does not increase for some time or when the persons have no more power for exciting the bridge.

A.2.9.3.3 Critical range of natural frequency

(1) The comfort criteria for pedestrian excitation should be verified for vertical and longitudinal vibrations if the corresponding natural frequencies of the bridge are situated within a given critical natural frequency limits.

NOTE 1 The critical natural frequency limits are 1,25 Hz and 2,3 Hz, unless the National Annex gives different values.

NOTE 2 For the proposed critical natural frequency limits, the reduction coefficient ψ_{W} for the load is read from prEN 1991-2:2021, Table G.2.

(2) If the bridge is especially sensitive to excitation, the comfort criteria should be verified for vertical and longitudinal vibrations also if the fundamental natural frequency is lower than 4,6 Hz, as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

NOTE In such cases, unacceptable vibrations can occur due to the second harmonic of pedestrian loading.

(3) The comfort criteria for pedestrian excitation should be verified for lateral or torsional vibrations if the corresponding natural frequencies of the bridge are situated within a given critical natural frequency limits.

NOTE 1 The critical natural frequency limits are 0,5 Hz and 1,2 Hz, unless the National Annex gives different values.

NOTE 2 For the proposed critical natural frequency limits, the reduction coefficient ψ_{W} for the load is read from prEN 1991-2:2021, Table G.2.

(4) The comfort criteria for jogger excitation should be verified for vertical vibrations if the corresponding natural frequencies of the bridge are situated within a given critical natural frequency limits.

NOTE 1 The critical natural frequency limits are 1,9 Hz and 3,5 Hz, unless the National Annex gives different values.

NOTE 2 For the proposed critical natural frequency limits, the reduction coefficient ψ_{j} for the load is read from prEN 1991-2:2021, Table G.3.

NOTE 3 The load model for joggers can be very severe and is not a systematic requirement. Very often, the crossing duration of joggers on the footbridge is relatively short and does not leave enough time for the resonance phenomenon to develop. In addition, any resonance that does occur affects the other pedestrians over a very short period. Therefore, depending on the expected bridge usage, it can be relevant to neglect the jogger excitation in such cases.

NOTE 4 The jogger excitation case does not cover exceptional events such as a popular marathon race.

(5) The ultimate limit state verification for intentional excitation should be verified for vertical vibrations if the corresponding natural frequencies of the bridge are between 1,7 Hz and 3,0 Hz, unless otherwise specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties depending on the usage of the bridge.

A.2.9.3.4 Vibration control devices

(1) The potential need for vibration control devices should be as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

NOTE There is uncertainty on the calculated system response due to uncertainties in the structural properties such as damping, hence final confirmation of vibration performance can sometimes only be achieved by measurements on the completed structure and, if necessary at this stage, by fitting a vibration control device.

A.2.9.4 Serviceability criteria for railway bridges

A.2.9.4.1 General

(1) Limits of deformation and vibration shall be taken into account for the design of new railway bridges in accordance with A.2.9.4.

NOTE 1 Excessive bridge deformations can endanger traffic by creating unacceptable changes in vertical and horizontal track geometry, excessive rail stresses and vibrations in bridge structures. Excessive vibrations can lead to ballast instability and unacceptable reduction in wheel rail contact forces. Excessive deformations can also affect the loads imposed on the track/bridge system, and create conditions which cause passenger discomfort.

NOTE 2 Deformation and vibration limits are either explicit or implicit in the bridge stiffness criteria given in A.2.9.4.1(2).

NOTE 3 Limits of deformation and vibration to be taken into account for the design of temporary railway bridges can be set by the National Annex.

NOTE 4 Special requirements for temporary bridges depending upon the conditions in which they are used (e.g. special requirements for skew bridges) can be set by the National Annex.

(2) Checks on bridge deformations shall be performed for traffic safety purposes for the following items:

— vertical accelerations of the deck (to avoid ballast instability and unacceptable reduction in wheel rail contact forces, see A.2.9.4.2.1);

— vertical deflection of the deck throughout each span (to ensure acceptable vertical track radii, passenger comfort and providing a minimum level of stiffness to assist with reducing future maintenance needs, see A.2.9.4.2.3(1) and A.2.9.4.3.2);

— unrestrained uplift at the bearings (to avoid premature bearing failure);

— vertical deflection of the end of the deck beyond bearings (to avoid destabilizing the track, limit uplift forces on rail fastening systems and limit additional rail stresses, see prEN 1991-2:2021, 8.5.4.5.2);

— twist of the deck measured along the centre line of each track on the approaches to a bridge and across a bridge (to minimize the risk of train derailment, see A.2.9.4.2.2);

NOTE 1 A.2.9.4.2.2 contains a mix of traffic safety and passenger comfort criteria that satisfy both traffic safety and passenger comfort requirements.

— rotation of the ends of each deck about a transverse axis or the relative total rotation between adjacent deck ends (to limit additional rail stresses (see prEN 1991-2:2021, 8.5.4), limit uplift forces on rail fastening systems and limit angular discontinuity at expansion devices and switch blades, see A.2.9.4.2.3(2));

— longitudinal displacement of the end of the upper surface of the deck due to longitudinal displacement and rotation of the deck end (to limit additional rail stresses and minimize disturbance to track ballast and adjacent track formation, see prEN 1991-2:2021, 8.5.4.5.2);

— horizontal transverse deflection (to ensure acceptable horizontal track radii, see A.2.9.4.2.4 Table A.2.13);

— horizontal rotation of a deck about a vertical axis at ends of a deck (to ensure acceptable horizontal track geometry and passenger comfort, see A.2.9.4.2.4 Table A.2.13);

— limits on the first natural frequency of lateral vibration of the span to avoid the occurrence of resonance between the lateral motion of vehicles on their suspension and the bridge, see A.2.9.4.2.4(4).

NOTE 2 There are other implicit stiffness criteria in the limits of bridge natural frequency given in prEN 1991-2:2021, 8.4.4 and when determining dynamic factors for real trains in accordance with prEN 1991-2:2021, 8.4.6.4 and prEN 1991-2:2021, Annex C.

(3) Checks on bridge deformations should be performed for passenger comfort, i.e. vertical deflection of the deck to limit coach body acceleration in accordance with A.2.9.4.3.

NOTE The limits given in A.2.9.4.2 and A.2.9.4.3 take into account the mitigating effects of track maintenance (for example to overcome the effects of the settlement of foundations, creep, etc.).

A.2.9.4.2 Criteria for traffic safety

A.2.9.4.2.1 Vertical acceleration of the deck

(1) Where a dynamic analysis is necessary, the maximum peak vertical acceleration of the deck due to rail traffic actions shall be verified at the serviceability limit state for the prevention of track instability, for traffic safety reasons.

NOTE The requirements for determining whether a dynamic analysis is necessary are given in prEN 1991-2:2021, 8.4.4.

(2) Where a dynamic analysis is necessary, it shall be in accordance with the requirements given in prEN 1991-2:2021, 8.4.6.

NOTE For characteristic rail traffic actions, see prEN 1991-2:2021, 8.4.6.1.

(3) The maximum peak values of bridge deck acceleration calculated along each track shall not exceed the following design values:

— a_{bt} for ballasted track,

— a_{df} for ballastless tracks with track and structural elements designed for high speed traffic,

for all members directly supporting the track considering frequencies (including consideration of associated mode shapes) up to the greater of:

— 30 Hz;

— 1,5 times the frequency of the fundamental mode of vibration of the member being considered;

— the frequency of the third mode of vibration of the member.

NOTE The limiting values of maximum peak vertical acceleration are as follows, unless the National Annex gives different values:

— a_{bt} = 3,5 m/s^2 in general;

— a_{bt} = 5,0 m/s^2 for a zone with a distance along the track of not exceeding 2 sleeper spacings;

— a_{df} = 5 m/s^2.

A.2.9.4.2.2 Deck twist

(1) The deck twist along each track shall be verified on the approach to the bridge, across the bridge and for the departure from the bridge (see A.2.9.4.1(2)).

(2) The twist of the bridge deck and approaches shall be calculated taking into account the characteristic values of load model 71 as well as SW/0 or SW/2 as appropriate multiplied by Φ_{dyn} and α and load model HSLM and real trains including centrifugal effects, all in accordance with prEN 1991-2:2021, 8.3, 8.4, 8.5.1 and 8.8.

(3) The requirements for verification methods and/or prescribed design solutions for approaches and transition zones should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

(4) The maximum twist t [mm/3 m] of a track gauge s_g [m] of 1,435 m measured over a length of 3 m (see Figure A.2.1) should not exceed the values given in Table A.2.12.

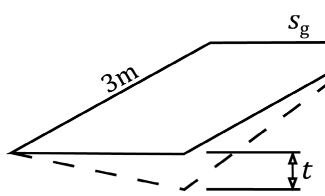


Figure A.2.1 — Definition of deck twist

Table A.2.12 — Limiting values of deck twist

Train speed V_T km/h	**Maximum twist t** mm/3 m
$V_T \leq 120$	$t \leq t_1$
$120 < V_T \leq 200$	$t \leq t_2$
$V_T > 200$	$t \leq t_3$

NOTE The limiting values for maximum twist t are as follows, unless the National Annex gives different values, including values for a track with a different gauge, where relevant:

— t_1 = 4,5;

— t_2 = 3,0;

— t_3 = 1,5.

(5) The total track twist due to any twist which is present in the track when the bridge is not subject to rail traffic actions (for example in a transition curve), plus the track twist due to the total deformation of the bridge resulting from rail traffic actions, shall not exceed t_T.

NOTE The value for t_T is 7,5 mm/3 m, unless the National Annex gives a different value.

A.2.9.4.2.3 Vertical deformation of the deck

(1) For all structural configurations loaded with the characteristic combinations of load model 71 and SW/0 as appropriate multiplied by the dynamic factor Φ_{dyn} and α and HSLM and real trains including centrifugal effects in accordance with prEN 1991-2:2021, 8.3, 8.4, 8.5.1 and 8.8 the maximum total vertical deflection measured along any track due to rail traffic actions shall not exceed $L/600$.

NOTE 1 Additional requirements for limiting vertical deformation for bridges can be given in the National Annex.

NOTE 2 Additional limitations on the deformation of bridge deck ends are implicit in prEN 1991-2:2021, 8.5.4.5.2.

(2) Minimum requirements for limits of angular rotations at the end of decks (shown as θ_1, θ_2 and θ_3 in Figure A.2.2) may be specified.

NOTE The additional limits of angular rotations can be set by the National Annex.

(3) Additional requirements for limits of angular rotations at the end of decks should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

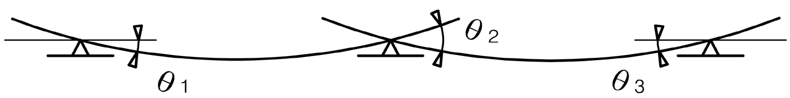


Figure A.2.2 — Definition of angular rotations at the end of decks

NOTE Limitations on the vertical displacement of bridge deck ends beyond bearings are given in prEN 1991-2:2021, 8.5.4.5.2.

A.2.9.4.2.4 Transverse deformation and vibration of the deck

(1) Transverse deformation and vibration of the deck shall be checked for characteristic combinations of load model 71 and SW/0 as appropriate multiplied by the dynamic factor Φ_{dyn} and α (or real train with the relevant dynamic factor if appropriate), wind loads, nosing force, centrifugal forces in accordance with prEN 1991-2:2021, Clause 8 and the effect of a transverse temperature differential across the bridge.

(2) The transverse deflection δ_h at the top of the deck should be limited to ensure:

— a horizontal angle of rotation of the end of a deck about a vertical axis not greater than the limiting values α_1 to α_3 given in Table A.2.13 (NDP); or

— the change of radius of the track across a deck is not greater than the limiting values r_{tr1} to r_{tr3} (for single deck) or r_{tr4} to r_{tr6} (for multi-deck) in Table A.2.13 (NDP); or

— at the end of a deck the differential transverse deflection between the deck and adjacent track formation or between adjacent decks does not exceed a limiting value $\Delta\delta_h$.

NOTE 1 The transverse deflection δ_h includes the deformation of the bridge deck and the substructure (including piers, piles and foundations).

NOTE 2 Limiting values for α_1 to α_3 and r_{tr1} to r_{tr3} are given in Table A.2.13 (NDP), unless the National Annex gives different values.

NOTE 3 The maximum differential transverse deflection $\Delta\delta_h$ can be set in the National Annex.

