BRITISH STANDARD

BS EN 1990:2002 +A1:2005

Incorporating corrigenda December 2008 and April 2010

Eurocode — Basis of structural design

ICS 91.010.30; 91.080.01

British Standards

National foreword

This British Standard is the UK implementation of EN 1990:2002+A1:2005, incorporating corrigenda December 2008 and April 2010. It supersedes DD ENV 1991-1:1996 which is withdrawn.

The start and finish of text introduced or altered by amendment is indicated in the text by tags. Tags indicating changes to CEN text carry the number of the CEN amendment. For example, text altered by CEN amendment A1 is indicated by [A1⟩ ⟨A1].

The start and finish of text introduced or altered by corrigendum is indicated in the text by tags. Text altered by CEN corrigendum December 2008 is indicated in the text by [AC1⟩ ⟨AC1].

The start and finish of text introduced or altered by corrigendum is indicated in the text by tags. Text altered by CEN corrigendum April 2010 is indicated in the text by [AC2⟩ ⟨AC2].

The UK participation in its preparation was entrusted by Technical Committee B/525, Building and Civil engineering structures, to Subcommittee B/525/1, Action, loadings and basis of design.

A list of organizations represented on this subcommittee can be obtained on request to its secretary.

Where a normative part of this EN allows for a choice to be made at the national level, the range and possible choice will be given in the normative text, and a Note will qualify it as a Nationally Determined Parameter (NDP). NDPs can be a specific value for a factor, a specific level or class, a particular method or a particular application rule if several are proposed in the EN.

To enable EN 1990 to be used in the UK, the NDPs will be published in a National Annex which will be incorporated by amendment into this British Standard in due course, after public consultation has taken place.

This publication does not purport to include all the necessary provisions of a contract. Users are responsible for its correct application.

Compliance with a British Standard cannot confer immunity from legal obligations.

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	31 July 2010	Implementation of CEN corrigendum April 2010

EUROPEAN STANDARD
NORME EUROPÉENNE
EUROPÄISCHE NORM

EN 1990:2002+A1

December 2005

ICS 91.010.30

Supersedes ENV 1991-1:1994
Incorporating corrigenda December 2008
and April 2010

English version

Eurocode - Basis of structural design

Eurocodes structuraux - Eurocodes: Bases de calcul des structures

Eurocode: Grundlagen der Tragwerksplanung

This European Standard was approved by CEN on 29 November 2001.

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. Up-to-date lists and bibliographical references concerning such national standards may be obtained on application to the Management Centre or to any CEN member.

This European Standard exists in three official versions (English, French, German). A version in any other language made by translation under the responsibility of a CEN member into its own language and notified to the Management Centre has the same status as the official versions.

CEN members are the national standards bodies of Austria, Belgium, Czech Republic, Denmark, Finland, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, Malta, Netherlands, Norway, Portugal, Spain, Sweden, Switzerland and United Kingdom.

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Contents Page

Foreword

This document (EN 1990:2002) has been prepared by Technical Committee CEN/TC 250 "Structural Eurocodes", the secretariat of which is held by BSI.

This European Standard shall be given the status of a national standard, either by publication of an identical text or by endorsement, at the latest by October 2002, and conflicting national standards shall be withdrawn at the latest by March 2010.

This document supersedes ENV 1991-1:1994.

CEN/TC 250 is responsible for all Structural Eurocodes.

According to the CEN/CENELEC Internal Regulations, the national standards organizations of the following countries are bound to implement this European Standard: Austria, Belgium, Czech Republic, Denmark, Finland, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, Malta, Netherlands, Norway, Portugal, Spain, Sweden, Switzerland and the United Kingdom.

Foreword to amendment A1

This European Standard (EN 1990:2002/A1:2005) has been prepared by Technical Committee CEN/TC 250 “Structural Eurocodes”, the secretariat of which is held by BSI.

This Amendment to the EN 1990:2002 shall be given the status of a national standard, either by publication of an identical text or by endorsement, at the latest by June 2006, and conflicting national standards shall be withdrawn at the latest by June 2006.

According to the CEN/CENELEC Internal Regulations, the national standards organizations of the following countries are bound to implement this European Standard: Austria, Belgium, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.

Background of the Eurocode programme

In 1975, the Commission of the European Community decided on an action programme in the field of construction, based on article 95 of the Treaty. The objective of the programme was the elimination of technical obstacles to trade and the harmonisation of technical specifications.

Within this action programme, the Commission took the initiative to establish a set of harmonised technical rules for the design of construction works which, in a first stage, would serve as an alternative to the AC2 national provisions AC2 in force in the Member States and, ultimately, would replace them.

For fifteen years, the Commission, with the help of a Steering Committee with Representatives of Member States, conducted the development of the Eurocodes programme, which led to the first generation of European codes in the 1980's.

In 1989, the Commission and the Member States of the EU and EFTA decided, on the basis of an agreement[1] between the Commission and CEN, to transfer the preparation and the publication of the Eurocodes to CEN through a series of Mandates, in order to provide them with a future status of European Standard (EN). This links *de facto* the Eurocodes with the provisions of all the Council's Directives and/or Commission's Decisions dealing with European standards (*e.g.* the Council Directive 89/106/EEC on construction products - CPD - and AC2 Council Directives 2004/17/EC and 2004/18/EC AC2 on public works and services and equivalent EFTA Directives initiated in pursuit of setting up the internal market).

The Structural Eurocode programme comprises the following standards generally consisting of a number of Parts:

EN 1990	Eurocode :	Basis of Structural 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

Eurocode standards recognise the responsibility of regulatory authorities in each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level where these continue to vary from State to State.

[1] Agreement between the Commission of the European Communities and the European Committee for Standardisation (CEN) concerning the work on EUROCODES for the design of building and civil engineering works (BC/CEN/03/89).

Status and field of application of Eurocodes

The Member States of the EU and EFTA recognise that Eurocodes serve as reference documents for the following purposes :

– as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 – Mechanical resistance and stability – and Essential Requirement N°2 – Safety in case of fire ;

– as a basis for specifying contracts for construction works and related engineering services ;

– as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs)

The Eurocodes, as far as they concern the construction works themselves, have a direct relationship with the Interpretative Documents[2] referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standards[3]. Therefore, technical aspects arising from the Eurocodes work need to be adequately considered by CEN Technical Committees and/or EOTA Working Groups working on product standards [AC2) and ETAGs (AC2] with a view to achieving ~~a~~ full compatibility of these technical specifications with the Eurocodes.

The Eurocode standards provide common structural design rules for everyday use for the design of whole structures and [AC2) parts of works and structural construction (AC2] products of both a traditional and an in-novative nature. Unusual forms of construction or design conditions are not specifically covered and additional expert consideration will be required by the designer in such cases.

National Standards implementing Eurocodes

The National Standards implementing Eurocodes will comprise the full text of the Eurocode (including any annexes), as published by CEN, which may be preceded by a National title page and National foreword, and may be followed by a National annex.

The National annex may only contain information on those parameters which are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned, i.e. :

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[2] According to Art. 3.3 of the CPD, the essential requirements (ERs) shall be given concrete form in interpretative documents for the creation of the necessary links between the essential requirements and the mandates for harmonised ENs and ETAGs/ETAs.

[3] According to Art. 12 of the CPD the interpretative documents shall :

a) give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;

b) indicate methods of correlating these classes or levels of requirement with the technical specifications, *e.g.* methods of calculation and of proof, technical rules for project design, etc. ;

c) serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.

The Eurocodes, *de facto*, play a similar role in the field of the ER 1 and a part of ER 2.

- values and/or classes where alternatives are given in the Eurocode,
- values to be used where a symbol only is given in the Eurocode,
- country specific data (geographical, climatic, etc.), *e.g.* snow map,
- the procedure to be used where alternative procedures are given in the Eurocode~~,~~.

It may also contain

- decisions on the application of informative annexes,
- references to non-contradictory complementary information to assist the user to apply the Eurocode.

Links between Eurocodes and harmonised technical specifications (ENs and ETAs) for products

There is a need for consistency between the harmonised technical specifications for construction products and the [AC2⟩ technical provisions ⟨AC2] for works[4]. Furthermore, all the information accompanying the CE Marking of the construction products which [AC2⟩ use the ⟨AC2] Euro-codes shall clearly mention which Nationally Determined Parameters have been taken into account.

Additional information specific to EN 1990

EN 1990 describes the Principles and requirements for safety, serviceability and durability of structures. It is based on the limit state concept used in conjunction with a partial factor method.

For the design of new structures, EN 1990 is intended to be used, for direct application, together with Eurocodes EN 1991 to 1999.

EN 1990 also gives guidelines for the aspects of structural reliability relating to safety, serviceability and durability :

- for design cases not covered by EN 1991 to EN 1999 (other actions, structures not treated, other materials) ;
- to serve as a reference document for other CEN TCs concerning structural matters.

EN 1990 is intended for use by :

- committees drafting standards for structural design and related product, testing and execution standards ;
- clients (*e.g.* for the formulation of their specific requirements on reliability levels and durability) ;
- designers and constructors ;
- relevant authorities.

EN 1990 may be used, when relevant, as a guidance document for the design of structures outside the scope of the Eurocodes EN 1991 to EN 1999, for :

- assessing other actions and their combinations ;
- modelling material and structural behaviour ;
- assessing numerical values of the reliability format.

[4] see Art.3.3 and Art.12 of the CPD, as well as 4.2, 4.3.1, 4.3.2 and 5.2 of ID 1.

Numerical values for partial factors and other reliability parameters are recommended as basic values that provide an acceptable level of reliability. They have been selected assuming that an appropriate level of workmanship and of quality management applies. When EN 1990 is used as a base document by other CEN/TCs the same values need to be taken.

National annex for EN 1990

This standard gives alternative procedures, values and recommendations for classes with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1990 should have a National annex containing all Nationally Determined Parameters to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

AC2 National choice is allowed in EN 1990 Annex A1 through; AC2

- A1.1(1)
- A1.2.1(1)
- A1.2.2 (Table A1.1)
- A1.3.1(1) (Tables A1.2(A) to (C))
- A1.3.1(5)
- A1.3.2 (Table A1.3)
- A1.4.2(2)

AC2 National choice is allowed in EN 1990 Annex A2 through:

General clauses

Clause	Item
A2.1 (1) NOTE 3	Use of Table 2.1 : Design working life
A2.2.1(2) NOTE 1	Combinations involving actions which are outside the scope of EN 1991
A2.2.6(1) NOTE 1	Values of ψ factors
A2.3.1(1)	Alteration of design values of actions for ultimate limit states
A2.3.1(5)	Choice of Approach 1, 2 or 3
A2.3.1(7)	Definition of forces due to ice pressure
A2.3.1(8)	Values of γ_P factors for prestressing actions where not specified in the relevant design Eurocodes
A2.3.1 Table A2.4(A) NOTES 1 and 2	Values of γ factors
A2.3.1 Table A2.4(B)	- NOTE 1 : choice between 6.10 and 6.10a/b - NOTE 2 : Values of γ and ξ factors - NOTE 4 : Values of γ_{Sd}

AC2

AC2

A2.3.1 Table A2.4(C)	Values of γ factors
A2.3.2(1)	Design values in Table A2.5 for accidental designs situations, design values of accompanying variable actions and seismic design situations
A2.3.2 Table A2.5 NOTE	Design values of actions
A2.4.1(1) NOTE 1 (Table A2.6) NOTE 2	Alternative γ values for traffic actions for the serviceability limit state Infrequent combination of actions
A2.4.1(2)	Serviceability requirements and criteria for the calculation of deformations

Clauses specific for road bridges

Clause	Item
A2.2.2 (1)	Reference to the infrequent combination of actions
A2.2.2(3)	Combination rules for special vehicles
A2.2.2(4)	Combination rules for snow loads and traffic loads
A2.2.2(6)	Combination rules for wind and thermal actions
A2.2.6(1) NOTE 2	Values of $\psi_{1,infq}$ factors
A2.2.6(1) NOTE 3	Values of water forces

Clauses specific for footbridges

Clause	Item
A2.2.3(2)	Combination rules for wind and thermal actions
A2.2.3(3)	Combination rules for snow loads and traffic loads
A2.2.3(4)	Combination rules for footbridges protected from bad weather
A2.4.3.2(1)	Comfort criteria for footbridges

Clauses specific for railway bridges

Clause	Item
A2.2.4(1)	Combination rules for snow loading on railway bridges
A2.2.4(4)	Maximum wind speed compatible with rail traffic
A2.4.4.1(1) NOTE 3	**Deformation and vibration requirements for temporary railway bridges**
A2.4.4.2.1(4)P	Peak values of deck acceleration for railway bridges and associated frequency range
A2.4.4.2.2 – Table A2.7 NOTE	Limiting values of deck twist for railway bridges

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A2.4.4.2.2(3)P	Limiting values of the total deck twist for railway bridges
A2.4.4.2.3(1)	Vertical deformation of ballasted and non ballasted railway bridges
A2.4.4.2.3(2)	Limitations on the rotations of non-ballasted bridge deck ends for railway bridges
A2.4.4.2.3(3)	Additional limits of angular rotations at the end of decks
A2.4.4.2.4(2) – Table A2.8 NOTE 3	Values of α_i and r_i factors
A2.4.4.2.4(3)	Minimum lateral frequency for railway bridges
A2.4.4.3.2(6)	Requirements for passenger comfort for temporary bridges

AC2

Section 1 General

1.1 Scope

(1) EN 1990 establishes Principles and requirements for the safety, serviceability and durability of structures, describes the basis for their design and verification and gives guidelines for related aspects of structural reliability.

(2) EN 1990 is intended to be used in conjunction with EN 1991 to EN 1999 for the structural design of buildings and civil engineering works, including geotechnical aspects, structural fire design, situations involving earthquakes, execution and temporary structures.

NOTE For the design of special construction works (*e.g.* nuclear installations, dams, etc.), other provisions than those in EN 1990 to EN 1999 might be necessary.

(3) EN 1990 is applicable for the design of structures where other materials or other actions outside the scope of EN 1991 to EN 1999 are involved.

(4) EN 1990 is applicable for the structural appraisal of existing construction, in developing the design of repairs and alterations or in assessing changes of use.

NOTE Additional or amended provisions might be necessary where appropriate.

1.2 Normative references

This European Standard incorporates by dated or undated reference, provisions from other publications. These normative references are cited at the appropriate places in the text and the publications are listed hereafter. For dated references, subsequent amendments to or revisions of any of these publications apply to this European Standard only when incorporated in it by amendment or revision. For undated references the latest edition of the publication referred to applies (including amendments).

NOTE The Eurocodes were published as European Prestandards. The following European Standards which are published or in preparation are cited in normative clauses :

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

1.3 Assumptions

(1) Design which employs the Principles and Application Rules is deemed to meet the requirements provided the assumptions given in EN 1990 to EN 1999 are satisfied (see Section 2).

(2) The general assumptions of EN 1990 are :

- the choice of the structural system and the design of the structure is made by appropriately qualified and experienced personnel;
- execution is carried out by personnel having the appropriate skill and experience;
- AC2 adequate supervision and quality control is provided during design and during execution of the work, i.e., factories, plants, and on site; AC2
- the construction materials and products are used as specified in EN 1990 or in EN 1991 to EN 1999 or in the relevant execution standards, or reference material or product specifications;
- the structure will be adequately maintained;
- the structure will be used in accordance with the design assumptions.

NOTE There may be cases when the above assumptions need to be supplemented.

1.4 Distinction between Principles and Application Rules

(1) Depending on the character of the individual clauses, distinction is made in EN 1990 between Principles and Application Rules.

(2) The Principles comprise :
- general statements and definitions for which there is no alternative, as well as ;
- requirements and analytical models for which no alternative is permitted unless specifically stated.

(3) The Principles are identified by the letter P following the paragraph number.

(4) The Application Rules are generally recognised rules which comply with the Principles and satisfy their requirements.

(5) It is permissible to use alternative design rules different from the Application Rules given in EN 1990 for works, provided that it is shown that the alternative rules accord with the relevant Principles and are at least equivalent with regard to the structural safety, serviceability and durability which would be expected when using the Eurocodes.

NOTE If an alternative design rule is substituted for an application rule, the resulting design cannot be claimed to be wholly in accordance with EN 1990 although the design will remain in accordance with the Principles of EN 1990. When EN 1990 is used in respect of a property listed in an Annex Z of a product standard or an ETAG, the use of an alternative design rule may not be acceptable for CE marking.

(6) In EN 1990, the Application Rules are identified by a number in brackets e.g. as this clause.

1.5 Terms and definitions

NOTE For the purposes of this European Standard, the terms and definitions are derived from ISO 2394, ISO 3898, ISO 8930, ISO 8402.

1.5.1 Common terms used in EN 1990 to EN 1999

1.5.1.1
construction works
everything that is constructed or results from construction operations

NOTE This definition accords with ISO 6707-1. The term covers both building and civil engineering works. It refers to the complete construction works comprising structural, non-structural and geotechnical elements.

1.5.1.2
type of building or civil engineering works
type of construction works designating its intended purpose, *e.g.* dwelling house, retaining wall, industrial building, road bridge

1.5.1.3
type of construction
indication of the principal structural material, *e.g.* reinforced concrete construction, steel construction, timber construction, masonry construction, steel and concrete composite construction

1.5.1.4
method of construction
manner in which the execution will be carried out, *e.g.* cast in place, prefabricated, cantilevered

1.5.1.5
construction material
material used in construction work, *e.g.* concrete, steel, timber, masonry

1.5.1.6
structure
organised combination of connected parts designed to carry loads and provide adequate rigidity

1.5.1.7
structural member
physically distinguishable part of a structure, *e.g.* a column, a beam, a slab, a foundation pile

1.5.1.8
form of structure
arrangement of structural members

NOTE Forms of structure are, for example, frames, suspension bridges.

1.5.1.9
structural system
load-bearing members of a building or civil engineering works and the way in which these members function together

1.5.1.10
structural model
idealisation of the structural system used for the purposes of analysis, design and verification

1.5.1.11
execution
all activities carried out for the physical completion of the work including procurement, the inspection and documentation thereof

NOTE The term covers work on site; it may also signify the fabrication of components off site and their subsequent erection on site.

1.5.2 Special terms relating to design in general

1.5.2.1
design criteria
quantitative formulations that describe for each limit state the conditions to be fulfilled

1.5.2.2
design situations
sets of physical conditions representing the real conditions occurring during a certain time interval for which the design will demonstrate that relevant limit states are not exceeded

1.5.2.3
transient design situation
design situation that is relevant during a period much shorter than the design working life of the structure and which has a high probability of occurrence

NOTE A transient design situation refers to temporary conditions of the structure, of use, or exposure, *e.g.* during construction or repair.

1.5.2.4
persistent design situation
design situation that is relevant during a period of the same order as the design working life of the structure

NOTE Generally it refers to conditions of normal use.

1.5.2.5
accidental design situation
design situation involving exceptional conditions of the structure or its exposure, including fire, explosion, impact or local failure

1.5.2.6
fire design
design of a structure to fulfil the required performance in case of fire

1.5.2.7
seismic design situation
design situation involving exceptional conditions of the structure when subjected to a seismic event

1.5.2.8
design working 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

1.5.2.9
hazard
for the purpose of EN 1990 to EN 1999, an unusual and severe event, *e.g.* an abnormal action or environmental influence, insufficient strength or resistance, or excessive deviation from intended dimensions

1.5.2.10
load arrangement
identification of the position, magnitude and direction of a free action

1.5.2.11
load case
compatible load arrangements, sets of deformations and imperfections considered simultaneously with fixed variable actions and permanent actions for a particular verification

1.5.2.12
limit states
states beyond which the structure no longer fulfils the relevant design criteria

1.5.2.13
ultimate limit states
states associated with collapse or with other similar forms of structural failure

NOTE They generally correspond to the maximum load-carrying resistance of a structure or structural member.

1.5.2.14
serviceability limit states
states that correspond to conditions beyond which specified service requirements for a structure or structural member are no longer met

1.5.2.14.1
irreversible serviceability limit states
serviceability limit states where some consequences of actions exceeding the specified service requirements will remain when the actions are removed

1.5.2.14.2
reversible serviceability limit states
serviceability limit states where no consequences of actions exceeding the specified service requirements will remain when the actions are removed

1.5.2.14.3
serviceability criterion
design criterion for a serviceability limit state

1.5.2.15
resistance
capacity of a member or component, or a cross-section of a member or component of a structure, to withstand actions without mechanical failure *e.g.* bending resistance, buckling resistance, tension resistance

1.5.2.16
strength
mechanical property of a material indicating its ability to resist actions, usually given in units of stress

1.5.2.17
reliability
ability of a structure or a structural member to fulfil the specified requirements, including the design working life, for which it has been designed. Reliability is usually expressed in probabilistic terms

NOTE Reliability covers safety, serviceability and durability of a structure.

1.5.2.18
reliability differentiation
measures intended for the socio-economic optimisation of the resources to be used to build construction works, taking into account all the expected consequences of failures and the cost of the construction works

1.5.2.19
basic variable
part of a specified set of variables representing physical quantities which characterise actions and environmental influences, geometrical quantities, and material properties including soil properties

1.5.2.20
maintenance
set of activities performed during the working life of the structure in order to enable it to fulfil the requirements for reliability

NOTE Activities to restore the structure after an accidental or seismic event are normally outside the scope of maintenance.

1.5.2.21
repair
activities performed to preserve or to restore the function of a structure that fall outside the definition of maintenance

1.5.2.22
nominal value
value fixed on non-statistical bases, for instance on acquired experience or on physical conditions

1.5.3 Terms relating to actions

1.5.3.1
action (*F*)
a) Set of forces (loads) applied to the structure (direct action);
b) Set of imposed deformations or accelerations caused for example, by temperature changes, moisture variation, uneven settlement or earthquakes (indirect action).

1.5.3.2
effect of action (*E*)
effect of actions (or action effect) on structural members, (*e.g.* internal force, moment, stress, strain) or on the whole structure (*e.g.* deflection, rotation)

1.5.3.3
permanent action (*G*)
action that is likely to act throughout a given reference period and for which the variation in magnitude with time is negligible, or for which the variation is always in the same direction (monotonic) until the action attains a certain limit value

1.5.3.4
variable action (*Q*)
action for which the variation in magnitude with time is neither negligible nor monotonic

1.5.3.5
accidental action (A)
action, usually of short duration but of significant magnitude, that is unlikely to occur on a given structure during the design working life

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

NOTE 2 Impact, snow, wind and seismic actions may be variable or accidental actions, depending on the available information on statistical distributions.

1.5.3.6
seismic action (A_E)
action that arises due to earthquake ground motions

1.5.3.7
geotechnical action
action transmitted to the structure by the ground, fill or groundwater

1.5.3.8
fixed action
action that has a fixed distribution and position over the structure or structural member such that the magnitude and direction of the action are determined unambiguously for the whole structure or structural member if this magnitude and direction are determined at one point on the structure or structural member

1.5.3.9
free action
action that may have various spatial distributions over the structure

1.5.3.10
single action
action that can be assumed to be statistically independent in time and space of any other action acting on the structure

1.5.3.11
static action
action that does not cause significant acceleration of the structure or structural members

1.5.3.12
dynamic action
action that causes significant acceleration of the structure or structural members

1.5.3.13
quasi-static action
dynamic action represented by an equivalent static action in a static model

1.5.3.14
characteristic value of an action (F_k)
principal representative value of an action

NOTE In so far as a characteristic value can be fixed on statistical bases, it is chosen so as to correspond to a prescribed probability of not being exceeded on the unfavourable side during a "reference period" taking into account the design working life of the structure and the duration of the design situation.

