

Eurocode 2: Design of concrete structures —

Part 1-1: General rules and rules for buildings

ICS 91.010.30; 91.080.40

National foreword

This British Standard is the UK implementation of EN 1992-1-1:2004+A1:2014, incorporating corrigendum January 2008 and November 2010. It supersedes BS EN 1992-1-1:2004 which is withdrawn.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. For example, text altered by CEN corrigendum January 2008 is indicated in the text by AC1 AC1 and CEN corrigendum November 2010 by AC2 AC2.

The UK participation in its preparation was entrusted by Technical Committee B/525, Building and civil engineering structures, to Subcommittee B/525/2, Structural use of concrete.

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 1992-1-1 to be used in the UK, the NDPs are published in a National Annex, which is available from BSI.

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.

This British Standard was published under the authority of the Standards Policy and Strategy Committee on 23 December 2004

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Published by BSI Standards Limited 2015

ISBN 978 0 580 83726 5

Amendments/corrigenda issued since publication

Date	Comments
30 June 2008	Implementation of CEN corrigendum January 2008
31 August 2011	Implementation of CEN corrigendum November 2010
31 January 2014	Expressions 6.60, 6.61 and 6.62 corrected
31 July 2015	Implementation of CEN amendment A1:2014

English version

Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings

Eurocode 2: Calcul des structures en béton - Partie 1-1 :
Règles générales et règles pour les bâtiments

Eurocode 2: Bemessung und konstruktion von Stahlbeton-
und Spannbetontragwerken - Teil 1-1: Allgemeine
Bemessungsregeln und Regeln für den Hochbau

This European Standard was approved by CEN on 16 April 2004.

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 Central Secretariat 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 Central Secretariat has the same status as the official versions.

CEN members are the national standards bodies of 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.



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

Management Centre: Avenue Marnix 17, B-1000 Brussels

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Foreword

This European Standard EN 1992, Eurocode 2: Design of concrete structures: General rules and rules for buildings, has been prepared by Technical Committee CEN/TC250 « Structural Eurocodes », the Secretariat of which is held by BSI. CEN/TC250 is responsible for all Structural Eurocodes.

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 June 2005, and conflicting National Standards shall be withdrawn at latest by March 2010.

This Eurocode supersedes ENV 1992-1-1, 1992-1-3, 1992-1-4, 1992-1-5, 1992-1-6 and 1992-3.

According to the CEN-CENELEC Internal Regulations, the National Standard Organisations of the following countries are bound to implement these 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.

Foreword to amendment A1

This document (EN 1992-1-1:2004/A1:2014) has been prepared by Technical Committee CEN/TC 250 “Structural Eurocodes”, the secretariat of which is held by BSI.

This Amendment to the European Standard EN 1992-1-1:2004 shall be given the status of a national standard, either by publication of an identical text or by endorsement, at the latest by December 2015, and conflicting national standards shall be withdrawn at the latest by December 2015.

Attention is drawn to the possibility that some of the elements of this document may be the subject of patent rights. CEN [and/or CENELEC] shall not be held responsible for identifying any or all such patent rights.

This document has been prepared under a mandate given to CEN by the European Commission and the European Free Trade Association.

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

Background to 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 national rules 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 1980s.

In 1989, the Commission and the Member States of the EU and EFTA decided, on the basis of an agreement¹ 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 Council Directives 93/37/EEC, 92/50/EEC and 89/440/EEC on public works and services and equivalent EFTA Directives initiated in pursuit of setting up the internal market).

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).

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

EN 1990	Eurocode 0:	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.

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 Documents² referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standards³. 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 with a view to achieving 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 component products of both a traditional and an innovative 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.

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.

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.* :

- 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 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 technical rules for works⁴. Furthermore, all the information accompanying the CE Marking of the construction products which refer to Eurocodes should clearly mention which Nationally Determined Parameters have been taken into account.

Additional information specific to EN 1992-1-1

EN 1992-1-1 describes the principles and requirements for safety, serviceability and durability of concrete structures, together with specific provisions for buildings. It is based on the limit state concept used in conjunction with a partial factor method.

For the design of new structures, EN 1992-1-1 is intended to be used, for direct application, together with other parts of EN 1992, Eurocodes EN 1990, 1991, 1997 and 1998.

EN 1992-1-1 also serves as a reference document for other CEN TCs concerning structural matters.

EN 1992-1-1 is intended for use by:

- committees drafting other 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.

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 1992-1-1 is used as a base document by other CEN/TCs the same values need to be taken.

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

National annex for EN 1992-1-1

This standard gives values with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1992-1-1 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.

National choice is allowed in EN 1992-1-1 through the following clauses:

2.3.3 (3)	5.10.3 (2)	9.2.2 (7)
2.4.2.1 (1)	5.10.8 (2)	9.2.2 (8)
2.4.2.2 (1)	5.10.8 (3)	9.3.1.1(3)
2.4.2.2 (2)	5.10.9 (1)P	9.5.2 (1)
2.4.2.2 (3)	6.2.2 (1)	9.5.2 (2)
2.4.2.3 (1)	6.2.2 (6)	9.5.2 (3)
2.4.2.4 (1)	6.2.3 (2)	9.5.3 (3)
2.4.2.4 (2)	6.2.3 (3)	9.6.2 (1)
2.4.2.5 (2)	6.2.4 (4)	9.6.3 (1)
3.1.2 (2)P	6.2.4 (6)	9.7 (1)
3.1.2 (4)	6.4.3 (6)	9.8.1 (3)
3.1.6 (1)P	6.4.4 (1)	9.8.2.1 (1)
3.1.6 (2)P	ⓘ A1 6.4.5 (1) ⓘ A1	9.8.3 (1)
3.2.2 (3)P	6.4.5 (3)	9.8.3 (2)
3.2.7 (2)	6.4.5 (4)	9.8.4 (1)
3.3.4 (5)	6.5.2 (2)	9.8.5 (3)
3.3.6 (7)	6.5.4 (4)	9.10.2.2 (2)
4.4.1.2 (3)	6.5.4 (6)	9.10.2.3 (3)
4.4.1.2 (5)	6.8.4 (1)	9.10.2.3 (4)
4.4.1.2 (6)	6.8.4 (5)	9.10.2.4 (2)
4.4.1.2 (7)	6.8.6 (1)	11.3.5 (1)P
4.4.1.2 (8)	ⓘ AC1 6.8.6 (3) ⓘ AC1	11.3.5 (2)P
4.4.1.2 (13)	6.8.7 (1)	11.3.7 (1)
4.4.1.3 (1)P	7.2 (2)	11.6.1 (1)
4.4.1.3 (3)	7.2 (3)	11.6.1 (2)
4.4.1.3 (4)	7.2 (5)	11.6.2 (1)
5.1.3 (1)P	7.3.1 (5)	11.6.4.1 (1)
5.2 (5)	7.3.2 (4)	12.3.1 (1)
5.5 (4)	7.3.4 (3)	12.6.3 (2)
5.6.3 (4)	7.4.2 (2)	A.2.1 (1)
5.8.3.1 (1)	8.2 (2)	A.2.1 (2)
5.8.3.3 (1)	8.3 (2)	A.2.2 (1)
5.8.3.3 (2)	8.6 (2)	A.2.2 (2)
5.8.5 (1)	8.8 (1)	A.2.3 (1)
5.8.6 (3)	9.2.1.1 (1)	C.1 (1)
5.10.1 (6)	9.2.1.1 (3)	C.1 (3)
5.10.2.1 (1)P	9.2.1.2 (1)	E.1 (2)
5.10.2.1 (2)	9.2.1.4 (1)	ⓘ AC1 J.1 (2) ⓘ AC1
5.10.2.2 (4)	9.2.2 (4)	J.2.2 (2)
5.10.2.2 (5)	9.2.2 (5)	J.3 (2)
	9.2.2 (6)	J.3 (3)

SECTION 1 GENERAL

1.1 Scope

1.1.1 Scope of Eurocode 2

(1)P Eurocode 2 applies to the design of buildings and civil engineering works in plain, reinforced and prestressed concrete. It complies with the principles and requirements for the safety and serviceability of structures, the basis of their design and verification that are given in EN 1990: Basis of structural design.

(2)P Eurocode 2 is only concerned with the requirements for resistance, serviceability, durability and fire resistance of concrete structures. Other requirements, e.g. concerning thermal or sound insulation, are not considered.

(3)P Eurocode 2 is intended to be used in conjunction with:

EN 1990: Basis of structural design
EN 1991: Actions on structures
hEN's: Construction products relevant for concrete structures
ENV 13670: Execution of concrete structures
EN 1997: Geotechnical design
EN 1998: Design of structures for earthquake resistance, when concrete structures are built in seismic regions.

(4)P Eurocode 2 is subdivided into the following parts:

Part 1.1: General rules and rules for buildings
Part 1.2: Structural fire design
Part 2: Reinforced and prestressed concrete bridges
Part 3: Liquid retaining and containing structures

1.1.2 Scope of Part 1-1 of Eurocode 2

(1)P Part 1-1 of Eurocode 2 gives a general basis for the design of structures in plain, reinforced and prestressed concrete made with normal and light weight aggregates together with specific rules for buildings.

(2)P The following subjects are dealt with in Part 1-1.

Section 1: General
Section 2: Basis of design
Section 3: Materials
Section 4: Durability and cover to reinforcement
Section 5: Structural analysis
Section 6: Ultimate limit states
Section 7: Serviceability limit states
Section 8: Detailing of reinforcement and prestressing tendons - General
Section 9: Detailing of members and particular rules

Section 10: Additional rules for precast concrete elements and structures

Section 11: Lightweight aggregate concrete structures

Section 12: Plain and lightly reinforced concrete structures

(3)P Sections 1 and 2 provide additional clauses to those given in EN 1990 “Basis of structural design”.

(4)P This Part 1-1 does not cover:

- the use of plain reinforcement
- resistance to fire;
- particular aspects of special types of building (such as tall buildings);
- particular aspects of special types of civil engineering works (such as viaducts, bridges, dams, pressure vessels, offshore platforms or liquid-retaining structures);
- no-fines concrete and aerated concrete components, and those made with heavy aggregate or containing structural steel sections (see Eurocode 4 for composite steel-concrete structures).

1.2 Normative references

(1)P The following normative documents contain provisions which, through references in this text, constitutive provisions of this European standard. For dated references, subsequent amendments to or revisions of any of these publications do not apply. However, parties to agreements based on this European standard are encouraged to investigate the possibility of applying the most recent editions of the normative documents indicated below. For undated references the latest edition of the normative document referred to applies.

1.2.1 General reference standards

EN 1990:	Basis of structural design
EN 1991-1-5:	Actions on structures: Thermal actions
EN 1991-1-6:	Actions on structures: Actions during execution

1.2.2 Other reference standards

EN1997:	Geotechnical design
EN 197-1:	Cement: Composition, specification and conformity criteria for common cements
EN 206-1:	Concrete: Specification, performance, production and conformity
EN 12390:	Testing hardened concrete
EN 10080:	Steel for the reinforcement of concrete
EN 10138:	Prestressing steels
EN ISO 17660 (all parts):	Welding – Welding of reinforcing steel
ENV 13670:	Execution of concrete structures
EN 13791:	Testing concrete
EN ISO 15630	Steel for the reinforcement and prestressing of concrete: Test methods

1.3 Assumptions

(1)P In addition to the general assumptions of EN 1990 the following assumptions apply:

- Structures are designed by appropriately qualified and experienced personnel.

- Adequate supervision and quality control is provided in factories, in plants, and on site.
- Construction is carried out by personnel having the appropriate skill and experience.
- The construction materials and products are used as specified in this Eurocode or in the relevant material or product specifications.
- The structure will be adequately maintained.
- The structure will be used in accordance with the design brief.
- The requirements for execution and workmanship given in ENV 13670 are complied with.