(3) The change of the radius of the track across a deck may be determined using Formula (A.2.1):

$$r_{rt} = \frac{L^2}{8\delta_h} \tag{A.2.1}$$

Table A.2.13 (NDP) — Maximum horizontal rotation and maximum change of radius of curvature

Train speed V_T km/h	Maximum horizontal rotation radian	Maximum change of radius of curvature m	
		Single-span deck	**Multi-span deck**
$V_T \leq 120$	$\alpha_1 = 0{,}003\,5$	$r_{tr1} = 1\,700$	$r_{tr4} = 3\,500$
$120 < V_T \leq 200$	$\alpha_2 = 0{,}002\,0$	$r_{tr2} = 6\,000$	$r_{tr5} = 9\,500$
$V_T > 200$	$\alpha_3 = 0{,}001\,5$	$r_{tr3} = 14\,000$	$r_{tr6} = 17\,500$

(4) The fundamental natural frequency of lateral vibration of a span should not be less than f_{h0}, when evaluated in each span for the mode of deformation of the deck considering the supports as fixed, i.e. not including the lateral flexibility of piers or abutments.

NOTE The value for f_{h0} is 1,2 Hz, unless the National Annex gives a different value.

A.2.9.4.2.5 Longitudinal displacement of the deck

(1) Where the rails are continuous over discontinuities in the support to the track (e.g. a discontinuous structure), then the longitudinal displacement of the deck shall be limited.

NOTE 1 Limitations on the longitudinal displacement of the ends of decks are given in prEN 1991-2:2021, 8.5.4.5.2.

NOTE 2 See also A.2.9.4.2.3.

A.2.9.4.3 Criteria for passenger comfort

A.2.9.4.3.1 Vertical acceleration

(1) The level of passenger comfort and associated limiting value for the vertical acceleration should be as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

NOTE 1 The limiting values for vertical acceleration for the different levels of passenger comfort are given in Table A.2.14.

NOTE 2 Passenger comfort depends on the vertical acceleration b_{V} inside the coach during travel on the approach to, passage over and departure from the bridge.

Table A.2.14 — Levels of comfort and associated vertical acceleration limits

Level of comfort	Limiting value of vertical acceleration b_{v} m/s^2
Very good	1,0
Good	1,3
Acceptable	2,0

(2) The requirements for passenger comfort for temporary bridges should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

(3) The vertical acceleration b_{V} may be determined by a dynamic vehicle/bridge interaction analysis (see A.2.9.4.3.3).

(4) For exceptional structures, e.g. continuous beams with widely varying span lengths or spans with wide variations in stiffness, a specific dynamic calculation should be carried out.

(5) Alternatively, other than for exceptional structures, for spans up to 120 m the vertical vehicle acceleration may be limited to the values given in Table A.2.14 by limiting the maximum vertical deflection δ_{V} along the centre line of the track of railway bridges (see A.2.9.4.3.2).

A.2.9.4.3.2 Vertical deflection

(1) Where vertical deflection is limited in order to limit vertical acceleration, the vertical deflections δ_{V} should be determined with load model 71 multiplied by the factor Φ_{dyn} and with the value of α = 1, in accordance with prEN 1991-2:2021, Clause 8.

(2) For bridges with two or more tracks, only one track should be loaded.

(3) The limiting values of vertical deflection should be taken from Figure A.2.3, for span length L [m], and modified using the clauses below to take account of:

— the train speed V_T [km/h];

— the number of spans; and

— the configuration of the bridge (simply supported beam, continuous beam).

NOTE The values of L/δ_V given in Figure A.2.3 are given for a succession of simply supported beams with three or more spans.

(4) The modification factors should not be applied to the limit of L/δ_V = 600.

(5) For spans longer than 120 m a special analysis should be undertaken.

NOTE The values of L/δ_V given in Figure A.2.3 are valid for span lengths up to 120 m.

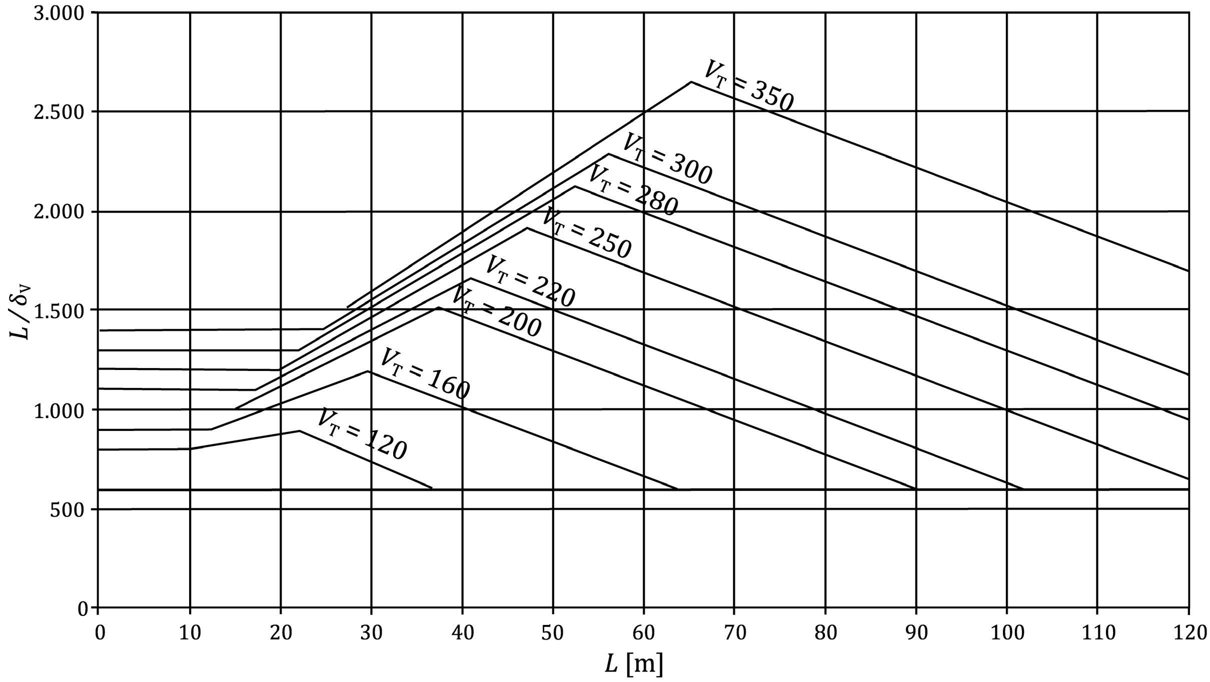


Figure A.2.3 — Maximum permissible vertical deflection δ_V for railway bridges with 3 or more successive simply supported spans corresponding to a permissible vertical acceleration of b_V = 1 [m/s^2] in a coach for speed V_T [km/h]

(6) The limiting values of vertical deflection may be modified for levels of comfort other than "very good" by dividing the values of L/δ_V given in Figure A.2.3 by $b_v^{'}$ [m/s^2] where $b_v^{'}$ is the maximum permissible vertical acceleration appropriate to the comfort level from Table A.2.14.

NOTE The limiting values of L/δ_V given in Figure A.2.3 are given for b_V = 1,0 m/s^2 which may be taken as providing a “very good” level of comfort.

(7) For a bridge comprising either a single span or a succession of two simply supported beams or two continuous spans the values of L/δ_V given in Figure A.2.3 should be multiplied by 0,7.

(8) For continuous beams with three or more spans the values of L/δ_{V} given in Figure A.2.3 should be multiplied by 0,9.

A.2.9.4.3.3 Dynamic vehicle/bridge interaction analysis

(1) Where a vehicle/bridge dynamic interaction analysis is required the analysis should take account of:

a) a series of vehicle speeds up to the maximum speed specified;

b) characteristic loading of the real trains specified for the individual project in accordance with prEN 1991-2:2021, 8.4.6.1.1;

c) dynamic mass interaction between vehicles in the real train and the structure;

d) the damping and stiffness characteristics of the vehicle suspension;

e) a sufficient number of vehicles to produce the maximum load effects in the longest span;

f) a sufficient number of spans in a structure with multiple spans to develop any resonance effects in the vehicle suspension.

(2) Requirements for taking track roughness into account in the vehicle/bridge dynamic interaction analysis should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

A.2.9.5 Foundation movements

(1) The design value of serviceability criteria $C_{\mathrm{d,SLS}}$ for foundation movement shall be selected during the design of the bridge.

NOTE 1 Serviceability criteria for foundation movement can include settlement, relative (or differential) settlement, rotation, tilt, relative deflection, angular distortion, deflection ratio, horizontal displacement and vibration amplitude. See EN 1997-1.

NOTE 2 Limiting values for foundation movement can be set by the National Annex.

(2) Effects of foundation movements should be taken into account if they are considered significant compared to the effects from direct actions or as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

NOTE Settlements vary monotonically (in the same direction) with time and are relevant from the time they give rise to effects in the structure (i.e. after the structure, or a part of it, becomes statically indeterminate). In addition, in the case of a concrete structure or a structure with concrete elements, there may be an interaction between the development of settlements and creep of concrete members.

(3) Where the structure is very sensitive to foundation movements, uncertainty in the assessment of these movements should be taken into account.

(4) Uneven settlements should be represented by a set of values corresponding to differences (compared to a reference level) of settlements between individual foundations or parts of foundations, $d_{\mathrm{set},i}$ (i is the number of the individual foundation or part of foundation).

(5) The differences of settlements of individual foundations or parts of foundations, $d_{\mathrm{set},i}$, should be taken into account as representative values in accordance with EN 1997 (all parts) with due regard for the construction process of the structure.

NOTE Methods for the assessment of settlements and associated uncertainties are given in EN 1997 (all parts).

(6) The actions and combinations of actions to be taken into account in the calculation of settlement should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

NOTE In some situations it can be necessary to take into account a proportion of the variable actions such as traffic.

(7) In the absence of control measures, the permanent action representing settlements should be determined as follows:

— the representative values $d_{\mathrm{set},i}$ are assigned to all individual foundations or parts of foundations;

— two individual foundations or parts of an individual foundation, selected in order to obtain the most unfavourable effect, are subject to a settlement $d_{\mathrm{set},i} \pm \Delta d_{\mathrm{set},i}$, where $\Delta d_{\mathrm{set},i}$ takes account of uncertainties attached to the assessment of settlements.

A.2.10 Fatigue

(1) The partial factor for fatigue actions γ_{Ff} should be used for fatigue verifications.

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

(2) Structural members designed using the damage tolerant method for fatigue shall be designed to permit inspection and maintenance.

NOTE Material related guidance on damage tolerant method is given in the other relevant Eurocodes. Regarding design to permit inspection and maintenance, see also A.2.5.

A.2.11 Bridge components

A.2.11.1 Tension components for cable supported bridges

(1) Tension components for cable supported bridges shall be in accordance with EN 1993-1-11.

(2) Where a cable supported bridge is erected using control of deformations during cable installation, then relevant permanent actions due to self-weight G and prestrain (prestressing) in the cables P may be treated as correlated and the same partial factor applied to G and P actions.

NOTE Applicable values of γ_{G} which are also used for $\gamma_{\mathrm{P}} = \gamma_{\mathrm{G}}$ are given in Table A.2.10 (NDP). See also EN 1993-1-11.

(3) In cases where self-weight G and prestress in the cables P are treated as uncorrelated, different partial factors should be applied to G and P actions.

NOTE Values of γ_{P} for the uncorrelated case are given in relevant material parts.

(4) Minimum strength requirements for elements connecting the main cables, hangers or stay cables to the structure should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

NOTE Strength of connecting elements lower than the strength of the main cables, hangers or stay cables can create a weak link and lead to strain and deformation localization.

(5) After completion of the bridge, the adjustment of each stay cable should be possible from at least one of its terminations.

(6) The range of adjustment should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

NOTE The range of adjustment can cover effects including, for example, unintended deviations of the dead load; deformations due to creep and shrinkage; geometrical imperfections; differential settlement of piers; etc.

(7) All stay cables/hangers for bridges should be replaceable.

(8) Replacement of one stay cable/hanger at a time should be verified for each stay cable/hanger.

(9) The actions to be considered for the stay cable/hanger replacement design situation should be specified.

NOTE Actions to be considered for the stay cable/hanger replacement design situation can be set by the National Annex.

(10) Accidental loss of one stay cable/hanger should be verified for each stay cable/hanger, including dynamic effects of the stay cable/hanger break, in accordance with EN 1993-1-11.

(11) Additional requirements for the accidental stay cable/hanger loss should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

NOTE For example, depending on the spacing of stay cables/hangers, it can be possible for scenarios such as a vehicle fire or vehicle impact to cause simultaneous loss of more than one stay cable/hanger.

(12) Additional requirements for an accidental situation where a fire in a vehicle affects the main cable of a suspension bridge should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

A.2.11.2 Bearings

(1) Forces and movements on structural bearings, and associated forces imposed by the bearings on the structure, shall be determined in accordance with Annex G, for non-seismic design situations.

A.2.11.3 Anti-seismic devices

(1) Anti-seismic devices shall be in accordance with the design requirements given in EN 1998 (all parts).