1.5.3.15
reference period
chosen period of time that is used as a basis for assessing statistically variable actions, and possibly for accidental actions

1.5.3.16
combination value of a variable action ($\psi_0 Q_k$)
value 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. It may be expressed as a determined part of the characteristic value by using a factor $\psi_0 \leq 1$

1.5.3.17
frequent value of a variable action ($\psi_1 Q_k$)
value determined - in so far as it can be fixed on statistical bases - so that either the total time, within the reference period, during which it is exceeded is only a small given part of the reference period, or the frequency of it being exceeded is limited to a given value. It may be expressed as a determined part of the characteristic value by using a factor $\psi_1 \leq 1$

AC2 NOTE For the frequent value of multi-component traffic actions see load groups in EN 1991-2. AC2

1.5.3.18
quasi-permanent value of a variable action ($\psi_2 Q_k$)
value determined so that the total period of time for which it will be exceeded is a large fraction of the reference period. It may be expressed as a determined part of the characteristic value by using a factor $\psi_2 \leq 1$

1.5.3.19
accompanying value of a variable action (ψQ_k)
value of a variable action that accompanies the leading action in a combination

NOTE The accompanying value of a variable action may be the combination value, the frequent value or the quasi-permanent value.

1.5.3.20
representative value of an action (F_{rep})
value used for the verification of a limit state. A representative value may be the characteristic value (F_k) or an accompanying value (ψF_k)

1.5.3.21
design value of an action (F_d)
value obtained by multiplying the representative value by the partial factor γ_f

NOTE The product of the representative value multiplied by the partial factor $\gamma_F = \gamma_{Sd} \times \gamma_f$ may also be designated as the design value of the action (See **6.3.2**).

1.5.3.22
combination of actions
set of design values used for the verification of the structural reliability for a limit state under the simultaneous influence of different actions

1.5.4 Terms relating to material and product properties

1.5.4.1
characteristic value (X_k or R_k)
value of a material or product property having a prescribed probability of not being attained in a hypothetical unlimited test series. 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

1.5.4.2
design value of a material or product property (X_d or R_d)
value obtained by dividing the characteristic value by a partial factor γ_m or γ_M, or, in special circumstances, by direct determination

1.5.4.3
nominal value of a material or product property (X_{nom} or R_{nom})
value normally used as a characteristic value and established from an appropriate document such as a European Standard or Prestandard

1.5.5 Terms relating to geometrical data

1.5.5.1
characteristic value of a geometrical property (a_k)
value usually corresponding to the dimensions specified in the design. Where relevant, values of geometrical quantities may correspond to some prescribed fractiles of the statistical distribution

1.5.5.2
design value of a geometrical property (a_d)
generally a nominal value. Where relevant, values of geometrical quantities may correspond to some prescribed fractile of the statistical distribution

NOTE The design value of a geometrical property is generally equal to the characteristic value. However, it may be treated differently in cases where the limit state under consideration is very sensitive to the value of the geometrical property, for example when considering the effect of geometrical imperfections on buckling. In such cases, the design value will normally be established as a value specified directly, for example in an appropriate European Standard or Prestandard. Alternatively, it can be established from a statistical basis, with a value corresponding to a more appropriate fractile (*e.g.* a rarer value) than applies to the characteristic value.

1.5.6 Terms relating to structural analysis

NOTE The definitions contained in the clause may not necessarily relate to terms used in EN 1990, but are included here to ensure a harmonisation of terms relating to structural analysis for EN 1991 to EN 1999.

1.5.6.1
structural analysis
procedure or algorithm for determination of action effects in every point of a structure

NOTE A structural analysis may have to be performed at three levels using different models : global analysis, member analysis, local analysis.

1.5.6.2
global analysis
determination, in a structure, of a consistent set of either internal forces and moments, or stresses, that are in equilibrium with a particular defined set of actions on the structure, and depend on geometrical, structural and material properties

1.5.6.3
first order linear-elastic analysis without redistribution
elastic structural analysis based on linear stress/strain or moment/curvature laws and performed on the initial geometry

1.5.6.4
first order linear-elastic analysis with redistribution
linear elastic analysis in which the internal moments and forces are modified for structural design, consistently with the given external actions and without more explicit calculation of the rotation capacity

1.5.6.5
second order linear-elastic analysis
elastic structural analysis, using linear stress/strain laws, applied to the geometry of the deformed structure

1.5.6.6
first order non-linear analysis
structural analysis, performed on the initial geometry, that takes account of the non-linear deformation properties of materials

NOTE First order non-linear analysis is either elastic with appropriate assumptions, or elastic-perfectly plastic (see 1.5.6.8 and 1.5.6.9), or elasto-plastic (see 1.5.6.10) or rigid-plastic (see 1.5.6.11).

1.5.6.7
second order non-linear analysis
structural analysis, performed on the geometry of the deformed structure, that takes account of the non-linear deformation properties of materials

NOTE Second order non-linear analysis is either elastic-perfectly plastic or elasto-plastic.

1.5.6.8
first order elastic-perfectly plastic analysis
structural analysis based on moment/curvature relationships consisting of a linear elastic part followed by a plastic part without hardening, performed on the initial geometry of the structure

1.5.6.9
second order elastic-perfectly plastic analysis
structural analysis based on moment/curvature relationships consisting of a linear elastic part followed by a plastic part without hardening, performed on the geometry of the displaced (or deformed) structure

1.5.6.10
AC2 **elasto-plastic analysis** AC2
structural analysis that uses stress-strain or moment/curvature relationships consisting of a linear elastic part followed by a plastic part with or without hardening

NOTE In general, it is performed on the initial structural geometry, but it may also be applied to the geometry of the displaced (or deformed) structure.

1.5.6.11
rigid plastic analysis
analysis, performed on the initial geometry of the structure, that uses limit analysis theorems for direct assessment of the ultimate loading

NOTE The moment/curvature law is assumed without elastic deformation and without hardening.

1.6 Symbols

AC2 For the purposes of this European Standard, the following symbols apply.

NOTE The notation used is based on ISO 3898:1987.

Latin upper case letters

A	Accidental action
A_{d}	Design value of an accidental action
A_{Ed}	Design value of seismic action $A_{Ed} = \gamma_I A_{Ek}$
A_{Ek}	Characteristic value of seismic action
C_{d}	Nominal value, or a function of certain design properties of materials
E	Effect of actions
E_{d}	Design value of effect of actions
$E_{\mathrm{d,dst}}$	Design value of effect of destabilising actions
$E_{\mathrm{d,stb}}$	Design value of effect of stabilising actions
F	Action
F_{d}	Design value of an action
F_{k}	Characteristic value of an action
F_{rep}	Representative value of an action AC2

AC2) F_w	Wind force (general symbol)
F_{wk}	Characteristic value of the wind force
F_W^*	Wind force compatible with road traffic
F_W^{**}	Wind force compatible with railway traffic
G	Permanent action
G_d	Design value of a permanent action
$G_\mathrm{d,inf}$	Lower design value of a permanent action
$G_\mathrm{d,sup}$	Upper design value of a permanent action
G_k	Characteristic value of a permanent action
$G_\mathrm{k,j}$	Characteristic value of permanent action j
$G_\mathrm{k,j,sup}$ / $G_\mathrm{k,j,inf}$	Upper/lower characteristic value of permanent action j
G_{set}	Permanent action due to uneven settlements
P	Relevant representative value of a prestressing action (see EN 1992 to EN 1996 and EN 1998 to EN 1999)
P_d	Design value of a prestressing action
P_k	Characteristic value of a prestressing action
P_m	Mean value of a prestressing action
Q	Variable action
Q_d	Design value of a variable action
Q_k	Characteristic value of a single variable action
$Q_\mathrm{k,1}$	Characteristic value of the leading variable action *1*
$Q_\mathrm{k,i}$	Characteristic value of the accompanying variable action *i*
Q_{Sn}	Characteristic value of snow load
R	Resistance
R_d	Design value of the resistance
R_k	Characteristic value of the resistance
T	Thermal climatic action (general symbol)
T_k	Characteristic value of the thermal climatic action
X	Material property
X_d	Design value of a material property
X_k	Characteristic value of a material property

Latin lower case letters

a_d	Design values of geometrical data
a_k	Characteristic values of geometrical data
a_nom	Nominal value of geometrical data
d_{set}	Difference in settlement of an individual foundation or part of a foundation compared to a reference level
u	Horizontal displacement of a structure or structural member
w	Vertical deflection of a structural member

Greek upper case letters

Δa	Change made to nominal geometrical data for particular design purposes, *e.g.* assessment of effects of imperfections
Δd_{set}	Uncertainty attached to the assessment of the settlement of a foundation or part of a foundation (AC2

AC2 *Greek lower case letters*

γ	Partial factor (safety or serviceability)
γ_{bt}	Maximum peak value of bridge deck acceleration for ballasted track
γ_{df}	Maximum peak value of bridge deck acceleration for direct fastened track
γ_{Gset}	Partial factor for permanent actions due to settlements, also accounting for model uncertainties
γ_{f}	Partial factor for actions, which takes account of the possibility of unfavourable deviations of the action values from the representative values
γ_{F}	Partial factor for actions, also accounting for model uncertainties and dimensional variations
γ_{g}	Partial factor for permanent actions, which takes account of the possibility of unfavourable deviations of the action values from the representative values
γ_{G}	Partial factor for permanent actions, also accounting for model uncertainties and dimensional variations
$\gamma_{G,j}$	Partial factor for permanent action *j*
$\gamma_{G,j,sup}$ / $\gamma_{G,j,inf}$	Partial factor for permanent action *j* in calculating upper/lower design values
γ_{I}	Importance factor (see EN 1998)
γ_{m}	Partial factor for a material property
γ_{M}	Partial factor for a material property, also accounting for model uncertainties and dimensional variations
γ_{P}	Partial factor for prestressing actions (see EN 1992 to EN 1996 and EN 1998 to EN 1999)
γ_{q}	Partial factor for variable actions, which takes account of the possibility of unfavourable deviations of the action values from the representative values
γ_{Q}	Partial factor for variable actions, also accounting for model uncertainties and dimensional variations
$\gamma_{Q,i}$	Partial factor for variable action *i*
γ_{Rd}	Partial factor associated with the uncertainty of the resistance model
γ_{Sd}	Partial factor associated with the uncertainty of the action and/or action effect model
η	Conversion factor
ξ	Reduction factor
ψ_{0}	Factor for combination value of a variable action
ψ_{1}	Factor for frequent value of a variable action
ψ_{2}	Factor for quasi-permanent value of a variable action AC2

Section 2 Requirements

2.1 Basic requirements

(1)P A structure shall be designed and executed in such a way that it will, during its intended life, with appropriate degrees of reliability and in an economical way
- sustain all actions and influences likely to occur during execution and use, and
- AC1 meet the specified serviceability requirements for a structure or a structural element.

NOTE See also 1.3, 2.1(7) and 2.4(1) P. AC1

(2)P A structure shall be designed to have adequate :
- structural resistance,
- serviceability, and
- durability.

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

NOTE See also EN 1991-1-2

(4)P A structure shall be designed and executed in such a way that it will not be damaged by events such as :
- explosion,
- impact, and
- the consequences of human errors,

to an extent disproportionate to the original cause.

NOTE 1 The events to be taken into account are those agreed for an individual project with the client and the relevant authority.

NOTE 2 Further information is given in EN 1991-1-7.

(5)P Potential damage shall be avoided or limited by appropriate choice of one or more of the following :
- avoiding, eliminating or reducing the hazards to which the structure can be subjected;
- selecting a structural form which has low sensitivity to the hazards considered ;
- selecting a structural form and design that can survive adequately the accidental removal of an individual member or a limited part of the structure, or the occurrence of acceptable localised damage ;
- avoiding as far as possible structural systems that can collapse without warning ;
- tying the structural members together.

(6) The basic requirements should be met :
- by the choice of suitable materials,
- by appropriate design and detailing, and
- by specifying control procedures for design, production, execution, and use

relevant to the particular project.

(7) The provisions of Section 2 should be interpreted on the basis that due skill and care appropriate to the circumstances is exercised in the design, based on such knowledge and good practice as is generally available at the time that the design of the structure is carried out.

2.2 Reliability management

(1)P The reliability required for structures within the scope of EN 1990 shall be achieved:

a) by design in accordance with EN 1990 to EN 1999 and
b) by
   – appropriate execution and
   – quality management measures.

NOTE See 2.2(5) and Annex B

(2) Different levels of reliability may be adopted *inter alia* :
– for structural resistance ;
– for serviceability.

(3) The choice of the levels of reliability for a particular structure should take account of the relevant factors, including :
– the possible cause and /or mode of attaining a limit state ;
– the possible consequences of failure in terms of risk to life, injury, potential economical losses ;
– public aversion to failure ;
– the expense and procedures necessary to reduce the risk of failure.

(4) The levels of reliability that apply to a particular structure may be specified in one or both of the following ways :
– by the classification of the structure as a whole ;
– by the classification of its components.

NOTE See also Annex B

(5) The levels of reliability relating to structural resistance and serviceability can be achieved by suitable combinations of :

a) preventative and protective measures (e.g. implementation of safety barriers, active and passive protective measures against fire, protection against risks of corrosion such as painting or cathodic protection) ;

b) measures relating to design calculations :
– representative values of actions ;
– the choice of partial factors ;

c) measures relating to quality management ;

d) measures aimed to reduce errors in design and execution of the structure, and gross human errors ;

e) other measures relating to the following other design matters :
– the basic requirements ;
– the degree of robustness (structural integrity) ;
– durability, including the choice of the design working life ;
– the extent and quality of preliminary investigations of soils and possible environmental influences ;
– the accuracy of the mechanical models used ;
– the detailing ;

f) efficient execution, *e.g.* in accordance with execution standards referred to in EN 1991 to EN 1999.

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

(6) The measures to prevent potential causes of failure and/or reduce their consequences may, in appropriate circumstances, be interchanged to a limited extent provided that the required reliability levels are maintained.

2.3 Design working life

(1) The design working life should be specified.

NOTE Indicative categories are given in Table 2.1. The values given in Table 2.1 may also be used for determining time-dependent performance (*e.g.* fatigue-related calculations). See also Annex A.

Table 2.1 - Indicative design working life

Design working life category	Indicative design working life (years)	Examples
1	10	Temporary structures (1)
2	10 to 25	Replaceable structural parts, e.g. gantry girders, bearings
3	15 to 30	Agricultural and similar structures
4	50	Building structures and other common structures
5	100	Monumental building structures, bridges, and other civil engineering structures
(1) Structures or parts of structures that can be dismantled with a view to being re-used should not be considered as temporary.		

2.4 Durability

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

(2) In order to achieve an adequately durable structure, the following should be taken into account :
– the intended or foreseeable use of the structure ;
– the required design criteria ;
– the expected environmental conditions ;
– the composition, properties and performance of the materials and products ;
– the properties of the soil ;
– the choice of the structural system ;
– the shape of members and the structural detailing ;
– the quality of workmanship, and the level of control ;
– the particular protective measures ;
– the intended maintenance during the design working life.

NOTE The relevant EN 1992 to EN 1999 specify appropriate measures to reduce deterioration.

(3)P The environmental conditions shall be identified at the design stage so that their significance can be assessed in relation to durability and adequate provisions can be made for protection of the materials used in the structure.

(4) The degree of any deterioration may be estimated on the basis of calculations, experimental investigation, experience from earlier constructions, or a combination of these considerations.

2.5 Quality management

(1) In order to provide a structure that corresponds to the requirements and to the assumptions made in the design, appropriate quality management measures should be in place. These measures comprise :
– definition of the reliability requirements,
– organisational measures and
– controls at the stages of design, execution, use and maintenance.

NOTE EN ISO 9001:2000 is an acceptable basis for quality management measures, where relevant.

Section 3 Principles of limit states design

3.1 General

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

NOTE In some cases, additional verifications may be needed, for example to ensure traffic safety.

(2) Verification of one of the two categories of limit states may be omitted provided that sufficient information is available to prove that it is satisfied by the other.

(3)P Limit states shall be related to design situations, see 3.2.

(4) Design situations should be classified as persistent, transient or accidental, see 3.2.

(5) Verification of limit states that are concerned with time dependent effects (*e.g.* fatigue) should be related to the design working life of the construction.

NOTE Most time dependent effects are cumulative.

3.2 Design situations

(1)P The relevant design situations shall be selected taking into account the circumstances under which the structure is required to fulfil its function.

(2)P Design situations shall be classified as follows :
- persistent design situations, which refer to the conditions of normal use ;
- transient design situations, which refer to temporary conditions applicable to the structure, *e.g.* during execution or repair ;
- accidental design situations, which refer to exceptional conditions applicable to the structure or to its exposure, *e.g.* to fire, explosion, impact or the consequences of localised failure ;
- seismic design situations, which refer to conditions applicable to the structure when subjected to seismic events.

NOTE Information on specific design situations within each of these classes is given in EN 1991 to EN 1999.

(3)P The selected design situations shall be sufficiently severe and varied so as to encompass all conditions that can reasonably be foreseen to occur during the execution and use of the structure.

3.3 Ultimate limit states

(1)P The limit states that concern :
– the safety of people, and/or
– the safety of the structure
shall be classified as ultimate limit states.

(2) In some circumstances, the limit states that concern the protection of the contents should be classified as ultimate limit states.

NOTE The circumstances are those agreed for a particular project with the client and the relevant authority.

(3) States prior to structural collapse, which, for simplicity, are considered in place of the collapse itself, may be treated as ultimate limit states.

(4)P The following ultimate limit states shall be verified where they are relevant :
– loss of equilibrium of the structure or any part of it, considered as a rigid body ;
– failure by excessive deformation, transformation of the structure or any part of it into a mechanism, rupture, loss of stability of the structure or any part of it, including supports and foundations ;
– failure caused by fatigue or other time-dependent effects.

AC2 NOTE Different sets of partial factors are associated with the various ultimate limit states, see 6.4.1. AC2

3.4 Serviceability limit states

(1)P The 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.

NOTE 1 In the context of serviceability, the term "appearance" is concerned with such criteria as high deflection and extensive cracking, rather than aesthetics.

NOTE 2 Usually the serviceability requirements are agreed for each individual project.

(2)P 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 aspects :
a) deformations that affect
   – the appearance,
   – the comfort of users, or
   – the functioning of the structure (including the functioning of machines or services),
or that cause damage to finishes or non-structural members ;

b) vibrations
   – that cause discomfort to people, or
   – that limit the functional effectiveness of the structure ;

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

NOTE Additional provisions related to serviceability criteria are given in the relevant EN 1992 to EN 1999.

3.5 Limit state design

(1)P Design for limit states shall be based on the use of structural and load models for relevant limit states.

(2)P It shall be verified that no limit state is exceeded when relevant design values for
– actions,
– material properties, or
– product properties, and
– geometrical data
are used in these models.

(3)P The verifications shall be carried out for all relevant design situations and load cases.

(4) The requirements of 3.5(1)P should be achieved by the partial factor method, described in section 6.

(5) As an alternative, a design directly based on probabilistic methods may be used.

NOTE 1 The relevant authority can give specific conditions for use.

NOTE 2 For a basis of probabilistic methods, see Annex C.

(6)P The selected design situations shall be considered and critical load cases identified.

(7) For a particular verification load cases should be selected, identifying compatible load arrangements, sets of deformations and imperfections that should be considered simultaneously with fixed variable actions and permanent actions.

(8)P Possible deviations from the assumed directions or positions of actions shall be taken into account.

(9) Structural and load models can be either physical models or mathematical models.

Section 4 Basic variables

4.1 Actions and environmental influences

4.1.1 Classification of actions

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

- permanent actions *(G)*, e.g. self-weight of structures, fixed equipment and road surfacing, and indirect actions caused by shrinkage and uneven settlements ;
- variable actions *(Q)*, e.g. imposed loads on building floors, beams and roofs, wind actions or snow loads ;
- accidental actions *(A),* e.g. explosions, or impact from vehicles.

NOTE Indirect actions caused by imposed deformations can be either permanent or variable.

(2) Certain actions, such as seismic actions and snow loads, may be considered as either accidental and/or variable actions, depending on the site location, see EN 1991 and EN 1998.

(3) Actions caused by water may be considered as permanent and/or variable actions depending on the variation of their magnitude with time.

(4)P Actions shall also be classified

- by their origin, as direct or indirect,
- by their spatial variation, as fixed or free, or
- by their nature and/or the structural response, as static or dynamic.

(5) An action should be described by a model, its magnitude being represented in the most common cases by one scalar which may have several representative values.

NOTE For some actions and some verifications, a more complex representation of the magnitudes of some actions may be necessary.

4.1.2 Characteristic values of actions

(1)P The characteristic value F_k of an action is its main representative value and shall be specified :

- as a mean value, an upper or lower value, or a nominal value (which does not refer to a known statistical distribution) (see EN 1991) ;
- in the project documentation, provided that consistency is achieved with methods given in EN 1991.

(2)P The characteristic value of a permanent action shall be assessed as follows :

- if the variability of G can be considered as small, one single value G_k may be used ;
- if the variability of G cannot be considered as small, two values shall be used : an upper value $G_{k,sup}$ and a lower value $G_{k,inf}$.

(3) The variability of G may be neglected if G does not vary significantly during the design working life of the structure and its coefficient of variation is small. G_k should then be taken equal to the mean value.

NOTE This coefficient of variation can be in the range of 0,05 to 0,10 depending on the type of structure.

(4) In cases when the structure is very sensitive to variations in G (e.g. some types of prestressed concrete structures), two values should be used even if the coefficient of variation is small. Then $G_{k,inf}$ is the 5% fractile and $G_{k,sup}$ is the 95% fractile of the statistical distribution for G, which may be assumed to be Gaussian.

(5) The self-weight of the structure may be represented by a single characteristic value and be calculated on the basis of the nominal dimensions and mean unit masses, see EN 1991-1-1.

NOTE For the settlement of foundations, see EN 1997.

(6) Prestressing (P) should be classified as a permanent action caused by either controlled forces and/or controlled deformations imposed on a structure. These types of prestress should be distinguished from each other as relevant (e.g. prestress by tendons, prestress by imposed deformation at supports).

NOTE The characteristic values of prestress, at a given time t, may be an upper value $P_{k,sup}(t)$ and a lower value $P_{k,inf}(t)$. For ultimate limit states, a mean value $P_m(t)$ can be used. Detailed information is given in EN 1992 to EN 1996 and EN 1999.

(7)P For variable actions, the characteristic value (Q_k) shall correspond to either :
- an upper value with an intended probability of not being exceeded or a lower value with an intended probability of being achieved, during some specific reference period;
- a nominal value, which may be specified in cases where a statistical distribution is not known.

NOTE 1 Values are given in the various Parts of EN 1991.

NOTE 2 The characteristic value of climatic actions is based upon the probability of 0,02 of its time-varying part being exceeded for a reference period of one year. This is equivalent to a mean return period of 50 years for the time-varying part. However in some cases the character of the action and/or the selected design situation makes another fractile and/or return period more appropriate.

(8) For accidental actions the design value A_d should be specified for individual projects.

NOTE See also EN 1991-1-7.

(9) For seismic actions the design value A_{Ed} should be assessed from the characteristic value A_{Ek} or specified for individual projects.

NOTE See also EN 1998.

(10) For multi-component actions the characteristic action should be represented by groups of values each to be considered separately in design calculations.

4.1.3 Other representative values of variable actions

(1)P Other representative values of a variable action shall be as follows :

(a) the combination value, represented as a product $\psi_0\, Q_k$, used for the verification of ultimate limit states and irreversible serviceability limit states (see section 6 and Annex C) ;
(b) the frequent value, represented as a product $\psi_1 Q_k$, used for the verification of ultimate limit states involving accidental actions and for verifications of reversible serviceability limit states ;

NOTE 1 For buildings, for example, the frequent value is chosen so that the time it is exceeded is 0,01 of the reference period ; for road traffic loads on bridges, the frequent value is assessed on the basis of a return period of one week.

AC2 NOTE 2 The infrequent value, represented as a product $\psi_{1,\text{infq}} Q_k$, may be used only for the verification of certain serviceability limit states specifically for concrete bridges The infrequent value which is defined only for road traffic loads (see EN 1991-2) is based on a return period of one year.

NOTE 3 For the frequent value of multi-component traffic actions see EN 1991-2. AC2

(c) the quasi-permanent value, represented as a product $\psi_2 Q_k$, used for the verification of ultimate limit states involving accidental actions and for the verification of reversible serviceability limit states. Quasi-permanent values are also used for the calculation of long-term effects.

NOTE For loads on building floors, the quasi-permanent value is usually chosen so that the proportion of the time it is exceeded is 0,50 of the reference period. The quasi-permanent value can alternatively be determined as the value averaged over a chosen period of time. In the case of wind actions or road traffic loads, the quasi-permanent value is generally taken as zero.