1.4 Distinction between principles and application rules

(1)P The rules given in EN 1990 apply.

1.5 Definitions

1.5.1 General

(1)P The terms and definitions given in EN 1990 apply.

1.5.2 Additional terms and definitions used in this Standard

1.5.2.1 Precast structures. Precast structures are characterised by structural elements manufactured elsewhere than in the final position in the structure. In the structure, elements are connected to ensure the required structural integrity.

1.5.2.2 Plain or lightly reinforced concrete members. Structural concrete members having no reinforcement (plain concrete) or less reinforcement than the minimum amounts defined in Section 9.

1.5.2.3 Unbonded and external tendons. Unbonded tendons for post-tensioned members having ducts which are permanently ungrouted, and tendons external to the concrete cross-section (which may be encased in concrete after stressing, or have a protective membrane).

1.5.2.4 Prestress. The process of prestressing consists in applying forces to the concrete structure by stressing tendons relative to the concrete member. "Prestress" is used globally to name all the permanent effects of the prestressing process, which comprise internal forces in the sections and deformations of the structure. Other means of prestressing are not considered in this standard.

1.6 Symbols

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

Note: The notation used is based on ISO 3898:1987

Latin upper case letters

A	Accidental action
A	Cross sectional area
A_c	Cross sectional area of concrete
A_p	Area of a prestressing tendon or tendons

A_s	Cross sectional area of reinforcement
$A_{s,min}$	minimum cross sectional area of reinforcement
A_{sw}	Cross sectional area of shear reinforcement
D	Diameter of mandrel
D_{Ed}	Fatigue damage factor
E	Effect of action
$E_c, E_{c(28)}$	Tangent modulus of elasticity of normal weight concrete at a stress of $\sigma_c = 0$ and at 28 days
$E_{c,eff}$	Effective modulus of elasticity of concrete
E_{cd}	Design value of modulus of elasticity of concrete
E_{cm}	Secant modulus of elasticity of concrete
$E_c(t)$	Tangent modulus of elasticity of normal weight concrete at a stress of $\sigma_c = 0$ and at time t
E_p	Design value of modulus of elasticity of prestressing steel
E_s	Design value of modulus of elasticity of reinforcing steel
EI	Bending stiffness
EQU	Static equilibrium
F	Action
F_d	Design value of an action
F_k	Characteristic value of an action
G_k	Characteristic permanent action
I	Second moment of area of concrete section
L	Length
M	Bending moment
M_{Ed}	Design value of the applied internal bending moment
N	Axial force
N_{Ed}	Design value of the applied axial force (tension or compression)
P	Prestressing force
P_0	Initial force at the active end of the tendon immediately after stressing
Q_k	Characteristic variable action
Q_{fat}	Characteristic fatigue load
R	Resistance
S	Internal forces and moments
S	First moment of area
SLS	Serviceability limit state
T	Torsional moment
T_{Ed}	Design value of the applied torsional moment
ULS	Ultimate limit state
V	Shear force
V_{Ed}	Design value of the applied shear force

Latin lower case letters

a	Distance
a	Geometrical data
Δa	Deviation for geometrical data
b	Overall width of a cross-section, or actual flange width in a T or L beam
b_w	Width of the web on T, I or L beams
d	Diameter ; Depth
d	Effective depth of a cross-section

d_g	Largest nominal maximum aggregate size
e	Eccentricity
f_c	Compressive strength of concrete
f_{cd}	Design value of concrete compressive strength
f_{ck}	Characteristic compressive cylinder strength of concrete at 28 days
f_{cm}	Mean value of concrete cylinder compressive strength
f_{ctk}	Characteristic axial tensile strength of concrete
f_{ctm}	Mean value of axial tensile strength of concrete
f_p	Tensile strength of prestressing steel
f_{pk}	Characteristic tensile strength of prestressing steel
$f_{p0,1}$	0,1% proof-stress of prestressing steel
$f_{p0,1k}$	Characteristic 0,1% proof-stress of prestressing steel
$f_{0,2k}$	Characteristic 0,2% proof-stress of reinforcement
f_t	Tensile strength of reinforcement
f_{tk}	Characteristic tensile strength of reinforcement
f_y	Yield strength of reinforcement
f_{yd}	Design yield strength of reinforcement
f_{yk}	Characteristic yield strength of reinforcement
f_{ywd}	Design yield of shear reinforcement
h	Height
h	Overall depth of a cross-section
i	Radius of gyration
k	Coefficient ; Factor
l	(or l or L) Length; Span
m	Mass
r	Radius
$1/r$	Curvature at a particular section
t	Thickness
t	Time being considered
t_0	The age of concrete at the time of loading
u	Perimeter of concrete cross-section, having area A_c
u, v, w	Components of the displacement of a point
x	Neutral axis depth
x, y, z	Coordinates
z	Lever arm of internal forces

Greek lower case letters

α	Angle ; ratio
β	Angle ; ratio; coefficient
γ	Partial factor
γ_A	Partial factor for accidental actions A
γ_C	Partial factor for concrete
γ_F	Partial factor for actions, F
$\gamma_{F,fat}$	Partial factor for fatigue actions
$\gamma_{C,fat}$	Partial factor for fatigue of concrete
γ_G	Partial factor for permanent actions, G
γ_M	Partial factor for a material property, taking account of uncertainties in the material property itself, in geometric deviation and in the design model used

γ_P	Partial factor for actions associated with prestressing, P
γ_Q	Partial factor for variable actions, Q
γ_S	Partial factor for reinforcing or prestressing steel
$\gamma_{S,fat}$	Partial factor for reinforcing or prestressing steel under fatigue loading
γ_f	Partial factor for actions without taking account of model uncertainties
γ_g	Partial factor for permanent actions without taking account of model uncertainties
γ_m	Partial factors for a material property, taking account only of uncertainties in the material property
δ	Increment/redistribution ratio
ζ	Reduction factor/distribution coefficient
ε_c	Compressive strain in the concrete
ε_{c1}	Compressive strain in the concrete at the peak stress f_c
ε_{cu}	Ultimate compressive strain in the concrete
ε_u	Strain of reinforcement or prestressing steel at maximum load
ε_{uk}	Characteristic strain of reinforcement or prestressing steel at maximum load
θ	Angle
λ	Slenderness ratio
μ	Coefficient of friction between the tendons and their ducts
ν	Poisson's ratio
ν	Strength reduction factor for concrete cracked in shear
ξ	Ratio of bond strength of prestressing and reinforcing steel
ρ	Oven-dry density of concrete in kg/m^3
ρ_{1000}	Value of relaxation loss (in %), at 1000 hours after tensioning and at a mean temperature of 20°C
ρ_l	Reinforcement ratio for longitudinal reinforcement
ρ_w	Reinforcement ratio for shear reinforcement
σ_c	Compressive stress in the concrete
σ_{cp}	Compressive stress in the concrete from axial load or prestressing
σ_{cu}	Compressive stress in the concrete at the ultimate compressive strain ε_{cu}
τ	Torsional shear stress
ϕ	Diameter of a reinforcing bar or of a prestressing duct
ϕ_n	Equivalent diameter of a bundle of reinforcing bars
$\varphi(t, t_0)$	Creep coefficient, defining creep between times t and t_0 , related to elastic deformation at 28 days
$\varphi(\infty, t_0)$	Final value of creep coefficient
ψ	Factors defining representative values of variable actions
ψ_0	for combination values
ψ_1	for frequent values
ψ_2	for quasi-permanent values

SECTION 2 BASIS OF DESIGN

2.1 Requirements

2.1.1 Basic requirements

(1)P The design of concrete structures shall be in accordance with the general rules given in EN 1990.

(2)P The supplementary provisions for concrete structures given in this section shall also be applied.

(3) The basic requirements of EN 1990 Section 2 are deemed to be satisfied for concrete structures when the following are applied together:

- limit state design in conjunction with the partial factor method in accordance with EN 1990,
- actions in accordance with EN 1991,
- combination of actions in accordance with EN 1990 and
- resistances, durability and serviceability in accordance with this Standard.

Note: Requirements for fire resistance (see EN 1990 Section 5 and EN 1992-1-2) may dictate a greater size of member than that required for structural resistance at normal temperature.

2.1.2 Reliability management

(1) The rules for reliability management are given in EN 1990 Section 2.

(2) A design using the partial factors given in this Eurocode (see 2.4) and the partial factors given in the EN 1990 annexes is considered to lead to a structure associated with reliability Class RC2.

Note: For further information see EN 1990 Annexes B and C.

2.1.3 Design working life, durability and quality management

(1) The rules for design working life, durability and quality management are given in EN 1990 Section 2.

2.2 Principles of limit state design

(1) The rules for limit state design are given in EN 1990 Section 3.

2.3 Basic variables

2.3.1 Actions and environmental influences

2.3.1.1 General

(1) Actions to be used in design may be obtained from the relevant parts of EN 1991.

Note 1: The relevant parts of EN1991 for use in design include:

EN 1991-1.1	Densities, self-weight and imposed loads
EN 1991-1.2	Fire actions
EN 1991-1.3	Snow loads
EN 1991-1.4	Wind loads
EN 1991-1.5	Thermal actions
EN 1991-1.6	Actions during execution
EN 1991-1.7	Accidental actions due to impact and explosions
EN 1991-2	Traffic loads on bridges
EN 1991-3	Actions induced by cranes and other machinery
EN 1991-4	Actions in silos and tanks

Note 2: Actions specific to this Standard are given in the relevant sections.

Note 3: Actions from earth and water pressure may be obtained from EN 1997.

Note 4: When differential movements are taken into account, appropriate estimate values of predicted movements may be used.

Note 5: Other actions, when relevant, may be defined in the design specification for a particular project.

2.3.1.2 Thermal effects

- (1) Thermal effects should be taken into account when checking serviceability limit states.
- (2) Thermal effects should be considered for ultimate limit states only where they are significant (e.g. fatigue conditions, in the verification of stability where second order effects are of importance, etc). In other cases they need not be considered, provided that the ductility and rotation capacity of the elements are sufficient.
- (3) Where thermal effects are taken into account they should be considered as variable actions and applied with a partial factor and ψ factor.

Note: The ψ factor is defined in the relevant annex of EN 1990 and EN 1991-1-5.

2.3.1.3 Differential settlements/movements

- (1) Differential settlements/movements of the structure due to soil subsidence should be classified as a permanent action, G_{set} which is introduced as such in combinations of actions. In general, G_{set} is represented by a set of values corresponding to differences (compared to a reference level) of settlements/movements between individual foundations or part of foundations, $d_{\text{set},i}$ (i denotes the number of the individual foundation or part of foundation).

Note: Where differential settlements are taken into account, appropriate estimate values of predicted settlements may be used.

- (2) The effects of differential settlements should generally be taken into account for the verification for serviceability limit states.
- (3) For ultimate limit states they should be considered only where they are significant (e.g. fatigue conditions, in the verification of stability where second order effects are of importance, etc). In other cases for ultimate limit states they need not be considered, provided that the ductility and rotation capacity of the elements are sufficient.
- (4) Where differential settlements are taken into account a partial safety factor for settlement effects should be applied.

Note: The value of the partial safety factor for settlement effects is defined in the relevant annex of EN1990.

2.3.1.4 Prestress

- (1)P The prestress considered in this Eurocode is applied by tendons made of high-strength steel (wires, strands or bars).
- (2) Tendons may be embedded in the concrete. They may be pre-tensioned and bonded or post-tensioned and bonded or unbonded.
- (3) Tendons may also be external to the structure with points of contact occurring at deviators and anchorages.
- (4) Provisions concerning prestress are found in 5.10.

2.3.2 Material and product properties

2.3.2.1 General

- (1) The rules for material and product properties are given in EN 1990 Section 4.
- (2) Provisions for concrete, reinforcement and prestressing steel are given in Section 3 or the relevant Product Standard.