A.2.11.4 Expansion joints

A.2.11.4.1 Scope

(1) A.2.11.4 should be applied for expansion joints that conform with a relevant harmonized technical specifications.

NOTE In rail bridges without public access, the traffic loading is carried over structural discontinuities by the rails or by rail expansion devices. Expansion joints or cover plates can be provided to retain and support the track and ballast across structural discontinuities.

(2) Forces and movements used in the specification and design of expansion joints should be defined in accordance with A.2.11.4.

(3) Effects of the expansion joint on the structure should be determined in accordance with A.2.11.4.

A.2.11.4.2 Performance characteristics

(1) The performance characteristics that are required for the expansion joint to be compatible with the performance of the bridge should be specified, using the categories defined in the European Technical Product Specification.

NOTE 1 The relevant EADs describe product performance characteristics. The following are dependent on the conceptual arrangement of the structure:

— movement capacity;

— user categories;

— action categories;

— minimum and maximum operating temperatures;

— design service life (including envisaged fatigue performance);

— seismic behaviour.

NOTE 2 Other performance characteristics which are not directly related to the structural design but which are relevant to the specification of the expansion joint include: maintainability, cleanability, resistance to wear, watertightness, safety in case of fire, release of dangerous substances, gaps and levels, skid resistance, drainage capacity, protection against noise, energy economy and durability.

A.2.11.4.3 Movement capacity

(1) The movement capacity for the expansion joint shall be specified.

(2) The movement capacity should be specified at ULS, unless otherwise specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

(3) Displacements and rotations at the location of the expansion joint should be calculated in accordance with G.7.4, as for bearings.

NOTE Displacements and rotations can arise from the behaviour of materials and effects of actions including traffic, thermal actions, creep, shrinkage, settlement, etc.

(4) The movement capacity should be stated in the longitudinal, transverse and vertical directions referenced to the joint direction.

NOTE 1 Where expansion joints are installed at a skew angle, longitudinal movements of the bridge deck result in both longitudinal and transverse displacements of the expansion joint.

NOTE 2 In wide or curved bridges, there can be an interaction between longitudinal bridge movements and transverse displacements of the expansion joint.

NOTE 3 Where the bridge has a longitudinal or transverse gradient, but the bearings are installed flat, movements of the bridge deck can result in a change of gradient across the expansion joint and an effective vertical displacement of the joint.

NOTE 4 Angular rotation of the deck-end can increase the movement experienced at the joint, due to the distance between the neutral axis and the vertical position of the expansion joint.

(5) The rotational movement capacity should be specified.

NOTE The rotational movement capacity can be relevant for the performance of an expansion joint.

(6) Where it is intended to retain the expansion joint in place during the replacement of bearings, then the vertical movement capacity of the expansion joint should be specified to include the allowance for lifting the bridge to replace the bearings (see G.3.4).

A.2.11.4.4 Actions applied to expansion joint

(1) The actions that apply to the expansion joint shall be specified.

NOTE The relevant EADs generally define the performance by the user category and the action category.

A.2.11.4.5 Effect of the expansion joint on the structure

(1) The connection between the expansion joint and the structure shall be designed to resist the forces transmitted by the expansion joint.

(2) Structural members that support the expansion joint shall be designed to resist the actions transmitted by the expansion joint.

NOTE prEN 1991-2:2021, 6.6.2(2) provides requirements for additional dynamic amplification factor in the vicinity of expansion joints.

(3) Structural members of a bridge to which expansion joint is connected should be designed such that the possible overloading of the expansion joint can occur without damaging the structure or the water impermeable layer of the bridge.

NOTE Supplementary guidance can be given in the National Annex.

A.2.11.4.6 Design of supporting members

(1) Structural members that support the expansion joint should be detailed to be compatible with the characteristics of the expansion joint.

NOTE Information about the expansion joint that is needed for the design of the works can include:

— nominal dimensions to respect in the abutment or in this part of the structure in order to allow a correct placement of the expansion joint, in particular the distance between the parts of the structure;

— conditions of levelling (height shift, change in slope, etc.) to the adjacent surface of the joint (e.g. safety in use for comfort);

— provisions for the connection to the bridge waterproofing and/or drainage;

— minimum grade of concrete in the connection area;

— allowable deformation of the adjacent structure;

— allowable location of butt joints of the expansion joint on site.

(2) Supporting members should be designed such that deflections under the frequent combination do not exceed the lesser of: 5 mm under the frequent combination of actions, or, the limit specified for the particular type of expansion joint.

A.2.11.4.7 Expansion joint schedule

(1) An expansion joint schedule should be prepared.

NOTE 1 The format of the expansion joint schedule can be given in the National Annex.

NOTE 2 The expansion joint schedule can include:

— structural performance characteristics including movement capacity, user categories, action categories, minimum and maximum operating temperatures, fatigue loading traffic categories, design service life, seismic behaviour;

- geometric arrangement of the expansion joints, including length of joint, relationship of the joint to the bridge articulation and bridge geometry;
- interface with other functional or structural features, including walkways, parapets, ducts and services, noise requirements;
- installation information, including prefixing of the expansion joint gap, adjustments for actual installation temperature, provisions for anchorage and connections, strength requirements for connection concrete;
- durability requirements, including access for inspection and maintenance, cleaning of the joint, drainage of the deck and joint, interface with waterproofing systems;
- protocol of the regular inspection, where all necessary prefixing actions and adjustment values are noted.

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) Clauses 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 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) that fulfils the provisions specified in the Eurocodes is achieved and the assumptions given in 1.2 are satisfied.

NOTE 1 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.

NOTE 2 The National Annex can differentiate between technical management measures for the structures covered in the different parts of Annex A.

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 For the selection of appropriate technical management measures, see B.8.

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 should be as specified by the relevant authority or, where not specified, as 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.

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.

(2) Additional project-specific requirements for design checking should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

(3) Self-checking shall be performed for all designs.

(4) Design checking should cover:

— loads, models for calculation of loads and design situations;

— structural and geotechnical models, calculation of effects of actions 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 assumptions 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.

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.		

(2) Additional project-specific requirements for levels of inspection should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

(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, in accordance with 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 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 Clauses 5 to 8 and Annex A are given. This Informative 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 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 Informative 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 the relevant authority.

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 (8).

(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 5.1).

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 actions, effects of actions, material resistances, and system-effects mean that the reliability-based approach gives a significantly better representation of the limit state than the partial factor design format.

NOTE Design situations that are not covered by the partial factor design format can include:

— situations where relevant actions or hazard scenarios are not covered by EN 1991 (all parts);

— 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.

NOTE Minimum conditions can be defined in the National Annex.

(6) Conditions for the use of reliability-based methods should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

(7) 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 the relevant authority.

(8) 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.

(9) 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

Method	Description	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, environmental and monetary losses, etc.)	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 characteristic values

C.3.2.3 Reliability-based and risk-informed approaches

NOTE Reliability-based and risk-informed approaches allow a more detailed representation of uncertainties.

(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 relevant prior information and available new evidence, using Bayesian probability theory.

(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 of populations.

C.3.3 Reliability-based design

C.3.3.1 General

(1) In the reliability-based approach, structural design decisions shall be based on 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 undesirable 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 failure surface of the limit state function g() may be represented by Formula (C.1):

$$\mathrm{g}\left(\boldsymbol{X}\left(t\right)\right)=0 \tag{C.1}$$

The time-variant basic variables $\boldsymbol{X}(t)$ may be represented by Formula (C.2):

$$\boldsymbol{X}\left(t\right)=X_1\left(t\right),X_2\left(t\right),\ldots \tag{C.2}$$

The domain of adverse (failure), events $\Omega(\boldsymbol{X}(t))$ is given by Formula (C.3):

$$\Omega\left(\boldsymbol{X}\left(t\right)\right)=\left\{\mathrm{g}(\boldsymbol{X}\left(t\right))<0\right\} \tag{C.3}$$

(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.3.2.

(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.

(2) Time-invariant reliability analysis may also be used for problems that can be transformed such that they do not depend on time.

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):

$$\boldsymbol{P}_{\mathrm{f}} = \int_{\Omega(\boldsymbol{X})} \mathrm{f}_{\boldsymbol{X}}(\boldsymbol{X})\boldsymbol{dx} \tag{C.4}$$

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 $\boldsymbol{X}$.

NOTE In a time-invariant reliability analysis, time-variable actions 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 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 $\boldsymbol{P}_{\mathbf{f}}$ and $\boldsymbol{\beta}$

$\boldsymbol{P}_{\mathbf{f}}$	$\mathbf{10^{-1}}$	$\mathbf{10^{-2}}$	$\mathbf{10^{-3}}$	$\mathbf{10^{-4}}$	$\mathbf{10^{-5}}$	$\mathbf{10^{-6}}$	$\mathbf{10^{-7}}$
$\beta = -\Phi^{-1}(P_{\mathrm{f}})$	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, the reliability requirements are 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).

— The service life cost optimal 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.

NOTE Both a target and a minimum reliability level are used in code calibration for a given set of structures. Individual cases in the set of structures with a lower reliability than the target are accepted.

(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 designers. For the last case, the requirements are relevant to the relevant national authorities, see Clause 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.

(2) This relative comparison should be made based on similar probabilistic models.

(3) 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 and 50-year reference periods for persistent and transient (fundamental) and fatigue design situations in ULS for structures included in the scope of Clauses A.1 and A.2 are given in Table C.3 (NDP), unless the National Annex gives different values.

NOTE 2 The partial factors given in Clauses A.1 and A.2 are expected to lead in general to a structure with a reliability index β for 50-year reference period greater than the values given in Table C.3 (NDP) for a 50-year reference period.

Table C.3 (NDP) — Target values for reliability index β for different consequence classes (for persistent and transient (fundamental) and fatigue design situations in ULS) relevant to structures in the scope of Clauses A.1 and A.2

Consequence class[a]	1-year reference period β	50-year reference period	
		β	$P_{f,50}$
CC3	5,2	4,3	~ 10^{-5}
CC2	4,7	3,8	~ 10^{-4}
CC1	4,2	3,3	~ 10^{-3}

[a] Regarding CC0 and CC4, see also 4.3(2) and 4.3(3).

NOTE 3 The β values given in Table C.3 (NDP) for 1-year reference period, corresponding to the values for 50-year reference period, 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 values for 1-year reference period, which is approached, for instance, in the case for wind dominated load combinations in structures with low variability in resistance.

Lower values of β for 1-year period 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.

NOTE 4 The minimum reliability level can be defined as a maximum deviation from the target.

(4) When referring to the 1-year reliability index β, the target should be met for every year of the required (or chosen) design service life (or remaining service life) of the structure.

C.3.4.3 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 50-year reference period and should be chosen as specified in C.3.4.2(3).

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 for reliability levels, taken as representative averages over the considered design situations.

NOTE Code calibration is performed by the relevant authority.

(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(3) should be used.

(4) 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.

(5) 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.

(6) The reliability targets specified according to C.3.4.2(3) 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):

$$R_{\mathrm{d}} - E_{\mathrm{d}} \geq 0 \tag{C.6}$$

where the design values of the resistance R_{d} and the effect of actions E_{d} are obtained from Formulae (C.7) and (C.8):

$$R_{\mathrm{d}} = R_{\mathrm{d}}\left(\boldsymbol{X}_{\mathrm{d}};\boldsymbol{a}_{\mathrm{d}};\theta_{\mathrm{d}};\boldsymbol{F}_{\mathrm{d,R}}\right) \tag{C.7}$$

$$E_{\mathrm{d}} = E_{\mathrm{d}}\left(\boldsymbol{F}_{\mathrm{d}};\boldsymbol{a}_{\mathrm{d}};\theta_{\mathrm{d}};\boldsymbol{X}_{\mathrm{d}}\right) \tag{C.8}$$

where

$\boldsymbol{X}_{\mathrm{d}}$ are vectors of design values of material properties independent of 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;

$\boldsymbol{F}_{\mathrm{d,R}}$ are vectors of design values of actions depending on the material properties, when relevant;

$\boldsymbol{F}_{\mathrm{d}}$ are vectors of design values of actions independent on the material properties.

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 actions (F_d) may be defined as the multiplication of the characteristic value F_k by a corresponding partial safety factor γ_F (i.e. $F_d = \gamma_F F_k$).

(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. $X_d = X_k / \gamma_M$).

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):

$$\gamma_F = \gamma_{Sd}\gamma_f \quad \text{(C.9)}$$

$$\gamma_M = \gamma_{Rd}\gamma_m \quad \text{(C.10)}$$

where

γ_{Sd} covers the model uncertainty in actions and action effects;

γ_f covers uncertainty in representative values of actions;

γ_{Rd} covers the model uncertainty in structural resistance, also accounting, when relevant, the bias in resistance model, see Annex D;

γ_m covers uncertainty in basic variables describing the resistance.