4.1.4 Representation of fatigue actions

(1) The models for fatigue actions should be those that have been established in the relevant parts of EN 1991 from evaluation of structural responses to fluctuations of loads performed for common structures (*e.g.* for simple span and multi-span bridges, tall slender structures for wind).

(2) For structures outside the field of application of models established in the relevant Parts of EN 1991, fatigue actions should be defined from the evaluation of measurements or equivalent studies of the expected action spectra.

NOTE For the consideration of material specific effects (for example, the consideration of mean stress influence or non-linear effects), see EN 1992 to EN 1999.

4.1.5 Representation of dynamic actions

AC2 (1) The load models defined by characteristic values, and fatigue load models, in EN 1991 may include the effects of accelerations caused by the actions either implicitly or explicitly by applying dynamic enhancement factors. AC2

NOTE Limits of use of these models are described in the various Parts of EN 1991.

(2) When dynamic actions cause significant acceleration of the structure, dynamic analysis of the system should be used. See 5.1.3 (6).

4.1.6 Geotechnical actions

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

4.1.7 Environmental influences

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

NOTE The EN 1992 to EN 1999 give the relevant measures.

(2) The effects of environmental influences should be taken into account, and where possible, be described quantitatively.

4.2 Material and product properties

(1) Properties of materials (including soil and rock) or products should be represented by characteristic values (see 1.5.4.1).

(2) When a limit state verification is sensitive to the variability of a material property, upper and lower characteristic values of the material property should be taken into account.

(3) Unless otherwise stated in EN 1991 to EN 1999 :

– where a low value of material or product property is unfavourable, the characteristic value should be defined as the 5% fractile value;

– where a high value of material or product property is unfavourable, the characteristic value should be defined as the 95% fractile value.

(4)P Material property values shall be determined from standardised tests performed under specified conditions. A conversion factor shall be applied where it is necessary to convert the test results into values which can be assumed to represent the behaviour of the material or product in the structure or the ground.

NOTE See annex D and EN 1992 to EN 1999

(5) Where insufficient statistical data are available to establish the characteristic values of a material or product property, nominal values may be taken as the characteristic values, or design values of the property may be established directly. Where upper or lower design values of a material or product property are established directly (*e.g.* friction factors, damping ratios), they should be selected so that more adverse values would affect the probability of occurrence of the limit state under consideration to an extent similar to other design values.

(6) Where an upper estimate of strength is required (*e.g.* for capacity design measures and for the tensile strength of concrete for the calculation of the effects of indirect actions) a characteristic upper value of the strength should be taken into account.

(7) The reductions of the material strength or product resistance to be considered resulting from the effects of repeated actions are given in EN 1992 to EN 1999 and can lead to a reduction of the resistance over time due to fatigue.

(8) The structural stiffness parameters (*e.g.* moduli of elasticity, creep coefficients) and thermal expansion coefficients should be represented by a mean value. Different values should be used to take into account the duration of the load.

NOTE In some cases, a lower or higher value than the mean for the modulus of elasticity may have to be taken into account (*e.g.* in case of instability).

(9) Values of material or product properties are given in EN 1992 to EN 1999 and in the relevant harmonised European technical specifications or other documents. If values are taken from product standards without guidance on interpretation being given in EN 1992 to EN 1999, the most adverse values should be used.

(10)P Where a partial factor for materials or products is needed, a conservative value shall be used, unless suitable statistical information exists to assess the reliability of the value chosen.

NOTE Suitable account may be taken where appropriate of the unfamiliarity of the application or materials/products used.

4.3 Geometrical data

(1)P Geometrical data shall be represented by their characteristic values, or (*e.g.* the case of imperfections) directly by their design values.

(2) The dimensions specified in the design may be taken as characteristic values.

(3) Where their statistical distribution is sufficiently known, values of geometrical quantities that correspond to a prescribed fractile of the statistical distribution may be used.

(4) Imperfections that should be taken into account in the design of structural members are given in EN 1992 to EN 1999.

(5)P Tolerances for connected parts that are made from different materials shall be mutually compatible.

Section 5 Structural analysis and design assisted by testing

5.1 Structural analysis

5.1.1 Structural modelling

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

(2) The structural models selected should be those appropriate for predicting structural behaviour with an acceptable level of accuracy. The structural models should also be appropriate to the limit states considered.

(3)P Structural models shall be based on established engineering theory and practice. If necessary, they shall be verified experimentally.

5.1.2 Static actions

(1)P The modelling for 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)P Boundary conditions applied to the model shall represent those intended in the structure.

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

NOTE Particular methods for dealing with effects of deformations are given in EN 1991 to EN 1999.

(4)P Indirect actions shall be introduced in the analysis as follows :
– in linear elastic analysis, directly or as equivalent forces (using appropriate modular ratios where relevant) ;
– in non-linear analysis, directly as imposed deformations.

5.1.3 Dynamic actions

(1)P The structural model to be used for determining the action effects shall be established taking account of all relevant structural members, their masses, strengths, stiffnesses and damping characteristics, and all relevant non structural members with their properties.

(2)P The boundary conditions applied to the model shall be representative of those intended in the structure.

(3) When it is appropriate to consider dynamic actions as quasi-static, the dynamic parts may be considered either by including them in the static values or by applying equivalent dynamic amplification factors to the static actions.

NOTE For some equivalent dynamic amplification factors, the natural frequencies are determined.

(4) In the case of ground-structure interaction, the contribution of the soil may be modelled by appropriate equivalent springs and dash-pots.

(5) Where relevant (*e.g.* for wind induced vibrations or seismic actions) the actions may be defined by a modal analysis based on linear material and geometric behaviour. For structures that have regular geometry, stiffness and mass distribution, provided that only the fundamental mode is relevant, an explicit modal analysis may be substituted by an analysis with equivalent static actions.

(6) The dynamic actions may be also expressed, as appropriate, in terms of time histories or in the frequency domain, and the structural response determined by appropriate methods.

(7) Where dynamic actions cause vibrations of a magnitude or frequencies that could exceed serviceability requirements, a serviceability limit state verification should be carried out.

NOTE Guidance for assessing these limits is given in Annex A and EN 1992 to EN 1999.

5.1.4 Fire design

(1)P The structural fire design analysis shall be based on design fire scenarios (see EN 1991-1-2), and shall consider models for the temperature evolution within the structure as well as models for the mechanical behaviour of the structure at elevated temperature.

(2) The required performance of the structure exposed to fire should be verified by either global analysis, analysis of sub-assemblies or member analysis, as well as the use of tabular data or test results.

(3) The behaviour of the structure exposed to fire should be assessed by taking into account either :
– nominal fire exposure, or
– modelled fire exposure,
as well as the accompanying actions.

NOTE See also EN 1991-1-2.

(4) The structural behaviour at elevated temperatures should be assessed in accordance with EN 1992 to EN 1996 and EN 1999, which give thermal and structural models for analysis.

(5) Where relevant to the specific material and the method of assessment :

– thermal models may be based on the assumption of a uniform or a non-uniform temperature within cross-sections and along members ;
– structural models may be confined to an analysis of individual members or may account for the interaction between members in fire exposure.

(6) The models of mechanical behaviour of structural members at elevated temperatures should be non-linear.

NOTE See also EN 1991 to EN 1999.

5.2 Design assisted by testing

(1) Design may be based on a combination of tests and calculations.

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

– if adequate calculation models are not available ;
– if a large number of similar components are to be used ;
– to confirm by control checks assumptions made in the design.

See Annex D.

(2)P Design assisted by test results shall achieve the level of reliability required for the relevant design situation. The statistical uncertainty due to a limited number of test results shall be taken into account.

(3) Partial factors (including those for model uncertainties) comparable to those used in EN 1991 to EN 1999 should be used.

Section 6 Verification by the partial factor method

6.1 General

(1)P When using the partial factor method, it shall be verified that, in all relevant design situations, no relevant limit state is exceeded when design values for actions or effects of actions and resistances are used in the design models.

(2) For the selected design situations and the relevant limit states the individual actions for the critical load cases should be combined as detailed in this section. However actions that cannot occur simultaneously, for example due to physical reasons, should not be considered together in combination.

(3) Design values should be obtained by using :
- the characteristic, or
- other representative values,

in combination with partial and other factors as defined in this section and EN 1991 to EN 1999.

(4) It can be appropriate to determine design values directly where conservative values should be chosen.

(5)P Design values directly determined on statistical bases shall correspond to at least the same degree of reliability for the various limit states as implied by the partial factors given in this standard.

6.2 Limitations

(1) The use of the Application Rules given in EN 1990 is limited to ultimate and serviceability limit state verifications of structures subject to static loading, including cases where the dynamic effects are assessed using equivalent quasi-static loads and dynamic amplification factors, including wind or traffic loads. For non-linear analysis and fatigue the specific rules given in various Parts of EN 1991 to EN 1999 should be applied.

6.3 Design values

6.3.1 Design values of actions

(1) The design value F_d of an action F can be expressed in general terms as :

$$F_\mathrm{d} = \gamma_\mathrm{f} F_\mathrm{rep} \tag{6.1a}$$

with :

$$F_{rep} = \psi F_k \tag{6.1b}$$

where :

F_k is the characteristic value of the action.

F_{rep} is the relevant representative value of the action.

γ_f is a partial factor for the action which takes account of the possibility of unfavourable deviations of the action values from the representative values.

ψ is either 1,00 or ψ_0, ψ_1 or ψ_2.

(2) For seismic actions the design value A_{Ed} should be determined taking account of the structural behaviour and other relevant criteria detailed in EN 1998.

6.3.2 Design values of the effects of actions

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

$$E_d = \gamma_{Sd} E\{\gamma_{f,i} F_{rep,i} \,; a_d\} \quad i \geq 1 \tag{6.2}$$

where :

a_d is the design values of the geometrical data (see 6.3.4) ;

γ_{Sd} is a partial factor taking account of uncertainties :
– in modelling the effects of actions ;
– in some cases, in modelling the actions.

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

(2) In most cases, the following simplification can be made :

$$E_d = E\{\gamma_{F,i} F_{rep,i} \,; a_d\} \quad i \geq 1 \tag{6.2a}$$

with :

$$\gamma_{F,i} = \gamma_{Sd} \times \gamma_{f,i} \tag{6.2b}$$

NOTE When relevant, *e.g.* where geotechnical actions are involved, partial factors $\gamma_{F,i}$ can be applied to the effects of individual actions or only one particular factor γ_F can be globally applied to the effect of the combination of actions with appropriate partial factors.

(3)P Where a distinction has to be made between favourable and unfavourable effects of permanent actions, two different partial factors shall be used ($\gamma_{G,inf}$ and $\gamma_{G,sup}$).

(4) For non-linear analysis (i.e. when the relationship between actions and their effects is not linear), the following simplified rules may be considered in the case of a single predominant action :

a) When the action effect increases more than the action, the partial factor γ_F should be applied to the representative value of the action.

b) When the action effect increases less than the action, the partial factor γ_F should be applied to the action effect of the representative value of the action.

NOTE Except for rope, cable and membrane structures, most structures or structural elements are in category a).

(5) In those cases where more refined methods are detailed in the relevant EN 1991 to EN 1999 (*e.g.* for prestressed structures), they should be used in preference to 6.3.2(4).

6.3.3 Design values of material or product properties

(1) The design value X_d of a material or product property can be expressed in general terms as :

$$X_d = \eta \frac{X_k}{\gamma_m} \tag{6.3}$$

where :

X_k is the characteristic value of the material or product property (see 4.2(3)) ;

η is the mean value of the conversion factor taking into account
- volume and scale effects,
- effects of moisture and temperature, and
- any other relevant parameters ;

γ_m is the partial factor for the material or product property to take account of :
- the possibility of an unfavourable deviation of a material or product property from its characteristic value ;
- the random part of the conversion factor η.

(2) Alternatively, in appropriate cases, the conversion factor η may be :
- implicitly taken into account within the characteristic value itself, or
- by using γ_M instead of γ_m (see expression (6.6b)).

NOTE The design value can be established by such means as :
- empirical relationships with measured physical properties, or
- with chemical composition, or
- from previous experience, or
- from values given in European Standards or other appropriate documents.

6.3.4 Design values of geometrical data

(1) Design values of geometrical data such as dimensions of members that are used to assess action effects and/or resistances may be represented by nominal values :

$$a_d = a_{nom} \tag{6.4}$$

(2)P Where the effects of deviations in geometrical data (*e.g.* inaccuracy in the load application or location of supports) are significant for the reliability of the structure (*e.g.* by second order effects) the design values of geometrical data shall be defined by :

$$a_d = a_{nom} \pm \Delta a \qquad (6.5)$$

where :

Δa takes account of :
- the possibility of unfavourable deviations from the characteristic or nominal values ;
- the cumulative effect of a simultaneous occurrence of several geometrical deviations.

NOTE 1 a_d can also represent geometrical imperfections where $a_{nom} = 0$ (i.e., $\Delta a \neq 0$).

NOTE 2 Where relevant, EN 1991 to EN 1999 provide further provisions.

(3) Effects of other deviations should be covered by partial factors
- on the action side (γ_F), and/or
- resistance side (γ_M).

NOTE Tolerances are defined in the relevant standards on execution referred to in EN 1990 to EN 1999.

6.3.5 Design resistance

(1) The design resistance R_d can be expressed in the following form :

$$R_d = \frac{1}{\gamma_{Rd}} R\{X_{d,i}; a_d\} = \frac{1}{\gamma_{Rd}} R\left\{\eta_i \frac{X_{k,i}}{\gamma_{m,i}}; a_d\right\} \quad i \geq 1 \qquad (6.6)$$

where :

γ_{Rd} is a partial factor covering uncertainty in the resistance model, plus geometric deviations if these are not modelled explicitly (see 6.3.4(2));

$X_{d,i}$ is the design value of material property *i*.

(2) The following simplification of expression (6.6) may be made :

$$R_d = R\left\{\eta_i \frac{X_{k,i}}{\gamma_{M,i}}; a_d\right\} \quad \mathrm{i} \geq 1 \qquad (6.6a)$$

where :

$$\gamma_{M,i} = \gamma_{Rd} \times \gamma_{m,i} \qquad (6.6b)$$

NOTE η_i may be incorporated in $\gamma_{M,i}$, see 6.3.3.(2).

(3) Alternatively to expression (6.6a), the design resistance may be obtained directly from the characteristic value of a material or product resistance, without explicit determination of design values for individual basic variables, using :

$$R_d = \frac{R_k}{\gamma_M} \tag{6.6c}$$

NOTE This is applicable to products or members made of a single material (e.g. steel) and is also used in connection with Annex D "Design assisted by testing".

(4) Alternatively to expressions (6.6a) and (6.6c), for structures or structural members that are analysed by non-linear methods, and comprise more than one material acting in association, or where ground properties are involved in the design resistance, the following expression for design resistance can be used :

$$R_d = \frac{1}{\gamma_{M,1}} R\left\{\eta_1 X_{k,1}; \eta_i X_{k,i(i>1)} \frac{\gamma_{m,1}}{\gamma_{m,i}}; a_d\right\} \tag{6.6d}$$

NOTE In some cases, the design resistance can be expressed by applying directly γ_M partial factors to the individual resistances due to material properties.

6.4 Ultimate limit states

6.4.1 General

(1)P The following ultimate limit states shall be verified as relevant :

a) EQU : Loss of static equilibrium of the structure or any part of it considered as a rigid body, where :
   - AC2 minor variations in the value or the spatial distribution of permanent actions from a single source are significant, and AC2
   - the strengths of construction materials or ground are generally not governing ;

b) STR : Internal failure or excessive deformation of the structure or structural members, including footings, piles, basement walls, etc., where the strength of construction materials of the structure governs ;

c) GEO : Failure or excessive deformation of the ground where the strengths of soil or rock are significant in providing resistance ;

d) FAT : Fatigue failure of the structure or structural members.

AC2 NOTE For fatigue design, the combinations of actions are given in EN 1992 to EN 1995, EN 1998 and EN 1999.

e) UPL : loss of equilibrium of the structure or the ground due to uplift by water pressure (buoyancy) or other vertical actions ;

NOTE See EN 1997.

f) HYD : hydraulic heave, internal erosion and piping in the ground caused by hydraulic gradients.

NOTE See EN 1997. AC2

(2)P The design values of actions shall be in accordance with Annex A.

6.4.2 Verifications of static equilibrium and resistance

(1)P When considering a limit state of static equilibrium of the structure (EQU), it shall be verified that :

$$E_{d,dst} \leq E_{d,stb} \tag{6.7}$$

where :

$E_{d,dst}$ is the design value of the effect of destabilising actions ;

$E_{d,stb}$ is the design value of the effect of stabilising actions.

(2) Where appropriate the expression for a limit state of static equilibrium may be supplemented by additional terms, including, for example, a coefficient of friction between rigid bodies.

(3)P When considering a limit state of rupture or excessive deformation of a section, member or connection (STR and/or GEO), it shall be verified that :

$$E_d \leq R_d \tag{6.8}$$

where :

E_d is the design value of the effect of actions such as internal force, moment or a vector representing several internal forces or moments ;

R_d is the design value of the corresponding resistance.

NOTE.1 Details for the methods STR and GEO are given in Annex A.

NOTE 2 Expression (6.8) does not cover all verification formats concerning buckling, i.e. failure that happens where second order effects cannot be limited by the structural response, or by an acceptable structural response. See EN 1992 to EN 1999.

6.4.3 Combination of actions (fatigue verifications excluded)

6.4.3.1 General

(1)P 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 :
– a leading variable action, or
– an accidental action.

(3) The combinations of actions should be in accordance with 6.4.3.2 to 6.4.3.4.

(4)P Where the results of a verification are very sensitive to variations of the magnitude of a permanent action from place to place in the structure, the unfavourable and the favourable parts of this action shall be considered as individual actions.

NOTE This applies in particular to the verification of static equilibrium and analogous limit states, see 6.4.2(2).

(5) Where several effects of one action (*e.g.* bending moment and normal force due to self-weight) are not fully correlated, the partial factor applied to any favourable component may be reduced.

NOTE For further guidance on this topic see the clauses on vectorial effects in EN 1992 to EN 1999.

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

NOTE For further guidance, see 5.1.2.4(P) and EN 1992 to EN 1999.

6.4.3.2 Combinations of actions for persistent or transient design situations (fundamental combinations)

(1) The general format of effects of actions should be :

$$E_d = \gamma_{Sd} E\{\gamma_{g,j} G_{k,j} \,;\, \gamma_p P \,;\, \gamma_{q,1} Q_{k,1} \,;\, \gamma_{q,i} \psi_{0,i} Q_{k,i}\} \quad j \geq 1 \,;\, i > 1 \tag{6.9a}$$

(2) The combination of effects of actions to be considered should be based on
– the design value of the leading variable action, and
– the design combination values of accompanying variable actions :

NOTE See also 6.4.3.2(4).

$$E_d = E\{\gamma_{G,j} G_{k,j} \,;\, \gamma_P P \,;\, \gamma_{Q,1} Q_{k,1} \,;\, \gamma_{Q,i} \psi_{0,i} Q_{k,i}\} \quad j \geq 1 \,;\, i > 1 \tag{6.9b}$$

(3) The combination of actions in brackets { }, in (6.9b) may either be expressed as :

$$\sum_{j\geq 1} \gamma_{G,j} G_{\mathrm{k,j}} \text{"+"} \gamma_{\mathrm{P}} P \text{"+"} \gamma_{\mathrm{Q,1}} Q_{\mathrm{k,1}} \text{"+"} \sum_{\mathrm{i>1}} \gamma_{\mathrm{Q,i}} \psi_{0,\mathrm{i}} Q_{\mathrm{k,i}} \tag{6.10}$$

or, alternatively for STR and GEO limit states, the less favourable of the two following expressions:

$$\begin{cases} \sum\limits_{j\geq 1} \gamma_{G,j} G_{k,j} \text{"+"} \gamma_P P \text{"+"} \gamma_{Q,1} \psi_{0,1} Q_{k,1} \text{"+"} \sum\limits_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} & \text{(6.10a)} \\ \sum\limits_{j\geq 1} \xi_j \gamma_{G,j} G_{k,j} \text{"+"} \gamma_P P \text{"+"} \gamma_{Q,1} Q_{k,1} \text{"+"} \sum\limits_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} & \text{(6.10b)} \end{cases}$$

Where :

"+ "	implies "to be combined with"
Σ	implies "the combined effect of"
ξ	is a reduction factor for unfavourable permanent actions G

NOTE Further information for this choice is given in Annex A.

(4) If the relationship between actions and their effects is not linear, expressions (6.9a) or (6.9b) should be applied directly, depending upon the relative increase of the effects of actions compared to the increase in the magnitude of actions (see also 6.3.2.(4)).

6.4.3.3 Combinations of actions for accidental design situations

(1) The general format of effects of actions should be :

$$E_d = E\left\{G_{k,j}\,;\,P\,;\,A_d\,;\,(\psi_{1,1}\text{ or }\psi_{2,1})Q_{k,1}\,;\,\psi_{2,i}Q_{k,i}\right\} \quad j\geq 1\,;\,i>1 \tag{6.11a}$$

(2) The combination of actions in brackets { } can be expressed as :

$$\sum_{j\geq 1} G_{k,j}\text{"+"}P\text{"+"}A_d\text{"+"}(\psi_{1,1}\text{ or }\psi_{2,1})Q_{k,1}\text{"+"}\sum_{i>1}\psi_{2,i}Q_{k,i} \tag{6.11b}$$

(3) The choice between $\psi_{1,1}Q_{k,1}$ or $\psi_{2,1}Q_{k,1}$ should be related to the relevant accidental design situation (impact, fire or survival after an accidental event or situation).

NOTE Guidance is given in the relevant Parts of EN 1991 to EN 1999.

(4) Combinations of actions for accidental design situations should either
– involve an explicit accidental action A (fire or impact), or
– refer to a situation after an accidental event ($A = 0$).

[AC2] For fire situations, apart from the temperature effect on the material properties, A_d should represent the design value of the indirect effects of thermal action due to fire. [AC2]

6.4.3.4 Combinations of actions for seismic design situations

(1) The general format of effects of actions should be :

$$E_d = E\left\{G_{k,j}\,;\,P\,;\,A_{Ed}\,;\,\psi_{2,i}Q_{k,i}\right\} \quad j\geq 1\,;\,i\geq 1 \tag{6.12a}$$

(2) The combination of actions in brackets { } can be expressed as :

$$\sum_{j\geq 1} G_{k,j}\text{"+"}P\text{"+"}A_{Ed}\text{"+"}\sum_{i\geq 1}\psi_{2,i}Q_{k,i} \tag{6.12b}$$

6.4.4 Partial factors for actions and combinations of actions

(1) The values of the γ and ψ factors for actions should be obtained from EN 1991 and from Annex A.

6.4.5 Partial factors for materials and products

(1) The partial factors for properties of materials and products should be obtained from EN 1992 to EN 1999.

6.5 Serviceability limit states

6.5.1 Verifications

(1)P It shall be verified that :

$$E_{\mathrm{d}} \leq C_{\mathrm{d}} \tag{6.13}$$

where :

C_{d} is the limiting design value of the relevant serviceability criterion.

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

6.5.2 Serviceability criteria

(1) The deformations to be taken into account in relation to serviceability requirements should be as detailed in the relevant Annex A according to the type of construction works, or agreed with the client or the National authority.

NOTE For other specific serviceability criteria such as crack width, stress or strain limitation, slip resistance, see EN 1991 to EN 1999.