2.3.2.2 Shrinkage and creep

- (1) Shrinkage and creep are time-dependent properties of concrete. Their effects should generally be taken into account for the verification of serviceability limit states.
- (2) The effects of shrinkage and creep should be considered at ultimate limit states only where their effects are significant, for example in the verification of ultimate limit states of stability where second order effects are of importance. In other cases these effects need not be considered for ultimate limit states, provided that ductility and rotation capacity of the elements are sufficient.
- (3) When creep is taken into account its design effects should be evaluated under the quasi-permanent combination of actions irrespective of the design situation considered i.e. persistent, transient or accidental.

Note: In most cases the effects of creep may be evaluated under permanent loads and the mean value of prestress.

2.3.3 Deformations of concrete

- (1)P The consequences of deformation due to temperature, creep and shrinkage shall be considered in design.
- (2) The influence of these effects is normally accommodated by complying with the application rules of this Standard. Consideration should also be given to:
 - minimising deformation and cracking due to early-age movement, creep and shrinkage through the composition of the concrete mix;
 - minimising restraints to deformation by the provision of bearings or joints;

- if restraints are present, ensuring that their influence is taken into account in design.

(3) In building structures, temperature and shrinkage effects may be omitted in global analysis provided joints are incorporated at every distance d_{joint} to accommodate resulting deformations.

Note: The value of d_{joint} is subject to a National Annex. The recommended value is 30 m. For precast concrete structures the value may be larger than that for cast in-situ structures, since part of the creep and shrinkage takes place before erection.

2.3.4 Geometric data

2.3.4.1 General

(1) The rules for geometric data are given in EN 1990 Section 4.

2.3.4.2 Supplementary requirements for cast in place piles

(1)P Uncertainties related to the cross-section of cast in place piles and concreting procedures shall be allowed for in design.

(2) In the absence of other provisions the diameter used in design calculations, of cast in place piles without permanent casing should be taken as:

- if $d_{\text{nom}} < 400$ mm $d = d_{\text{nom}} - 20$ mm
- if $400 \leq d_{\text{nom}} \leq 1000$ mm $d = 0,95 \cdot d_{\text{nom}}$
- if $d_{\text{nom}} > 1000$ mm $d = d_{\text{nom}} - 50$ mm

Where d_{nom} is the nominal diameter of the pile.

2.4 Verification by the partial factor method

2.4.1 General

(1) The rules for the partial factor method are given in EN 1990 Section 6.

2.4.2 Design values

2.4.2.1 Partial factor for shrinkage action

(1) Where consideration of shrinkage actions is required for ultimate limit state a partial factor, γ_{SH} , should be used.

Note: The value of γ_{SH} for use in a Country may be found in its National Annex. The recommended value is 1,0.

2.4.2.2 Partial factors for prestress

(1) Prestress in most situations is intended to be favourable and for the ultimate limit state verification the value of $\gamma_{\text{P,fav}}$ should be used. The design value of prestress may be based on the mean value of the prestressing force (see EN 1990 Section 4).

Note: The value of $\gamma_{\text{P,fav}}$ for use in a Country may be found in its National Annex. The recommended value for persistent and transient design situations is 1,0. This value may also be used for fatigue verification.

(2) In the verification of the limit state for stability with external prestress, where an increase of the value of prestress can be unfavourable, $\gamma_{P,unfav}$ should be used.

Note: The value of $\gamma_{P,unfav}$ in the stability limit state for use in a Country may be found in its National Annex. The recommended value for global analysis is 1,3.

(3) In the verification of local effects $\gamma_{P,unfav}$ should also be used.

Note: The value of $\gamma_{P,unfav}$ for local effects for use in a Country may be found in its National Annex. The recommended value is 1,2. The local effects of the anchorage of pre-tensioned tendons are considered in 8.10.2.

2.4.2.3 Partial factor for fatigue loads

(1) The partial factor for fatigue loads is $\gamma_{F,fat}$.

Note: The value of $\gamma_{F,fat}$ for use in a Country may be found in its National Annex. The recommended value is 1,0.

2.4.2.4 Partial factors for materials

(1) Partial factors for materials for ultimate limit states, γ_C and γ_S should be used.

Note: The values of γ_C and γ_S for use in a Country may be found in its National Annex. The recommended values for 'persistent & transient' and 'accidental, design situations are given in Table 2.1N. These are not valid for fire design for which reference should be made to EN 1992-1-2.

For fatigue verification the partial factors for persistent design situations given in Table 2.1N are recommended for the values of $\gamma_{C,fat}$ and $\gamma_{S,fat}$.

Table 2.1N: Partial factors for materials for ultimate limit states

Design situations	γ_C for concrete	γ_S for reinforcing steel	γ_S for prestressing steel
Persistent & Transient	1,5	1,15	1,15
Accidental	1,2	1,0	1,0

(2) The values for partial factors for materials for serviceability limit state verification should be taken as those given in the particular clauses of this Eurocode.

Note: The values of γ_C and γ_S in the serviceability limit state for use in a Country may be found in its National Annex. The recommended value for situations not covered by particular clauses of this Eurocode is 1,0.

(3) Lower values of γ_C and γ_S may be used if justified by measures reducing the uncertainty in the calculated resistance.

Note: Information is given in Informative Annex A.

2.4.2.5 Partial factors for materials for foundations

(1) Design values of strength properties of the ground should be calculated in accordance with EN 1997.

(2) The partial factor for concrete γ_C given in 2.4.2.4 (1) should be multiplied by a factor, k_f , for calculation of design resistance of cast in place piles without permanent casing.

Note: The value of k_f for use in a Country may be found in its National Annex. The recommended value is 1,1.

2.4.3 Combinations of actions

(1) The general formats for combinations of actions for the ultimate and serviceability limit states are given in EN 1990, Section 6.

Note 1: Detailed expressions for combinations of actions are given in the normative annexes of EN 1990, i.e. Annex A1 for buildings, A2 for bridges, etc. with relevant recommended values for partial factors and representative values of actions given in the notes.

Note 2: Combination of actions for fatigue verification is given in 6.8.3.

(2) For each permanent action either the lower or the upper design value (whichever gives the more unfavourable effect) should be applied throughout the structure (e.g. self-weight in a structure).

Note: There may be some exceptions to this rule (e.g. in the verification of static equilibrium, see EN 1990 Section 6). In such cases a different set of partial factors (Set A) may be used. An example valid for buildings is given in Annex A1 of EN 1990.

2.4.4 Verification of static equilibrium - EQU

(1) The reliability format for the verification of static equilibrium also applies to design situations of EQU, such as holding down devices or the verification of the uplift of bearings for continuous beams.

Note: Information is given in Annex A of EN 1990.

2.5 Design assisted by testing

(1) The design of structures or structural elements may be assisted by testing.

Note: Information is given in Section 5 and Annex D of EN 1990.

2.6 Supplementary requirements for foundations

(1)P Where ground-structure interaction has significant influence on the action effects in the structure, the properties of the soil and the effects of the interaction shall be taken into account in accordance with EN 1997-1.

(2) Where significant differential settlements are likely their influence on the action effects in the structure should be checked.

Note 1: Annex G may be used to model the soil -structure interaction.

Note 2: Simple methods ignoring the effects of ground deformation are normally appropriate for the majority of structural designs.

(3) Concrete foundations should be sized in accordance with EN 1997-1.

(4) Where relevant, the design should include the effects of phenomena such as subsidence, heave, freezing, thawing, erosion, etc.

2.7 Requirements for fastenings

(1) The local and structural effects of fasteners should be considered.

Note: The requirements for the design of fastenings are given in the Technical Specification 'Design of Fastenings for Use in Concrete' (under development). This Technical Specification will cover the design of the following types of fasteners:

cast-in fasteners such as:

- headed anchors,
- channel bars,

and post-installed fasteners such as:

- expansion anchors,
- undercut anchors,
- concrete screws,
- bonded anchors,
- bonded expansion anchors and
- bonded undercut anchors.

The performance of fasteners should comply with the requirements of a CEN Standard or should be demonstrated by a European Technical Approval.

The Technical Specification 'Design of Fastenings' for Use in Concrete' includes the local transmission of loads into the structure.

In the design of the structure the loads and additional design requirements given in Annex A of that Technical Specification should be taken into account.

SECTION 3 MATERIALS

3.1 Concrete

3.1.1 General

(1)P The following clauses give principles and rules for normal and high strength concrete.

(2) Rules for lightweight aggregate concrete are given in Section 11.

3.1.2 Strength

(1)P The compressive strength of concrete is denoted by concrete strength classes which relate to the characteristic (5%) cylinder strength f_{ck} , or the cube strength $f_{ck,cube}$, in accordance with EN 206-1.

(2)P The strength classes in this code are based on the characteristic cylinder strength f_{ck} determined at 28 days with a maximum value of C_{max} .

Note: The value of C_{max} for use in a Country may be found in its National Annex. The recommended value is C90/105.

(3) The characteristic strengths for f_{ck} and the corresponding mechanical characteristics necessary for design, are given in Table 3.1.

(4) In certain situations (e.g. prestressing) it may be appropriate to assess the compressive strength for concrete before or after 28 days, on the basis of test specimens stored under other conditions than prescribed in EN 12390.

If the concrete strength is determined at an age $t > 28$ days the values α_{cc} and α_{ct} defined in 3.1.6 (1)P and 3.1.6 (2)P should be reduced by a factor k_t .

Note: The value of k_t for use in a Country may be found in its National Annex. The recommended value is 0,85.

(5) It may be required to specify the concrete compressive strength, $f_{ck}(t)$, at time t for a number of stages (e.g. demoulding, transfer of prestress), where

$$\begin{aligned} f_{ck}(t) &= f_{cm}(t) - 8 \text{ (MPa)} & \text{for } 3 < t < 28 \text{ days.} \\ f_{ck}(t) &= f_{ck} & \text{for } t \geq 28 \text{ days} \end{aligned}$$

More precise values should be based on tests especially for $t \leq 3$ days

(6) The compressive strength of concrete at an age t depends on the type of cement, temperature and curing conditions. For a mean temperature of 20°C and curing in accordance with EN 12390 the compressive strength of concrete at various ages $f_{cm}(t)$ may be estimated from Expressions (3.1) and (3.2).

$$f_{cm}(t) = \beta_{cc}(t) f_{cm} \tag{3.1}$$

with

$$\beta_{cc}(t) = \exp \left\{ s \left[1 - \left(\frac{28}{t} \right)^{1/2} \right] \right\} \tag{3.2}$$

where:

$f_{cm}(t)$ is the mean concrete compressive strength at an age of t days

- f_{cm} is the mean compressive strength at 28 days according to Table 3.1
 $\beta_{cc}(t)$ is a coefficient which depends on the age of the concrete t
 t is the age of the concrete in days
 s is a coefficient which depends on the type of cement:
 = 0,20 for cement of strength Classes CEM 42,5 R, CEM 52,5 N and CEM 52,5 R (Class R)
 = 0,25 for cement of strength Classes CEM 32,5 R, CEM 42,5 N (Class N)
 = 0,38 for cement of strength Classes CEM 32,5 N (Class S)

Note: $\exp\{ \}$ has the same meaning as $e^{()}$

Where the concrete does not conform with the specification for compressive strength at 28 days the use of Expressions (3.1) and (3.2) is not appropriate.

This clause should not be used retrospectively to justify a non conforming reference strength by a later increase of the strength.

For situations where heat curing is applied to the member see 10.3.1.1 (3).

(7)P The tensile strength refers to the highest stress reached under concentric tensile loading. For the flexural tensile strength reference should be made to 3.1.8 (1).