(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):

$$Y_k = F_Y^{-1}(p) \quad \text{(C.11)}$$

where

Y is the basic variable, representing e.g. action, resistance and/or model uncertainty;

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, i.e. representing both time-dependent and time-independent uncertainties.

(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 format method, see C.4.4.2, and the optimization method, see C.4.4.3, may be used to determine partial safety factors.

NOTE The partial factors of the Eurocodes on the resistance side are mainly based on the design value method. The partial factors for actions given in Annex A are partly calibrated to previous standards and experience and partly calibrated using theoretical methods.

C.4.4.2 The design value format 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 format method' in Formula (C.12):

$$Y_\mathrm{d} = F_Y^{-1}\left(\Phi\left(-\alpha_Y \beta\right)\right) \tag{C.12}$$

where

Y	is the basic variable, representing e.g. action, resistance and/or model uncertainty;
Y_d	is the design value of the basic variable Y;
F_Y	is the cumulative probability distribution function of the basic variable Y;
α_Y	with $\lvert\alpha_Y\rvert \leq 1$ is a sensitivity factor indicating the importance of Y in the reliability estimation;
β	is the target value for the 50-year reliability index according to the reliability requirement in C.3.4.

(2) When the design value format method is used then it should be applied both for actions and resistances.

(3) 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:

$$Y_\mathrm{k} = m_Y + \Phi^{-1}\left(p\right)\sigma_Y = m_Y\left(1+\Phi^{-1}\left(p\right)V_Y\right) \tag{C.13}$$

— Design value:

$$Y_\mathrm{d} = m_Y - \alpha_Y \beta \sigma_Y = m_Y\left(1-\alpha_Y \beta V_Y\right) \tag{C.14}$$

Log-normal distribution:

— Characteristic value:

$$Y_\mathrm{k} = m_Y e^{\left(-\frac{1}{2}\ln(1+V_Y^2)\right)+\Phi^{-1}\left(p\right)\sqrt{\ln(1+V_Y^2)})} \tag{C.15}$$

$$Y_{\mathrm{k}} \simeq m_Y e^{\left(\Phi^{-1}(p)V_Y\right)} \quad \text{for } V_Y < 0,2 \tag{C.16}$$

— Design value:

$$Y_{\mathrm{d}} = m_Y e^{\left(-\frac{1}{2}\ln(1+V_Y^2)\right) - \alpha_Y \beta \sqrt{\ln(1+V_Y^2)})} \tag{C.17}$$

$$Y_{\mathrm{d}} \simeq m_Y e^{(-\alpha_Y \beta V_Y)} \quad \text{for } V_Y < 0,2 \tag{C.18}$$

Gumbel distribution:

— Characteristic value:

$$Y_{\mathrm{k}} = m_Y \left(1 - V_Y \frac{\sqrt{6}}{\pi}\left(0,5772 + \ln(-\ln(p))\right)\right) \tag{C.19}$$

— Design value:

$$Y_{\mathrm{d}} = m_Y \left(1 - V_Y \frac{\sqrt{6}}{\pi}\left(0,5772 + \ln(-\ln(\Phi\left(-\alpha_Y \beta\right)))\right)\right) \tag{C.20}$$

where

m_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 Y 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 for a 50-year reference period, provided that Formula (C.21) is satisfied:

$$0,16 < \sigma_{\mathrm{E}} / \sigma_{\mathrm{R}} < 7,6 \tag{C.21}$$

where

— if Y represents a strength related variable: α_{Y} = 0,8;

— if Y represents a leading action related variable: α_{Y} = -0,7;

— If Y is dominating the reliability problem: α_{Y} = 1 (resistance); α_{Y} = -1 (action).

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.

NOTE 3 Formulae (C.19) and (C.20) are only applicable if the time-independent part of the uncertainty is negligible. If the time-independent part of the uncertainty is not negilible, both characteristic value and design value can be found by use of reliability methods separating the time-independent and time-dependent parts.

NOTE 4 In Formula (C.19) the reference period is 1 year. In Formula (C.20) the reference period is the reference period for the reliability index, e.g. 50 years.

(4) When the action model contains several basic variables, Formula (C.12) should be used for the leading variable only. For the accompanying actions the design values can be defined by Formula (C.22):

$$P\left(E > E_{\mathrm{d}}\right)=\Phi\left(-0,4\cdot 0,7\beta\right)=\Phi\left(-0,28\beta\right) \tag{C.22}$$

(5) Formulae (C.19) and (C.20) should only be used when the uncertainty of time-dependent part of a climatic action is substantially larger than the uncertainty of the time-independent part.

(6) 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 β equal to the 50-year target value, the reference period is 50 years, with β equal to the 1-year target value, 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, chosen according to C.3.4.2, is best achieved.

NOTE 1 The calibration procedure involves the following steps:

a) selecting of a set of comparable reference structures;

b) selecting and specifying a set of reliability elements, e.g. partial factors, Ψ factors;

c) designing the structures according to the selected set of reliability elements;

d) calculating the reliability indices β_i for the designed structures;

e) calculating the difference: $D=\sum_i w_i\left(\beta_i-\beta\right)^2$ where w_i is the weight factor i and β is the target reliabiliy index;

f) 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) for actions where the uncertainties associated with the time-variant part are larger than the uncertainties associated with the time-invariant parts:

— Q_1 dominating:

$$\psi_{0,2}=\frac{F^{-1}_{Q_{2,\max,\tau_1}}\left\{\Phi\left(\alpha_{Q_{2,\max,\tau_1}}\beta_{\mathrm{t}}\right)\right\}}{F^{-1}_{Q_{2,\max,T}}\left\{\Phi\left(\alpha_{Q_{2,\max,T}}\beta_{\mathrm{t}}\right)\right\}} \quad \text{with} \quad F_{Q_{2,\max,\tau_1}}\left(q\right)=F^{\left(\tau_1/T\right)}_{Q_{2,\max,T}}\left(q\right) \tag{C.23}$$

— Q_2 dominating:

$$\psi_{0,1} = \frac{F^{-1}_{Q_{1,\tau_1}}\left\{\Phi\left(\alpha_{Q_{1,\tau_1}}\beta_{\mathrm{t}}\right)\right\}}{F^{-1}_{Q_{1,\max,T}}\left\{\Phi\left(\alpha_{Q_{1,\max,T}}\beta_{\mathrm{t}}\right)\right\}} \quad \text{with} \quad F_{Q_{1,\tau_1}}(q) = F^{(\tau_1/T)}_{Q_{1,\max,T}}(q) \tag{C.24}$$

where

$F_{Q_{i,\max,T}}$	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 action intensity is assumed to be constant (i = 1; 2);
$F_{Q_{2,\max,\tau_1}}(q)$	is the extreme value distribution function for Q_2 for reference period τ_1;
$F_{Q_{1,\tau_1}}(q)$	is the cumulative distribution of Q_1 for reference period τ_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	is the target reliability for the reference period T.

NOTE 1 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 2 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 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 Informative Annex gives guidance on the procedure to be followed to directly determine by testing either the resistance-side parameters of the design equation for 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 parts of EN 1991 or in specialist literature.

NOTE 2 This Informative 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 Informative Annex do not cover testing of geotechnical materials for which the relevant part of 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 Informative Annex provides guidance on the application of 7.3(3), 8.1(3), and 8.3.6(2).

NOTE This Informative 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 actions or impact loads;

b) tests to obtain specific material properties using specified testing procedures; for instance 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;

NOTE For further guidance, see EN 1991 (all parts).

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) and d), the design values to be used should wherever practicable be derived from the test results by applying acceptable conventional statistical techniques. See D.5 to D.8.

NOTE Special techniques might be needed in order to evaluate type c) test results.

(3) Test types e), f) and 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) and 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 organization and other relevant parties, if any.

NOTE Other relevant parties, if any, can be described in the National Annex.

(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;

— if 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 suitable 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 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 relevant 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.7. 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 Formulae given in 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) 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):

$$X_{\mathrm{d}} = \eta_{\mathrm{d}} \frac{X_{\mathrm{k}}(n)}{\gamma_{\mathrm{M}}} = \frac{\eta_{\mathrm{d}}}{\gamma_{\mathrm{M}}} m_X \{1 - k_n V_X\} \tag{D.1}$$

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 "V_X known" should be used if the coefficient of variation V_X, 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):

$$s_X^2 = \frac{1}{n-1}\Sigma\left(x_i - m_X\right)^2 \tag{D.2}$$

$$V_X = s_X / m_X \tag{D.3}$$

where

x_i is the i-th measurement of the variable X.

(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 % fractile 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):

$$X_{\mathrm{d}} = \eta_{\mathrm{d}} m_X \left\{1 - k_{\mathrm{d},n} V_X\right\} \tag{D.4}$$

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 Table D.2 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 Formula (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 Clause 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.5(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.

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):

$$r_{\mathrm{t}} = \mathrm{g}_{\mathrm{rt}}\left(\underline{X}\right) \tag{D.6}$$

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 $\left(\underline{X}\right)$ 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 $r_{\mathrm{e},i}$ from the tests, the actual measured properties should be substituted into the resistance function and the theoretical values $r_{\mathrm{t},i}$ obtained for each test *i*.

(2) The points representing pairs of corresponding values ($r_{\mathrm{t},i}$, $r_{\mathrm{e},i}$) should be plotted on a diagram, as indicated in Figure D.1.

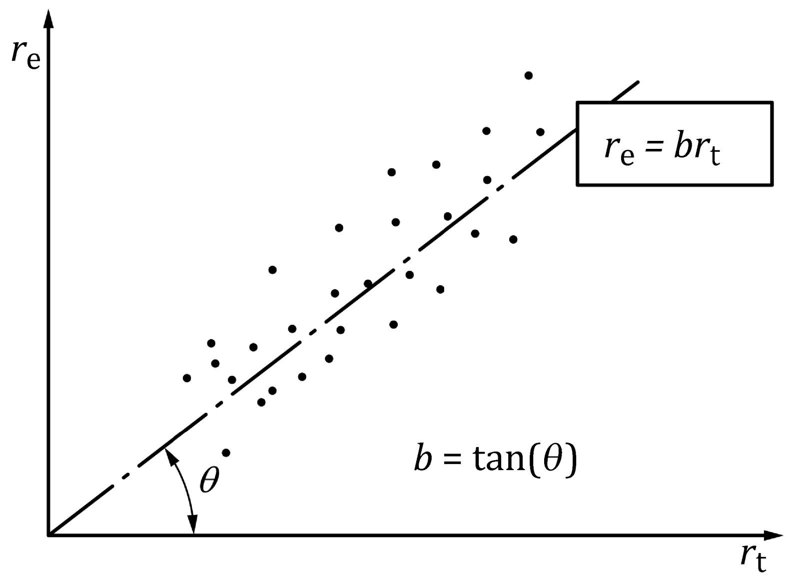


Figure D.1 — r_e - r_t diagram

(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):

$$r = br_t\delta \tag{D.7}$$

where

b is the slope;

δ is the error term with mean value one and coefficient of variation V_δ.

