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EUROPEAN STANDARD

NORME EUROPÉENNE

EUROPÄISCHE NORM

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Descriptors: Buildings, concrete structures, computation, building codes, rules of calculation

English version

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

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

Eurocode 2: Planung von Stahlbeton- und Spannbetontragwerken - Teil 1: Grundlagen und Anwendungsregeln für den Hochbau

CEN

European Committee for Standardization
Comité Européen de Normalisation
Europäishes Komitee für Normung

Central Secretariat: rue de Stassart, 36 B-1050 Brussels

Foreword

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

The text of the draft standard was submitted to the formal vote and was approved by CEN as EN 1992-1-1 on **YYYY-MM-DD**.

No existing European Standard is superseded.

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 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 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 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).

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

[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).

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 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 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.

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

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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.

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 works[4]. Furthermore, all the information accompanying the CE Marking of the construction products which refer to Eurocodes shall 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.

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

[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.

engineering works to be constructed in the relevant country.

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

2.3.2.2 (4)	5.10.8 (2)	9.5.3 (3)
2.4.2.1 (1)	5.10.8 (3)	9.6.2 (1)
2.4.2.2 (1)	5.10.9 (1)P	9.6.3 (1)
2.4.2.2 (2)	6.2.2 (1)	9.7 (1)
2.4.2.2 (3)	6.2.3 (2)	9.8.1 (3)
2.4.2.3 (1)	6.2.3 (3)	9.8.2.1 (1)
2.4.2.4 (1)	6.4.4 (1)	9.8.3 (1)
2.4.2.4 (2)	6.5.2 (2)	9.8.3 (2)
2.4.2.5 (2)	6.8.4 (1)	9.8.5 (3)
3.1.2 (2)P	6.8.4 (5)	9.8.5 (4)
3.1.6 (1)P	6.8.7 (1)	9.10.2.2 (2)
3.1.6 (2)P	7.2 (1)P	9.10.2.3 (3)
3.2.7 (2)	7.2 (2)P	9.10.2.3 (4)
3.3.6 (7)	7.3.1 (5)	9.10.2.4 (2)
4.4.1.2 (3)	7.4.2 (2)	11.3.5 (1)P
4.4.1.2 (5)	8.3 (1)P	11.3.5 (2)P
4.4.1.2 (6)	8.6 (2)	11.6.1 (1)
4.4.1.2 (7)	8.8 (1)	12.2.1 (1)
4.4.1.2 (8)	9.2.1.1 (1)	12.3.1 (1)
4.4.1.2 (11)	9.2.1.1 (3)	12.6.3 (2)
4.4.1.3 (2)	9.2.1.2 (1)	A.1 (1)
4.4.1.3 (3)	9.2.1.4 (1)	A.2.1 (1)
5.2 (5)	9.2.2 (4)	A.2.1 (2)
5.6.3 (4)	9.2.2 (5)	A.2.2 (1)
5.8.5 (1)	9.2.2 (6)	A.2.2 (2)
5.8.6 (3)	9.2.2 (7)	A.2.3 (1)
5.10.1 (6)	9.2.2 (8)	C.1 (1)
5.10.2.1 (1)P	9.2.4 (3)	C.1 (3)
5.10.2.1 (2)	9.3.1.1(3)	D.4 (1)
5.10.2.2 (4)	9.5.2 (1)	I.1.2 (2)
5.10.2.2 (5)	9.5.2 (2)	I.2 (2)
5.10.3 (2)	9.5.2 (3)	I.2 (3)

Contents List

Annexes

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
EN 13670:	Execution of concrete structures
EN 1997:	Geotechnical design
EN 1998:	Design of structures for earthquake resistance, when composite structures are built in seismic regions.

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

Part 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 of Eurocode 2

(1)P Part 1 of Eurocode 2 gives a general basis for the design of structures in 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.

Section 1:	Introduction
Section 2:	Basis of Design
Section 3:	Materials
Section 4:	Durability
Section 5:	Structural Analysis
Section 6:	Ultimate Limit State
Section 7:	Serviceability States
Section 8:	Detailing of Reinforcement
Section 9:	Detailing of Members
Section 10:	Additional rules for precast concrete elements and structures

Section 11: Lightweight aggregate Concrete Structures
Section 12: Lightly Reinforced concrete Structures

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

(5)P This Part 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 1991-1-5:200 :	Actions on structures: Thermal actions
EN 1991-1-6:200 :	Actions on structures: Actions during execution

1.2.2 Other reference standards

EN1997:	Geotechnical design
EN 197:	Cement: Composition, specification and conformity criteria for common cements
EN 206:	Concrete: Specification, performance, production and conformity
EN 12350:	Testing fresh concrete
EN 10080:	Steel for the reinforcement of concrete
EN 10138:	Prestressing steels
EN ISO 17760:	Permitted welding process for reinforcement
ENV 13670:	Execution of concrete structures
EN 13791:	Testing concrete
EN ISO 15630	Steel for the reinforcement and prestressing of concrete: Test methods

[*Drafting Note: This list will require updating at time of publication*]

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.