(8) Where the tensile strength is determined as the splitting tensile strength, $f_{ct,sp}$, an approximate value of the axial tensile strength, f_{ct} , may be taken as:

$$f_{ct} = 0,9f_{ct,sp} \quad (3.3)$$

(9) The development of tensile strength with time is strongly influenced by curing and drying conditions as well as by the dimensions of the structural members. As a first approximation it may be assumed that the tensile strength $f_{ctm}(t)$ is equal to:

$$f_{ctm}(t) = (\beta_{cc}(t))^{\alpha} \cdot f_{ctm} \quad (3.4)$$

where $\beta_{cc}(t)$ follows from Expression (3.2) and

$$\alpha = 1 \text{ for } t < 28$$

$$\alpha = 2/3 \text{ for } t \geq 28. \text{ The values for } f_{ctm} \text{ are given in Table 3.1.}$$

Note: Where the development of the tensile strength with time is important it is recommended that tests are carried out taking into account the exposure conditions and the dimensions of the structural member.

3.1.3 Elastic deformation

(1) The elastic deformations of concrete largely depend on its composition (especially the aggregates). The values given in this Standard should be regarded as indicative for general applications. However, they should be specifically assessed if the structure is likely to be sensitive to deviations from these general values.

(2) The modulus of elasticity of a concrete is controlled by the moduli of elasticity of its components. Approximate values for the modulus of elasticity E_{cm} , secant value between $\sigma_c = 0$ and $0,4f_{cm}$, for concretes with quartzite aggregates, are given in Table 3.1. For limestone and sandstone aggregates the value should be reduced by 10% and 30% respectively. For basalt aggregates the value should be increased by 20%.

Note: A Country's National Annex may refer to non-contradictory complementary information.

Table 3.1 Strength and deformation characteristics for concrete

Strength classes for concrete															Analytical relation / Explanation
f_{ck} (MPa)	12	16	20	25	30	35	40	45	50	55	60	70	80	90	
$f_{ck,cube}$ (MPa)	15	20	25	30	37	45	50	55	60	67	75	85	95	105	2.8
f_{cm} (MPa)	20	24	28	33	38	43	48	53	58	63	68	78	88	98	$f_{cm} = f_{ck}+8(\text{MPa})$
f_{ctm} (MPa)	1,6	1,9	2,2	2,6	2,9	3,2	3,5	3,8	4,1	4,2	4,4	4,6	4,8	5,0	$f_{ctm}=0,30\times f_{ck}^{(2/3)}\leq C50/60$ $f_{ctm}=2,12\ln(1+(f_{cm}/10))$ > C50/60
$f_{ctk,0.05}$ (MPa)	1,1	1,3	1,5	1,8	2,0	2,2	2,5	2,7	2,9	3,0	3,1	3,2	3,4	3,5	$f_{ctk,0.05} = 0,7\times f_{ctm}$ 5% fractile
$f_{ctk,0.95}$ (MPa)	2,0	2,5	2,9	3,3	3,8	4,2	4,6	4,9	5,3	5,5	5,7	6,0	6,3	6,6	$f_{ctk,0.95} = 1,3\times f_{ctm}$ 95% fractile
E_{cm} (GPa)	27	29	30	31	33	34	35	36	37	38	39	41	42	44	$E_{cm} = 22[(f_{cm})/10]^{0.3}$ (f_{cm} in MPa)
ε_{c1} (‰)	1,8	1,9	2,0	2,1	2,2	2,25	2,3	2,4	2,45	2,5	2,6	2,7	2,8	2,8	see Figure 3.2 $\varepsilon_{c1}^{(0)}/\varepsilon_{c1}^{(0)}/100 = 0,7 f_{cm}^{0.31} \leq 2,8$
ε_{cu1} (‰)					3,5					3,2	3,0	2,8	2,8	2,8	see Figure 3.2 for $f_{ck} \geq 50$ Mpa $\varepsilon_{cu1}^{(0)}/\varepsilon_{cu1}^{(0)}/100=2,8+27[(98-f_{cm})/100]^4$
ε_{c2} (‰)					2,0					2,2	2,3	2,4	2,5	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa $\varepsilon_{c2}^{(0)}/\varepsilon_{c2}^{(0)}/100=2,0+0,085(f_{ck}-50)^{0.53}$
ε_{cu2} (‰)					3,5					3,1	2,9	2,7	2,6	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa $\varepsilon_{cu2}^{(0)}/\varepsilon_{cu2}^{(0)}/100=2,6+35[(90-f_{ck})/100]^4$
n					2,0					1,75	1,6	1,45	1,4	1,4	for $f_{ck}\geq 50$ Mpa $n=1,4+23,4[(90-f_{ck})/100]^4$
ε_{c3} (‰)					1,75					1,8	1,9	2,0	2,2	2,3	see Figure 3.4 for $f_{ck}\geq 50$ Mpa $\varepsilon_{c3}^{(0)}/\varepsilon_{c3}^{(0)}/100=1,75+0,55[(f_{ck}-50)/40]$
ε_{cu3} (‰)					3,5					3,1	2,9	2,7	2,6	2,6	see Figure 3.4 for $f_{ck} \geq 50$ Mpa $\varepsilon_{cu3}^{(0)}/\varepsilon_{cu3}^{(0)}/100=2,6+35[(90-f_{ck})/100]^4$

(3) Variation of the modulus of elasticity with time can be estimated by:

$$E_{cm}(t) = (f_{cm}(t) / f_{cm})^{0,3} E_{cm} \quad (3.5)$$

where $E_{cm}(t)$ and $f_{cm}(t)$ are the values at an age of t days and E_{cm} and f_{cm} are the values determined at an age of 28 days. The relation between $f_{cm}(t)$ and f_{cm} follows from Expression (3.1).

(4) Poisson's ratio may be taken equal to 0,2 for uncracked concrete and 0 for cracked concrete.

(5) Unless more accurate information is available, the linear coefficient of thermal expansion may be taken equal to $10 \cdot 10^{-6} \text{ K}^{-1}$.

3.1.4 Creep and shrinkage

(1)P Creep and shrinkage of the concrete depend on the ambient humidity, the dimensions of the element and the composition of the concrete. Creep is also influenced by the maturity of the concrete when the load is first applied and depends on the duration and magnitude of the loading.

(2) The creep coefficient, $\varphi(t, t_0)$ is related to E_c , the tangent modulus, which may be taken as $1,05 E_{cm}$. Where great accuracy is not required, the value found from Figure 3.1 may be considered as the creep coefficient, provided that the concrete is not subjected to a compressive stress greater than $0,45 f_{ck}(t_0)$ at an age t_0 , the age of concrete at the time of loading.

Note: For further information, including the development of creep with time, Annex B may be used.

(3) The creep deformation of concrete $\varepsilon_{cc}(\infty, t_0)$ at time $t = \infty$ for a constant compressive stress σ_c applied at the concrete age t_0 , is given by:

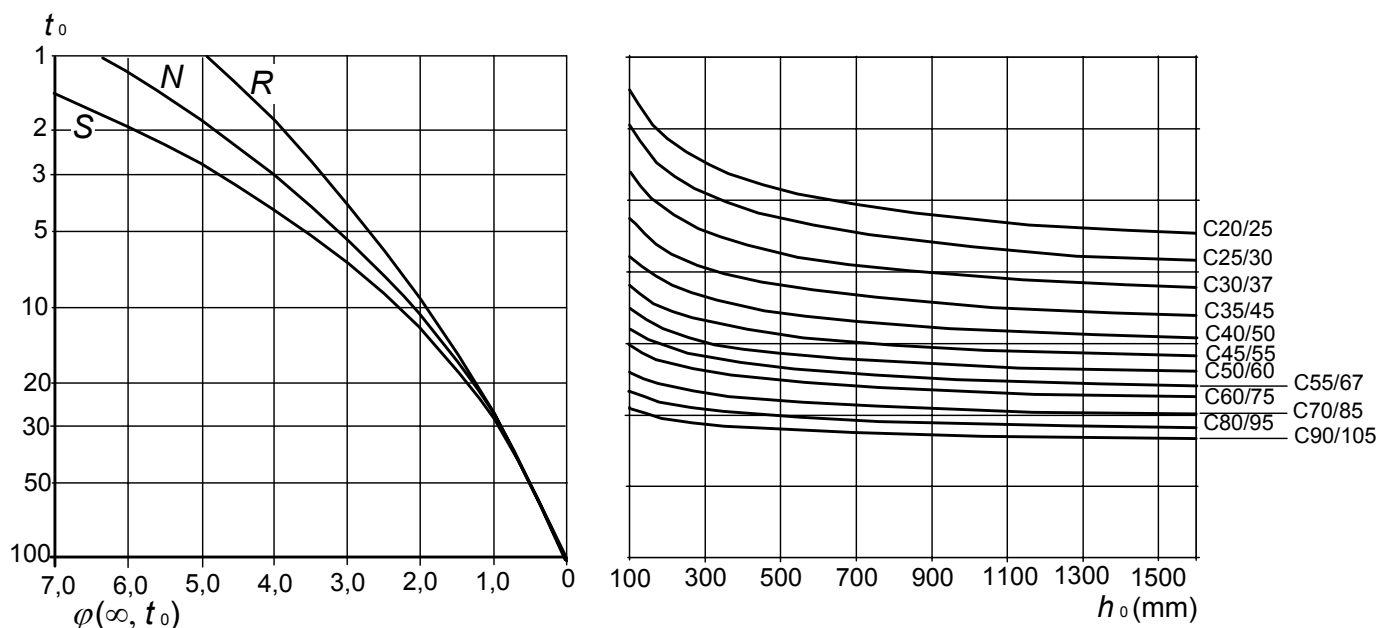
$$\varepsilon_{cc}(\infty, t_0) = \varphi(\infty, t_0) \cdot (\sigma_c / E_c) \quad (3.6)$$

(4) When the compressive stress of concrete at an age t_0 exceeds the value $0,45 f_{ck}(t_0)$ then creep non-linearity should be considered. Such a high stress can occur as a result of pretensioning, e.g. in precast concrete members at tendon level. In such cases the non-linear notional creep coefficient should be obtained as follows:

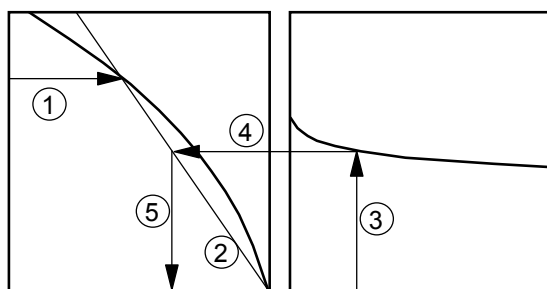
$$\boxed{\text{AC1}} \varphi_{nl}(\infty, t_0) = \varphi(\infty, t_0) \exp(1,5 (k_\sigma - 0,45)) \quad (3.7)$$

where:

$\varphi_{nl}(\infty, t_0)$ is the non-linear notional creep coefficient, which replaces $\varphi(\infty, t_0)$
 k_σ is the stress-strength ratio $\sigma_c / f_{ck}(t_0)$, where σ_c is the compressive stress and $f_{ck}(t_0)$ is the characteristic concrete compressive strength at the time of loading. AC1