(2) When the resistance function is linear, the mean value of the theoretical resistance function, calculated using the mean values $\underline{X}_m$ of the basic variables, may be obtained from Formula (D.8):

$$r_m = br_t\left(\underline{X}_m\right)\delta = bg_{rt}(\underline{X}_m)\delta \tag{D.8}$$

(3) The product $b\delta_i$ for each experimental value $r_{e,i}$ should be determined from Formula (D.9):

$$b\delta_i = \frac{r_{e,i}}{r_{t,i}} \tag{D.9}$$

(4) From the values of $b\delta_i$, values of Δ_i should be obtained from Formula (D.10):

$$\Delta_i = \ln(b\delta_i) = \ln(r_{e,i}) - \ln(r_{t,i}) \tag{D.10}$$

(3) The estimated value $\overline{\Delta}$ for E(Δ) should be obtained from Formula (D.11):

$$\overline{\Delta} = \frac{1}{n}\sum_{i=1}^{n}\Delta_i \tag{D.11}$$

(4) The estimated value $s_\Delta{}^2$ for $\sigma_\Delta{}^2$ should be obtained from Formula (D.12):

$$s_\Delta^2 = \frac{1}{n-1}\sum_{i=1}^{n}\left(\Delta_i - \overline{\Delta}\right)^2 \tag{D.12}$$

(7) Formula (D.13) may be used as the slope b:

$$b = \exp\left(\overline{\Delta} + \frac{1}{2}s_\Delta^2\right) \tag{D.13}$$

D.8.2.2.4 Step 4 - Estimate the coefficient of variation of the errors

(1) 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) The scatter of the ($r_{e,i}$, $r_{t,i}$) values may 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 $r = br_t\delta = b\{X_1 \cdot X_2 \cdot ... \cdot X_j\}\delta$, the mean value $E(r)$ may be obtained from Formula (D.15):

$$E(r) = b\left\{E(X_1)\cdot E(X_2)\cdot ...\cdot E(X_j)\right\} = b\mathrm{g}_{\mathrm{rt}}(\underline{X}_{\mathrm{m}}) \quad \text{(D.15)}$$

and the coefficient of variation V_r may be obtained from the product function in Formula (D.16):

$$V_{\mathrm{r}}^2 = \left(V_\delta^2 + 1\right)\left[\prod_{i=1}^{j}\left(V_{X_i}^2 + 1\right)\right] - 1 \quad \text{(D.16)}$$

(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:

$$V_{\mathrm{r}}^2 = V_\delta^2 + V_{\mathrm{rt}}^2 \quad \text{(D.17)}$$

with $V_{X_i}^2$ as given in Formula (D.18):

$$V_{\mathrm{rt}}^2 = \sum_{i=1}^{j} V_{X_i}^2 \quad \text{(D.18)}$$

(3) If the resistance function is a more complex function of the form $r = br_t\delta = b\mathrm{g}_{rt}(X_1, ..., X_j)\delta$, the mean value $E(r)$ may be obtained from Formula (D.19):

$$E(r) = b\mathrm{g}_{\mathrm{rt}}\left(E(X_1), ..., E(X_j)\right) = b\mathrm{g}_{\mathrm{rt}}(\underline{X}_{\mathrm{m}}) \quad \text{(D.19)}$$

and the coefficient of variation V_{rt} may be obtained from Formula (D.20):

$$V_{\mathrm{rt}}^2 = \frac{VAR[\mathrm{g}_{\mathrm{rt}}(\underline{X})]}{\mathrm{g}_{\mathrm{rt}}^2(\underline{X}_{\mathrm{m}})} \cong \frac{1}{\mathrm{g}_{\mathrm{rt}}^2(\underline{X}_{\mathrm{m}})} \times \sum_{i=1}^{j}\left(\frac{\partial \mathrm{g}_{\mathrm{rt}}}{\partial X_i}\sigma_i\right)^2 \quad \text{(D.20)}$$

(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), the characteristic resistance r_k should be obtained from Formulae (D.21) to (D.26):

$$r_{\mathrm{k}} = b\mathrm{g}_{\mathrm{rt}}(\underline{X}_{\mathrm{m}})\exp\left(-k_\infty \alpha_{\mathrm{rt}} Q_{\mathrm{rt}} - k_n \alpha_\delta Q_\delta - 0{,}5Q^2\right) \quad \text{(D.21)}$$

with

$$Q_{\mathrm{rt}} = \sigma_{\ln(r_{\mathrm{t}})} = \sqrt{\ln(V_{\mathrm{rt}}^2 + 1)} \quad \text{(D.22)}$$

$$Q_\delta = \sigma_{\ln(\delta)} = \sqrt{\ln(V_\delta^2 + 1)} \tag{D.23}$$

$$Q = \sigma_{\ln(r)} = \sqrt{\ln(V_r^2 + 1)} \tag{D.24}$$

$$\alpha_{rt} = \frac{Q_{rt}}{Q} \tag{D.25}$$

$$\alpha_\delta = \frac{Q_\delta}{Q} \tag{D.26}$$

where

k_∞ is the value of k_n for $n \to \infty$, $k_\infty = 1{,}64$;

α_{rt} is the weighting factor for Q_{rt};

k_n is the characteristic fractile factor from Table D.1 for the case V_X unknown;

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):

$$r_k = b g_{rt}\left(\underline{X}_m\right) \exp\left(-k_\infty Q - 0{,}5Q^2\right) \tag{D.27}$$

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 $k_{d,n} = \alpha_R \beta = 0{,}8 \times 3{,}8 = 3{,}04$.

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):

$$r_d = b g_{rt}\left(\underline{X}_m\right) \exp\left(-k_{d,\infty} \alpha_{rt} Q_{rt} - k_{d,n}\, \alpha_\delta\, Q_\delta - 0{,}5Q^2\right) \tag{D.28}$$

where

$k_{d,\infty}$ is the value of $k_{d,n}$ for $n \to \infty$, $k_{d,\infty} = 3{,}04$;

$k_{d,n}$ is the design fractile factor from Table D.2 for the case “V_X unknown”.

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):

$$r_{\mathrm{d}} = b g_{\mathrm{rt}}\left(\underline{X}_{\mathrm{m}}\right) \exp\left(-k_{d,\infty} Q - 0,5 Q^2\right) \tag{D.29}$$

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):

$$r_{\mathrm{k}} = \eta_{\mathrm{k}} r_{\mathrm{e}} \tag{D.30}$$

where

η_{k} is a reduction factor applicable in the case of prior knowledge that may be obtained from Formula (D.31):

$$\eta_{\mathrm{k}} = 0,9 \exp\left(-2,31 V_{\mathrm{r}} - 0,5\, V_{\mathrm{r}}^2\right) \tag{D.31}$$

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):

$$r_{\mathrm{k}} = \eta_{\mathrm{k}} r_{\mathrm{em}} \tag{D.32}$$

where

η_{k} is a reduction factor applicable in the case of prior knowledge that may be obtained from Formula (D.33):

$$\eta_{\mathrm{k}} = \exp\left(-2,0 V_{\mathrm{r}} - 0,5 V_{\mathrm{r}}^2\right) \tag{D.33}$$

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):

$$\left|r_{\mathrm{ee}} - r_{\mathrm{em}}\right| \leq 0,10 r_{\mathrm{em}} \tag{D.34}$$

(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 annex

(1) This Informative Annex provides additional guidance to that given in 4.4, for enhancing the robustness of buildings and bridges during execution and use.

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 Informative 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 Informative 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.

NOTE Whilst the strategies and methods given in this Informative 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.

(2) Design for identified accidental actions should be undertaken in accordance with EN 1991 (all parts) 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 Informative Annex and designing for identified accidental actions in accordance with EN 1991 (all parts), 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 (all parts) a target level of reliability is expected to be achieved.

Table E.1 —Design for identified accidental actions and design strategies for enhanced robustness

Design for accidental actions (EN 1991 (all parts)) 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 sufficient ductility, resistance and deformation capacity and redundancy, or applying prescriptive design rules	Key members i.e. designing selected members to resist notional action(s)	Segmentation i.e. separation into distinct parts

(3) Design for situations after an accidental event should be performed on the damaged structure (i.e. not including the damaged members) using the combination of actions according to 8.3.4.3, with A_d = 0.

NOTE 1 This can apply as part of a verification of an accidental event to check against structural collapse at system level.

NOTE 2 Design strategies and design methods for enhanced robustness are given in E.3 and E.4.

E.3 Design strategies

(1) Strategies for designing structures for enhanced robustness may be selected from the following (see Table E.1):

a) Creation of alternative load paths:

— by providing sufficient ductility, resistance and deformation capacity under local damage scenarios and redundancy to the structure; and/or

— applying prescriptive design rules, such as for tying;

b) Key members: designing selected members to resist notional action(s);

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 Appropriate 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.

NOTE 3 Design strategies can also be considered as: (i) making an initial local failure less probable; (ii) limiting the total damage following assumed cases of initial local failure.

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, as agreed for a specific project by the relevant parties, satisfy the requirements for CC2 appropriately adapted and in addition consider, where appropriate: a) potential initial failure events; b) propagation of failure; c) resulting consequences; d) risks.
CC2	When specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties, either: a) For buildings: use of prescriptive design rules for ties to provide integrity, ductility and alternative load paths; or b) Design of particular structural members as ‘Key members’; and/or c) Segmentation.
CC1	No design methods to provide enhanced robustness need be applied.

NOTE 1 Prescriptive design rules are given in other Eurocodes.

NOTE 2 Values of actions for designing horizontal and vertical ties can be found in other Eurocodes.

NOTE 3 Design methods given for high consequence class structures can be used with low consequence class structures. For some special structures such use of higher level methods can be specified in the other 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.

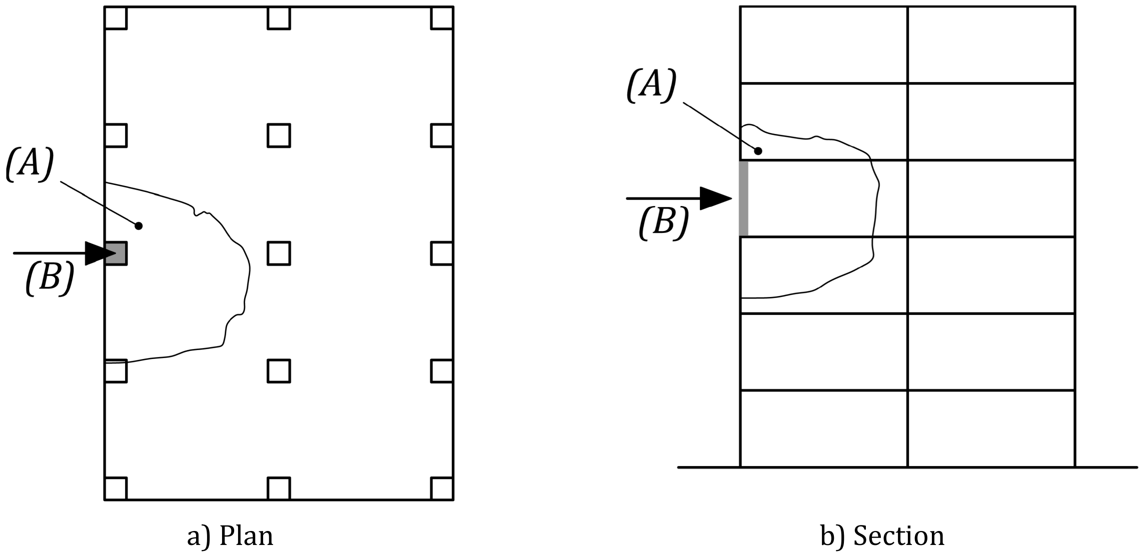
NOTE 1 Reasons for using alternatives to prescriptive designs rules include situations where they are unsuitable for a particular structure or where alternative approaches are more economically advantageous.

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.

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.



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.

NOTE 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 fatigue

F.1Use of this annex

(1) This Informative Annex provides guidance on rain-flow and reservoir counting methods for the determination of stress ranges due to 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 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 (maxima) and valleys (minima);

d) extract the maxima (peaks) in decreasing order (1, 2, …., 6) and the minima (valleys) in ascending order (1 ', 2', …, 6 ') of claim of the constraints: the path of “each drop of water” on a “dry part of the guide” makes it possible to define a half-cycle and and to classify it in a range of stresses σ(t) : i.e. each path ends when the drop falls outside the guide or when it encounters a “weted part of the guide”, see Figure F.3 (the process is finished when the whole guide is humidified and the cycles are obtained by coupling the half-cyles).

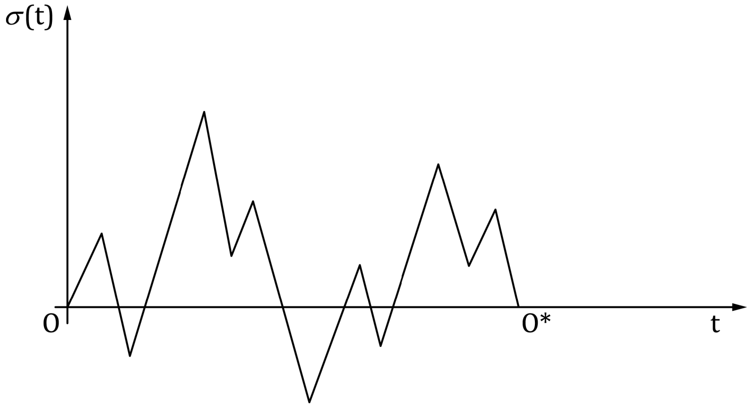


Figure F.1 — Rainflow method - Stress history

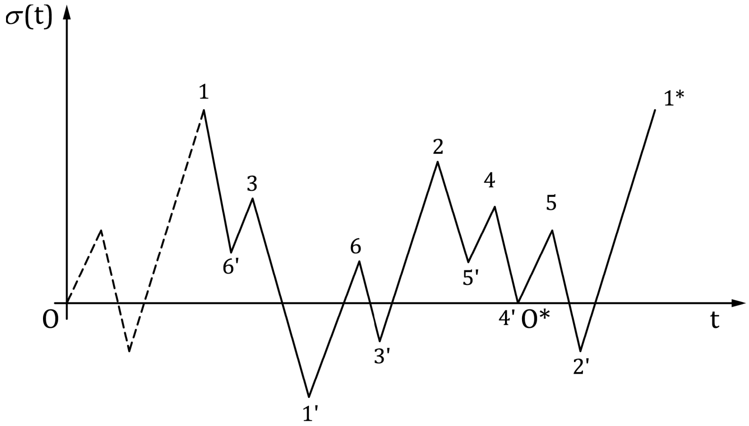


Figure F.2 — Rainflow method - Modified stress history

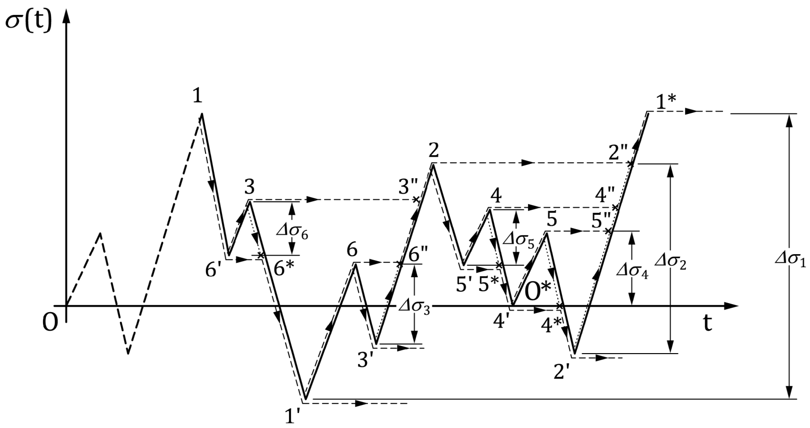


Figure F.3 — Example of rainflow method

F.4Reservoir counting method

(1) The reservoir counting method consists in the following steps:

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) empty the reservoir starting with the valleys with the lowest minima then proceed in ascending order (1’, 2’, …, 6’) untill the reservoir is empty, see c) (each evacuation operation corresponds to a cycle and the evacuated water level corresponds to the range of constraints). See Figure F.6.