6.5.3 Combination of actions

(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) The combinations of actions for serviceability limit states are defined symbolically by the following expressions (see also 6.5.4) :

NOTE It is assumed, in these expressions, that all partial factors are equal to 1. See Annex A and EN 1991 to EN 1999.

a) Characteristic combination :

$$E_d = E\left\{G_{k,j}\,;P\,;Q_{k,1}\,;\psi_{0,i}Q_{k,i}\right\} \quad j \geq 1\,; i > 1 \tag{6.14a}$$

in which the combination of actions in brackets { } (called the characteristic combination), can be expressed as :

$$\sum_{j\geq 1} G_{k,j} \text{"+"} P \text{"+"} Q_{k,1} \text{"+"} \sum_{i>1} \psi_{0,i} Q_{k,i} \qquad (6.14b)$$

NOTE The characteristic combination is normally used for irreversible limit states.

b) Frequent combination :

$$E_d = E\{G_{k,j}\ ;\ P\ ;\ \psi_{1,1}Q_{k,1}\ ;\ \psi_{2,i}Q_{k,i}\} \quad j \geq 1\ ;\ i > 1 \qquad (6.15a)$$

in which the combination of actions in brackets { }, (called the frequent combination), can be expressed as :

$$\sum_{j\geq 1} G_{k,j} \text{"+"} P \text{"+"} \psi_{1,1} Q_{k,1} \text{"+"} \sum_{i>1} \psi_{2,i} Q_{k,i} \qquad (6.15b)$$

NOTE The frequent combination is normally used for reversible limit states.

c) Quasi-permanent combination :

$$E_d = E\{G_{k,j}\ ;\ P\ ;\ \psi_{2,i}Q_{k,i}\} \quad j \geq 1\ ;\ i \geq 1 \qquad (6.16a)$$

in which the combination of actions in brackets { }, (called the quasi-permanent combination), can be expressed as :

$$\sum_{j\geq 1} G_{k,j} \text{"+"} P \text{"+"} \sum_{i\geq 1} \psi_{2,i} Q_{k,i} \qquad (6.16b)$$

where the notation is as given in 1.6 and 6.4.3(1).

NOTE The quasi-permanent combination is normally used for long-term effects and the appearance of the structure.

(3) For the representative value of the prestressing action (i.e. P_k or P_m), reference should be made to the relevant design Eurocode for the type of prestress under consideration.

(4)P Effects of actions due to imposed deformations shall be considered where relevant.

NOTE In some cases expressions (6.14) to (6.16) require modification. Detailed rules are given in the relevant Parts of EN 1991 to EN 1999.

6.5.4 Partial factors for materials

(1) For serviceability limit states the partial factors γ_M for the properties of materials should be taken as 1,0 except if differently specified in EN 1992 to EN 1999.

Annex A1
(normative)
Application for Buildings

A1.1 Field of application

(1) This annex A1 gives rules and methods for establishing combinations of actions for buildings. It also gives the recommended design values of permanent, variable and accidental actions and ψ factors to be used in the design of buildings.

NOTE Guidance may be given in the National annex with regard to the use of Table 2.1 (design working life).

A1.2 Combinations of actions

A1.2.1 General

(1) Effects of actions that cannot exist simultaneously due to physical or functional reasons should not be considered together in combinations of actions.

NOTE 1 Depending on its uses and the form and the location of a building, the combinations of actions may be based on not more than two variable actions.

NOTE 2 Where modifications of A1.2.1(2) and A1.2.1(3) are necessary for geographical reasons, these can be defined in the National annex.

(2) The combinations of actions given in expressions 6.9a to 6.12b should be used when verifying ultimate limit states.

(3) The combinations of actions given in expressions 6.14a to 6.16b should be used when verifying serviceability limit states.

(4) Combinations of actions that include prestressing forces should be dealt with as detailed in EN 1992 to EN 1999.

A1.2.2 Values of ψ factors

(1) Values of ψ factors should be specified.

AC2) NOTE Recommended values of ψ factors for the more common actions may be obtained from Table A1.1. For ψ factors during execution see EN 1991-1-6 Annex A1.(AC2

Table A1.1 - Recommended values of ψ factors for buildings

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A : domestic, residential areas	0,7	0,5	0,3
Category B : office areas	0,7	0,5	0,3
Category C : congregation areas	0,7	0,7	0,6
Category D : shopping areas	0,7	0,7	0,6
Category E : storage areas	1,0	0,9	0,8
Category F : traffic area, vehicle weight ≤ 30kN	0,7	0,7	0,6
Category G : traffic area, 30kN < vehicle weight ≤ 160kN	0,7	0,5	0,3
Category H : roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)*			
Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
NOTE The ψ values may be set by the National annex. * For countries not mentioned below, see relevant local conditions.			

A1.3 Ultimate limit states

A1.3.1 Design values of actions in persistent and transient design situations

(1) The design values of actions for ultimate limit states in the persistent and transient design situations (expressions 6.9a to 6.10b) should be in accordance with Tables A1.2(A) to (C).

NOTE The values in Tables A1.2 ((A) to (C)) can be altered *e.g.* for different reliability levels in the National annex (see Section 2 and Annex B).

(2) In applying Tables A1.2(A) to A1.2(C) in cases when the limit state is very sensitive to variations in the magnitude of permanent actions, the upper and lower characteristic values of actions should be taken according to 4.1.2(2)P.

(3) Static equilibrium (EQU, see 6.4.1) for building structures should be verified using the design values of actions in Table A1.2(A).

(4) Design of structural members (STR, see 6.4.1) not involving geotechnical actions should be verified using the design values of actions from Table A1.2(B).

(5) Design of structural members (footings, piles, basement walls, etc.) (STR) involving geotechnical actions and the resistance of the ground (GEO, see 6.4.1) should be verified using one of the following three approaches supplemented, for geotechnical actions and resistances, by EN 1997 :

– Approach 1: Applying in separate calculations design values from Table A1.2(C) and Table A1.2(B) to the geotechnical actions as well as the other actions on/from the structure. In common cases, the sizing of foundations is governed by Table A1.2(C) and the structural resistance is governed by Table A1.2(B) ;

NOTE In some cases, application of these tables is more complex, see EN 1997.

– Approach 2 : Applying design values from Table A1.2(B) to the geotechnical actions as well as the other actions on/from the structure ;

– Approach 3 : Applying design values from Table A1.2(C) to the geotechnical actions and, simultaneously, applying partial factors from Table A1.2(B) to the other actions on/from the structure,

NOTE The use of approaches 1, 2 or 3 is chosen in the National annex.

(6) Overall stability for building structures (e.g. the stability of a slope supporting a building) should be verified in accordance with EN 1997.

AC2 (7) Hydraulic (HYD) and buoyancy (UPL) failure (e.g. in the bottom of an excavation for a building structure) should be verified in accordance with EN 1997. AC2

AC2 Table A1.2(A) - Design values of actions (EQU) (Set A)

Persistent and transient design situations	Permanent actions		Leading variable action (*)	Accompanying variable actions	
	Unfavourable	Favourable		Main (if any)	Others
(Eq. 6.10)	$\gamma_{G,j,sup}G_{k,j,sup}$	$\gamma_{G,j,inf}G_{k,j,inf}$	$\gamma_{Q,1}\ Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

(*) Variable actions are those considered in Table A1.1

NOTE 1 The γ values may be set by the National annex. The recommended set of values for γ are :

$\gamma_{G,j,sup} = 1{,}10$

$\gamma_{G,j,inf} = 0{,}90$

$\gamma_{Q,1} = 1{,}50$ where unfavourable (0 where favourable)

$\gamma_{Q,i} = 1{,}50$ where unfavourable (0 where favourable)

NOTE 2 In cases where the verification of static equilibrium also involves the resistance of structural members, as an alternative to two separate verifications based on Tables A1.2(A) and A1.2(B), a combined verification, based on Table A1.2(A), may be adopted, if allowed by the National annex, with the following set of recommended values. The recommended values may be altered by the National annex.

$\gamma_{G,j,sup} = 1{,}35$

$\gamma_{G,j,inf} = 1{,}15$

$\gamma_{Q,1} = 1{,}50$ where unfavourable (0 where favourable)

$\gamma_{Q,i} = 1{,}50$ where unfavourable (0 where favourable)

provided that applying $\gamma_{G,j,inf} = 1{,}00$ both to the favourable part and to the unfavourable part of permanent actions does not give a more unfavourable effect.

AC2

AC2 Table A1.2(B) - Design values of actions (STR/GEO) (Set B)

Persistent and transient design situations	Permanent actions		Leading variable action	Accompanying variable actions (*)	
	Unfavourable	Favourable		Main (if any)	Others
(Eq. 6.10)	$\gamma_{G,j,sup}G_{k,j,sup}$	$\gamma_{G,j,inf}G_{k,j,inf}$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

Persistent and transient design situations	Permanent actions		Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable	Action	Main	Others
(Eq. 6.10a)	$\gamma_{G,j,sup}G_{k,j,sup}$	$\gamma_{G,j,inf}G_{k,j,inf}$		$\gamma_{Q,1}\psi_{0,1}Q_{k,1}$	$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$
(Eq. 6.10b)	$\xi\gamma_{G,j,sup}G_{k,j,sup}$	$\gamma_{G,j,inf}G_{k,j,inf}$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

(*) Variable actions are those considered in Table A1.1

NOTE 1 The choice between 6.10, or 6.10a and 6.10b will be in the National annex. In case of 6.10a and 6.10b, the National annex may in addition modify 6.10a to include permanent actions only.

NOTE 2 The γ and ξ values may be set by the National annex. The following values for γ and ξ are recommended when using expressions 6.10, or 6.10a and 6.10b.

$\gamma_{G,j,sup} = 1,35$
$\gamma_{G,j,inf} = 1,00$
$\gamma_{Q,1} = 1,50$ where unfavourable (0 where favourable)
$\gamma_{Q,i} = 1,50$ where unfavourable (0 where favourable)
$\xi = 0,85$ (so that $\xi\gamma_{G,j,sup} = 0,85 \times 1,35 \cong 1,15$).
See also EN 1991 to EN 1999 for γ values to be used for imposed deformations.

NOTE 3 The characteristic values of all permanent actions from one source are multiplied by $\gamma_{G,sup}$ if the total resulting action effect is unfavourable and $\gamma_{G,inf}$ if the total resulting action effect is favourable. For example, all actions originating from the self weight of the structure may be considered as coming from one source; this also applies if different materials are involved.

NOTE 4 For particular verifications, the values for γ_G and γ_Q may be subdivided into γ_g and γ_q and the model uncertainty factor γ_{Sd}. A value of γ_{Sd} in the range 1,05 to 1,15 can be used in most common cases and can be modified in the National annex. AC2

AC2 Table A1.2(C) - Design values of actions (STR/GEO) (Set C)

Persistent and transient design situation	Permanent actions		Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable		Main (if any)	Others
(Eq. 6.10)	$\gamma_{G,j,sup}G_{k,j,sup}$	$\gamma_{G,j,inf}G_{k,j,inf}$	$\gamma_{Q,1}\ Q_{k,1}$		$\gamma_{Q,i}\,\psi_{0,i}Q_{k,i}$

(*) Variable actions are those considered in Table A1.1

NOTE The γ values may be set by the National annex. The recommended set of values for γ are :

$\gamma_{G,j,sup}$ = 1,00
$\gamma_{G,j,inf}$ = 1,00
$\gamma_{Q,1}$ = 1,30 where unfavourable (0 where favourable)
$\gamma_{Q,i}$ = 1,30 where unfavourable (0 where favourable)

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A1.3.2 Design values of actions in the accidental and seismic design situations

(1) The partial factors for actions for the ultimate limit states in the accidental and seismic design situations (expressions 6.11a to 6.12b) should be 1,0. ψ values are given in Table A1.1.

NOTE For the seismic design situation see also EN 1998.

Table A1.3 - Design values of actions for use in accidental and seismic combinations of actions

AC2

Design situation	Permanent actions		Leading accidental or seismic action	Accompanying variable actions (**)	
	Unfavourable	Favourable		Main (if any)	Others
Accidental (*) (Eq. 6.11a/b)	$G_{k,j,sup}$	$G_{k,j,inf}$	A_d	$\psi_{1,1}$ or $\psi_{2,1}Q_{k,1}$	$\psi_{2,i}\ Q_{k,i}$
Seismic (Eq. 6.12a/b)	$G_{k,j,sup}$	$G_{k,j,inf}$	$A_{Ed} = \gamma_I A_{Ek}$		$\psi_{2,i}\ Q_{k,i}$

(*) In the case of accidental design situations, the main variable action may be taken with its frequent or, as in seismic combinations of actions, its quasi-permanent values. The choice will be in the National annex, depending on the accidental action under consideration. See also EN 1991-1-2.

(**) Variable actions are those considered in Table A1.1.

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A1.4 Serviceability limit states

A1.4.1 Partial factors for actions

(1) For serviceability limit states the partial factors for actions should be taken as 1,0 except if differently specified in EN 1991 to EN 1999.

Table A1.4 - Design values of actions for use in the combination of actions

AC2
Combination	Permanent actions G_d		Variable actions Q_d	
	Unfavourable	Favourable	Leading	Others
Characteristic	$G_{k,j,sup}$	$G_{k,j,inf}$	$Q_{k,1}$	$\psi_{0,i}Q_{k,i}$
Frequent	$G_{k,j,sup}$	$G_{k,j,inf}$	$\psi_{1,1}Q_{k,1}$	$\psi_{2,i}Q_{k,i}$
Quasi-permanent	$G_{k,j,sup}$	$G_{k,j,inf}$	$\psi_{2,1}Q_{k,1}$	$\psi_{2,i}Q_{k,i}$

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A1.4.2 Serviceability criteria

(1) Serviceability limit states in buildings should take into account criteria related, for example, to floor stiffness, differential floor levels, storey sway or/and building sway and roof stiffness. Stiffness criteria may be expressed in terms of limits for vertical deflections and for vibrations. Sway criteria may be expressed in terms of limits for horizontal displacements.

(2) The serviceability criteria should be specified for each project and agreed with the client.

NOTE The serviceability criteria may be defined in the National annex.

(3)P The serviceability criteria for deformations and vibrations shall be defined :
– depending on the intended use ;
– in relation to the serviceability requirements in accordance with 3.4 ;
– independently of the materials used for supporting structural member.

A1.4.3 Deformations and horizontal displacements

(1) Vertical and horizontal deformations should be calculated in accordance with EN 1992 to EN 1999, by using the appropriate combinations of actions according to expressions (6.14a) to (6.16b) taking into account the serviceability requirements given in 3.4(1). Special attention should be given to the distinction between reversible and irreversible limit states.

(2) Vertical deflections are represented schematically in Figure. A1.1.

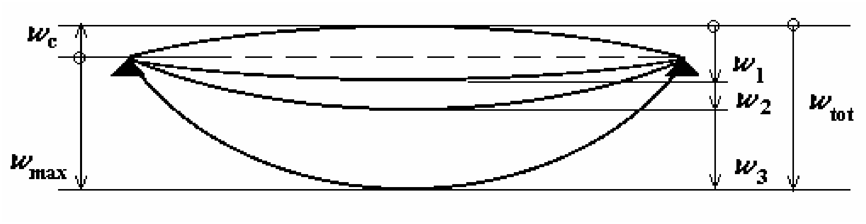

Figure A1.1 - Definitions of vertical deflections

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 expressions (6.14a) to (6.16b)
w_2	Long-term part of the deflection under permanent loads
w_3	Additional part of the deflection due to the variable actions of the relevant combination of actions according to expressions (6.14a) to (6.16b)
w_{tot}	Total deflection as sum of w_1 , w_2 , w_3
w_{max}	Remaining total deflection taking into account the precamber

(3) If the functioning or damage of the structure or to finishes, or to non-structural members (*e.g.* partition walls, claddings) is being considered, the verification for deflection should take account of those effects of permanent and variable actions that occur after the execution of the member or finish concerned.

NOTE Guidance on which expression (6.14a) to (6.16b) to use is given in 6.5.3 and EN 1992 to EN 1999.

(4) If the appearance of the structure is being considered, the quasi-permanent combination (expression 6.16b) should be used.

(5) If the comfort of the user, or the functioning of machinery are being considered, the verification should take account of the effects of the relevant variable actions.

(6) 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.

(7) Horizontal displacements are represented schematically in Figure A1.2.

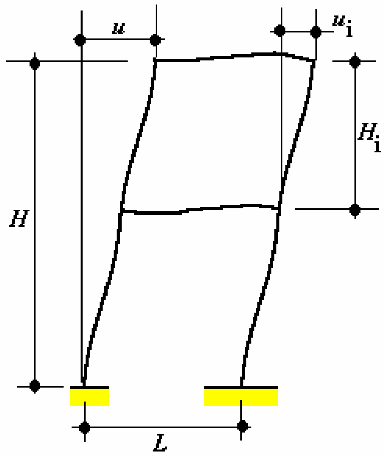

Figure A1.2 - Definition of horizontal displacements

Key :

u Overall horizontal displacement over the building height H

u_i Horizontal displacement over a storey height H_i

A1.4.4 Vibrations

(1) To achieve satisfactory vibration behaviour of buildings and their structural members under serviceability conditions, the following aspects, amongst others, should be considered :

a) the comfort of the user;
b) the functioning of the structure or its structural members (*e.g.* cracks in partitions, damage to cladding, sensitivity of building contents to vibrations).

Other aspects should be considered for each project and agreed with the client.

(2) For the serviceability limit state of a structure or a structural member not to be exceeded when subjected to vibrations, the natural frequency of vibrations of the structure or structural member should be kept above appropriate values which depend upon the function of the building and the source of the vibration, and agreed with the client and/or the relevant authority.

(3) If the natural frequency of vibrations 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 For further guidance, see EN 1991-1-1, EN 1991-1-4 and ISO 10137.

(4) Possible sources of vibration that should be considered include walking, synchronised movements of people, machinery, ground borne vibrations from traffic, and wind actions. These, and other sources, should be specified for each project and agreed with the client.

AC2 18) Modification to the Annexes

At the end of Annex A1 and before Annex B, add the following Annex A2: AC2

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Annex A2
(normative)
Application for bridges

National Annex for EN 1990 Annex A2

National choice is allowed in EN 1990 Annex A2 through the following clauses:

General clauses

Clause	Item
A2.1 (1) NOTE 3	Use of Table 2.1: Design working life
A2.2.1(2) NOTE 1	Combinations involving actions which are outside the scope of EN 1991
A2.2.6(1) NOTE 1	Values of ψ factors
A2.3.1(1)	Alteration of design values of actions for ultimate limit states
A2.3.1(5)	Choice of Approach 1, 2 or 3
A2.3.1(7)	Definition of forces due to ice pressure
A2.3.1(8)	Values of γ_P factors for prestressing actions where not specified in the relevant design Eurocodes
A2.3.1 Table A2.4(A) NOTES 1 and 2	Values of γ factors
A2.3.1 Table A2.4(B)	- NOTE 1: choice between 6.10 and 6.10a/b - NOTE 2: Values of γ and ξ factors - NOTE 4: Values of γ_{Sd}
A2.3.1 Table A2.4 (C)	Values of γ factors
A2.3.2(1)	Design values in Table A2.5 for accidental design situations, design values of accompanying variable actions and seismic design situations
A2.3.2 Table A2.5 NOTE	Design values of actions
A2.4.1(1) NOTE 1 (Table A2.6) NOTE 2	Alternative γ values for traffic actions for the serviceability limit state Infrequent combination of actions
A2.4.1(2)	Serviceability requirements and criteria for the calculation of deformations

Clauses specific for road bridges

Clause	Item
A2.2.2 (1)	Reference to the infrequent combination of actions
A2.2.2(3)	Combination rules for special vehicles
A2.2.2(4)	Combination rules for snow loads and traffic loads
A2.2.2(6)	Combination rules for wind and thermal actions
A2.2.6(1) NOTE 2	Values of $\psi_{1,infq}$ factors
A2.2.6(1) NOTE 3	Values of water forces

Clauses specific for footbridges

Clause	Item
A2.2.3(2)	Combination rules for wind and thermal actions
A2.2.3(3)	Combination rules for snow loads and traffic loads
A2.2.3(4)	Combination rules for footbridges protected from bad weather

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A2.4.3.2(1)	Comfort criteria for footbridges

Clauses specific for railway bridges

Clause	Item
A2.2.4(1)	Combination rules for snow loading on railway bridges
A2.2.4(4)	Maximum wind speed compatible with rail traffic
A2.4.4.1(1) NOTE 3	Deformation and vibration requirements for temporary railway bridges
A2.4.4.2.1(4)P	Peak values of deck acceleration for railway bridges and associated frequency range
A2.4.4.2.2 – Table A2.7 NOTE	Limiting values of deck twist for railway bridges
A2.4.4.2.2(3)P	Limiting values of the total deck twist for railway bridges
A2.4.4.2.3(1)	Vertical deformation of ballasted and non ballasted railway bridges
A2.4.4.2.3(2)	Limitations on the rotations of non ballasted bridge deck ends for railway bridges
A2.4.4.2.3(3)	Additional limits of angular rotations at the end of decks
A2.4.4.2.4(2) – Table A2.8 NOTE 3	Values of α_i and r_i factors
A2.4.4.2.4(3)	Minimum lateral frequency for railway bridges
A2.4.4.3.2(6)	Requirements for passenger comfort for temporary bridges

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A2.1 Field of application

AC2 *Text deleted* AC2

(1) This Annex A2 to EN 1990 gives rules and methods for establishing combinations of actions for serviceability and ultimate limit state verifications (except fatigue verifications) with the recommended design values of permanent, variable and accidental actions and ψ factors to be used in the design of road bridges, footbridges and railway bridges. It also applies to actions during execution. Methods and rules for verifications relating to some material-independent serviceability limit states are also given.

NOTE 1 Symbols, notations, Load Models and groups of loads are those used or defined in the relevant section of EN 1991-2.

NOTE 2 Symbols, notations and models of construction loads are those defined in EN 1991-1-6.

NOTE 3 Guidance may be given in the National Annex with regard to the use of Table 2.1 (design working life).

NOTE 4 Most of the combination rules defined in clauses A2.2.2 to A2.2.5 are simplifications intended to avoid needlessly complicated calculations. They may be changed in the National Annex or for the individual project as described in A2.2.1 to A2.2.5.

NOTE 5 This Annex A2 to EN 1990 does not include rules for the determination of actions on structural bearings (forces and moments) and associated movements of bearings or give rules for the analysis of bridges involving ground-structure interaction that may depend on movements or deformations of structural bearings.

(2) The rules given in this Annex A2 to EN 1990 may not be sufficient for:

- bridges that are not covered by EN 1991-2 (for example bridges under an airport runway, mechanically-moveable bridges, roofed bridges, bridges carrying water, etc.),
- bridges carrying both road and rail traffic, and
- other civil engineering structures carrying traffic loads (for example backfill behind a retaining wall). A1

AC2 *Text deleted* AC2

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A2.2 Combinations of actions

A2.2.1 General

(1) Effects of actions that cannot occur simultaneously due to physical or functional reasons need not be considered together in combinations of actions.

(2) Combinations involving actions which are outside the scope of EN 1991 (e.g. due to mining subsidence, particular wind effects, water, floating debris, flooding, mud slides, avalanches, fire and ice pressure) should be defined in accordance with EN 1990, 1.1(3).

NOTE 1 Combinations involving actions that are outside the scope of EN 1991 may be defined either in the National Annex or for the individual project.

NOTE 2 For seismic actions, see EN 1998.

NOTE 3 For water actions exerted by currents and debris effects, see also EN 1991-1-6.

(3) The combinations of actions given in expressions 6.9a to 6.12b should be used when verifying ultimate limit states.

NOTE Expressions 6.9a to 6.12b are not for the verification of the limit states due to fatigue. For fatigue verifications, see EN 1991 to EN 1999.

(4) The combinations of actions given in expressions 6.14a to 6.16b should be used when verifying serviceability limit states. Additional rules are given in A2.4 for verifications regarding deformations and vibrations.

(5) Where relevant, variable traffic actions should be taken into account simultaneously with each other in accordance with the relevant sections of EN 1991-2.

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

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

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

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

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

(10) Snow loads and wind actions need not be considered simultaneously with loads arising from construction activity Q_{ca} (i.e. loads due to working personnel).

NOTE For an individual project it may be necessary to agree the requirements for snow loads and wind actions to be taken into account simultaneously with other construction loads (e.g. actions due to heavy equipment or cranes) during some transient design situations. See also EN 1991-1-3, 1-4 and 1-6.

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

(12) The inclusion of prestressing actions in combinations of actions should be in accordance with A2.3.1(8) and EN 1992 to EN 1999.

(13) Effects of uneven settlements should be taken into account if they are considered significant compared to the effects from direct actions.

NOTE The individual project may specify limits on total settlement and differential settlement.

(14) Where the structure is very sensitive to uneven settlements, uncertainty in the assessment of these settlements should be taken into account.