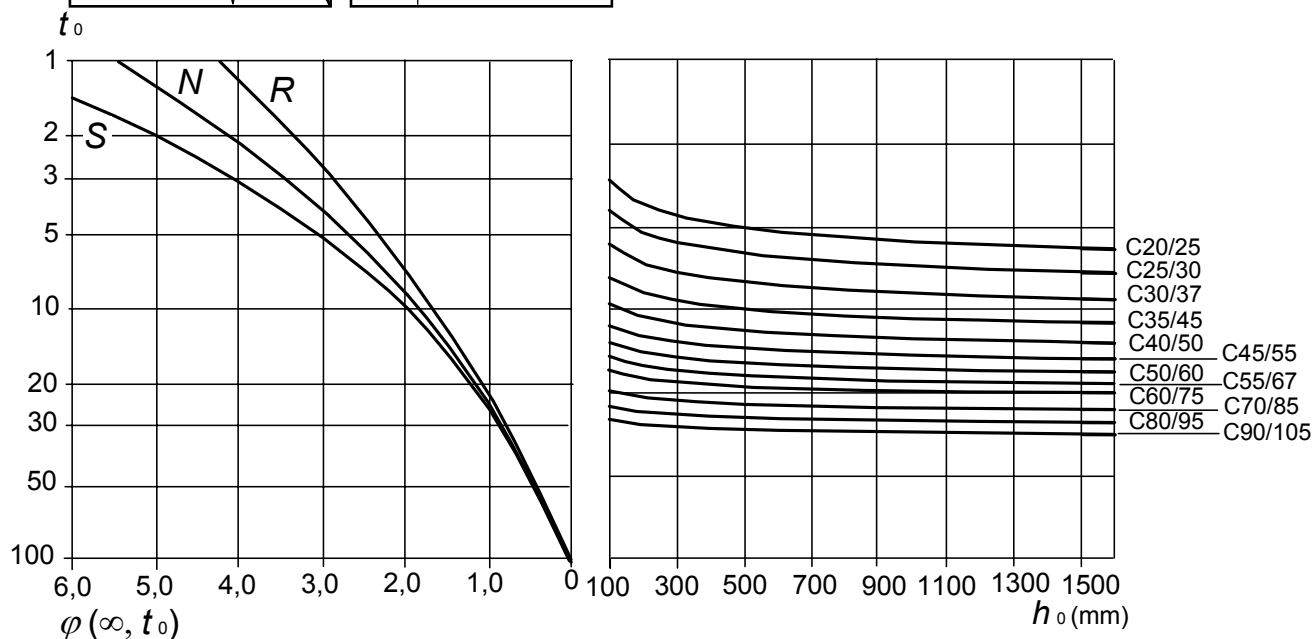


a) inside conditions - RH = 50%



Note:

- intersection point between lines 4 and 5 can also be above point 1
- for $t_0 > 100$ it is sufficiently accurate to assume $t_0 = 100$ (and use the tangent line)



b) outside conditions - RH = 80%

Figure 3.1: Method for determining the creep coefficient $\varphi(\infty, t_0)$ for concrete under normal environmental conditions

(5) The values given in Figure 3.1 are valid for ambient temperatures between -40°C and $+40^{\circ}\text{C}$ and a mean relative humidity between $\text{RH} = 40\%$ and $\text{RH} = 100\%$. The following symbols are used:

- $\varphi(\infty, t_0)$ is the final creep coefficient
- t_0 is the age of the concrete at time of loading in days
- h_0 is the notional size $= 2A_c / u$, where A_c is the concrete cross-sectional area and u is the perimeter of that part which is exposed to drying
- S is Class S, according to 3.1.2 (6)
- N is Class N, according to 3.1.2 (6)
- R is Class R, according to 3.1.2 (6)

(6) The total shrinkage strain is composed of two components, the drying shrinkage strain and the autogenous shrinkage strain. The drying shrinkage strain develops slowly, since it is a function of the migration of the water through the hardened concrete. The autogenous shrinkage strain develops during hardening of the concrete: the major part therefore develops in the early days after casting. Autogenous shrinkage is a linear function of the concrete strength. It should be considered specifically when new concrete is cast against hardened concrete. Hence the values of the total shrinkage strain ε_{cs} follow from

$$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca} \quad (3.8)$$

where:

- ε_{cs} is the total shrinkage strain
- ε_{cd} is the drying shrinkage strain
- ε_{ca} is the autogenous shrinkage strain

The final value of the drying shrinkage strain, $\varepsilon_{cd,\infty}$ is equal to $k_h \cdot \varepsilon_{cd,0}$. $\varepsilon_{cd,0}$ may be taken from Table 3.2 (expected mean values, with a coefficient of variation of about 30%).

Note: The formula for $\varepsilon_{cd,0}$ is given in Annex B.

Table 3.2 Nominal unrestrained drying shrinkage values $\varepsilon_{cd,0}$ (in ‰) for concrete with cement CEM Class N

$f_{ck}/f_{ck,cube}$ (MPa)	Relative Humidity (in ‰)					
	20	40	60	80	90	100
20/25	0.62	0.58	0.49	0.30	0.17	0.00
40/50	0.48	0.46	0.38	0.24	0.13	0.00
60/75	0.38	0.36	0.30	0.19	0.10	0.00
80/95	0.30	0.28	0.24	0.15	0.08	0.00
90/105	0.27	0.25	0.21	0.13	0.07	0.00

The development of the drying shrinkage strain in time follows from:

$$\varepsilon_{cd}(t) = \beta_{ds}(t, t_s) \cdot k_h \cdot \varepsilon_{cd,0} \quad (3.9)$$

where

- k_h is a coefficient depending on the notional size h_0 according to Table 3.3

Table 3.3 Values for k_h in Expression (3.9)

h_0	k_h
100	1.0
200	0.85
300	0.75
≥ 500	0.70

$$\beta_{ds}(t, t_s) = \frac{(t - t_s)}{(t - t_s) + 0,04\sqrt{h_0^3}} \quad (3.10)$$

where:

t is the age of the concrete at the moment considered, in days

t_s is the age of the concrete (days) at the beginning of drying shrinkage (or swelling).
Normally this is at the end of curing.

h_0 is the notional size (mm) of the cross-section
 $= 2A_c/u$

where:

A_c is the concrete cross-sectional area

u is the perimeter of that part of the cross section which is exposed to drying

The autogenous shrinkage strain follows from:

$$\varepsilon_{ca}(t) = \beta_{as}(t) \varepsilon_{ca}(\infty) \quad (3.11)$$

where:

$$\varepsilon_{ca}(\infty) = 2,5 (f_{ck} - 10) 10^{-6} \quad (3.12)$$

and

$$\beta_{as}(t) = 1 - \exp(-0,2t^{0,5}) \quad (3.13)$$

where t is given in days.

3.1.5 Stress-strain relation for non-linear structural analysis

(1) The relation between σ_c and ε_c shown in Figure 3.2 (compressive stress and shortening strain shown as absolute values) for short term uniaxial loading is described by the Expression (3.14):

$$\frac{\sigma_c}{f_{cm}} = \frac{k\eta - \eta^2}{1 + (k - 2)\eta} \quad (3.14)$$

where:

$$\eta = \varepsilon_c / \varepsilon_{c1}$$

ε_{c1} is the strain at peak stress according to Table 3.1

$$k = 1,05 E_{cm} \times |\varepsilon_{c1}| / f_{cm} \quad (f_{cm} \text{ according to Table 3.1})$$

Expression (3.14) is valid for $0 < |\varepsilon_c| < |\varepsilon_{cu1}|$ where ε_{cu1} is the nominal ultimate strain.

(2) Other idealised stress-strain relations may be applied, if they adequately represent the behaviour of the concrete considered.

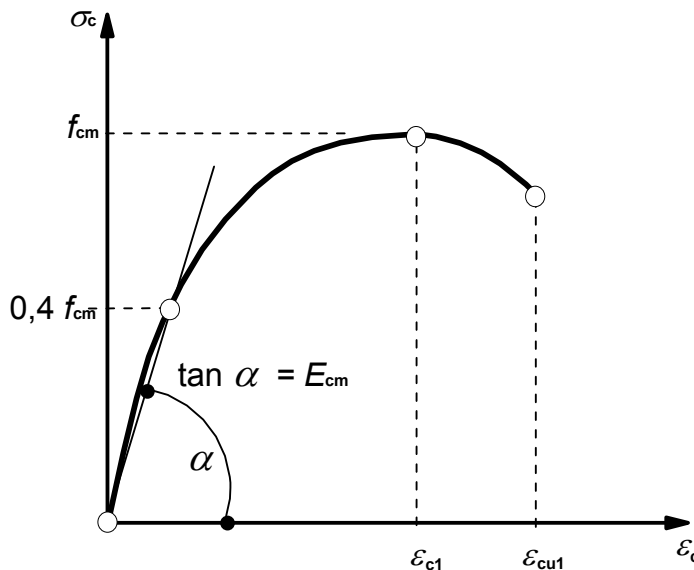


Figure 3.2: Schematic representation of the stress-strain relation for structural analysis (the use $0,4f_{cm}$ for the definition of E_{cm} is approximate).

3.1.6 Design compressive and tensile strengths

(1)P The value of the design compressive strength is defined as

$$f_{cd} = \alpha_{cc} f_{ck} / \gamma_C \quad (3.15)$$

where:

γ_C is the partial safety factor for concrete, see 2.4.2.4, and

α_{cc} is the coefficient taking account of long term effects on the compressive strength and of unfavourable effects resulting from the way the load is applied.

Note: The value of α_{cc} for use in a Country should lie between 0,8 and 1,0 and may be found in its National Annex. The recommended value is 1.

(2)P The value of the design tensile strength, f_{ctd} , is defined as

$$f_{ctd} = \alpha_{ct} f_{ctk,0,05} / \gamma_C \quad (3.16)$$

where:

γ_C is the partial safety factor for concrete, see 2.4.2.4, and

α_{ct} is a coefficient taking account of long term effects on the tensile strength and of unfavourable effects, resulting from the way the load is applied.

Note: The value of α_{ct} for use in a Country may be found in its National Annex. The recommended value is 1,0.

3.1.7 Stress-strain relations for the design of cross-sections

(1) For the design of cross-sections, the following stress-strain relationship may be used, see Figure 3.3 (compressive strain shown positive):

$$\sigma_c = f_{cd} \left[1 - \left(1 - \frac{\varepsilon_c}{\varepsilon_{c2}} \right)^n \right] \quad \text{for } 0 \leq \varepsilon_c \leq \varepsilon_{c2} \quad (3.17)$$

$$\sigma_c = f_{cd} \quad \text{for } \varepsilon_{c2} \leq \varepsilon_c \leq \varepsilon_{cu2} \quad (3.18)$$

where:

n is the exponent according to Table 3.1

ε_{c2} is the strain at reaching the maximum strength according to Table 3.1

ε_{cu2} is the ultimate strain according to Table 3.1

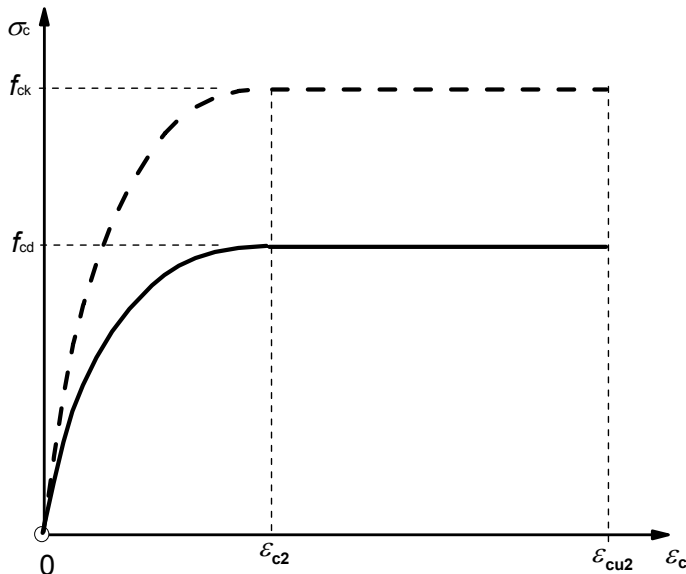


Figure 3.3: Parabola-rectangle diagram for concrete under compression.

(2) Other simplified stress-strain relationships may be used if equivalent to or more conservative than the one defined in (1), for instance bi-linear according to Figure 3.4 (compressive stress and shortening strain shown as absolute values) with values of ε_{c3} and ε_{cu3} according to Table 3.1.