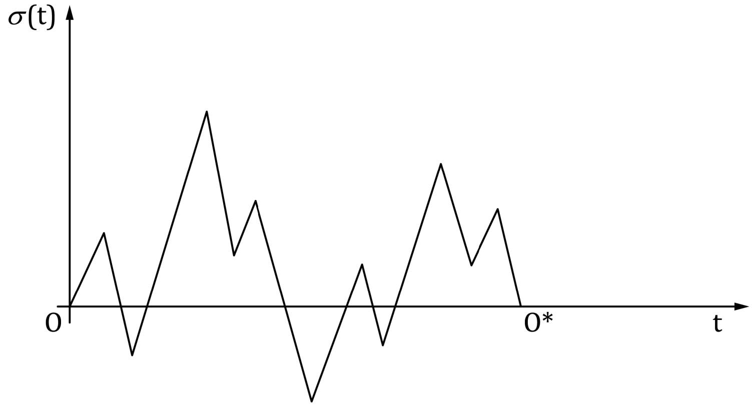


Figure F.4 — Reservoir method - Stress history

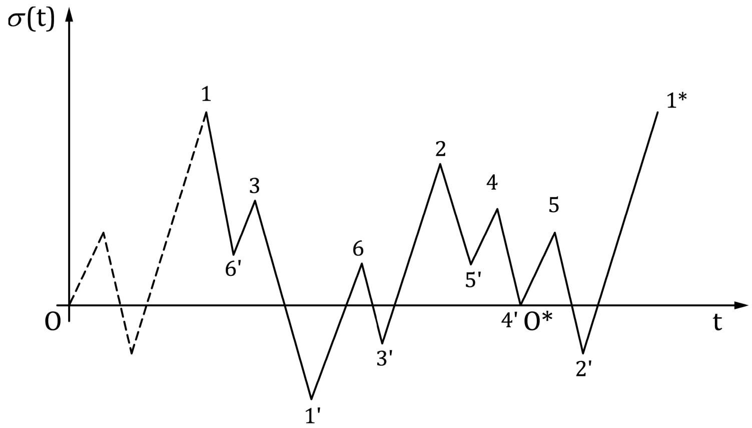


Figure F.5 — Reservoir method - Modified stress history

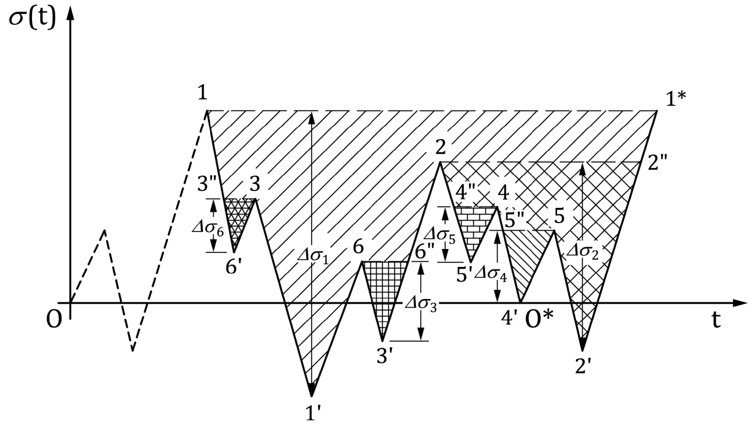


Figure F.6 — Example of reservoir method

Annex G
(normative)

Basis of design for bearings

G.1 Use of this annex

(1) This Normative Annex contains additional provisions to 8.3.3 for the design value of bearing actions and the design effects used to specify bearings.

G.2 Scope and field of application

(1) This Normative Annex may be used as the basis of design where bearings are included in other types of structure, subject to additional or amended provisions as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

NOTE 1 Minimum requirements for additional or amended provisions for bearings in other types of structure can be provided in the National Annex.

NOTE 2 For seismic design requirements, which can be more onerous than non-seismic requirements for the design of bearings, see EN 1998 (all parts). For anti-seismic devices, see EN 15129 or a relevant EAD.

(2) This Normative Annex shall be used to determine the forces imposed by the bearings on bridges in non-seismic design situations.

(3) This Normative Annex shall be used to determine the bearing specification for bridges, including forces and movements, for the design of bearings according to a relevant harmonised technical specification.

NOTE 1 A relevant harmonized technical specification can be EN 1337 (all parts) or an EAD.

NOTE 2 Movements (in structural bearings) can include translations and rotations and are enabled by sliding, rolling, rocking or deformation of the structural bearing.

G.3 General rules

G.3.1 Basic requirements

(1) A structure shall be designed considering the actions imposed by the bearings including those actions caused by the functioning of the bearing.

NOTE The functioning of the bearing (e.g. movement of the bearing) can lead to restraining effects such as frictional restraint, which affect other parts of the structure and/or other bearings.

(2) The forces and movements that are applied to a bearing under the effect of actions applied to the structure shall be obtained at ultimate and serviceability limit states.

(3) At ultimate limit state, the strength and stability of bearings should be adequate to withstand the ultimate design loads and movements of the structure, unless the conditions in (4) are met.

(4) Forces and movements at ultimate limit state may be neglected in the design of the bearing if an alternative load path is provided and the structure is designed to carry this alternative load path, assuming that a bearing fails at ultimate limit state and does not carry load, and the structure is not damaged by failure of the bearing.

(5) At serviceability limit state, the bearings should not suffer damage that would affect their correct functioning, or incur excessive maintenance costs during their design service life.

(6) Support systems should be designed in such a way that individual bearings are not subject to tensile forces in the SLS characteristic combination and that no significant redistribution of bearing forces to other bearings occurs due to accumulation of installation tolerances.

NOTE The National Annex can give criteria under which tensile forces are permitted in bearings and support systems. Special devices can be designed that are outside the scope of the relevant harmonised technical specifications, for example, tie-down systems.

G.3.2 Uncertainties

(1) The forces and movements for the bearing should account for uncertainties including the following:

— the installation procedure of the bearings;

— the construction sequence of the structure;

— the initial temperature T_0 assumed when the bearings are installed (see EN 1991-1-5);

— the range of initial bridge temperature, ΔT_0 (see EN 1991-1-5);

— foundation tilting and settlement; and

— the accuracy of the position of the superstructure, piers and abutments.

G.3.3 Bearing layout and schedule

G.3.3.1 Bearing layout

(1) A bearing layout shall be prepared in accordance with a relevant harmonised technical specification.

(2) The bearing layout should indicate the orientation of bearings in relation to the structure.

(3) The bearing layout should show the direction of a preset in relation to the overall structure, where the preset is not symmetric.

G.3.3.2 Bearing schedule

(1) A bearing schedule shall be prepared.

NOTE 1 The format of the bearing schedules are given as Table G.1 (NDP) and Table G.2 (NDP), unless the National Annex gives different formats for bearing schedules, and include:

— characteristic values of bearing forces and movements resulting from each individual action (see Table G.1 (NDP));

— design values of bearing movements and bearing loads for the relevant combinations for the ultimate limit state (see Table G.2(NDP));

— bearing movements and bearing forces for the relevant combinations for the serviceability limit state (see Table G.2(NDP));

— design value of the lowest and highest operating temperature of the bearing which can be taken as the minimum and maximum shade air temperature in accordance with EN 1991-1-5 unless the bearing is exposed to high solar radiation;

— further functional and structural features as required in a relevant harmonised technical specification).

NOTE 2 The local coordinate system used in Table G.1 (NDP) and Table G.2 (NDP) is shown in Figure G.1.

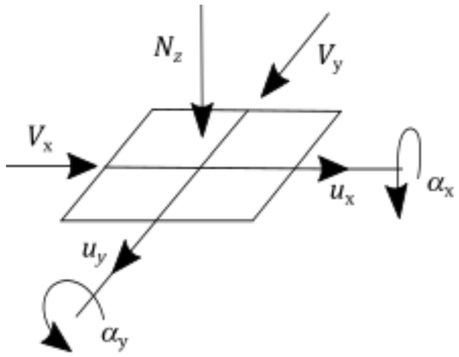


Figure G.1 — Local coordinate system used in Table G.1 (NDP) and Table G.2 (NDP)

(2) The sizing of the sliding plate of a bearing should be specified under a defined combination of actions.

NOTE The combination to be used is the ULS persistent design situation, unless the National Annex gives a different choice.

(3) If bearing loads and movements during construction exceed the service condition values, they should be shown separately.

Table G.1 (NDP) — Bearing schedule with specification of characteristic values concerning individual actions

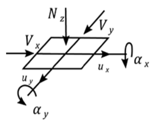
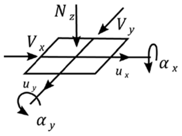
 Local coordinate system	Project: Bearing no.:	This list contains extreme characteristic values for loads and movements in the service condition. If bearing loads and movements during construction exceed the service condition values, they are to be shown separately.													
		Bearing reactions and movements													
		Nz [kN]		Vx [kN]		Vy [kN]		ux [mm]		uy [mm]		αx [mrad]		αy [mrad]	
		max.	min.	max.	min.	max.	min.	max.	min.	max.	min.	max.	min.	max.	min.
Permanent actions															
1.1	Structural self weight														
1.2	Weight of elements other than structural														
1.3	Prestressing														
1.4	Creep														
1.5	Shrinkage														
Variable actions															
2.1	Traffic loads														
2.2	Special vehicles in combination with traffic loads														
2.3	Centrifugal forces														
2.4	Braking and acceleration														
2.5	Pedestrian traffic														
2.6	Wind on structure without traffic														
2.7	Wind on structure including traffic														
2.8	Range of uniform temperature														
2.9	Vertical temperature difference														
2.10	Horizontal temperature difference														
2.11	Settlements														
2.12	Bearing resistance/ friction														
2.13	Bearing replacement														
2.14	Compressive and tensile loading effects of traffic														
Seismic actions															
3.1	Significant Damage (SD)														
3.2	Damage Limitation (DL)														
Accidental actions															
4.1	Derailment														
4.2	Impact														

Table G.2 (NDP) — Bearing schedule with specification of bearing loads and movements for the ultimate and serviceability limit states

Project:									
Bearing no.:									
			This list contains concurrent design values of loads and movements on the bearing in the service condition. If bearing loads and movements during construction exceed the service condition values, they have to be shown separately. Extreme values (envelope approach) displayed in grey fields.						
			Related design values of bearing loads and movements						
			N_z	V_x	V_y	u_x	u_y	α_x	α_y
Local coordinate system			kN	mm	kN	mm	mm	mrad	mrad
Bearing loads and movements at the ultimate limit state persistent design situation									
Bearing loads for the ULS persistent design situation									
1.1		max. $N_{z,d}$							
1.2		min. $N_{z,d}$							
1.3		max. $V_{x,d}$							
1.4		min. $V_{x,d}$							
1.5		max. $V_{y,d}$							
1.6		min. $V_{y,d}$							
Movements for the ULS persistent design situation									
2.1		max. $u_{x,d}$							
2.2		min. $u_{x,d}$							
2.3		max. $u_{y,d}$							
2.4		min. $u_{y,d}$							
2.5		max. $\alpha_{x,d}$							
2.6		min. $\alpha_{x,d}$							
2.7		max. $\alpha_{y,d}$							
2.8		min. $\alpha_{y,d}$							
Bearing loads and movements at the serviceability limit state (characteristic combination)									
Bearing loads for the characteristic combination									
3.1		max. $N_{z,d}$							
3.2		min. $N_{z,d}$							
3.3		max. $V_{x,d}$							
3.4		min. $V_{x,d}$							
3.5		max. $V_{y,d}$							
3.6		min. $V_{y,d}$							

Movements for the characteristic combination									
4.1		max. $u_{x,d}$							
4.2		min. $u_{x,d}$							
4.3		max. $u_{y,d}$							
4.4		min. $u_{y,d}$							
4.5		max. $\alpha_{x,d}$							
4.6		min. $\alpha_{x,d}$							
4.7		max. $\alpha_{y,d}$							
4.8		min. $\alpha_{y,d}$							

G.3.4 Replacement of bearings

(1) Bearings and support systems shall be designed in such a way, that the bearings or individual bearing parts can be inspected, maintained and replaced.