(15) Uneven settlements on the structure due to soil subsidence 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. G_{set} 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, $d_{set,i}$ (i is the number of the individual foundation or part of foundation).

NOTE 1 Settlements are mainly caused by permanent loads and backfill. Variable actions may have to be taken into account for some individual projects.

NOTE 2 Settlements vary monotonically (in the same direction) with time and need to be taken into account 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.

(16) The differences of settlements of individual foundations or parts of foundations, $d_{set,i}$, should be taken into account as best-estimate predicted values in accordance with EN 1997 with due regard for the construction process of the structure.

NOTE Methods for the assessment of settlements are given in EN 1997

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

- the best-estimate predicted values $d_{set,i}$ 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 $d_{set,i} \pm \Delta d_{set,i}$, where $\Delta d_{set,i}$ takes account of uncertainties attached to the assessment of settlements.

A2.2.2 Combination rules for road bridges

(1) The infrequent values of variable actions may be used for certain serviceability limit states of concrete bridges.

NOTE The National Annex may refer to the infrequent combination of actions. The expression of this combination of actions is:

$$E_d = E\left\{G_{k,j}\ ;P\ ;\psi_{1,\mathrm{infq}}Q_{k,1}\ ;\psi_{1,i}Q_{k,i}\right\} \quad j \geq 1\ ; i > 1 \qquad \text{(A2.1a)}$$

in which the combination of actions in brackets { } may be expressed as:

$$\sum_{j\geq 1} G_{k,j}\text{"+"}P\text{"+"}\psi_{1,\mathrm{infq}}Q_{\mathrm{k},1}\text{"+"}\sum_{i>1}\psi_{1,i}Q_{\mathrm{k,i}} \qquad \text{(A2.1b)}$$

(2) Load Model 2 (or associated group of loads gr1b) and the concentrated load Q_{fwk} (see 5.3.2.2 in EN 1991-2) on footways need not be combined with any other variable non traffic action.

(3) Neither snow loads nor wind actions need 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.

NOTE The combination rules for special vehicles (see EN 1991-2, Annex A, Informative) with normal traffic (covered by LM1 and LM2) and other variable actions may be referenced as appropriate in the National Annex or agreed for the individual project.

(4) Snow loads need 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.

NOTE Geographical areas where snow loads may have to be combined with groups of loads gr1a and gr1b in combinations of actions may be specified in the National Annex.

(5) No wind action greater than the smaller of F_W^* and $\psi_0 F_{Wk}$ should be combined with Load Model 1 or with the associated group of loads gr1a.

NOTE For wind actions, see EN1991-1-4.

(6) Wind actions and thermal actions need not be taken into account simultaneously unless otherwise specified for local climatic conditions.

NOTE Depending upon the local climatic conditions a different simultaneity rule for wind and thermal actions may be defined either in the National Annex or for the individual project.

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A2.2.3 Combination rules for footbridges

(1) The concentrated load Q_{fwk} need not be combined with any other variable actions that are not due to traffic.

(2) Wind actions and thermal actions need not be taken into account simultaneously unless otherwise specified for local climatic conditions.

NOTE Depending upon the local climatic conditions a different simultaneity rule for wind and thermal actions may be defined either in the National Annex or for the individual project.

(3) Snow loads need not be combined with groups of loads gr1 and gr2 for footbridges unless otherwise specified for particular geographical areas and certain types of footbridges.

NOTE Geographical areas, and certain types of footbridges, where snow loads may have to be combined with groups of loads gr1 and gr2 in combinations of actions may be specified in the National Annex.

(4) For footbridges on which pedestrian and cycle traffic is fully protected from all types of bad weather, specific combinations of actions should be defined.

NOTE Such combinations of actions may be given as appropriate in the National Annex or agreed for the individual project. Combinations of actions similar to those for buildings (see Annex A1), the imposed loads being replaced by the relevant group of loads and the ψ factors for traffic actions being in accordance with Table A2.2, are recommended.

A2.2.4 Combination rules for railway bridges

(1) Snow loads need 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.

NOTE Geographical areas, and certain types of railway bridges, where snow loads may have to be taken into account in combinations of actions are to be specified in the National Annex.

(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;
- AC2 vertical rail traffic actions excluding dynamic factor and lateral rail traffic actions from the "unloaded train" defined in EN 1991-2 (6.3.4 and 6.5) with wind forces for checking stability." AC2

(3) Wind action need not be combined with:
- groups of loads gr 13 or gr 23;
- groups of loads gr 16, gr 17, gr 26, gr 27 and Load Model SW/2 (see EN 1991-2, 6.3.3).

(4) No wind action greater than the smaller of F_W^{**} and $\psi_0 F_{Wk}$ should be combined with traffic actions.

NOTE The National Annex may give the limits of the maximum wind speed(s) compatible with rail traffic for determining F_W^{**}. See also EN 1991-1-4.

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(5) Actions due to aerodynamic effects of rail traffic (see EN 1991-2, 6.6) and wind actions should be combined together. Each action should be considered individually as a leading variable action.

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

(7) 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.

A2.2.5 Combinations of actions for accidental (non seismic) 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 need be taken into account in the same combination.

(2) For an accidental design situation concerning impact from traffic (road or rail traffic) under the bridge, the loads due to the traffic on the bridge should be taken into account in the combinations as accompanying actions with their frequent value.

NOTE 1 For actions due to impact from traffic, see EN 1991-1-7.

NOTE 2 Additional combinations of actions for other accidental design situations (e.g. combination of road or rail traffic actions with avalanche, flood or scour effects) may be agreed for the individual project.

NOTE 3 Also see 1) in Table A2.1.

(3) 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.

NOTE 1 For actions due to impact from traffic, see EN 1991-1-7.

NOTE 2 Actions for accidental design situations due to impact from rail traffic running on the bridge including derailment actions are specified in EN1991-2, 6.7.1.

(4) Accidental design situations involving ship collisions against bridges should be identified.

NOTE For ship impact, see EN1991-1-7. Additional requirements may be specified for the individual project.

A2.2.6 Values of ψ factors

(1) Values of ψ factors should be specified.

NOTE 1 The ψ values may be set by the National Annex. Recommended values of ψ factors for the groups of traffic loads and the more common other actions are given in:

Table A2.1 for road bridges,
Table A2.2 for footbridges, and
Table A2.3 for railway bridges, both for groups of loads and individual components of traffic actions.

A1⟩ **Table A2.1 – Recommended values of ψ factors for road bridges**

Action	Symbol		ψ_0	ψ_1	ψ_2
Traffic loads (see EN 1991-2, Table 4.4)	gr1a (LM1+pedestrian or cycle-track loads) 1)	TS	0,75	0,75	0
		UDL	0,40	0,40	0
		Pedestrian+cycle-track loads 2)	0,40	0,40	0
	gr1b (Single axle)		0	0,75	0
	gr2 (Horizontal forces)		0	0	0
	gr3 (Pedestrian loads)		0	AC1⟩0,40⟨AC1	0
	gr4 (LM4 – Crowd loading))		0	AC2⟩ - ⟨AC2	0
	gr5 (LM3 – Special vehicles))		0	AC2⟩ - ⟨AC2	0
Wind forces	F_{Wk} - Persistent design situations		0,6	0,2	0
	F_{Wk} - Execution		0,8	-	0
	F_W^*		1,0	-	-
Thermal actions	T_k		0,6 3)	0,6	0,5
Snow loads	$Q_{Sn,k}$ (during execution)		0,8	-	-
Construction loads	Q_c		1,0	-	1,0

1) The recommended values of ψ_0, ψ_1 and ψ_2 for gr1a and gr1b are given for road traffic corresponding to adjusting 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, 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.

2) The combination value of the pedestrian and cycle-track load, mentioned in Table 4.4a of EN 1991-2, is a "reduced" value. ψ_0 and ψ_1 factors are applicable to this value.

3) The recommended ψ_0 value for thermal actions may in most cases be reduced to 0 for ultimate limit states EQU, STR and GEO. See also the design Eurocodes.

NOTE 2 When the National Annex refers to the infrequent combination of actions for some serviceability limit states of concrete bridges, the National Annex may define the values of $\psi_{1,infq}$. The recommended values of $\psi_{1,infq}$ are :

– 0,80 for gr1a (LM1), gr1b (LM2), gr3 (pedestrian loads), gr4 (LM4, crowd loading) and T (thermal actions);
– 0,60 for F_{Wk} in persistent design situations;
– 1,00 in other cases (i.e. the characteristic value is used as the infrequent value).

NOTE 3 The characteristic values of wind actions and snow loads during execution are defined in EN 1991-1-6. Where relevant, representative values of water forces (F_{wa}) may be defined in the National Annex or for the individual project.

Table A2.2 – Recommended values of ψ factors for footbridges

Action	Symbol	ψ_0	ψ_1	ψ_2
Traffic loads	gr1	0,40	0,40	0
	Q_{fwk}	0	0	0
	gr2	0	0	0
Wind forces	F_{Wk}	0,3	0,2	0
Thermal actions	T_k	0,6 1)	0,6	0,5
Snow loads	$Q_{Sn,k}$ (during execution)	0,8	-	0
Construction loads	Q_c	1,0	-	1,0

1) The recommended ψ_0 value for thermal actions may in most cases be reduced to 0 for ultimate limit states EQU, STR and GEO. See also the design Eurocodes.

NOTE 4 For footbridges, the infrequent value of variable actions is not relevant. ⟨A1

Table A2.3 – Recommended values of ψ factors for railway bridges

Actions			ψ_0	ψ_1	ψ_2[4)]
Individual components of traffic actions[5)]	LM 71		0,80	1)	0
	SW/0		0,80	1)	0
	SW/2		0	1,00	0
	Unloaded train		1,00	–	–
	HSLM		1,00	1,00	0
	Traction and braking Centrifugal forces 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
			0,80	1)	0
	Horizontal earth pressure due to traffic load surcharge		0,80	0,50	0
	Aerodynamic effects				
Main traffic actions (groups of loads)	gr11 (LM71 + SW/0)	Max. vertical 1 with max. longitudinal			
	gr12 (LM71 + SW/0)	Max. vertical 2 with max. transverse			
	gr13 (Braking/traction)	Max. longitudinal			
	gr14 (Centrifugal/nosing)	Max. lateral	0,80	0,80	0
	gr15 (Unloaded train)	Lateral stability with “unloaded train”			
	gr16 (SW/2)	SW/2 with max. longitudinal			
	gr17 (SW/2)	SW/2 with max. transverse			
	gr21 (LM71 + SW/0)	Max. vertical 1 with max. longitudinal			
	gr22 (LM71 + SW/0)	Max. vertical 2 with max transverse			
	gr23 (Braking/traction)	Max. longitudinal	0,80	0,70	0
	gr24 (Centrifugal/nosing)	Max. lateral			
	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		0,80	0,50	0
	General maintenance loading for non public footpaths		0,80	0,50	0
Wind forces [2)]	F_{Wk}		0,75	0,50	0
	F_W^{**}		1,00	0	0
Table continued on next page					

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Table continued from previous page				
Thermal actions 3)	T_k	0,60	0,60	0,50
Snow loads	$Q_{Sn,k}$ (during execution)	0,8	-	0
Construction loads	Q_c	1,0	-	1,0

1) 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.
2) When wind forces act simultaneously with traffic actions, the wind force $\psi_0 F_{Wk}$ should be taken as no greater than F_W^{**} (see EN 1991-1-4). See A2.2.4(4).
3) See EN 1991-1-5.
4) If deformation is being considered for Persistent and Transient design situations, ψ_2 should be taken equal to 1,00 for rail traffic actions. For seismic design situations, see Table A2.5.
5) Minimum coexistent favourable vertical load with individual components of rail traffic actions (e.g. centrifugal, traction or braking) is 0,5LM71, etc.

NOTE 5 For specific design situations (e.g. calculation of bridge camber for aesthetics and drainage consideration, calculation of clearance, etc.) the requirements for the combinations of actions to be used may be defined for the individual project.

NOTE 6 For railway bridges, the infrequent value of variable actions is not relevant.

AC2 (2) For railway bridges, a unique ψ value should be applied to one group of loads as defined in EN 1991-2, and taken as equal to the ψ value applicable to the leading component of the group. AC2

(3) For railway bridges, where groups of loads are used the groups of loads defined in EN 1991-2, 6.8.2, Table 6.11 should be used.

(4) Where relevant, combinations of individual traffic actions (including individual components) should be taken into account for railway bridges. Individual traffic actions may also have to be taken into account, for example for the design of bearings, for the assessment of maximum lateral and minimum vertical traffic loading, bearing restraints, maximum overturning effects on abutments (especially for continuous bridges), etc., see Table A2.3. AC2

NOTE Individual traffic actions may also have to be taken into account, for example for the design of bearings, for the assessment of maximum lateral and minimum vertical traffic loading, bearing restraints, maximum overturning effects on abutments (especially for continuous bridges), etc., see Table A2.3.

A2.3 Ultimate limit states

NOTE Verification for fatigue excluded.

A2.3.1 Design values of actions in persistent and transient design situations

(1) The design values of actions for ultimate limit states in the persistent and transient design situations (expressions 6.9a to 6.10b) should be in accordance with Tables A2.4(A) to (C).

NOTE The values in Tables A2.4(A) to (C) may be changed in the National Annex (e.g. for different reliability levels see Section 2 and Annex B).

(2) In applying Tables A2.4(A) to A2.4(C) in cases when the limit state is very sensitive to variations in the magnitude of permanent actions, the upper and lower characteristic values of these actions should be taken according to 4.1.2(2)P.

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(3) Static equilibrium (EQU, see 6.4.1 and 6.4.2(2)) for bridges should be verified using the design values of actions in Table A2.4(A).

(4) Design of structural members (STR, see 6.4.1) not involving geotechnical actions should be verified using the design values of actions in Table A2.4(B).

(5) Design of structural members (footings, piles, piers, side walls, wing walls, flank walls and front walls of abutments, ballast retention walls, etc.) (STR) involving geotechnical actions and the resistance of the ground (GEO, see 6.4.1) should be verified using one only of the following three approaches supplemented, for geotechnical actions and resistances, by EN 1997:

– Approach 1: Applying in separate calculations design values from Table A2.4(C) and Table A2.4(B) to the geotechnical actions as well as the actions on/from the structure;

– Approach 2: Applying design values of actions from Table A2.4(B) to the geotechnical actions as well as the actions on/from the structure;

– Approach 3: Applying design values of actions from Table A2.4(C) to the geotechnical actions and, simultaneously, applying design values of actions from Table A2.4(B) to the actions on/from the structure.

NOTE The choice of approach 1, 2 or 3 is given in the National Annex.

(6) Site stability (e.g. the stability of a slope supporting a bridge pier) should be verified in accordance with EN 1997.

(7) Hydraulic (HYD) and buoyancy (UPL) failure (e.g. in the bottom of an excavation for a bridge foundation), if relevant, should be verified in accordance with EN 1997.

NOTE For water actions and debris effects, see EN 1991-1-6. General and local scour depths may have to be assessed for the individual project. Requirements for taking account of forces due to ice pressure on bridge piers, etc., may be defined as appropriate in the National Annex or for the individual project.

(8) The γ_P values to be used for prestressing actions should be specified for the relevant representative values of these actions in accordance with EN 1990 to EN 1999.

NOTE In the cases where γ_P values are not provided in the relevant design Eurocodes, these values may be defined as appropriate in the National Annex or for the individual project. They depend, *inter alia*, on:
- the type of prestress (see the Note in 4.1.2(6))
- the classification of prestress as a direct or an indirect action (see 1.5.3.1)
- the type of structural analysis (see 1.5.6)
- the unfavourable or favourable character of the prestressing action and the leading or accompanying character of prestressing in the combination.

See also EN1991-1-6 during execution.

AC2 Table A2.4(A) - Design values of actions (EQU) (Set A)

Persistent and transient design situation	Permanent actions		Prestress	Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable			Main (if any)	Others
(Eq. 6.10)	$\gamma_{G,j,sup} G_{k,j,sup}$	$\gamma_{G,j,inf} G_{k,j,inf}$	$\gamma_P P$	$\gamma_{Q,1}\ Q_{k,1}$		$\gamma_{Q,i} \psi_{0,i} Q_{k,i}$

(*) Variable actions are those considered in Tables A2.1 to A2.3.

NOTE 1 The γ values for the persistent and transient design situations may be set by the National Annex.

For persistent design situations, the recommended set of values for γ are:
$\gamma_{G,sup} = 1,05$
$\gamma_{G,inf} = 0,95^{(1)}$
$\gamma_Q = 1,35$ for road and pedestrian traffic actions, where unfavourable (0 where favourable)
$\gamma_Q = 1,45$ for rail traffic actions, where unfavourable (0 where favourable)
$\gamma_Q = 1,50$ for all other variable actions for persistent design situations, where unfavourable (0 where favourable).
γ_P = recommended values defined in the relevant design Eurocode.

For transient design situations during which there is a risk of loss of static equilibrium, $Q_{k,1}$ represents the dominant destabilising variable action and $Q_{k,i}$ represents the relevant accompanying destabilising variable actions.

During execution, if the construction process is adequately controlled, the recommended set of values for γ are:
$\gamma_{G,sup} = 1,05$
$\gamma_{G,inf} = 0,95^{(1)}$
$\gamma_Q = 1,35$ for construction loads where unfavourable (0 where favourable)
$\gamma_Q = 1,50$ for all other variable actions, where unfavourable (0 where favourable)

(1) Where a counterweight is used, the variability of its characteristics may be taken into account, for example, by one or both of the following recommended rules:
– applying a partial factor $\gamma_{G,\text{inf}} = 0,8$ 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. For steel bridges during launching, the variation of the counterweight position is often taken equal to ± 1 m.

NOTE 2 For the verification of uplift of bearings of continuous bridges or in cases where the verification of static equilibrium also involves the resistance of structural elements (for example where the loss of static equilibrium is prevented by stabilising systems or devices, e.g. anchors, stays or auxiliary columns), as an alternative to two separate verifications based on Tables A2.4(A) and A2.4(B), a combined verification, based on Table A2.4(A), may be adopted. The National Annex may set the γ values. The following values of γ are recommended:
$\gamma_{G,sup} = 1,35$
$\gamma_{G,inf} = 1,25$
$\gamma_Q = 1,35$ for road and pedestrian traffic actions, where unfavourable (0 where favourable)
$\gamma_Q = 1,45$ for rail traffic actions, where unfavourable (0 where favourable)
$\gamma_Q = 1,50$ for all other variable actions for persistent design situations, where unfavourable (0 where favourable)
$\gamma_Q = 1,35$ for all other variable actions, where unfavourable (0 where favourable)
provided that applying $\gamma_{G,inf} = 1,00$ both to the favourable part and to the unfavourable part of permanent actions does not give a more unfavourable effect.
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AC2 Table A2.4(B) - Design values of actions (STR/GEO) (Set B)

Persistent and transient design situation	Permanent actions		Prestress	Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable			Main (if any)	Others
(Eq. 6.10)	$\gamma_{G,j,sup}G_{k,j,sup}$	$\gamma_{G,j,inf}G_{k,j,inf}$	$\gamma_P P$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

Persistent and transient design situation	Permanent actions		Prestress	Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable			Main (if any)	Others
(Eq. 6.10a)	$\gamma_{G,j,sup}G_{k,j,sup}$	$\gamma_{G,j,inf}G_{k,j,inf}$	$\gamma_P P$		$\gamma_{Q,1}\psi_{0,1}Q_{k,1}$	$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$
(Eq. 6.10b)	$\xi\gamma_{G,j,sup}G_{k,j,sup}$	$\gamma_{G,j,inf}G_{k,j,inf}$	$\gamma_P P$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

(*) Variable actions are those considered in Tables A2.1 to A2.3.

NOTE 1 The choice between 6.10, or 6.10a and 6.10b will be in the National Annex. In the case of 6.10a and 6.10b, the National Annex may in addition modify 6.10a to include permanent actions only.

NOTE 2 The γ and ξ values may be set by the National Annex. The following values for γ and ξ are recommended when using expressions 6.10, or 6.10a and 6.10b:

$\gamma_{G,sup} = 1,35$[1)]

$\gamma_{G,inf} = 1,00$

$\gamma_Q = 1,35$ when Q represents unfavourable actions due to road or pedestrian traffic (0 when favourable)

$\gamma_Q = 1,45$ when Q represents unfavourable actions due to rail traffic, for groups of loads 11 to 31 (except 16, 17, 26[3)] and 27[3)]), load models LM71, SW/0 and HSLM and real trains, when considered as individual leading traffic actions (0 when favourable)

$\gamma_Q = 1,20$ when Q represents unfavourable actions due to rail traffic, for groups of loads 16 and 17 and SW/2 (0 when favourable)

$\gamma_Q = 1,50$ for other traffic actions and other variable actions [2)]

$\xi = 0,85$ (so that $\xi\gamma_{G,sup} = 0,85 \times 1,35 \cong 1,15$).

$\gamma_{Gset} = 1,20$ in the case of a linear elastic analysis, and $\gamma_{Gset} = 1,35$ in the case of a non linear analysis, for design situations where actions due to uneven settlements may have unfavourable effects. For design situations where actions due to uneven settlements may have favourable effects, these actions are not to be taken into account.

See also EN 1991 to EN 1999 for γ values to be used for imposed deformations.

γ_P = recommended values defined in the relevant design Eurocode.

[1)] This value covers: self-weight of structural and non structural elements, ballast, soil, ground water and free water, removable loads, etc.

[2)] This value covers: variable horizontal earth pressure from soil, ground water, free water and ballast, traffic load surcharge earth pressure, traffic aerodynamic actions, wind and thermal actions, etc.

[3)] For rail traffic actions for groups of loads 26 and 27 $\gamma_Q = 1,20$ may be applied to individual components of traffic actions associated with SW/2 and $\gamma_Q = 1,45$ may be applied to individual components of traffic actions associated with load models LM71, SW/0 and HSLM, etc.

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NOTE 3 The characteristic values of all permanent actions from one source are multiplied by $\gamma_{G,sup}$ if the total resulting action effect is unfavourable and $\gamma_{G,inf}$ if the total resulting action effect is favourable. For example, all actions originating from the self-weight of the structure may be considered as coming from one source; this also applies if different materials are involved. See however A2.3.1(2).
NOTE 4 For particular verifications, the values for γ_G and γ_Q may be subdivided into γ_g and γ_q and the model uncertainty factor γ_{Sd}. A value of γ_{Sd} in the range 1,0–1,15 may be used in most common cases and may be modified in the National Annex.
NOTE 5 Where actions due to water are not covered by EN 1997 (e.g. flowing water), the combinations of actions to be used may be specified for the individual project.

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AC2) Table A2.4(C) - Design values of actions (STR/GEO) (Set C)

Persistent and transient design situation	Permanent actions		Prestress	Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable			Main (if any)	Others
(Eq. 6.10)	$\gamma_{G,j,sup} G_{k,j,sup}$	$\gamma_{G,j,inf} G_{k,j,inf}$	$\gamma_P P$	$\gamma_{Q,1}\ Q_{k,1}$		$\gamma_{Q,i} \psi_{0,i} Q_{k,i}$
(*) Variable actions are those considered in Tables A2.1 to A2.3						

NOTE The γ values may be set by the National Annex. The recommended set of values for γ are:
$\gamma_{G,sup} = 1,00$
$\gamma_{G,inf} = 1,00$
$\gamma_{Gset} = 1,00$
$\gamma_Q = 1,15$ for road and pedestrian traffic actions where unfavourable (0 where favourable)
$\gamma_Q = 1,25$ for rail traffic actions where unfavourable (0 where favourable)
$\gamma_Q = 1,30$ for the variable part of horizontal earth pressure from soil, ground water, free water and ballast, for traffic load surcharge horizontal earth pressure, where unfavourable (0 where favourable)
$\gamma_Q = 1,30$ for all other variable actions where unfavourable (0 where favourable)
$\gamma_{Gset} = 1,00$ in the case of linear elastic or non linear analysis, for design situations where actions due to uneven settlements may have unfavourable effects. For design situations where actions due to uneven settlements may have favourable effects, these actions are not to be taken into account.
γ_P = recommended values defined in the relevant design Eurocode.