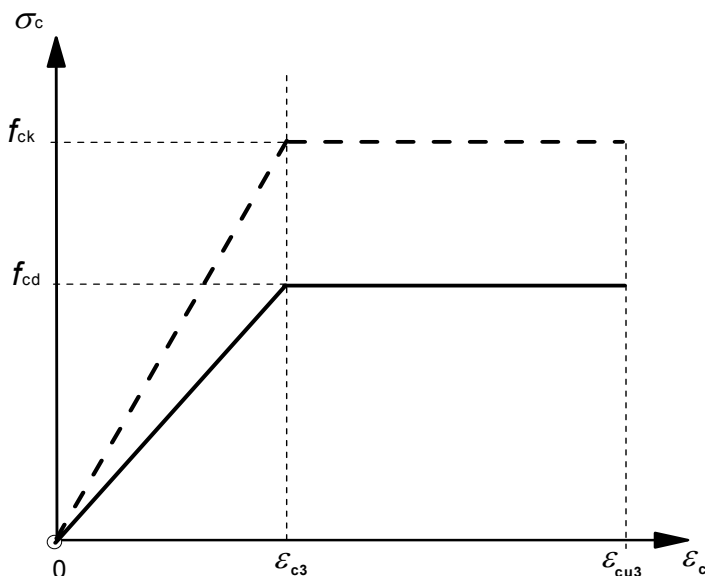


Figure 3.4: Bi-linear stress-strain relation.

(3) A rectangular stress distribution (as given in Figure 3.5) may be assumed. The factor λ ,

defining the effective height of the compression zone and the factor η , defining the effective strength, follow from:

$$\lambda = 0,8 \quad \text{for } f_{ck} \leq 50 \text{ MPa} \quad (3.19)$$

$$\lambda = 0,8 - (f_{ck} - 50)/400 \quad \text{for } 50 < f_{ck} \leq 90 \text{ MPa} \quad (3.20)$$

and

$$\eta = 1,0 \quad \text{for } f_{ck} \leq 50 \text{ MPa} \quad (3.21)$$

$$\eta = 1,0 - (f_{ck} - 50)/200 \quad \text{for } 50 < f_{ck} \leq 90 \text{ MPa} \quad (3.22)$$

Note: If the width of the compression zone decreases in the direction of the extreme compression fibre, the value ηf_{cd} should be reduced by 10%.

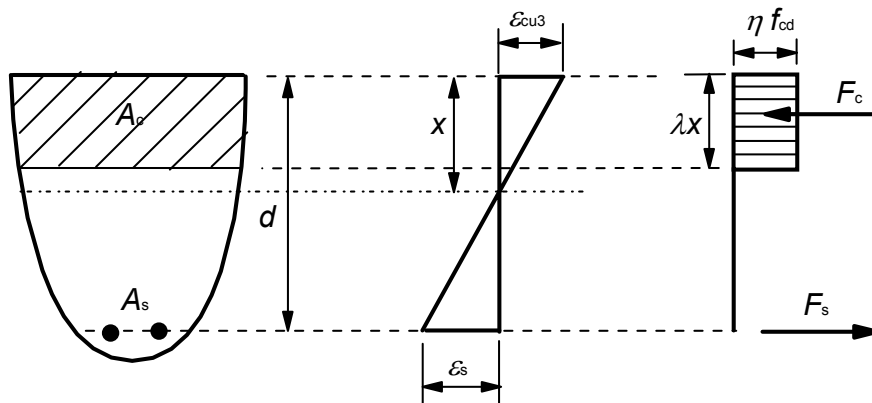


Figure 3.5: Rectangular stress distribution

3.1.8 Flexural tensile strength

(1) The mean flexural tensile strength of reinforced concrete members depends on the mean axial tensile strength and the depth of the cross-section. The following relationship may be used:

$$f_{ctm,fl} = \max \{ (1,6 - h/1000) f_{ctm}; f_{ctm} \} \quad (3.23)$$

where:

h is the total member depth in mm

f_{ctm} is the mean axial tensile strength following from Table 3.1.

The relation given in Expression (3.23) also applies for the characteristic tensile strength values.

3.1.9 Confined concrete

(1) Confinement of concrete results in a modification of the effective stress-strain relationship: higher strength and higher critical strains are achieved. The other basic material characteristics may be considered as unaffected for design.

(2) In the absence of more precise data, the stress-strain relation shown in Figure 3.6 (compressive strain shown positive) may be used, with increased characteristic strength and strains according to:

$$f_{ck,c} = f_{ck} (1,000 + 5,0 \sigma_2 / f_{ck}) \quad \text{for } \sigma_2 \leq 0,05 f_{ck} \quad (3.24)$$

$$f_{ck,c} = f_{ck} (1,125 + 2,50 \sigma_2/f_{ck}) \quad \text{for } \sigma_2 > 0,05f_{ck} \quad (3.25)$$

$$\varepsilon_{c2,c} = \varepsilon_{c2} (f_{ck,c}/f_{ck})^2 \quad (3.26)$$

$$\varepsilon_{cu2,c} = \varepsilon_{cu2} + 0,2 \sigma_2/f_{ck} \quad (3.27)$$

where σ_2 ($= \sigma_3$) is the effective lateral compressive stress at the ULS due to confinement and ε_{c2} and ε_{cu2} follow from Table 3.1. Confinement can be generated by adequately closed AC₂ links or cross-ties, which can reach the plastic condition due to lateral extension of the concrete. AC₂

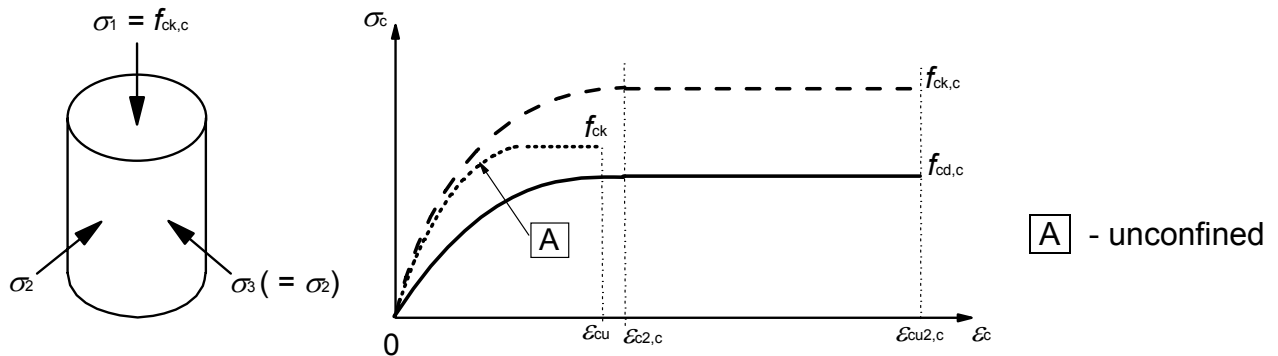


Figure 3.6: Stress-strain relationship for confined concrete

3.2 Reinforcing steel

3.2.1 General

(1)P The following clauses give principles and rules for reinforcement which is in the form of bars, de-coiled rods, welded fabric and lattice girders. They do not apply to specially coated bars.

(2)P The requirements for the properties of the reinforcement are for the material as placed in the hardened concrete. If site operations can affect the properties of the reinforcement, then those properties shall be verified after such operations.

(3)P Where other steels are used, which are not in accordance with EN10080, the properties shall be verified to be in accordance with 3.2.2 to 3.2.6 and Annex C.

(4)P The required properties of reinforcing steels shall be verified using the testing procedures in accordance with EN 10080.

Note: EN 10080 refers to a yield strength R_e , which relates to the characteristic, minimum and maximum values based on the long-term quality level of production. In contrast f_{yk} is the characteristic yield stress based on only that reinforcement used in a particular structure. There is no direct relationship between f_{yk} and the characteristic R_e . However the methods of evaluation and verification of yield strength given in EN 10080 provide a sufficient check for obtaining f_{yk} .

(5) The application rules relating to lattice girders (see EN 10080 for definition) apply only to those made with ribbed bars. Lattice girders made with other types of reinforcement may be given in an appropriate European Technical Approval.

3.2.2 Properties

(1)P The behaviour of reinforcing steel is specified by the following properties:

- yield strength (f_{yk} or $f_{0,2k}$)
- maximum actual yield strength ($f_{y,max}$)
- tensile strength (f_t)
- ductility (ε_{uk} and f_t/f_{yk})
- bendability
- bond characteristics (f_R : See Annex C)
- section sizes and tolerances
- fatigue strength
- weldability
- shear and weld strength for welded fabric and lattice girders

(2)P This Eurocode applies to ribbed and weldable reinforcement, including fabric. The permitted welding methods are given in Table 3.4.

Note 1: The properties of reinforcement required for use with this Eurocode are given in Annex C.

Note 2: The properties and rules for the use of indented bars with precast concrete products may be found in the relevant product standard.

(3)P The application rules for design and detailing in this Eurocode are valid for a specified yield strength range, $f_{yk} = 400$ to 600 MPa.

Note: The upper limit of f_{yk} within this range for use within a Country may be found in its National Annex.

(4)P The surface characteristics of ribbed bars shall be such to ensure adequate bond with the concrete.

(5) Adequate bond may be assumed by compliance with the specification of projected rib area, f_R .

Note: Minimum values of the relative rib area, f_R , are given in the Annex C.

(6)P The reinforcement shall have adequate bendability to allow the use of the minimum mandrel diameters specified in Table 8.1 and to allow rebending to be carried out.

Note: For bend and rebend requirements see Annex C.

3.2.3 Strength

(1)P The yield strength f_{yk} (or the 0,2% proof stress, $f_{0,2k}$) and the tensile strength f_{tk} are defined respectively as the characteristic value of the yield load, and the characteristic maximum load in direct axial tension, each divided by the nominal cross sectional area.

3.2.4 Ductility characteristics

(1)P The reinforcement shall have adequate ductility as defined by the ratio of tensile strength to the yield stress, $(f_t/f_y)_k$ and the elongation at maximum force, ε_{uk} .

(2) Figure 3.7 shows stress-strain curves for typical hot rolled and cold worked steel.

[AC1] Note: Values of $k = (f_t/f_y)_k$ and ε_{uk} for Class A, B and C are given in Annex C. **[AC1]**

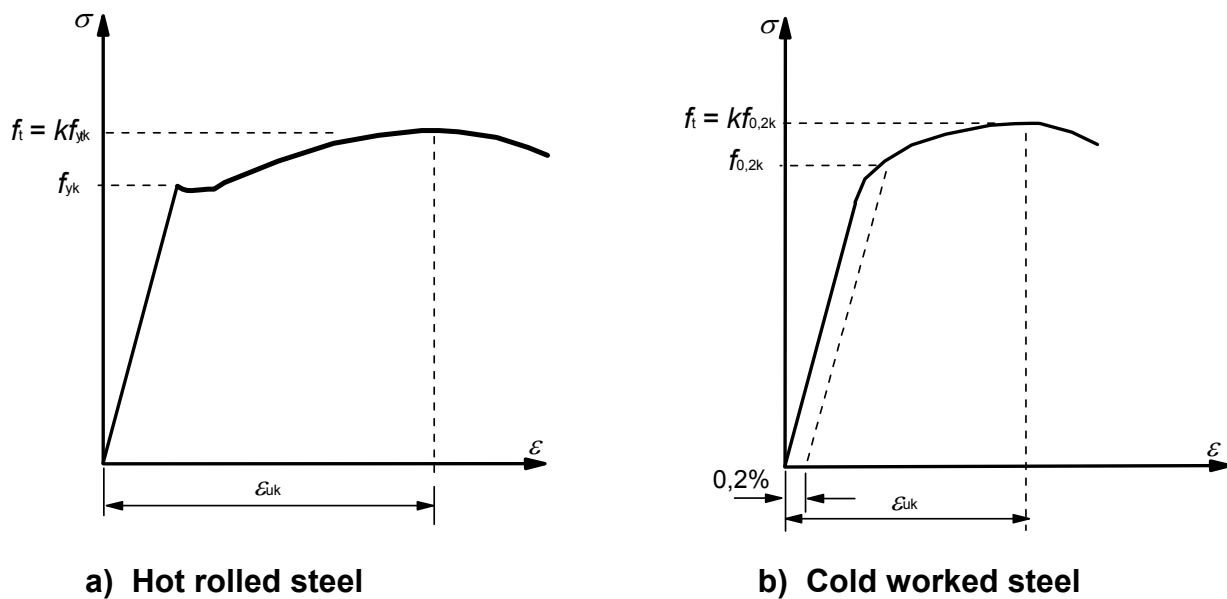


Figure 3.7: Stress-strain diagrams of typical reinforcing steel (absolute values are shown for tensile stress and strain)

3.2.5 Welding

(1)P Welding processes for reinforcing bars shall be in accordance with Table 3.4 and the weldability shall be in accordance with EN10080.