(2) To enable bearings or individual bearing parts to be replaced, the structure shall be designed in such a way that it can be lifted by means of jacks to provide a minimum lifting clearance.

NOTE The minimum lifting clearance of the superstructure is 10 mm relative to the unloaded bearing condition, unless the National Annex gives a different value.

(3) The actions to be considered for the jacking force should be specified.

NOTE Actions to be considered for the jacking force can be set by the National Annex.

(4) If the structural system of the bridge is affected due to the replacement of the bearings, the effect should be considered in the design.

(5) In case of replacement of bearings or bearing parts intended to withstand tensile forces, the structure shall be designed for such replacement.

G.4 Principles of limit state design

G.4.1 Design situations

(1) Bearing forces and movements shall be determined taking into account the distinction between transient and persistent design situations, considering if bearings are installed before completion of the bridge superstructure and/or if deformations of the superstructure during execution need to be taken into account.

NOTE 1 Transient design situations are those associated with execution up to the completion of the superstructure or up until installation of the bearings, and those associated with replacement of bearings.

NOTE 2 Persistent design situations are those resulting from variable and time-dependent actions after completion of the superstructure and installation of the bearings.

NOTE 3 Transient design situations can be governing for the bearing design, for example, elastomeric bearings subject to high rotations under low vertical loads during installation of beams.

(2) Bearing forces and movements in persistent design situations shall include the effects of construction stages after the bearings are fixed to the structure.

(3) Particular transient design situations for verification of bearings and the effect of bearings on the structure should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

NOTE For example, for steel bridges the effects of installation of hot asphalt can be taken into account as a transient design situation.

G.5 Basic variables – Actions and environmental influences

G.5.1 Classification of actions

(1) Bearing forces and movements should be treated as an effect of combinations of actions.

NOTE Bearing forces are applied as an effect of other actions, for example, thermal actions or longitudinal traffic actions that result in movement of a bearing. Therefore, where a combination of actions results in movement of a bearing, then the bearing force is also applied in that combination using its design value as defined in G.7.1. Bearing forces are not treated as independent permanent or variable actions and hence combination factors ψ are not applied directly to bearing forces.

G.5.2 Representative values of actions

(1) The characteristic values of actions that are used to derive bearing forces and movements shall be taken from the other relevant Eurocodes.

(2) For transient design situations, the characteristic values of variable actions may be reduced due to the limited duration of the design situation, as defined in the other relevant Eurocodes.

NOTE 1 EN 1991-1-6 contains details of actions during execution.

NOTE 2 prEN 1991-2:2021, 8.8.4 contains details of the traffic load in transient design situations.

G.5.3 Specific types of action

G.5.3.1 Actions during installation (transient design situation)

(1) When installing the superstructure by sliding or rolling, not only the bearing friction, but also effects of the bridge's longitudinal inclination and the misalignment of piers shall be considered.

NOTE For information about launched bridges, see EN 1991-1-6.

G.5.3.2 Effects from settlements (persistent design situation)

(1) If settlements contribute to the actions, they shall be taken into account.

G.6 Structural analysis - Effects of deformation of piers and abutments

(1) If deformation of piers and abutments generate redistribution of actions in the support system (and the bearings), the actions on bearings shall be calculated with the combination of actions based on the second-order theory.

(2) When calculating the pier and abutment deformations, the geometric imperfections may be reduced by multiplying with the factor k_{φ}.

NOTE The value of k_{φ} is 0,5, unless the National Annex gives a different value.

G.7 Verification by the partial factor method

G.7.1 Design values of bearing force

G.7.1.1 Restraining effects and eccentricities

(1) The restraining effects and eccentricities inside the bearings due to movements shall be taken from a relevant harmonised technical specification.

G.7.1.2 Forces arising from the resistance of sliding elements

(1) The force due to relative movement of bearing sliding surfaces should be modelled as a friction force taking a limiting value based on the coefficient of static friction and the normal force, irrespective of the magnitude of movement, with a sign opposing the direction of movement.

(2) The design value of the bearing force F_{bd} due to bearing friction arising from movement of sliding elements shall be determined using Formula (G.1):

$$F_{\mathrm{bd}} = \gamma_{\mathrm{Sd}} \mu_{\mathrm{d}} N_{\mathrm{d}} \tag{G.1}$$

where

γ_{Sd} is a partial factor that takes account of uncertainties in modelling the effects of actions;

μ_d is the design value of the bearing friction coefficient, given in G.7.3.1;

N_d is the design value of the normal forces at the bearing in the applicable combination.

NOTE 1 The partial factor γ_{Sd} represents uncertainties in modelling the effect of the friction force on the structure, over and above the uncertainties in calculating the vertical forces that are used to derive the friction force. The value of γ_{Sd} is 1,0 unless the National Annex gives a different value.

NOTE 2 The design value at SLS is obtained using partial factors γ_F = 1,0. See 8.4.2.

(3) The upper design value of the bearing force F_{bd} may be limited based on the maximum possible reaction force consistent with compatible movements of the superstructure and support system.

EXAMPLE This can apply to tall slender piers close to the centre of movement of the deck.

(4) Where the bearing force acts favourably, the lower design value of the bearing force F_{bd} due to bearing friction shall be taken as zero.

NOTE See G.7.3.1 where friction acts both favourably and unfavourably on the same structural member.

(5) The bearing friction force should not be considered to resist an externally applied force.

G.7.1.3 Forces arising from the shear resistance of elastomeric elements

(1) The force due to deformation of elastomeric material in bearings should be modelled as a force proportional to the magnitude of movement, with a sign opposing the direction of movement.

(2) The design value of the bearing force F_{bd} due to deformation of elastomeric elements shall be determined using Formula (G.2) where these properties are known:

$$F_{\mathrm{bd}} = \gamma_{\mathrm{Sd}} \sum_{i=1}^{n} \frac{G_{\mathrm{de},i} A_{\mathrm{eff},i}}{t_{\mathrm{el;eff},i}} u_{\mathrm{d},i} \tag{G.2}$$

where

γ_{Sd} is a partial factor that takes account of uncertainties in modelling the effects of actions;

n is the number of elastomer layers;

$G_{de,i}$ is the design value of the shear modulus of the elastomer for layer i, see G.7.3.2;

$A_{eff,i}$ is the effective plan area for layer i of the elastomeric bearing, determined in accordance with EN 1337-3;

$t_{el;eff,i}$ is the effective thickness of elastomer layer i, determined in accordance with a relevant harmonized technical specification;

$u_{d,i}$ is the design value of the relative displacement of elastomer layer i.

NOTE 1 The partial factor γ_{Sd} represents uncertainties in modelling the effect of the shear force on the structure, over and above the uncertainties in calculating the displacement that is used to derive the friction force. The value of γ_{Sd} is 1,0 unless the National Annex gives a different value.

NOTE 2 The design value at SLS is obtained using partial factors γ_F = 1,0. See 8.4.2.

(3) Where the bearing behaviour can be represented by a set of overall parameters, or the specific construction of the bearing is not known, then Formula (G.3) may be applied.

$$F_{bd} = \gamma_{Sd} \left(\frac{G_d A_{eff}}{t_{el;eff}} \right) u_d \qquad \text{(G.3)}$$

where

A_{eff} is the effective plan area of the elastomeric bearing, determined in accordance with a relevant harmonised technical specification;

$t_{el;eff}$ is the effective thickness of the elastomer element, determined in accordance with a relevant harmonised technical specification.

G.7.2 Design values of actions

(1) The design values of movements and forces shall be determined on the basis of the combinations given in A.2.

(2) The design values of movements at bearings resulting from creep and shrinkage should be obtained from the characteristic values of creep and shrinkage obtained from the other relevant Eurocodes.

NOTE See EN 1992-1-1.

G.7.3 Design values of material properties

G.7.3.1 Friction coefficient at sliding bearings

(1) The design value of bearing friction coefficient μ_d used to calculate the maximum friction force at sliding bearings, shall be determined from Formula (G.4).

$$\mu_d = \mu_{max} \tag{G.4}$$

where

μ_{max} is the maximum coefficient of friction of the sliding surfaces given the contact pressure resulting from N_d, in accordance with a relevant harmonised technical specification.

NOTE 1 The coefficient of friction depends on many factors, including the type of material of the sliding surfaces and contact pressure. The coefficient of friction can also depend on wear, temperature and speed of sliding. Relevant harmonised technical spesifications provide different coefficients of friction for different materials and contact pressures.

NOTE 2 μ_{max} represents the long term maximum static coefficient of friction, taking into account wear and low temperature. Lower values of coefficient of friction can be present when the sliding surfaces are new.

(2) Where bearing friction acts in opposite directions, such that the friction forces at some bearings are relieved by the friction forces at other bearings, then the resultant bearing force acting at a fixed point may be determined using modified unfavourable and favourable values of the coefficient of friction in accordance with Formulae (G.5) and (G.6).

$$\mu_d = 0,5\mu_{max}\left(1+\alpha_n\right) \tag{G.5}$$

$$\mu_{d,fav} = 0,5\mu_{max}\left(1-\alpha_n\right) \tag{G.6}$$

where

μ_d is the unfavourable coefficient of friction;

$\mu_{d,fav}$ is the favourable coefficient of friction;

α_n is a factor depending on the type of bearing and number of bearings in accordance with Table G.3;

n is the minimum of: number of bearings with unfavourable forces, or number of bearings with favourable forces; n can be different from the number of piers if there are multiple bearings at each pier.

Table G.3 — Modification factor α_n related to number of bearings

Number of bearings *n*	**Modification factor α_n**
≤ 4	1
4 < n < 10	(16-n)/12
≥ 10	0,5

G.7.3.2 Shear modulus at elastomeric bearings

(1) The design shear modulus of the elastomer G_{de} should be taken in accordance with a relevant harmonised technical specification.

NOTE The design shear modulus G_{de} depends on the hardness of the elastomer, temperature effects and dynamic effects.

(2) Where bearing forces act in opposite directions, such that the forces at some bearings counteract the forces at other bearings, then the displacements of the structure and bearing forces should be calculated based on upper and lower values of the design shear modulus for the respective directions of movement.

NOTE Upper and lower values can be based on the tolerances for shear modulus G_{exp} given in a relevant harmonised technical specification, unless the National Annex gives different values, where G_{exp} is used as the nominal shear modulus G_{nom}, and is used to derive upper and lower values of G_{de} using the temperature and dynamic factors k_{temp} and k_{dyn} in accordance with a relevant harmonised technical specification.

G.7.4 Design values of movement range

G.7.4.1 Design contraction and expansion movements

(1) The design movement range should be defined in relation to an initial reference point for the position of the bearing, considering the design contraction movement and the design expansion movement from the initial reference point.

(2) The design movement range should be calculated for the most onerous of short-term and long-term situations considering time-dependent movements of the structure.

(3) The design contraction movement of the bearing $d_{d,con}$ and the design expansion movement of the bearing $d_{d,exp}$ should be calculated according to Formulae (G.7) and (G.8).

$$d_{\mathrm{d,con}} = \Delta d + d_{\mathrm{Gd}} + d_{\mathrm{Qd,con}} + d_{\mathrm{Pd}} \tag{G.7}$$

$$d_{\mathrm{d,exp}} = \Delta d - d_{\mathrm{Gd}} + d_{\mathrm{Qd,exp}} - d_{\mathrm{Pd}} \tag{G.8}$$

where

Δd represents the geometric uncertainty, see G.7.4.2;

d_{Gd} is the design movement due to permanent effects, see G.7.4.3;

$d_{Qd,con}$ is the design contraction movement due to variable actions, see G.7.4.4;

$d_{Qd,exp}$ is the design expansion movement due to variable actions, see G.7.4.4;

d_{Pd} is the design movement due to prestressing, see G.7.4.5.

NOTE The terms d_{Gd} and d_{Pd} are given illustrative signs in Formulae (G.7) and (G.8). The direction of movement is to be consistent with other movements applied in the Formulae.