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A1) A2.3.2 Design values of actions in the accidental and seismic design situations

(1) The partial factors for actions for the ultimate limit states in the accidental and seismic design situations (expressions 6.11a to 6.12b) are given in Table A2.5. ψ values are given in Tables A2.1 to A2.3.

NOTE For the seismic design situation see also EN 1998.

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Table A2.5 - Design values of actions for use in accidental and seismic combinations of actions

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Design situation	Permanent actions		Prestress	Accidental or seismic action	Accompanying variable actions (**)	
	Unfavourable	Favourable			Main (if any)	Others
Accidental(*) (Eq. 6.11a/b)	$G_{k,j,sup}$	$G_{k,j,inf}$	P	A_d	$\psi_{1,1}Q_{k,1}$ or $\psi_{2,1}Q_{k,1}$	$\psi_{2,i}\ Q_{k,i}$
Seismic(***) (Eq. 6.12a/b)	$G_{k,j,sup}$	$G_{k,j,inf}$	P	$A_{Ed} = \gamma_I A_{Ek}$		$\psi_{2,i}\ Q_{k,i}$

(*) In the case of accidental design situations, the main variable action may be taken with its frequent or, as in seismic combinations of actions, its quasi-permanent values. The choice will be in the National Annex, depending on the accidental action under consideration.

(**) Variable actions are those considered in Tables A2.1 to A2.3.

(***) The National Annex or the individual project may specify particular seismic design situations. For railway bridges only one track need be loaded and load model SW/2 may be neglected.

NOTE The design values in this Table A2.5 may be changed in the National Annex. The recommended values are γ= 1,0 for all non seismic actions. ⟨AC2

A1⟩ (2) Where, in special cases, one or several variable actions need to be considered simultaneously with the accidental action, their representative values should be defined.

NOTE As an example, in the case of bridges built by the cantilevered method, some construction loads may be considered as simultaneous with the action corresponding to the accidental fall of a prefabricated unit. The relevant representative values may be defined for the individual project.

(3) For execution phases during which there is a risk of loss of static equilibrium, the combination of actions should be as follows:

$$\sum_{j\geq1} G_{\mathrm{kj,sup}}\text{"+"}\sum_{j\geq1} G_{\mathrm{kj,inf}}\text{"+"}P\text{"+"}A_{\mathrm{d}}\text{"+"}\psi_2 Q_{\mathrm{c,k}} \qquad \text{(A2.2)}$$

where:

$Q_{c,k}$ is the characteristic value of construction loads as defined in EN 1991-1-6 (i.e. the characteristic value of the relevant combination of groups Q_{ca}, Q_{cb}, Q_{cc}, Q_{cd}, Q_{ce} and Q_{cf}).

A2.4 Serviceability and other specific limit states

A2.4.1 General

(1) For serviceability limit states the design values of actions should be taken from Table A2.6 except if differently specified in EN1991 to EN1999.

NOTE 1 γ factors for traffic and other actions for the serviceability limit state may be defined in the National Annex. The recommended design values are given in Table A2.6, with all γ factors being taken as 1,0.

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Table A2.6 - Design values of actions for use in the combination of actions

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Combination	Permanent actions G_d		Prestress	Variable actions Q_d	
	Unfavourable	Favourable		Leading	Others
Characteristic	$G_{k,j,sup}$	$G_{k,j,inf}$	P	$Q_{k,1}$	$\psi_{0,i}Q_{k,i}$
Frequent	$G_{k,j,sup}$	$G_{k,j,inf}$	P	$\psi_{1,1}Q_{k,1}$	$\psi_{2,i}Q_{k,i}$
Quasi-permanent	$G_{k,j,sup}$	$G_{k,j,inf}$	P	$\psi_{2,1}Q_{k,1}$	$\psi_{2,i}Q_{k,i}$

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A1) NOTE 2 The National Annex may also refer to the infrequent combination of actions.

(2) The serviceability criteria should be defined in relation to the serviceability requirements in accordance with 3.4 and EN 1992 to EN 1999. Deformations should be calculated in accordance with EN 1991 to EN 1999 by using the appropriate combinations of actions according to expressions (6.14a) to (6.16b) (see Table A2.6) taking into account the serviceability requirements and the distinction between reversible and irreversible limit states.

NOTE Serviceability requirements and criteria may be defined as appropriate in the National Annex or for the individual project.

A2.4.2 Serviceability criteria regarding deformation and vibration for road bridges

(1) Where relevant, requirements and criteria should be defined for road bridges concerning:
- uplift of the bridge deck at supports,
- damage to structural bearings.

NOTE Uplift at the end of a deck can jeopardise traffic safety and damage structural and non structural elements. Uplift may be avoided by using a higher safety level than usually accepted for serviceability limit states.

(2) Serviceability limit states during execution should be defined in accordance with EN 1990 to EN 1999

(3) Requirements and criteria should be defined for road bridges concerning deformations and vibrations, where relevant.

NOTE 1 The verification of serviceability limit states concerning deformation and vibration needs to be considered only in exceptional cases for road bridges. The frequent combination of actions is recommended for the assessment of deformation.

NOTE 2 Vibrations of road bridges may 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 may have to be considered. Fatigue may also have to be taken into account.

A2.4.3 Verifications concerning vibration for footbridges due to pedestrian traffic

NOTE For vibrations due to wind actions, see EN 1991-1-4.

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A2.4.3.1 Design situations and associated traffic assumptions

(1) The design situations (see 3.2) should be selected depending on the pedestrian traffic to be admitted on the individual footbridge during its design working life.

NOTE The design situations may take into account the way the traffic will be authorised, regulated and controlled, depending on the individual project.

(2) Depending on the deck area or the part of the deck area under consideration, the presence of a group of about 8 to 15 persons walking normally should be taken into account for design situations considered as persistent design situations.

(3) Depending on the deck area or the part of the deck area under consideration, other traffic categories, associated with design situations which may be persistent, transient or accidental, should be specified when relevant, including:
– the presence of streams of pedestrians (significantly more than 15 persons),
– occasional festive or choreographic events.

NOTE 1 These traffic categories and the relevant design situations may have to be agreed for the individual project, not only for bridges in highly populated urban areas, but also in the vicinity of railway and bus stations, schools, or any other places where crowds may congregate, or any important building with public admittance.

NOTE 2 The definition of design situations corresponding to occasional festive or choreographic events depends on the expected degree of control of them by a responsible owner or authority. No verification rule is provided in the present clause and special studies may need to be considered. Some information on the relevant design criteria may be found in the appropriate literature.

A2.4.3.2 Pedestrian comfort criteria (for serviceability)

(1) The comfort criteria should be defined in terms of maximum acceptable acceleration of any part of the deck.

NOTE The criteria may be defined as appropriate in the National Annex or for the individual project. The following accelerations (m/s^2) are the recommended maximum values for any part of the deck:
i) 0,7 for vertical vibrations,
ii) 0,2 for horizontal vibrations due to normal use,
iii) 0,4 for exceptional crowd conditions.

(2) A verification of the comfort criteria should be performed if the fundamental frequency of the deck is less than:
- 5 Hz for vertical vibrations,
- 2,5 Hz for horizontal (lateral) and torsional vibrations.

NOTE The data used in the calculations, and therefore the results, are subject to very high uncertainties. When the comfort criteria are not satisfied with a significant margin, it may be necessary to make provision in the design for the possible installation of dampers in the structure after its completion. In such cases the designer should consider and identify any requirements for commissioning tests.

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A2.4.4 Verifications regarding deformations and vibrations for railway bridges

A2.4.4.1 General

(1) This clause A2.4.4 gives the limits of deformation and vibration to be taken into account for the design of new railway bridges.

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 A2.4.4.1(2)P.

NOTE 3 The National Annex may specify limits of deformation and vibration to be taken into account for the design of temporary railway bridges. The National Annex may give special requirements for temporary bridges depending upon the conditions in which they are used (e.g. special requirements for skew bridges).

(2)P 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 A2.4.4.2.1),
– vertical deflection of the deck throughout each span (to ensure acceptable vertical track radii and generally robust structures – see A2.4.4.2.3(3)),
– unrestrained uplift at the bearings (to avoid premature bearing failure),
– vertical deflection of the end of the deck beyond bearings (to avoid destabilising the track, limit uplift forces on rail fastening systems and limit additional rail stresses – see A2.4.4.2.3(1) and EN1991-2, 6.5.4.5.2),
– twist of the deck measured along the centre line of each track on the approaches to a bridge and across a bridge (to minimise the risk of train derailment – see A2.4.4.2.2),

NOTE A2.4.4.2.2 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 EN 1991-2, 6.5.4), limit uplift forces on rail fastening systems and limit angular discontinuity at expansion devices and switch blades – see A2.4.4.2.3(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 minimise disturbance to track ballast and adjacent track formation – see EN 1991-2, 6.5.4.5.2),
– horizontal transverse deflection (to ensure acceptable horizontal track radii – see A2.4.4.2.4, Table A2.8),
– horizontal rotation of a deck about a vertical axis at ends of a deck (to ensure acceptable horizontal track geometry and passenger comfort – see A2.4.4.2.4, Table A2.8),
– 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 A2.4.4.2.4(3). A1

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NOTE There are other implicit stiffness criteria in the limits of bridge natural frequency given in EN 1991-2, 6.4.4 and when determining dynamic factors for real trains in accordance with EN 1991-2, 6.4.6.4 and EN1991-2 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 A2.4.4.3.

(4) The limits given in A2.4.4.2 and A2.4.4.3 take into account the mitigating effects of track maintenance (for example to overcome the effects of the settlement of foundations, creep, etc.).

A2.4.4.2 Criteria for traffic safety

A2.4.4.2.1 Vertical acceleration of the deck

(1)P To ensure traffic safety, where a dynamic analysis is necessary, the verification of maximum peak deck acceleration due to rail traffic actions shall be regarded as a traffic safety requirement checked at the serviceability limit state for the prevention of track instability.

(2) The requirements for determining whether a dynamic analysis is necessary are given in EN 1991-2, 6.4.4.

(3)P Where a dynamic analysis is necessary, it shall comply with the requirements given in EN 1991-2, 6.4.6.

NOTE Generally only characteristic rail traffic actions in accordance with EN1991-2, 6.4.6.1 need to be considered.

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

i) γ_{bt} for ballasted track;
ii) γ_{df} for direct fastened tracks with track and structural elements designed for high speed traffic

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

i) 30 Hz;
ii) 1,5 times the frequency of the fundamental mode of vibration of the member being considered;
iii) the frequency of the third mode of vibration of the member.

NOTE The values and the associated frequency limits may be defined in the National Annex. The recommended values are:
$\gamma_{bt} = 3,5\ m/s^2$
$\gamma_{df} = 5\ m/s^2$

A2.4.4.2.2 Deck twist

(1)P The twist of the bridge deck 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 and α and Load Model HSLM including centrifugal effects, all in accordance with EN1991-2, 6.

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Twist shall be checked on the approach to the bridge, across the bridge and for the departure from the bridge (see A2.4.4.1(2)P).

(2) The maximum twist t [mm/3m] of a track gauge s [m] of 1,435 m measured over a length of 3 m (Figure A2.1) should not exceed the values given in Table A2.7:

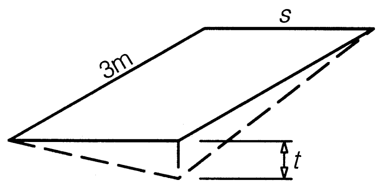

Figure A2.1 - Definition of deck twist

Table A2.7 – Limiting values of deck twist

Speed range V (km/h)	Maximum twist t (mm/3m)
$V \leq 120$	$t \leq t_1$
$120 < V \leq 200$	$t \leq t_2$
$V > 200$	$t \leq t_3$

NOTE The values for t may be defined in the National Annex.
The recommended values for the set of t are:
$t_1 = 4{,}5$
$t_2 = 3{,}0$
$t_3 = 1{,}5$
Values for a track with a different gauge may be defined in the National Annex.

(3) P The total track twist due to any twist which may be 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 may be defined in the National Annex. The recommended value for t_T is 7,5 mm/3m.

A2.4.4.2.3 Vertical deformation of the deck

(1) For all structure configurations loaded with the classified characteristic vertical loading in accordance with EN 1991-2, 6.3.2 (and where required classified SW/0 and SW/2 in accordance with EN 1991-2, 6.3.3) the maximum total vertical deflection measured along any track due to rail traffic actions should not exceed L/600.

NOTE Additional requirements for limiting vertical deformation for ballasted and non ballasted bridges may be specified as appropriate in the National Annex or for the individual project.

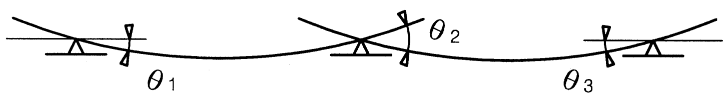

Figure A2.2 - Definition of angular rotations at the end of decks

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(2) Limitations on the rotations of ballasted bridge deck ends are implicit in EN 1991-2, 6.5.4.

NOTE The requirements for non ballasted structures may be specified in the National Annex.

(3) Additional limits of angular rotations at the end of decks in the vicinity of expansion devices, switches and crossings, etc., should be specified.

NOTE The additional limits of angular rotations may be defined in the National Annex or for the individual project.

(4) Limitations on the vertical displacement of bridge deck ends beyond bearings are given in EN1991-2, 6.5.4.5.2.

A2.4.4.2.4 Transverse deformation and vibration of the deck

(1)P 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 and α (or real train with the relevant dynamic factor if appropriate), wind loads, nosing force, centrifugal forces in accordance with EN1991-2, 6 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 values given in Table A2.8, or
- the change of radius of the track across a deck is not greater than the values in Table A2.8, 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 the specified value.

NOTE The maximum differential transverse deflection may be specified in the National Annex or for the individual project.

Table A2.8 - Maximum horizontal rotation and maximum change of radius of curvature

Speed range V (km/h)	Maximum horizontal rotation (radian)	Maximum change of radius of curvature (m)	
		Single deck	Multi-deck bridge
$V \leq 120$	α_1	r_1	r_4
$120 < V \leq 200$	α_2	r_2	r_5
$V > 200$	α_3	r_3	r_6

NOTE 1 The change of the radius of curvature may be determined using:

$$r = \frac{L^2}{8\delta_h} \qquad \text{(A2.7)}$$

NOTE 2 The transverse deformation includes the deformation of the bridge deck and the substructure (including piers, piles and foundations).

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NOTE 3 The values for the set of α_i and r_i may be defined in the National Annex. The recommended values are:
$\alpha_1 = 0{,}0035$; $\alpha_2 = 0{,}0020$; $\alpha_3 = 0{,}0015$;
$r_1 = 1700$; $r_2 = 6000$; $r_3 = 14000$;
$r_4 = 3500$; $r_5 = 9500$; $r_6 = 17500$

(3) The first natural frequency of lateral vibration of a span should not be less than f_{h0}.

NOTE The value for f_{h0} may be defined in the National Annex. The recommended value is:
$f_{h0} = 1{,}2$ Hz.

A2.4.4.2.5 Longitudinal displacement of the deck

(1) Limitations on the longitudinal displacement of the ends of decks are given in EN1991-2, 6.5.4.5.2.

NOTE Also see A2.4.4.2.3.

A2.4.4.3 Limiting values for the maximum vertical deflection for passenger comfort

A2.4.4.3.1 Comfort criteria

(1) 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.

(2) The levels of comfort and associated limiting values for the vertical acceleration should be specified.

NOTE These levels of comfort and associated limiting values may be defined for the individual project. Recommended levels of comfort are given in Table A2.9.

Table A2.9 - Recommended levels of comfort

Level of comfort	Vertical acceleration b_v (m/s^2)
Very good	1,0
Good	1,3
Acceptable	2,0

A2.4.4.3.2 Deflection criteria for checking passenger comfort

(1) To limit vertical vehicle acceleration to the values given in A2.4.4.3.1(2) values are given in this clause for the maximum permissible vertical deflection δ along the centre line of the track of railway bridges as a function of:
- the span length L [m],
- the train speed V [km/h],
- the number of spans and
- the configuration of the bridge (simply supported beam, continuous beam).

Alternatively the vertical acceleration b_v may be determined by a dynamic vehicle/bridge interaction analysis (see A2.4.4.3.3).

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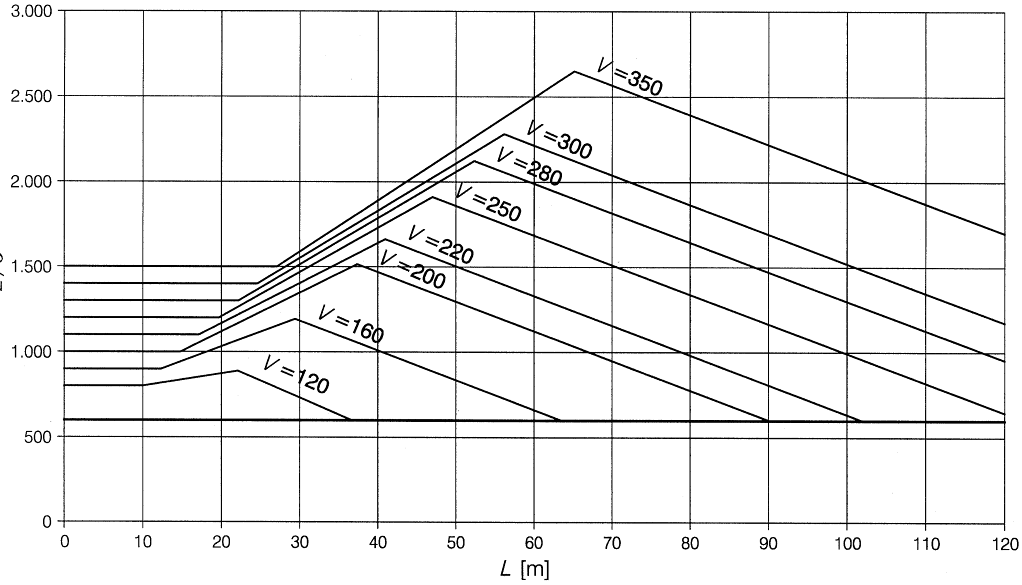
(2) The vertical deflections δ should be determined with Load Model 71 multiplied by the factor Φ and with the value of α=1, in accordance with EN1991-2, Section 6.

For bridges with two or more tracks only one track should be loaded.

(3) 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.



The factors listed in A2.4.4.3.2.(5) should not be applied to the limit of L/δ= 600.

Figure A2.3 - Maximum permissible vertical deflection δ for railway bridges with 3 or more successive simply supported spans corresponding to a permissible vertical acceleration of b_v = 1 m/s² in a coach for speed V [km/h]

(4) The limiting values of L/δ given in Figure A2.3 are given for b_v = 1,0 m/s² which may be taken as providing a "very good" level of comfort.
For other levels of comfort and associated maximum permissible vertical accelerations b'_v the values of L/δ given in Figure A2.3 may be divided by b'_v [m/s²].

(5) The values of L/δ given in Figure A2.3 are given for a succession of simply supported beams with three or more spans.
For a bridge comprising of either a single span or a succession of two simply supported beams or two continuous spans the values of L/δ given in Figure A2.3 should be multiplied by 0,7.
For continuous beams with three or more spans the values of L/δ given in Figure A2.3 should be multiplied by 0,9.

(6) The values of L/δ given in Figure A2.3 are valid for span lengths up to 120 m. For longer spans a special analysis is necessary.

NOTE The requirements for passenger comfort for temporary bridges may be defined in the National Annex or for the individual project.

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A2.4.4.3.3 Requirements for a dynamic vehicle/bridge interaction analysis for checking passenger comfort

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

iv) a series of vehicle speeds up to the maximum speed specified,
v) characteristic loading of the real trains specified for the individual project in accordance with EN1991-2, 6.4.6.1.1,
vi) dynamic mass interaction between vehicles in the real train and the structure,
vii) the damping and stiffness characteristics of the vehicle suspension,
viii) a sufficient number of vehicles to produce the maximum load effects in the longest span,
ix) a sufficient number of spans in a structure with multiple spans to develop any resonance effects in the vehicle suspension.

NOTE Any requirements for taking track roughness into account in the vehicle/bridge dynamic interaction analysis may be defined for the individual project.

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Annex B
(informative)
Management of Structural Reliability for Construction Works

B1 Scope and field of application

(1) This annex provides additional guidance to 2.2 (Reliability management) and to appropriate clauses in EN 1991 to EN 1999.

NOTE Reliability differentiation rules have been specified for particular aspects in the design Eurocodes, e.g. in EN 1992, EN 1993, EN 1996, EN 1997 and EN 1998.

(2) The approach given in this Annex recommends the following procedures for the management of structural reliability for construction works (with regard to ULSs, excluding fatigue) :

a) In relation to 2.2(5)b, classes are introduced and are based on the assumed consequences of failure and the exposure of the construction works to hazard. A procedure for allowing moderate differentiation in the partial factors for actions and resistances corresponding to the classes is given in B3.

NOTE Reliability classification can be represented by β indexes (see Annex C) which takes account of accepted or assumed statistical variability in action effects and resistances and model uncertainties.

b) In relation to 2.2(5)c and 2.2(5)d, a procedure for allowing differentiation between various types of construction works in the requirements for quality levels of the design and execution process are given in B4 and B5.

NOTE Those quality management and control measures in design, detailing and execution which are given in B4 and B5 aim to eliminate failures due to gross errors, and ensure the resistances assumed in the design.

(3) The procedure has been formulated in such a way so as to produce a framework to allow different reliability levels to be used, if desired.

B2 Symbols

In this annex the following symbols apply.

K_{FI}	Factor applicable to actions for reliability differentiation
β	Reliability index

B3 Reliability differentiation

B3.1 Consequences classes

(1) For the purpose of reliability differentiation, consequences classes (CC) may be established by considering the consequences of failure or malfunction of the structure as given in Table B1.

Table B1 - Definition of consequences classes

Consequences Class	Description	Examples of buildings and civil engineering works
CC3	**High** consequence for loss of human life, *or* economic, social or environmental consequences **very great**	Grandstands, public buildings where consequences of failure are high (e.g. a concert hall)
CC2	**Medium** consequence for loss of human life, economic, social or environmental consequences **considerable**	Residential and office buildings, public buildings where consequences of failure are medium (e.g. an office building)
CC1	**Low** consequence for loss of human life, *and* economic, social or environmental consequences **small or negligible**	Agricultural buildings where people do not normally enter (e.g. storage buildings), greenhouses

(2) The criterion for classification of consequences is the importance, in terms of consequences of failure, of the structure or structural member concerned. See B3.3

(3) Depending on the structural form and decisions made during design, particular members of the structure may be designated in the same, higher or lower consequences class than for the entire structure.

NOTE At the present time the requirements for reliability are related to the structural members of the construction works.

B3.2 Differentiation by β values

(1) The reliability classes (RC) may be defined by the β reliability index concept.

(2) Three reliability classes RC1, RC2 and RC3 may be associated with the three consequences classes CC1, CC2 and CC3.

(3) Table B2 gives recommended minimum values for the reliability index associated with reliability classes (see also annex C).

Table B2 - Recommended minimum values for reliability index β (ultimate limit states)

Reliability Class	Minimum values for β	
	1 year reference period	50 years reference period
RC3	5,2	4,3
RC2	4,7	3,8
RC1	4,2	3,3

NOTE A design using EN 1990 with the partial factors given in annex A1 and EN 1991 to EN 1999 is considered generally to lead to a structure with a β value greater than 3,8 for a 50 year reference period. Reliability classes for members of the structure above RC3 are not further considered in this Annex, since these structures each require individual consideration.

B3.3 Differentiation by measures relating to the partial factors

(1) One way of achieving reliability differentiation is by distinguishing classes of γ_F factors to be used in fundamental combinations for persistent design situations. For example, for the same design supervision and execution inspection levels, a multiplication factor K_{FI}, see Table B3, may be applied to the partial factors.

Table B3 - K_{FI} factor for actions

K_{FI} factor for actions	Reliability class		
	RC1	RC2	RC3
K_{FI}	0,9	1,0	1,1

NOTE In particular, for class RC3, other measures as described in this Annex are normally preferred to using K_{FI} factors. K_{FI} should be applied only to unfavourable actions.