Table 3.4: Permitted welding processes and examples of application

Loading case	Welding method	Bars in tension ¹	Bars in compression ¹
Predominantly static (see 6.8.1 (2))	flash-welding	butt joint	
	manual metal arc welding and metal arc welding with filling electrode	butt joint with $\phi \geq 20$ mm, splice, lap, cruciform joints ³ , joint with other steel members	
	 metal arc active welding 	splice, lap, cruciform ³ joints & joint with other steel members	
		-	butt joint with $\phi \geq 20$ mm
	friction welding	butt joint, joint with other steels	
	resistance spot welding	lap joint ⁴ cruciform joint ^{2, 4}	
Not predominantly static (see 6.8.1 (2))	flash-welding	butt joint	
	 manual metal arc welding 	-	butt joint with $\phi \geq 14$ mm
	metal arc active welding	-	butt joint with $\phi \geq 14$ mm
	resistance spot welding	lap joint ⁴ cruciform joint ^{2, 4}	
Notes:			
1. Only bars with approximately the same nominal diameter may be welded together.			
2. Permitted ratio of mixed diameter bars $\geq 0,57$			
3. For bearing joints $\phi \leq 16$ mm			
4. For bearing joints $\phi \leq 28$ mm			

AC1 (2)P All welding of reinforcing bars shall be carried out in accordance with EN ISO 17660. AC1

(3)P The strength of the welded joints along the anchorage length of welded fabric shall be sufficient to resist the design forces.

(4) The strength of the welded joints of welded fabric may be assumed to be adequate if each welded joint can withstand a shearing force not less than 25% of a force equivalent to the specified characteristic yield stress times the nominal cross sectional area. This force should be based on the area of the thicker wire if the two are different.

3.2.6 Fatigue

(1)P Where fatigue strength is required it shall be verified in accordance with EN 10080.

Note : Information is given in Annex C.

3.2.7 Design assumptions

(1) Design should be based on the nominal cross-section area of the reinforcement and the design values derived from the characteristic values given in 3.2.2.

(2) For normal design, either of the following assumptions may be made (see Figure 3.8):

[AC1] a) an inclined top branch with a strain limit of ε_{ud} and a maximum stress of kf_{yk}/γ_s at ε_{uk} , **[AC1]** where $k = (f_t/f_y)_k$,

b) a horizontal top branch without the need to check the strain limit.

Note 1: The value of ε_{ud} for use in a Country may be found in its National Annex. The recommended value is $0,9\varepsilon_{uk}$

Note 2: The value of $(f_t/f_y)_k$ is given in Annex C.

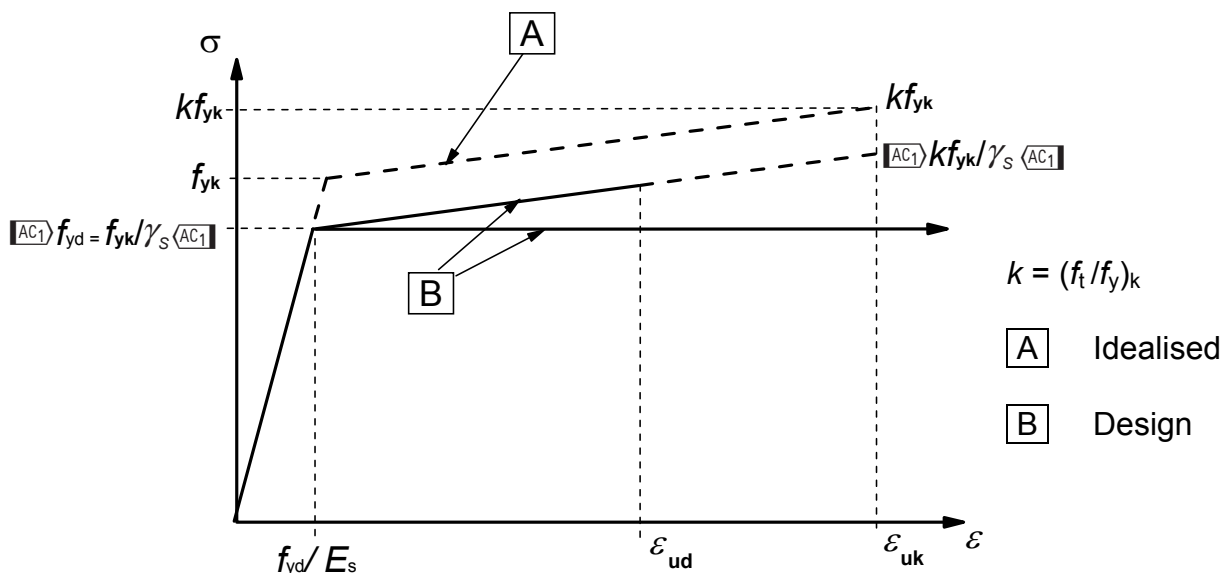


Figure 3.8: Idealised and design stress-strain diagrams for reinforcing steel (for tension and compression)

(3) The mean value of density may be assumed to be 7850 kg/m^3 .

(4) The design value of the modulus of elasticity, E_s may be assumed to be 200 GPa.

3.3 Prestressing steel

3.3.1 General

(1)P This clause applies to wires, bars and strands used as prestressing tendons in concrete structures.

(2)P Prestressing tendons shall have an acceptably low level of susceptibility to stress corrosion.

(3) The level of susceptibility to stress corrosion may be assumed to be acceptably low if the prestressing tendons comply with the criteria specified in EN 10138 or given in an appropriate European Technical Approval.

(4) The requirements for the properties of the prestressing tendons are for the materials as placed in their final position in the structure. Where the methods of production, testing and attestation of conformity for prestressing tendons are in accordance with EN 10138 or given in an appropriate European Technical Approval it may be assumed that the requirements of this Eurocode are met.

(5)P For steels complying with this Eurocode, tensile strength, 0,1% proof stress, and elongation at maximum load are specified in terms of characteristic values; these values are designated respectively f_{pk} , $f_{p0,1k}$ and ϵ_{uk} .

Note: EN 10138 refers to the characteristic, minimum and maximum values based on the long-term quality level of production. In contrast $f_{p0,1k}$ and f_{pk} are the characteristic proof stress and tensile strength based on only that prestressing steel required for the structure. There is no direct relationship between the two sets of values. However the characteristic values for 0,1% proof force, $F_{p0,1k}$ divided by the cross-section area, S_n given in EN 10138 together with the methods for evaluation and verification provide a sufficient check for obtaining the value of $f_{p0,1k}$.

(6) Where other steels are used, which are not in accordance with EN 10138, the properties may be given in an appropriate European Technical Approval.

(7)P Each product shall be clearly identifiable with respect to the classification system in 3.3.2 (2)P.

(8)P The prestressing tendons shall be classified for relaxation purposes according to 3.3.2 (4)P or given in an appropriate European Technical Approval.

(9)P Each consignment shall be accompanied by a certificate containing all the information necessary for its identification with regard to (i) - (iv) in 3.3.2 (2)P and additional information where necessary.

(10)P There shall be no welds in wires and bars. Individual wires of strands may contain staggered welds made only before cold drawing.

(11)P For coiled prestressing tendons, after uncoiling a length of wire or strand the maximum bow height shall comply with EN 10138 unless given in an appropriate European Technical Approval.

3.3.2 Properties

(1)P The properties of prestressing steel are given in EN 10138, Parts 2 to 4 or European Technical Approval.

(2)P The prestressing tendons (wires, strands and bars) shall be classified according to:

- (i) Strength, denoting the value of the 0,1% proof stress ($f_{p0,1k}$) and the value of the ratio of tensile $\overline{A_1}$ strength to proof strength ($f_p / f_{p0,1k}$) $\overline{A_1}$ and elongation at maximum load (ε_{uk})
- (ii) Class, indicating the relaxation behaviour
- (iii) Size
- (iv) Surface characteristics.

(3)P The actual mass of the prestressing tendons shall not differ from the nominal mass by more than the limits specified in EN 10138 or given in an appropriate European Technical Approval.

(4)P In this Eurocode, three classes of relaxation are defined:

- Class 1: wire or strand - ordinary relaxation
- Class 2: wire or strand - low relaxation
- Class 3: hot rolled and processed bars

Note: Class 1 is not covered by EN 10138.

(5) The design calculations for the losses due to relaxation of the prestressing steel should be based on the value of ρ_{1000} , the relaxation loss (in %) at 1000 hours after tensioning and at a mean temperature of 20 °C (see EN 10138 for the definition of the isothermal relaxation test).

Note: The value of ρ_{1000} is expressed as a percentage ratio of the initial stress and is obtained for an initial stress equal to $0,7f_p$, where f_p is the actual tensile strength of the prestressing steel samples. For design calculations, the characteristic tensile strength (f_{pk}) is used and this has been taken into account in the following expressions.

(6) The values for ρ_{1000} can be either assumed equal to 8% for Class 1, 2,5% for Class 2, and 4% for Class 3, or taken from the certificate.

(7) The relaxation loss may be obtained from the manufacturers test certificates or defined as the percentage ratio of the variation of the prestressing stress over the initial prestressing stress, should be determined by applying one of the Expressions below. Expressions (3.28) and (3.29) apply for wires or strands for ordinary prestressing and low relaxation tendons respectively, whereas Expression (3.30) applies for hot rolled and processed bars.

$$\text{Class 1} \quad \frac{\Delta\sigma_{pr}}{\sigma_{pi}} = 5,39 \rho_{1000} e^{6,7 \mu} \left(\frac{t}{1000} \right)^{0,75 (1-\mu)} 10^{-5} \quad (3.28)$$

$$\text{Class 2} \quad \frac{\Delta\sigma_{pr}}{\sigma_{pi}} = 0,66 \rho_{1000} e^{9,1 \mu} \left(\frac{t}{1000} \right)^{0,75 (1-\mu)} 10^{-5} \quad (3.29)$$

$$\text{Class 3} \quad \frac{\Delta\sigma_{pr}}{\sigma_{pi}} = 1,98 \rho_{1000} e^{8 \mu} \left(\frac{t}{1000} \right)^{0,75 (1-\mu)} 10^{-5} \quad (3.30)$$

Where

$\Delta\sigma_{pr}$ is absolute value of the relaxation losses of the prestress

σ_{pi} For post-tensioning σ_{pi} is the absolute value of the initial prestress $\sigma_{pi} = \sigma_{pm0}$ (see also 5.10.3 (2));

- For pre-tensioning σ_{pi} is the maximum tensile stress applied to the tendon minus the immediate losses occurred during the stressing process see 5.10.4 (1) (i)
- t is the time after tensioning (in hours)
- $\mu = \sigma_{pi} / f_{pk}$, where f_{pk} is the characteristic value of the tensile strength of the prestressing steel
- ρ_{1000} is the value of relaxation loss (in %), at 1000 hours after tensioning and at a mean temperature of 20°C.