G.7.4.2 Geometric uncertainty

(1) Uncertainties regarding the bearing position should be represented as a geometrical uncertainty Δd on bearing position.

NOTE 1 The value of Δd is ± 20 mm, unless the National Annex gives a different value. The value of geometric uncertainty can be defined in the National Annex as an equivalent temperature.

NOTE 2 The geometric uncertainty Δd is included as an addition to both the design expansion movement and the design contraction movement.

(2) An increased value of geometric uncertainty should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

G.7.4.3 Design movement due to permanent effects

(1) The design value of movement due to time-dependent effects d_{Gd} should include creep, shrinkage, locked-in movements from execution, settlement, etc.

NOTE 1 Different values can be appropriate in the short-term and long-term.

NOTE 2 Permanent effects can result in an offset from the initial reference point either in the expansion or in the contraction direction.

(2) Movements of the bearings during execution before the structure becomes restrained (see G.4.1) should be included in the design value of movement due to permanent effects as a value $d_{\mathrm{execution}}$.

(3) In the case of structures with statically indeterminate bearing supports in the longitudinal direction (e.g. groups of piers with fixed bearings), uncertainties regarding the reference temperature during installation of the fixed bearing group should be considered for the determination of the horizontal bearing forces resulting from changes in the temperature of the superstructure.

G.7.4.4 Design movement due to variable actions

(1) Design movements due to variable actions $d_{\mathrm{Qd,con}}$ and $d_{\mathrm{Qd,exp}}$ should be calculated from a relevant combination of actions.

NOTE Variable actions such as temperature and traffic can result in movements in both the expansion direction and the contraction direction.

(2) Where temperature is included in relevant combinations of actions, the design movements should be calculated based on the design value of the maximum fall of the uniform temperature component resulting in contraction $\Delta T_{\mathrm{d,con}}$ in accordance with Formula (G.9) and the design value of the maximum rise of the uniform temperature component resulting in expansion $\Delta T_{\mathrm{d,exp}}$ in accordance with Formula (G.10).

$$\Delta T_{\mathrm{d,con}} = \gamma_{\mathrm{Q,T}} \Delta T_{\mathrm{N,con}} \tag{G.9}$$

$$\Delta T_{\mathrm{d,exp}} = \gamma_{\mathrm{Q,T}} \Delta T_{\mathrm{N,exp}} \tag{G.10}$$

where

$\gamma_{\mathrm{Q,T}}$ is the partial factor for thermal actions at the applicable combination;

$\Delta T_{\mathrm{N,con}}$ is the characteristic value of the maximum fall of the uniform temperature component resulting in contraction (see EN 1991-1-5);

$\Delta T_{\mathrm{N,exp}}$ is the characteristic value of the maximum rise of the uniform temperature component resulting in expansion (see EN 1991-1-5).

NOTE 1 The characteristic values of temperature rise and fall, $\Delta T_{\mathrm{N,con}}$ and $\Delta T_{\mathrm{N,exp}}$, include the range of initial bridge temperature ΔT_0 (see EN 1991-1-5).

NOTE 2 The design value at SLS is obtained using partial factors γ_F = 1,0. See 8.4.2.

G.7.4.5 Design movement due to prestress

(1) The design movement due to prestress d_{Pd} should include the instantaneous deformations due to the application of prestress.

NOTE Prestress can result in an offset from the initial reference point either in the expansion or in the contraction direction.

(2) The design movement due to prestress d_{Pd} should not include time-dependent effects such as creep which are included in the movements due to permanent effects.

G.7.5 Reaction forces at fixed points

G.7.5.1 Rigid fixed point in one abutment, other piers free-sliding bearings

(1) If the horizontal deformation of the abutment at the fixed point can be neglected, and all the rest of the bearing are sliding bearings, the applied horizontal force F_{bd} at the fixed point of the support system resulting from the resistance of the whole support system together with the horizontal load Q_{Lk} out of accelerating and braking as leading variable action should be calculated according to Formula (G.11):

$$F_{\mathrm{bd}} = \gamma_{\mathrm{Q}} Q_{\mathrm{Lk}} + \gamma_{\mathrm{Sd}} \mu_{\mathrm{d}} \sum N_{\mathrm{d},i} \qquad \text{(G.11)}$$

where

F_{bd} is the design value of the bearing force at the fixed point;

$\gamma_Q Q_{Lk}$ is the design value of the horizontal force resulting from acceleration and braking (see prEN 1991-2:2021, 6.4.1);

γ_{Sd} is a partial factor that takes account of uncertainties in modelling the effects of actions (see G.7.1);

μ_d is the design value of the bearing friction, given in G.7.3.1;

$N_{d,i}$ is the design value of the normal forces at the sliding bearings of support i (see G.7.1).

NOTE 1 The applicable combination of actions to be used in calculating N_d is given in Formula (G.12), unless the National Annex makes a different choice.

$$N_{\mathrm{d}} = \gamma_{\mathrm{G}} G_{\mathrm{k}} + \gamma_{\mathrm{P}} P_{\mathrm{k}} + \sum \gamma_{\mathrm{Q},j} \psi_{2,j} Q_{\mathrm{k},j} \qquad \text{(G.12)}$$

where

N_d is the design value of the normal forces at the sliding bearings;

$\gamma_G G_k$ is the design value of the vertical force due to permanent actions;

$\gamma_P P_k$ is the design value of the vertical force on supports due to prestress;

$\gamma_{Q,j}$ is the partial factor for $Q_{k,j}$;

$\Psi_{2,j} Q_{k,j}$ is the quasi-permanent value of the vertical accompanying variable action j.

NOTE 2 Wind action is not combined with traffic accelerating and braking actions, see A.2.7.

G.7.5.2 Intermediate rigid fixed point, other piers free-sliding bearings

(1) If the horizontal deformation of the rigid pier at the fixed point can be neglected, and all the rest of the bearings are sliding bearings, the applied horizontal force F_{bd} at the fixed point of the support system resulting from the resistance of the whole support system together with the horizontal load Q_{Lk} out of accelerating and braking as leading variable action should be calculated according to Formula (G.13):

$$F_{bd} = \gamma_Q Q_{Lk} + \gamma_{Sd} \left[\mu_d \sum N_{d,i} - \mu_{d,fav} \sum N_{d,j} \right] \tag{G.13}$$

where

F_{bd}	is the design value of the bearing force at the fixed point;
$\gamma_Q Q_{Lk}$	is the design value of the horizontal force resulting from acceleration and braking (see prEN 1991-2:2021, 6.4.1);
γ_{Sd}	is the partial factor that takes account of uncertainties in modelling the effects of actions (see G.7.1);
μ_d, $\mu_{d,fav}$	is the design coefficient of friction with unfavourable and favourable effect according to G.7.3.1;
$N_{d,i}$	is the design value of the normal forces at the sliding bearings of support *i* (see G.7.1);
$N_{d,j}$	is the design value of the normal forces of the permanent actions and the prestress at the sliding bearings of support *j*.

NOTE 1 The applicable combination of actions to be used in calculating $N_{d,i}$ for unfavourable friction forces is given in Formula (G.12), unless the National Annex makes a different choice.

NOTE 2 The applicable combination of actions to be used in calculating $N_{d,j}$ for favourable friction forces is given in Formula (G.14), unless the National Annex makes a different choice.

$$N_{d,j} = \gamma_G G_{k,j} + \gamma_P P_{k,j} \tag{G.14}$$

where

$G_{k,j}$	is the characteristic value of the vertical force on support *j* due to permanent actions;
$P_{k,j}$	is the characteristic value of the vertical force on supports due to prestress.

NOTE 3 As the permanent actions are applied to the whole deck, a single value of γ_G is applied for the unfavourable and the favourable part of the bridge, applying the single-source principle. The favourable part of the friction is considered by the use of $\mu_{d,fav}$ instead of μ_d.

G.7.5.3 Intermediate elastic fixed point, other piers free

(1) If the horizontal deformation at the fixed point cannot be neglected, then further analysis should be undertaken to determine whether variable actions including vertical traffic action are included in the calculation of friction force.

(2) The release of friction forces due to the deformation of intermediate elastic fixed point may be taken into account.

Annex H
(informative)

Verifications concerning vibration of footbridges due to pedestrian traffic

H.1 Use of this annex

(1) This Informative Annex provides complementary guidance to A.2.9.3 for vibration of footbridges.

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.

H.2 Scope and field of application

(1) This Informative Annex covers verifications concerning vibration of footbridges due to pedestrian traffic.

(2) This Informative Annex does not cover criteria for synchronized activity and lock-in criteria for horizontal vibrations.

(3) The assessment of human-induced vibrations should be considered to ensure that:

— vibrations due to pedestrian traffic are acceptable for the users;

— the lock-in phenomenon does not arise;

— the footbridge does not exceed ULS when subjected to intentional excitation.

H.3 Dynamic load models and traffic classes

(1) Assessment of vibrations of footbridges should be based on relevant dynamic load models.

(2) The dynamic load models may be defined based on a set of traffic classes that describe the density of traffic on the bridge.

NOTE Guidance on dynamic load models and corresponding traffic classes, intended for use in conjunction with the comfort limits defined in this Annex, is provided in prEN 1991-2:2021, Annex G and includes the following dynamic load models:

— load model for pedestrian stream;

— load model for single pedestrian or group of pedestrians;

— load model for single jogger or group of joggers; and

— load model for intentional excitation.

H.4 Comfort criteria

(1) The required pedestrian comfort level should be represented by a limiting acceleration in the vertical and horizontal directions selected from Table H.1, as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

Table H.1— Comfort levels

Comfort level	Degree of comfort	$a_{\mathrm{lim,v}}$ m/s^2	$a_{\mathrm{lim,h}}$ m/s^2
CL3	Maximum	≤ 0,5	≤ 0,1
CL2	Medium	≤ 1,0	≤ 0,3
CL1	Minimum	≤ 2,5	≤ 0,8
CL0	No limit set	-	-

NOTE 1 The comfort level can be selected based on factors including:

— design situation (number of people walking on the bridge, frequency of use);

— bridge location;

— presence of alternative routes;

— height of structure;

— parapet design (sturdiness, transparency);

— quality of walking surface;

— exposure time;

— nature of terrain underneath;

— user perception towards vibration (walking, standing, sitting);

— expectation of vibration due to bridge appearance.

NOTE 2 The acceleration limits given in Table H.1 are for comfort criteria associated to normal service loads.

(2) CL0 (no limit set) may be chosen for bridges that are located in spots of low pedestrian traffic frequency or with dominant bicycle traffic as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

(3) Comfort criteria for synchronized activity and lock-in criteria for horizontal vibrations should be assessed separately.

H.5 Design situations

(1) Vertical and horizontal accelerations should be checked for one or more design situations when relevant natural frequencies are within the limits stated in A.2.9.3.3.

(2) Design situations should be represented by a comfort level to be achieved under a particular traffic class.

NOTE 1 For traffic classes, see prEN 1991-2:2021, Annex G.

NOTE 2 Some design situations might occur once in the design service life of a footbridge, like the inauguration of the bridge, while others occur daily, such as commuter traffic. Table H.2 gives an example of some typical traffic situations which can occur on footbridges. The expected type of pedestrian traffic and the traffic density, together with the comfort requirements, has a significant effect on the required dynamic behaviour of the bridge.

Table H.2 — Example of a specification of multiple design situations

Design situation	Description	Traffic class (prEN 1991-2:2021, Table G.1)	Expected occurrence	Comfort level (see Table H.1)
DS1	Inauguration of the bridge	TC4(A)[a]	Once during the design service life	CL1
DS2	Commuter traffic	TC3(A)[b] and TC2(B)[c]	Daily	CL3
DS3	Occasional jogger	TC3(C)[d]	Weekly	CL2
…	…	…	…	…

[a] TC4(A) is a load model for pedestrian stream with density of 1,0 P/m^2.

[b] TC3(A) is a load model for pedestrian stream with density of 0,5 P/m^2.

[c] TC2(B) is a load model for group of 2 pedestrians.

[d] TC3(C) is a load model for 1 jogger.

Bibliography

References contained in possibilities (i.e. “can” clauses) and notes

The following documents are cited informatively in the document, for example in "can" clauses and in notes.

EN 1337 (all parts), *Structural bearings*

EN 15129, *Anti-seismic devices*

EN ISO 9000:2015, *Quality management systems - Fundamentals and vocabulary (ISO 9000:2015)*

ISO 2394:2015, *General principles on reliability for structures*

ISO 3898:2013, *Bases for design of structures - Names and symbols of physical quantities and generic quantities*

ISO 6707-1:2020, *Buildings and civil engineering works - Vocabulary - Part 1: General terms*

ISO 10137, *Bases for design of structures - Serviceability of buildings and walkways against vibrations*

ISO 12491, *Statistical methods for quality control of building materials and components*