(2) Reliability differentiation may also be applied through the partial factors on resistance γ_M. However, this is not normally used. An exception is in relation to fatigue verification (see EN 1993). See also B6.

(3) Accompanying measures, for example the level of quality control for the design and execution of the structure, may be associated to the classes of γ_F. In this Annex, a three level system for control during design and execution has been adopted. Design supervision levels and inspection levels associated with the reliability classes are suggested.

(4) There can be cases (*e.g.* lighting poles, masts, etc.) where, for reasons of economy, the structure might be in RC1, but be subjected to higher corresponding design supervision and inspection levels.

B4 Design supervision differentiation

(1) Design supervision differentiation consists of various organisational quality control measures which can be used together. For example, the definition of design supervision

level (B4(2)) may be used together with other measures such as classification of designers and checking authorities (B4(3)).

(2) Three possible design supervision levels (DSL) are shown in Table B4. The design supervision levels may be linked to the reliability class selected or chosen according to the importance of the structure and in accordance with National requirements or the design brief, and implemented through appropriate quality management measures. See 2.5.

Table B4 - Design supervision levels (DSL)

Design Supervision Levels	Characteristics	Minimum recommended requirements for checking of calculations, drawings and specifications
DSL3 relating to RC3	Extended supervision	Third party checking : Checking performed by an organisation different from that which has prepared the design
DSL2 relating to RC2	Normal supervision	Checking by different persons than those originally responsible and in accordance with the procedure of the organisation.
DSL1 Relating to RC1	Normal supervision	Self-checking: Checking performed by the person who has prepared the design

(3) Design supervision differentiation may also include a classification of designers and/or design inspectors (checkers, controlling authorities, etc.), depending on their competence and experience, their internal organisation, for the relevant type of construction works being designed.

NOTE The type of construction works, the materials used and the structural forms can affect this classification.

(4) Alternatively, design supervision differentiation can consist of a more refined detailed assessment of the nature and magnitude of actions to be resisted by the structure, or of a system of design load management to actively or passively control (restrict) these actions.

B5 Inspection during execution

(1) Three inspection levels (IL) may be introduced as shown in Table B5. The inspection levels may be linked to the quality management classes selected and implemented through appropriate quality management measures. See 2.5. Further guidance is available in relevant execution standards referenced by EN 1992 to EN 1996 and EN 1999.

Table B5 - Inspection levels (IL)

Inspection Levels	Characteristics	Requirements
IL3 Relating to RC3	Extended inspection	Third party inspection
IL2 Relating to RC2	Normal inspection	Inspection in accordance with the procedures of the organisation
IL1 Relating to RC1	Normal inspection	Self inspection

NOTE Inspection levels define the subjects to be covered by inspection of products and execution of works including the scope of inspection. The rules will thus vary from one structural material to another, and are to be given in the relevant execution standards.

B6 Partial factors for resistance properties

(1) A partial factor for a material or product property or a member resistance may be reduced if an inspection class higher than that required according to Table B5 and/or more severe requirements are used.

NOTE For verifying efficiency by testing see section 5 and Annex D.

NOTE Rules for various materials may be given or referenced in EN 1992 to EN 1999.

NOTE Such a reduction, which allows for example for model uncertainties and dimensional variation, is not a reliability differentiation measure : it is only a compensating measure in order to keep the reliability level dependent on the efficiency of the control measures.

Annex C
(informative)
Basis for Partial Factor Design and Reliability Analysis

C1 Scope and Field of Applications

(1) This annex provides information and theoretical background to the partial factor method described in Section 6 and annex A. This Annex also provides the background to annex D, and is relevant to the contents of annex B.

(2) This annex also provides information on

– the structural reliability methods ;

– the application of the reliability-based method to determine by calibration design values and/or partial factors in the design expressions ;

– the design verification formats in the Eurocodes.

C2 Symbols

In this annex the following symbols apply.

Latin upper case letters

P_f	Failure probability
Prob(.)	Probability
P_s	survival probability

Latin lower case letters

a	geometrical property
g	performance function

Greek upper case letters

Φ	cumulative distribution function of the standardised Normal distribution

Greek lower case letters

α_E	FORM (First Order Reliability Method) sensitivity factor for effects of actions
α_R	FORM (First Order Reliability Method) sensitivity factor for resistance
β	reliability index
θ	model uncertainty
μ_X	mean value of X

σ_X standard deviation of X
V_X coefficient of variation of X

C3 Introduction

(1) In the partial factor method the basic variables (i.e. actions, resistances and geometrical properties) through the use of partial factors and ψ factors are given design values, and a verification made to ensure that no relevant limit state has been exceeded. See C7.

NOTE Section 6 describes the design values for actions and the effects of actions, and design values of material and product properties and geometrical data.

(2) In principle numerical values for partial factors and ψ factors can be determined in either of two ways :

a) On the basis of calibration to a long experience of building tradition.

NOTE For most of the partial factors and the ψ factors proposed in the currently available Eurocodes this is the leading Principle.

b) On the basis of statistical evaluation of experimental data and field observations. (This should be carried out within the framework of a probabilistic reliability theory.)

(3) When using method 2b), either on its own or in combination with method 2a), ultimate limit states partial factors for different materials and actions should be calibrated such that the reliability levels for representative structures are as close as possible to the target reliability index. See C6.

C4 Overview of reliability methods

(1) Figure C1 presents a diagrammatic overview of the various methods available for calibration of partial factor (limit states) design equations and the relation between them.

(2) The probabilistic calibration procedures for partial factors can be subdivided into two main classes :
– full probabilistic methods (Level III), and
– first order reliability methods (FORM) (Level II).

NOTE 1 Full probabilistic methods (Level III) give in principle correct answers to the reliability problem as stated. Level III methods are seldom used in the calibration of design codes because of the frequent lack of statistical data.

NOTE 2 The level II methods make use of certain well defined approximations and lead to results which for most structural applications can be considered sufficiently accurate.

(3) In both the Level II and Level III methods the measure of reliability should be identified with the survival probability $P_s = (1 - P_f)$, where P_f is the failure probability for the considered failure mode and within an appropriate reference period. If the calculated

failure probability is larger than a pre-set target value P_0, then the structure should be considered to be unsafe.

NOTE The 'probability of failure' and its corresponding reliability index (see C5) are only notional values that do not necessarily represent the actual failure rates but are used as operational values for code calibration purposes and comparison of reliability levels of structures.

(4) The Eurocodes have been primarily based on method *a* (see Figure C1). Method *c* or equivalent methods have been used for further development of the Eurocodes.

NOTE An example of an equivalent method is design assisted by testing (see annex D).

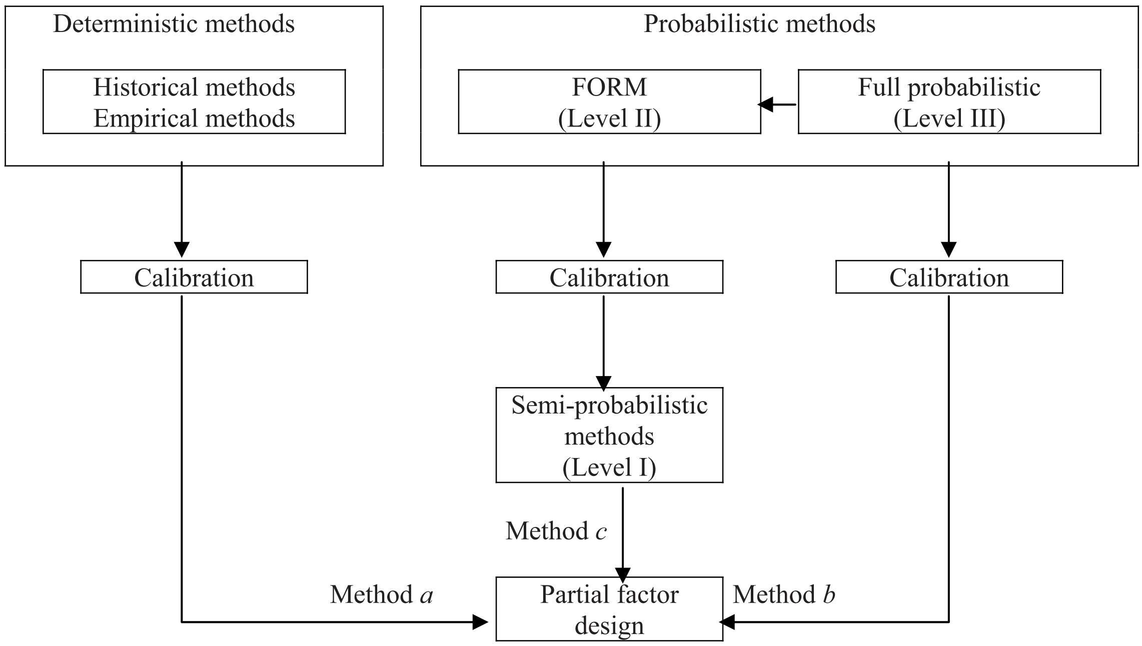


Figure C1 - Overview of reliability methods

C5 Reliability index β

(1) In the Level II procedures, an alternative measure of reliability is conventionally defined by the reliability index β which is related to P_f by :

$$P_f = \Phi(-\beta) \qquad \text{(C.1)}$$

where Φ is the cumulative distribution function of the standardised Normal distribution. The relation between Φ and β is given in Table C1.

Table C1 - Relation between β and P_f

P_f	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}
β	1,28	2,32	3,09	3,72	4,27	4,75	5,20

(2) The probability of failure P_f can be expressed through a performance function g such that a structure is considered to survive if $g > 0$ and to fail if $g \leq 0$:

$$P_{\mathrm{f}} = \mathrm{Prob}(g \leq 0) \qquad \text{(C.2a)}$$

If R is the resistance and E the effect of actions, the performance function g is :

$$g = R - E \qquad \text{(C.2b)}$$

with R, E and g random variables.

(3) If g is Normally distributed, β is taken as :

$$\beta = \frac{\mu_g}{\sigma_g} \qquad \text{(C.2c)}$$

where :

μ_{g} is the mean value of g, and
σ_{g} is its standard deviation,

so that :

$$\mu_g - \beta\sigma_g = 0 \qquad \text{(C.2d)}$$

and

$$P_f = \mathrm{Prob}(g \leq 0) = \mathrm{Prob}(g \leq \mu_g - \beta\sigma_g) \qquad \text{(C.2e)}$$

For other distributions of g, β is only a conventional measure of the reliability $P_{\mathrm{s}} = (1 - P_{\mathrm{f}})$.

C6 Target values of reliability index β

(1) Target values for the reliability index β for various design situations, and for reference periods of 1 year and 50 years, are indicated in Table C2. The values of β in Table C2 correspond to levels of safety for reliability class RC2 (see Annex B) structural members.

NOTE 1 For these evaluations of β

– Lognormal or Weibull distributions have usually been used for material and structural resistance parameters and model uncertainties ;
– Normal distributions have usually been used for self-weight ;
– For simplicity, when considering non-fatigue verifications, Normal distributions have been used for variable actions. Extreme value distributions would be more appropriate.

NOTE 2 When the main uncertainty comes from actions that have statistically independent maxima in each year, the values of β for a different reference period can be calculated using the following expression :

$$\Phi(\beta_n) = [\Phi(\beta_1)]^n \qquad \text{(C.3)}$$

where :

β_n is the reliability index for a reference period of n years,
β_1 is the reliability index for one year.

Table C2 - Target reliability index β for Class RC2 structural members [1)]

Limit state	Target reliability index	
	1 year	50 years
Ultimate	4,7	3,8
Fatigue		1,5 to 3,8 [2)]
Serviceability (irreversible)	2,9	1,5
[1)] See Annex B [2)] Depends on degree of inspectability, reparability and damage tolerance.		

(2) The actual frequency of failure is significantly dependent upon human error, which are not considered in partial factor design (See Annex B). Thus β does not necessarily provide an indication of the actual frequency of structural failure.

C7 Approach for calibration of design values

(1) In the design value method of reliability verification (see Figure C1), design values need to be defined for all the basic variables. A design is considered to be sufficient if the limit states are not reached when the design values are introduced into the analysis models. In symbolic notation this is expressed as :

$$E_d < R_d \qquad \text{(C.4)}$$

where the subscript '*d*' refers to design values. This is the practical way to ensure that the reliability index β is equal to or larger than the target value.

E_d and R_d can be expressed in partly symbolic form as :

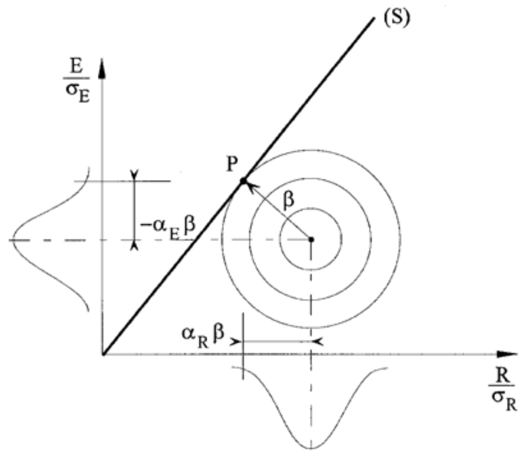
$$E_d = E\ \{F_{d1}, F_{d2}, \ldots\ a_{d1}, a_{d2}, \ldots\ \theta_{d1}, \theta_{d2}\ , \ldots\} \qquad \text{(C.5a)}$$

$$R_d = R\ \{X_{d1}, X_{d2}, \ldots\ a_{d1}, a_{d2}, \ldots\ \theta_{d1}, \theta_{d2}, \ldots\} \qquad \text{(C.5b)}$$

where :

E is the action effect ;
R is the resistance ;
F is an action ;
X is a material property ;
a is a geometrical property ;
θ is a model uncertainty.

For particular limit states (*e.g.* fatigue) a more general formulation may be necessary to express a limit state.



(S) failure boundary $g = R - E = 0$
P design point

Figure C2 - Design point and reliability index β according to the first order reliability method (FORM) for Normally distributed uncorrelated variables

(2) Design values should be based on the values of the basic variables at the FORM design point, which can be defined as the point on the failure surface (g = 0) closest to the average point in the space of normalised variables (as diagrammatically indicated in Figure C2).

(3) The design values of action effects E_d and resistances R_d should be defined such that the probability of having a more unfavourable value is as follows :

$$P(E > E_d) = \Phi(+\alpha_E\beta) \qquad \text{(C.6a)}$$
$$P(R \leq R_d) = \Phi(-\alpha_R\beta) \qquad \text{(C.6b)}$$

where :

β is the target reliability index (see C6).

α_E and α_R, with $|\alpha| \leq 1$, are the values of the FORM sensitivity factors. The value of α is negative for unfavourable actions and action effects, and positive for resistances.

α_E and α_R may be taken as - 0,7 and 0,8, respectively, provided

$$0{,}16 < \sigma_E/\sigma_R < 7{,}6 \qquad \text{(C.7)}$$

where σ_E and σ_R are the standard deviations of the action effect and resistance, respectively, in expressions (C.6a) and (C.6b). This gives :

$$P(E > E_d) = \Phi(-0{,}7\beta) \qquad \text{(C.8a)}$$

$$P(R \leq R_d) = \Phi(-0{,}8\beta) \qquad \text{(C.8b)}$$

(4) Where condition (C.7) is not satisfied $\alpha = \pm$ 1,0 should be used for the variable with the larger standard deviation, and $\alpha = \pm$ 0,4 for the variable with the smaller standard deviation.

(5) When the action model contains several basic variables, expression (C.8a) should be used for the leading variable only. For the accompanying actions the design values may be defined by :

$$P(E > E_d) = \Phi(-0{,}4 \times 0{,}7 \times \beta) = \Phi(-0{,}28\beta) \qquad \text{(C.9)}$$

NOTE For β = 3,8 the values defined by expression (C.9) correspond approximately to the 0,90 fractile.

(6) The expressions provided in Table C3 should be used for deriving the design values of variables with the given probability distribution.

Table C3 - Design values for various distribution functions

Distribution	Design values
Normal	$\mu - \alpha\beta\sigma$
Lognormal	$\mu \exp(-\alpha\beta V)$ for $V = \sigma/\mu < 0{,}2$
Gumbel	$u - \frac{1}{a}\ln\{-\ln\Phi(-\alpha\beta)\}$ where $u = \mu - \frac{0{,}577}{a}$; $a = \frac{\pi}{\sigma\sqrt{6}}$

NOTE In these expressions μ, σ and V are, respectively, the mean value, the standard deviation and the coefficient of variation of a given variable. For variable actions, these should be based on the same reference period as for β.

(7) One method of obtaining the relevant partial factor is to divide the design value of a variable action by its representative or characteristic value.

C8 Reliability verification formats in Eurocodes

(1) In EN 1990 to EN 1999, the design values of the basic variables, X_d and F_d, are usually not introduced directly into the partial factor design equations. They are introduced in terms of their representative values X_{rep} and F_{rep}, which may be :

– characteristic values, i.e. values with a prescribed or intended probability of being exceeded, e.g. for actions, material properties and geometrical properties (see 1.5.3.14, 1.5.4.1 and 1.5.5.1, respectively) ;

– nominal values, which are treated as characteristic values for material properties (see 1.5.4.3) and as design values for geometrical properties (see 1.5.5.2).

(2) The representative values X_{rep} and F_{rep}, should be divided and/or multiplied, respectively, by the appropriate partial factors to obtain the design values X_d and F_d.

NOTE See also expression (C.10).

(3) Design values of actions F, material properties X and geometrical properties a are given in expressions (6.1), (6.3) and (6.4), respectively.

Where an upper value for design resistance is used (see 6.3.3), the expression (6.3) takes the form :

$$X_d = \eta \gamma_{fM} X_{k,sup} \quad \text{(C.10)}$$

where γ_{fM} is an appropriate factor greater than 1.

NOTE Expression (C.10) may be used for capacity design.

(4) Design values for model uncertainties may be incorporated into the design expressions through the partial factors γ_{Sd} and γ_{Rd} applied on the total model, such that :

$$E_d = \gamma_{Sd} E\{\gamma_{gj} G_{kj}; \gamma_P P; \gamma_{q1} Q_{k1}; \gamma_{qi} \psi_{0i} Q_{ki}; a_d ...\} \quad \text{(C.11)}$$

$$R_d = R\{\eta X_k / \gamma_m; a_d ...\} / \gamma_{Rd} \quad \text{(C.12)}$$

(5) The coefficient ψ which takes account of reductions in the design values of variable actions, is applied as ψ_0 , ψ_1 or ψ_2 to simultaneously occurring, accompanying variable actions.

(6) The following simplifications may be made to expression (C.11) and (C.12), when required.

a) On the loading side (for a single action or where linearity of action effects exists) :

$$E_d = E\ \{\gamma_{F,i} F_{rep,i}, a_d\} \quad \text{(C.13)}$$

b) On the resistance side the general format is given in expressions (6.6), and further simplifications may be given in the relevant material Eurocode. The simplifications should only be made if the level of reliability is not reduced.

NOTE Non–linear resistance and actions models, and multi-variable action or resistance models, are commonly encountered in Eurocodes. In such instances, the above relations become more complex.

C9 Partial factors in EN 1990

(1) The different partial factors available in EN 1990 are defined in 1.6.

(2) The relation between individual partial factors in Eurocodes is schematically shown Figure C3.

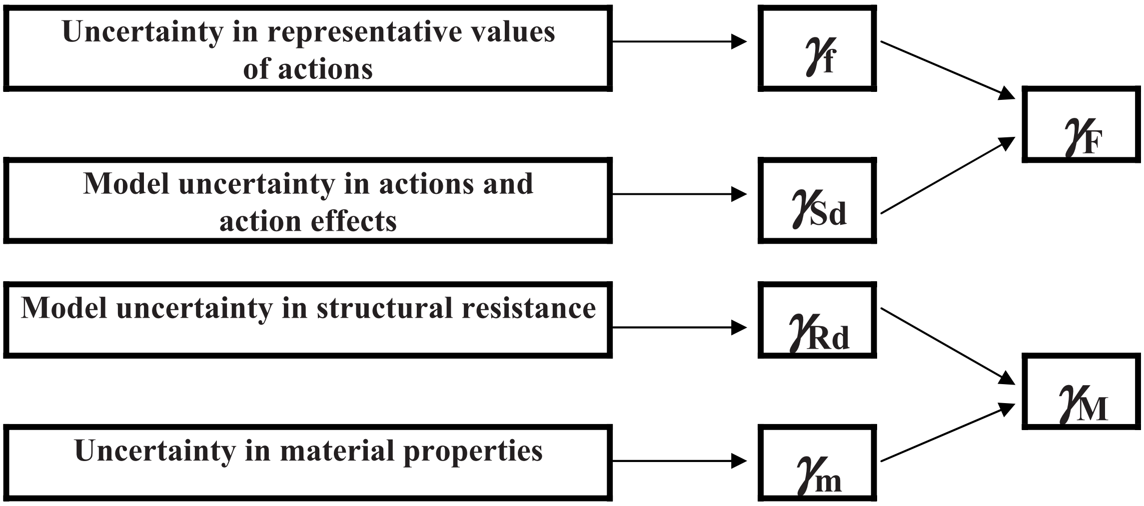


Figure C3 - Relation between individual partial factors

C10 ψ_0 factors

(1) Table C4 gives expressions for obtaining the ψ_0 factors (see Section 6) in the case of two variable actions.

(2) The expressions in Table C4 have been derived by using the following assumptions and conditions :

– the two actions to be combined are independent of each other ;

– the basic period (T_1 or T_2) for each action is constant ; T_1 is the greater basic period ;

– 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.

(3) The distribution functions in Table C4 refer to the maxima within the reference period T. These distribution functions are total functions which consider the probability that an action value is zero during certain periods.

Table C4 - Expressions for ψ_0 for the case of two variable actions

Distribution	$\psi_0 = F_{\text{accompanying}} / F_{\text{leading}}$
General	$\dfrac{F_s^{-1}\left\{\Phi(0{,}4\beta')^{N_1}\right\}}{F_s^{-1}\left\{\Phi(0{,}7\beta)^{N_1}\right\}}$ with $\beta' = -\Phi^{-1}\left\{\Phi(-0{,}7\beta)/N_1\right\}$
Approximation for very large N_1	$\dfrac{F_s^{-1}\left\{\exp\left[-N_1\Phi(-0{,}4\beta')\right]\right\}}{F_s^{-1}\left\{\Phi(0{,}7\beta)\right\}}$ with $\beta' = -\Phi^{-1}\left\{\Phi(-0{,}7\beta)/N_1\right\}$
Normal (approximation)	$\dfrac{1+\left(0{,}28\beta - 0{,}7\ln N_1\right)V}{1+0{,}7\beta V}$
Gumbel (approximation)	$\dfrac{1-0{,}78V\left[0{,}58+\ln\left(-\ln\Phi\left(0{,}28\beta\right)\right)+\ln N_1\right]}{1-0{,}78V\left[0{,}58+\ln\left(-\ln\Phi(0{,}7\beta)\right)\right]}$
$F_s(.)$ is the probability distribution function of the extreme value of the accompanying action in the reference period T ; $\Phi(.)$ is the standard Normal distribution function ; T is the reference period ; T_1 is the greater of the basic periods for actions to be combined ; N_1 is the ratio T/T_1, approximated to the nearest integer ; β is the reliability index ; V is the coefficient of variation of the accompanying action for the reference period.	

Annex D
(informative)
Design assisted by testing

D1 Scope and field of application

(1) This annex provides guidance on 3.4, 4.2 and 5.2.

(2) This annex is not intended to replace acceptance rules given in harmonised European product specifications, other product specifications or execution standards.

D2 Symbols

In this annex, the following symbols apply.