Note: Where the relaxation losses are calculated for different time intervals (stages) and greater accuracy is required, reference should be made to Annex D.

- (8) The long term (final) values of the relaxation losses may be estimated for a time t equal to 500 000 hours (i.e. around 57 years).
- (9) Relaxation losses are very sensitive to the temperature of the steel. Where heat treatment AC_1 is applied (e.g. by steam), 10.3.2.1 applies. Otherwise where this temperature is greater than AC_1 50°C the relaxation losses should be verified.

3.3.3 Strength

- (1)P The 0,1% proof stress ($f_{p0,1k}$) and the specified value of the tensile strength (f_{pk}) are defined as the characteristic value of the 0,1% proof load and the characteristic maximum load in axial tension respectively, divided by the nominal cross sectional area as shown in Figure 3.9.

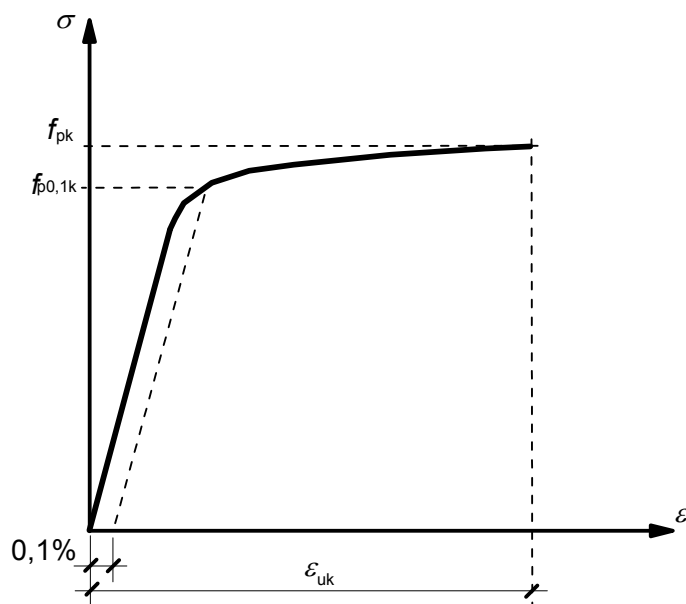


Figure 3.9: Stress-strain diagram for typical prestressing steel (absolute values are shown for tensile stress and strain)

3.3.4 Ductility characteristics

- (1)P The prestressing tendons shall have adequate ductility, as specified in EN 10138.
- (2) Adequate ductility in elongation may be assumed if the prestressing tendons obtain the specified value of the elongation at maximum load given in EN 10138.

(3) Adequate ductility in bending may be assumed if the prestressing tendons satisfy the requirements for bendability of EN ISO 15630.

(4) Stress-strain diagrams for the prestressing tendons, based on production data, shall be prepared and made available by the producer as an annex to the certificate accompanying the consignment (see 3.3.1 (9)P).

(5) Adequate ductility in tension may be assumed for the prestressing tendons if $\frac{f_p}{f_{p0,1k}} \geq k$.

Note: The value of k for use in a Country may be found in its National Annex. The recommended value is 1,1.

3.3.5 Fatigue

(1)P Prestressing tendons shall have adequate fatigue strength.

(2)P The fatigue stress range for prestressing tendons shall be in accordance with EN 10138 or given in an appropriate European Technical Approval.

3.3.6 Design assumptions

(1)P Structural analysis is performed on the basis of the nominal cross-section area of the prestressing steel and the characteristic values $f_{p0,1k}$, f_{pk} and ε_{uk} .

(2) The design value for the modulus of elasticity, E_p may be assumed equal to 205 GPa for wires and bars. The actual value can range from 195 to 210 GPa, depending on the manufacturing process. Certificates accompanying the consignment should give the appropriate value.

(3) The design value for the modulus of elasticity, E_p may be assumed equal to 195 GPa for strand. The actual value can range from 185 GPa to 205 GPa, depending on the manufacturing process. Certificates accompanying the consignment should give the appropriate value.

(4) The mean density of prestressing tendons for the purposes of design may normally be taken as 7850 kg/m³

(5) The values given above may be assumed to be valid within a temperature range between -40°C and +100°C for the prestressing steel in the finished structure.

(6) The design value for the steel stress, f_{pd} , is taken as $f_{p0,1k}/\gamma_s$ (see Figure 3.10).

(7) For cross-section design, either of the following assumptions may be made (see Figure 3.10):

- an inclined branch, with a strain limit ε_{ud} . The design may also be based on the actual stress/strain relationship, if this is known, with stress above the elastic limit reduced analogously with Figure 3.10, or
- a horizontal top branch without strain limit.

Note: The value of ε_{ud} for use in a Country may be found in its National Annex. The recommended value is 0,9 ε_{uk} . If more accurate values are not known the recommended values are $\varepsilon_{ud} = 0,02$ and $f_{p0,1k}/f_{pk} = 0,9$.

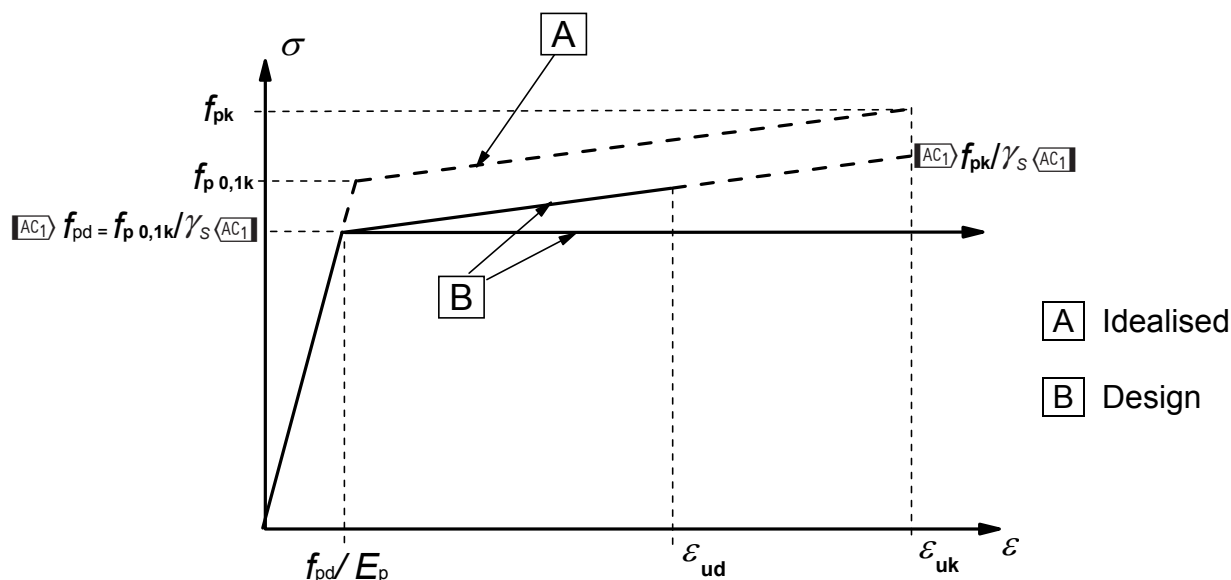


Figure 3.10: Idealised and design stress-strain diagrams for prestressing steel (absolute values are shown for tensile stress and strain)

3.3.7 Prestressing tendons in sheaths

(1)P Prestressing tendons in sheaths (e.g. bonded tendons in ducts, unbonded tendons etc.) shall be adequately and permanently protected against corrosion (see 4.3).

(2)P Prestressing tendons in sheaths shall be adequately protected against the effects of fire (see EN 1992-1-2).

3.4 Prestressing devices

3.4.1 Anchorages and couplers

3.4.1.1 General

(1)P 3.4.1 applies to anchoring devices (anchorages) and coupling devices (couplers) for application in post-tensioned construction, where:

- (i) anchorages are used to transmit the forces in tendons to the concrete in the anchorage zone
- (ii) couplers are used to connect individual lengths of tendon to make continuous tendons.

(2)P Anchorages and couplers for the prestressing system considered shall be in accordance with the relevant European Technical Approval.

(3)P Detailing of anchorage zones shall be in accordance with 5.10, 8.10.3 and 8.10.4.

3.4.1.2 Mechanical properties

3.4.1.2.1 Anchored tendons

(1)P Prestressing tendon anchorage assemblies and prestressing tendon coupler assemblies shall have strength, elongation and fatigue characteristics sufficient to meet the requirements of the design.

- (2) This may be assumed provided that:
- (i) The geometry and material characteristics of the anchorage and coupler components are in accordance with the appropriate European Technical Approval and that their premature failure is precluded.
 - (ii) Failure of the tendon is not induced by the connection to the anchorage or coupler.
 - (iii) The elongation at failure of the assemblies $\geq 2\%$.
 - (iv) Tendon-anchorage assemblies are not located in otherwise highly-stressed zones.
 - (v) Fatigue characteristics of the anchorage and coupler components are in accordance with the appropriate European Technical Approval.

3.4.1.2.2 Anchorage devices and anchorage zones

(1)P The strength of the anchorage devices and zones shall be sufficient for the transfer of the tendon force to the concrete and the formation of cracks in the anchorage zone shall not impair the function of the anchorage.

3.4.2 External non-bonded tendons

3.4.2.1 General

(1)P An external non-bonded tendon is a tendon situated outside the original concrete section and is connected to the structure by anchorages and deviators only.

(2)P The post-tensioning system for the use with external tendons shall be in accordance with the appropriate European Technical Approval.

(3) Reinforcement detailing should follow the rules given in 8.10.

3.4.2.2 Anchorages

(1) The minimum radius of curvature of the tendon in the anchorage zone for non-bonded tendons should be given in the appropriate European Technical Approval.

SECTION 4 DURABILITY AND COVER TO REINFORCEMENT

4.1 General

(1)P A durable structure shall meet the requirements of serviceability, strength and stability throughout its design working life, without significant loss of utility or excessive unforeseen maintenance (for general requirements see also EN 1990).

(2)P The required protection of the structure shall be established by considering its intended use, design working life (see EN 1990), maintenance programme and actions.

(3)P The possible significance of direct and indirect actions, environmental conditions (4.2) and consequential effects shall be considered.

Note: Examples include deformations due to creep and shrinkage (see 2.3.2).

(4) Corrosion protection of steel reinforcement depends on density, quality and thickness of concrete cover (see 4.4) and cracking (see 7.3). The cover density and quality is achieved by controlling the maximum water/cement ratio and minimum cement content (see EN 206-1) and may be related to a minimum strength class of concrete.

Note: Further information is given in Annex E.

(5) Where metal fastenings are inspectable and replaceable, they may be used with protective coatings in exposed situations. Otherwise, they should be of corrosion resistant material.

(6) Further requirements to those given in this Section should be considered for special situations (e.g. for structures of temporary or monumental nature, structures subjected to extreme or unusual actions etc.).

4.2 Environmental conditions

(1)P Exposure conditions are chemical and physical conditions to which the structure is exposed in addition to the mechanical actions.

(2) Environmental conditions are classified according to Table 4.1, based on EN 206-1.

(3) In addition to the conditions in Table 4.1, particular forms of aggressive or indirect action should be considered including:

chemical attack, arising from e.g.

- the use of the building or the structure (storage of liquids, etc)
- solutions of acids or sulfate salts (EN 206-1, ISO 9690)
- chlorides contained in the concrete (EN 206-1)
- alkali-aggregate reactions (EN 206-1, National Standards)

physical attack, arising from e.g.

- temperature change
- abrasion (see 4.4.1.2 (13))
- water penetration (EN 206-1).