Latin upper case letters

$E(.)$	Mean value of (.)
V	Coefficient of variation [$V =$ (standard deviation) / (mean value)]
V_X	Coefficient of variation of X
V_δ	Estimator for the coefficient of variation of the error term δ
$\underline{X}$	Array of j basic variables $X_1 \ldots X_j$
$X_{k(n)}$	Characteristic value, including statistical uncertainty for a sample of size n with any conversion factor excluded
$\underline{X}_m$	Array of mean values of the basic variables
$\underline{X}_n$	Array of nominal values of the basic variables

Latin lower case letters

b	Correction factor
b_i	Correction factor for test specimen i
$g_{rt}(\underline{X})$	Resistance function (of the basic variables $\underline{X}$) used as the design model
$k_{d,n}$	Design fractile factor
k_n	Characteristic fractile factor
m_X	Mean of the n sample results
n	Number of experiments or numerical test results
r	Resistance value
r_d	Design value of the resistance
r_e	Experimental resistance value
r_{ee}	Extreme (maximum or minimum) value of the experimental resistance [i.e. value of r_e that deviates most from the mean value r_{em}]
r_{ei}	Experimental resistance for specimen i
r_{em}	Mean value of the experimental resistance
r_k	Characteristic value of the resistance
r_m	Resistance value calculated using the mean values $\underline{X}_m$ of the basic variables
r_n	Nominal value of the resistance
r_t	Theoretical resistance determined from the resistance function $g_{rt}(\underline{X})$

r_{ti}	Theoretical resistance determined using the measured parameters $\underline{X}$ for specimen i
s	Estimated value of the standard deviation σ
s_Δ	Estimated value of σ_Δ
s_δ	Estimated value of σ_δ

Greek upper case letters

Φ	Cumulative distribution function of the standardised Normal distribution
Δ	Logarithm of the error term δ $[\Delta_i = \ln(\delta_i)]$
$\overline{\Delta}$	Estimated value for $E(\Delta)$

Greek lower case letters

α_E	FORM (First Order Reliability Method) sensitivity factor for effects of actions
α_R	FORM (First Order Reliability Method) sensitivity factor for resistance
β	Reliability index
γ_M*	Corrected partial factor for resistances $[\gamma_M^* = r_n/r_d$ so $\gamma_M^* = k_c\,\gamma_M]$
δ	Error term
δ_i	Observed error term for test specimen i obtained from a comparison of the experimental resistance r_{ei} and the mean value corrected theoretical resistance br_{ti}
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
σ	Standard deviation $[\sigma = \sqrt{\text{variance}}]$
σ_Δ^2	Variance of the term Δ

D3 Types of tests

(1) A distinction needs to be made between the following types of tests :

a) tests to establish directly the ultimate resistance or serviceability properties of structures or structural members for given loading conditions. Such tests can be performed, for example, for fatigue loads or impact loads ;

b) tests to obtain specific material properties using specified testing procedures ; for instance, ground testing in situ or in the laboratory, or the testing of new materials ;

c) tests to reduce uncertainties in parameters in load or load effect models; for instance, by wind tunnel testing, or in tests to identify actions from waves or currents ;

d) tests to reduce uncertainties in parameters used in resistance models ; for instance, by testing structural members or assemblies of structural members (*e.g.* roof or floor structures) ;

e) control tests to check the identity or quality of delivered products or the consistency of production characteristics ; for instance, testing of cables for bridges, or concrete cube testing ;

f) tests carried out during execution in order to obtain information needed for part of the execution ; for instance, testing of pile resistance, testing of cable forces during execution ;

g) control tests to check the behaviour of an actual structure or of structural members after completion, *e.g.* to find the elastic deflection, vibrational frequencies or damping ;

(2) For test types (a), (b), (c), (d), the design values to be used should wherever practicable be derived from the test results by applying accepted statistical techniques. See D5 to D8.

NOTE Special techniques might be needed in order to evaluate type (c) test results.

(3) Test types (e), (f), (g) may be considered as acceptance tests where no test results are available at the time of design. Design values should be conservative estimates which are expected to be able to meet the acceptance criteria (tests (e), (f), (g)) at a later stage.

D4 Planning of tests

(1) Prior to the carrying out of tests, a test plan should be agreed with the testing organisation. This plan should contain the objectives of the test and all specifications necessary for the selection or production of the test specimens, the execution of the tests and the test evaluation. The test plan should cover :
– objectives and scope,
– prediction of test results,
– specification of test specimens and sampling,
– loading specifications,
– testing arrangement,
– measurements,
– evaluation and reporting of the tests.

Objectives and scope : The objective of the tests should be clearly stated, *e.g.* the required properties, the influence of certain design parameters varied during the test and the range of validity. Limitations of the test and required conversions (*e.g.* scaling effects) should be specified.

Prediction of test results : All properties and circumstances that can influence the prediction of test results should be taken into account, including :
– geometrical parameters and their variability,
– geometrical imperfections,
– material properties,
– parameters influenced by fabrication and execution procedures,
– scale effects of environmental conditions taking into account, if relevant, any sequencing.

The expected modes of failure and/or calculation models, together with the corresponding variables should be described. If there is a significant doubt about which failure modes might be critical, then the test plan should be developed on the basis of accompanying pilot tests.

NOTE Attention needs to be given to the fact that a structural member can possess a number of fundamentally different failure modes.

Specification of test specimen and sampling : Test specimens should be specified, or obtained by sampling, in such a way as to represent the conditions of the real structure.

Factors to be taken into account include :
– dimensions and tolerances,
– material and fabrication of prototypes,
– number of test specimens,
– sampling procedures,
– restraints.

The objective of the sampling procedure should be to obtain a statistically representative sample.
Attention should be drawn to any difference between the test specimens and the product population that could influence the test results.

Loading specifications : 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.

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.

Where structural behaviour depends upon the effects of one or more actions that will not be varied systematically, then those effects should be specified by their representative values.

Testing arrangement : The test equipment should be relevant for the type of tests and the expected range of measurements. Special attention should be given to measures to obtain sufficient strength and stiffness of the loading and supporting rigs, and clearance for deflections, etc.

Measurements : Prior to the testing, all relevant properties to be measured for each individual test specimen should be listed. Additionally a list should be made :

a) of measurement-locations,

b) of procedures for recording results, including if relevant :
   – time histories of displacements,
   – velocities,
   – accelerations,
   – strains,
   – forces and pressures,
   – required frequency,
   – accuracy of measurements, and
   – appropriate measuring devices.

Evaluation and reporting the test : For specific guidance, see D5 to D8. Any Standards on which the tests are based should be reported.

D5 Derivation of 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 in one of the following ways :

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 D7.2 and D8.2) ;

b) by direct determination of the design value, implicitly or explicitly accounting for the conversion of results and the total reliability required (see D7.3 and D8.3).

NOTE In general method a) is to be preferred provided the value of the partial factor is determined from the normal design procedure (see (3) below).

(2) The derivation of a characteristic value from tests (Method (a)) should take into account :

a) the scatter of test data ;
b) statistical uncertainty associated with the number of tests ;
c) prior statistical knowledge.

(3) The partial factor to be applied to a characteristic value should be taken from the appropriate Eurocode provided there is sufficient similarity between the tests and the usual field of application of the partial factor as used in numerical verifications.

(4) If the response of the structure or structural member or the resistance of the material depends on influences not sufficiently covered by the tests such as :
– time and duration effects,
– scale and size effects,
– different environmental, loading and boundary conditions,
– resistance effects,
then the calculation model should take such influences into account as appropriate.

(5) In special cases where the method given in D5(1)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 in expression (C.8a) ;
– where appropriate, the required design working life ;
– prior knowledge from similar cases.

NOTE Further information may be found in D6, D7 and D8.

D6 General principles for statistical evaluations

(1) When evaluating test results, the behaviour of test specimens and failure modes should be compared with theoretical predictions. When significant deviations from a prediction occur, an explanation should be sought : this might involve additional testing, perhaps under different conditions, or modification of the theoretical model.

(2) 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. The methods given in this Annex may be used only when the following conditions are satisfied :
– the statistical data (including prior information) are taken from identified populations which are sufficiently homogeneous ; and
– 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, make it possible to present an interpretation as statistical (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 D7. This interpretation will still need to use some prior information about the parameter ; however, this will normally be less than above.
– when a series of tests is carried out in order to calibrate a model (as a function) and one or more associated parameters, a classical statistical interpretation is possible.

(3) The result of a test evaluation should be considered valid only for the specifications and load characteristics considered in the tests. If the results are to be extrapolated to cover other design parameters and loading, additional information from previous tests or from theoretical bases should be used.

D7 Statistical determination of a single property

D7.1 General

(1) This clause gives working expressions for deriving design values from test types (a) and (b) of D3(3) for a single property (for example, a strength) when using evaluation methods (a) and (b) of D5(1).

NOTE The expressions presented here, which use Bayesian procedures with “vague” prior distributions, lead to almost the same results as classical statistics with confidence levels equal to 0,75.

(2) The single property X may represent
a) a resistance of a product,
b) a property contributing to the resistance of a product.

(3) In case a) the procedure D7.2 and D7.3 can be applied directly to determine characteristic or design or partial factor values.

(4) In case b) it should be considered that the design value of the resistance should also include :
- the effects of other properties,
- the model uncertainty,
- other effects (scaling, volume, etc.)

(5) The tables and expressions in D7.2 and D7.3 are based on the following assumptions:
– all variables follow either a Normal or a log-normal distribution ;
– there is no prior knowledge about the value of the mean ;
– 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 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.

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, should be assumed to be not smaller than 0,10.

D7.2 Assessment via the characteristic value

(1) The design value of a property X should be found by using :

$$X_d = \eta_d \frac{X_{k(n)}}{\gamma_m} = \frac{\eta_d}{\gamma_m} m_X \{1 - k_n V_X\} \qquad \text{(D.1)}$$

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.

The value of k_n can be found from Table D1.

(2) When using table D1, one of two cases should be considered as follows.

– 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. What is 'comparable' needs to be determined by engineering judgement (see D7.1(3)).

– 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 :

$$s_x^2 = \frac{1}{n-1} \Sigma (x_i - m_X)^2 \qquad \text{(D.2)}$$

$$V_X = s_X / m_X \qquad \text{(D.3)}$$

(3) The partial factor γ_m should be selected according to the field of application of the test results.

Table D1 : Values of k_n for the 5% characteristic value

n	1	2	3	4	5	6	8	10	20	30	∞
V_X known	2,31	2,01	1,89	1,83	1,80	1,77	1,74	1,72	1,68	1,67	1,64
V_X unknown	-	-	3,37	2,63	2,33	2,18	2,00	1,92	1,76	1,73	1,64

NOTE 1 This table is based on the Normal distribution.
NOTE 2 With a log-normal distribution expression (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, $s_y = \sqrt{\frac{1}{n-1} \Sigma (\ln x_i - m_y)^2}$

D7.3 Direct assessment of the design value for ULS verifications

(1) The design value X_d for X should be found by using :

$$X_d = \eta_d m_X \{1 - k_{d,n} V_X\} \qquad \text{(D.4)}$$

In this case, η_d should cover all uncertainties not covered by the tests.

(2) $k_{d,n}$ should be obtained from table D2.

Table D2 - Values of $k_{d,n}$ for the ULS design value.

n	1	2	3	4	5	6	8	10	20	30	∞
V_X known	4,36	3,77	3,56	3,44	3,37	3,33	3,27	3,23	3,16	3,13	3,04
V_X unknown	-	-	-	11,40	7,85	6,36	5,07	4,51	3,64	3,44	3,04

NOTE 1 This table is based on the assumption that the design value corresponds to a product $\alpha_R\beta$ = 0,8×3,8 = 3,04 (see annex C) and that X is Normally distributed. This gives a probability of observing a lower value of about 0,1 %.

NOTE 2 With a log-normal distribution, expression (D.4) becomes :
$X_d = \eta_d \exp\left[m_y - k_{d,n} s_y\right]$

D8 Statistical determination of resistance models

D8.1 General

(1) This clause is mainly intended to define procedures (methods) for calibrating resistance models and for deriving design values from tests type d) (see D3(1)). Use will be made of available prior information (knowledge or assumptions).

(2) Based on the observation of actual behaviour in tests and on theoretical considerations, a "design model" should be developed, leading to the derivation of a resistance function. The validity of this model should be then checked by means of a statistical interpretation of all available test data. If necessary the design model is then adjusted until 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 will need to be combined with the deviations of the other variables in the resistance function in order to obtain an overall indication of deviation. These other variables 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.

(5) In D5(1) two different methods are distinguished. These methods are given in D8.2 and D8.3 respectively. Additionally, some possible simplifications are given in D8.4.

These methods are presented as a number of discrete steps and some assumptions regarding the test population are made and explained ; these assumptions are to be considered to be no more than recommendations covering some of the commoner cases.

D8.2 Standard evaluation procedure (Method (a))

D8.2.1 General

(1) For the standard evaluation procedure the following assumptions are made :
a) the resistance function is a function of a number of independent variables $\underline{X}$;
b) a sufficient number of test results is available ;
c) all relevant geometrical and material properties are measured ;
d) there is no correlation (statistical dependence) between the variables in the resistance function ;
e) all variables follow either a Normal or a log-normal distribution.

NOTE Adopting a log-normal distribution for a variable has the advantage that no negative values can occur.

(2) The standard procedure for method D5(1)a) comprises the seven steps given in D8.2.2.1 to D8.2.2.7.

D8.2.2 Standard procedure

D8.2.2.1 Step 1 : Develop a design model

(1) Develop a design model for the theoretical resistance r_t of the member or structural detail considered, represented by the resistance function :

$$r_t = g_{rt}(\underline{X}) \qquad \text{(D.5)}$$

(2) The resistance function should cover all relevant basic variables $\underline{X}$ that affect the resistance at the relevant limit state.

(3) All basic parameters should be measured for each test specimen i (assumption (c) in D8.2.1) and should be available for use in the evaluation.

D8.2.2.2 Step 2 : Compare experimental and theoretical values

(1) Substitute the actual measured properties into the resistance function so as to obtain theoretical values r_{ti} to form the basis of a comparison with the experimental values r_{ei} from the tests.

(2) The points representing pairs of corresponding values (r_{ti}, r_{ei}) should be plotted on a diagram, as indicated in figure D1.

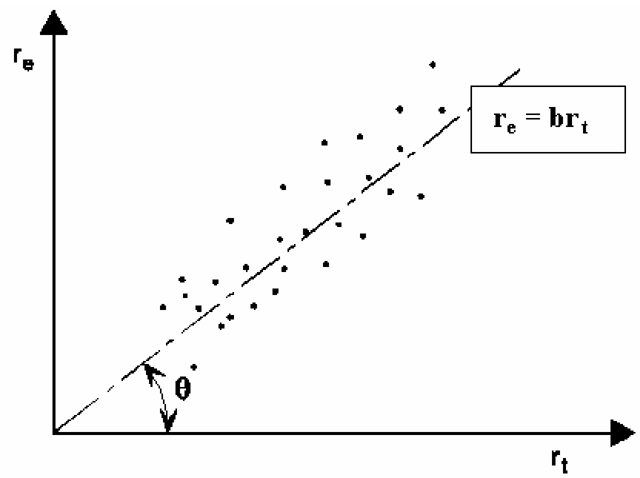


Figure D1 - r_e - r_t diagram

(3) If the resistance function is exact and complete, then all of the points will lie on the line $\theta = \pi/4$. In practice the points will show some scatter, but the causes of any systematic deviation from that line should be investigated to check whether this indicates errors in the test procedures or in the resistance function.

D8.2.2.3 Step 3 : Estimate the mean value correction factor b

(1) Represent the probabilistic model of the resistance r in the format :

$$r = b\, r_t \delta \qquad \text{(D.6)}$$

where :

b is the "Least Squares" best-fit to the slope, given by $b = \dfrac{\sum r_e r_t}{\sum r_t^2}$ (D.7)

(2) The mean value of the theoretical resistance function, calculated using the mean values $\underline{X}_m$ of the basic variables, can be obtained from :

$$r_m = b\, r_t(\underline{X}_m)\,\delta = b g_{rt}(\underline{X}_m)\,\delta \qquad \text{(D.8)}$$

D8.2.2.4 Step 4 : Estimate the coefficient of variation of the errors

(1) The error term δ_i for each experimental value r_{ei} should be determined from expression (D.9) :

$$\delta_i = \frac{r_{ei}}{b\, r_{ti}} \qquad \text{(D.9)}$$

(2) From the values of δ_i an estimated value for V_δ should be determined by defining :

$$\Delta_i = \ln(\delta_i) \qquad \text{(D.10)}$$

(3) The estimated value $\overline{\Delta}$ for $E(\Delta)$ should be obtained from :

$$\overline{\Delta} = \frac{1}{n}\sum_{i=1}^{n} \Delta_i \qquad \text{(D.11)}$$

(4) The estimated value s_Δ^2 for σ_Δ^2 should be obtained from :

$$s_\Delta^2 = \frac{1}{n-1}\sum_{i=1}^{n}\left(\Delta_i - \overline{\Delta}\right)^2 \qquad \text{(D.12)}$$

(5) The expression :

$$V_\delta = \sqrt{\exp(s_\Delta^2) - 1} \qquad \text{(D.13)}$$

may be used as the coefficient of variation V_δ of the δ_i error terms.

D8.2.2.5 Step 5 : Analyse compatibility

(1) The compatibility of the test population with the assumptions made in the resistance function should be analysed.

(2) If the scatter of the (r_{ei} , r_{ti}) values is too high to give economical design resistance functions, this scatter may be reduced in one of the following ways :

a) by correcting the design model to take into account parameters which had previously been ignored ;
b) 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 sub-set by analysing each subset using the standard procedure. The disadvantage of splitting the test results into sub-sets is that the number of test results in each sub-set can become very small.

(4) When determining the fractile factors k_n (see step 7), the k_n value for the sub-sets 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.

D8.2.2.6 Step 6 : Determine the coefficients of variation V_{Xi} of the basic variables

(1) If it can be shown that the test population is fully representative of the variation in reality, then the coefficients of variation V_{Xi} of the basic variables in the resistance function may be determined from the test data. However, since this is not generally the case, the coefficients of variation V_{Xi} will normally need to be determined on the basis of some prior knowledge.

D8.2.2.7 Step 7 : Determine the characteristic value r_k of the resistance

(1) If the resistance function for j basic variables is a product function of the form :

$$r = b\, r_t \delta = b\,\{X_1 \times X_2 \ldots X_j\}\,\delta$$

the mean value $E(r)$ may be obtained from :

$$E(r) = b\,\{E(X_1) \times E(X_2) \ldots E(X_j)\} = b\, g_{rt}(\underline{X}_m) \tag{D.14a}$$

and the coefficient of variation V_r may be obtained from the product function :

$$V_r^2 = (V_\delta^2 + 1)\left[\prod_{i=1}^{j}\left(V_{Xi}^2 + 1\right)\right] - 1 \tag{D.14b}$$

(2) Alternatively, for small values of V_δ^2 and V_{Xi}^2 the following approximation for V_r may be used :

$$V_r^2 = V_\delta^2 + V_{rt}^2 \tag{D.15a}$$

with :

$$V_{rt}^2 = \sum_{i=1}^{j} V_{Xi}^2 \qquad \text{(D.15b)}$$

(3) If the resistance function is a more complex function of the form :

$r = b\, r_t \delta = b\, g_{rt}(X_1, ..., X_j)\, \delta$

the mean value $E(r)$ may be obtained from :

$$E(r) = b\, g_{rt}(E(X_1), ..., E(X_j)) = b\, g_{rt}(\underline{X}_m) \qquad \text{(D.16a)}$$

and the coefficient of variation V_{rt} may be obtained from :

$$V_{rt}^2 = \frac{VAR[g_{rt}(\underline{X})]}{g_{rt}^2(\underline{X}_m)} \cong \frac{1}{g_{rt}^2(\underline{X}_m)} \times \sum_{i=1}^{j}\left(\frac{\partial g_{rt}}{\partial X_i}\sigma_i\right)^2 \qquad \text{(D.16b)}$$

(4) If the number of tests is limited (say $n < 100$) allowance should be made in the distribution of Δ for statistical uncertainties. The distribution should be considered as a central t-distribution with the parameters $\overline{\Delta}$, V_Δ and n.

(5) In this case the characteristic resistance r_k should be obtained from :

$$r_k = b\, g_{rt}(\underline{X}_m) \exp(- k_\infty \alpha_{rt} Q_{rt} - k_n \alpha_\delta Q_\delta - 0{,}5\, Q^2) \qquad \text{(D.17)}$$

with :

$$Q_{rt} = \sigma_{\ln(rt)} = \sqrt{\ln\left(V_{rt}^2 + 1\right)} \qquad \text{(D.18a)}$$

$$Q_\delta = \sigma_{\ln(\delta)} = \sqrt{\ln\left(V_\delta^2 + 1\right)} \qquad \text{(D.18b)}$$

$$Q = \sigma_{\ln(r)} = \sqrt{\ln\left(V_r^2 + 1\right)} \qquad \text{(D.18c)}$$

$$\alpha_{rt} = \frac{Q_{rt}}{Q} \qquad \text{(D.19a)}$$

$$\alpha_\delta = \frac{Q_\delta}{Q} \qquad \text{(D.19b)}$$

where :

k_n is the characteristic fractile factor from table D1 for the case V_X unknown ;
k_∞ is the value of k_n for $n \to \infty$ $[k_\infty = 1{,}64]$;
α_{rt} is the weighting factor for Q_{rt}
α_δ is the weighting factor for Q_δ

NOTE The value of V_δ is to be estimated from the test sample under consideration.

(6) If a large number of tests ($n \geq 100$) is available, the characteristic resistance r_k may be obtained from :

$$r_k = b\, g_{rt}(\underline{X}_m) \exp(-\, k_\infty Q - 0{,}5\, Q^2) \quad \text{(D.20)}$$

D8.3 Standard evaluation procedure (Method (b))

(1) In this case the procedure is the same as in D8.2, excepted that step 7 is adapted by replacing the characteristic fractile factor k_n by the design fractile factor $k_{d,n}$ equal to the product $\alpha_R\beta$ assessed at $0{,}8 \times 3{,}8 = 3{,}04$ as commonly accepted (see Annex C) to obtain the design value r_d of the resistance.

(2) For the case of a limited number of tests the design value r_d should be obtained from :

$$r_d = b g_{rt}(\underline{X}_m) \exp(-k_{d,\infty}\, \alpha_{rt}\, Q_{rt} - k_{d,n}\, \alpha_\delta\, Q_\delta - 0{,}5\, Q^2) \quad \text{(D.21)}$$

where :

$k_{d,n}$ is the design fractile factor from table D2 for the case "V_X unknown" ;
$k_{d,\infty}$ is the value of $k_{d,n}$ for $n \rightarrow \infty$ $[k_{d,\infty} = 3{,}04]$.

NOTE The value of V_δ is to be estimated from the test sample under consideration.

(2) For the case of a large number of tests the design value r_d may be obtained from :

$$r_d = b g_{rt}(\underline{X}_m) \exp(-\, k_{d,\infty} Q - 0{,}5\, Q^2) \quad \text{(D.22)}$$

D8.4 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 :

$$r_k = \eta_k\, r_e \quad \text{(D.23)}$$

where :

η_k is a reduction factor applicable in the case of prior knowledge that may be obtained from :

$$\eta_k = 0{,}9 \exp(-2{,}31\, V_r - 0{,}5\, V_r^2) \quad \text{(D.24)}$$

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 :

$$r_k = \eta_k \, r_{em} \tag{D.25}$$

where :

η_k is a reduction factor applicable in the case of prior knowledge that may be obtained from :

$$\eta_k = \exp(-2{,}0\, V_r - 0{,}5\, V_r^2) \tag{D.26}$$

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 :

$$\left| r_{ee} - r_{em} \right| \leq 0{,}10\, r_{em} \tag{D.27}$$

(4) The values of the coefficient of variation V_r given in table D3 may be assumed for the types of failure to be specified (*e.g.* in the relevant design Eurocode), leading to the listed values of η_k according to expressions (D.24) and (D.26).

Table D3 - 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

Bibliography

ISO 2394	General principles on reliability for structures
ISO 2631:1997	Mechanical vibration and shock - Evaluation of human exposure to whole-body vibration
ISO 3898	Basis for design of structures – Notations - General symbols
ISO 6707−1	Building and civil engineering - Vocabulary - Part 1 : General terms
ISO 8930	General principles on reliability for structures - List of equivalent terms
EN ISO 9001:2000	Quality management systems – Requirements (ISO 9001:2000)
ISO 10137	Basis for design of structures – Serviceability of buildings against vibrations
ISO 8402	Quality management and quality assurance – Vocabulary

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