(2)P The design procedures are valid only when the requirements for execution and workmanship given in prENV 13670 are also 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 at the ultimate limit state

Latin lower case letters

a Distance
a Geometrical data
Δa Additive or reducing safety element 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 cylinder 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 strength of stirrups
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	Time at initial loading of the concrete
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 safety factor
γ_A	Partial safety factors for accidental actions A
γ_C	Partial safety factors for concrete material properties
γ_F	Partial safety factors for actions, F
γ_G	Partial safety factors for permanent actions, G
γ_M	Partial safety factors for a material property, taking account of uncertainties in the material property itself and in the design model used
γ_P	Partial safety factors for actions associated with prestressing, P

γ_Q	Partial safety factors for variable actions, Q
γ_s	Partial safety factors for the properties of reinforcement or prestressing steel
$\gamma_{s,fat}$	Partial safety factors for the properties of reinforcement or prestressing steel under fatigue loading
γ_f	Partial safety factors for actions without taking account of model uncertainties
γ_g	Partial safety factors for permanent actions without taking account of model uncertainties
γ_m	Partial safety factors for a material property, taking account only of uncertainties in the material property
δ	Increment
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.

2.1.3 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 settlements are taken into account, appropriate estimate values of predicted settlements may be used.

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

2.3.1.2 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 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 the 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 irrespectively 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.

(4) In building structures, temperature and shrinkage effects may be omitted in global analysis provided joints are included at a maximum distance apart of l_{joint} to accommodate resulting deformations.

Note: The value of l_{joint} for use in a Country may be found in its National Annex. The recommended value is 30 m. For precast concrete structures the value may be larger than that for insitu structures, since part of the creep and shrinkage takes place before erection.

2.3.3 Geometric data

2.3.3.1 General

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

2.3.3.2 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) The diameter, used in design calculations, of cast in place piles without permanent casing should be taken as:

- if $d_{nom} < 400$ mm $\quad d = d_{nom} - 20$ mm
- if $400 \leq d_{nom} \leq 1000$ mm $\quad d = 0{,}95.d_{nom}$
- if $d_{nom} > 1000$ mm $\quad d = d_{nom} - 50$ 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 factors 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 $\gamma_{P,fav}$ 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 $\gamma_{P,fav}$ 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 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.

2.4.2.3 Partial factors 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

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 2.4.1.4 (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 Combination 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, e.g. for the design of holding down anchors 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 E 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 designed in accordance with EN 1997-1.

(4) Where relevant, the design should include for 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 Standard 'Design of Fastenings for Use in Concrete' (under development). This Standard will cover the design of the following types of fasteners:
- cast-in fasteners such as headed anchors,
- channel bars,
- sockets and shear lugs

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

The Standard '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 this Eurocode 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)P 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.

(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)P 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).

(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).

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

with

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

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 rapid hardening high strength cements (R) (CEM 42,5R, CEM 52,5)
= 0,25 for normal and rapid hardening cements (N) (CEM 32,5R, CEM 42,5)
= 0,38 for slow hardening cements (S) (CEM 32,5)

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.

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

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

(7)P The tensile strength refers to the highest stress reached under concentric tensile loading. For the flexural tensile strength reference is 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:

$$f_{ct} = 0{,}9 f_{ct,sp} \tag{3.3}$$

(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:

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

where $\beta_{cc}(t)$ follows from Expression (3.2) and
$\alpha = 1$ for $t < 28$
$\alpha = 2/3$ for $t \geq 28$. The values for f_{ctm} 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.

Table 3.1 Stress 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	
f_{cm} (MPa)	20	24	28	33	38	43	48	53	58	63	68	78	88	98	$f_{cm} = f_{ck}+8$(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	$f_{ctm}=0,30\times f_{ck}^{(2/3)}$ ≤C50/60 $f_{ctm}=2,12\cdot\ln(1+(f_{cm}/10))$ > 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	32	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 ε_{c1} ($^0/_{00}$) $= 0,7\ f_{cm}^{0,31} < 2.8$
ε_{cu1} (‰)	3,5									3,2	3,0	2,8	2,8	2,8	see Figure 3.2 for $f_{ck} \geq 50$ Mpa ε_{cu1}($^0/_{00}$)$=2,8+27[(98-f_{cm})/100]^4$
ε_{c2} (‰)	2,0									2,2	2,3	2,4	2, 5	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa ε_{c2}($^0/_{00}$)$=2,0+0,085(f_{ck}-50)^{0,53}$
ε_{cu2} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa ε_{cu2}($^0/_{00}$)$=2,6+35[(90-f_{ck})/100]^4$
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 ε_{c3}($^0/_{00}$)$=1,75+0,55[(f_{ck}-50)/40]$
ε_{cu3} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	see Figure 3.4 for $f_{ck} \geq 50$ Mpa ε_{cu3}($^0/_{00}$)$=2,6+35[(90-f_{ck})/100]^4$

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

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

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 $10 \cdot 10^{-6}\ K^{-1}$.

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. Any estimation of the creep coefficient $\varphi(t, t_0)$, and of the shrinkage strain, ε_{cs}, should take these parameters into account.

(2) Where great accuracy is not required, the value found from Figure 3.1 may be considered as the final creep coefficient $\varphi(\infty, t_0)$, provided that the concrete is not subjected to a compressive stress greater than 0,45 $f_{ck}(t_0)$ at an age t_0 at first loading. The final creep coefficient $\varphi(\infty, t_0)$ is related to E_{cm} according to Table 3.1.

Note: For greater accuracy, 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 with time may be calculated from:

$$\varepsilon_{cc}(\infty,t_0) = \varphi(\infty,t_0).\ (\sigma_c/E_{c0}) \tag{3.6}$$

where E_{c0} is the tangent modulus of elasticity at time t_0,

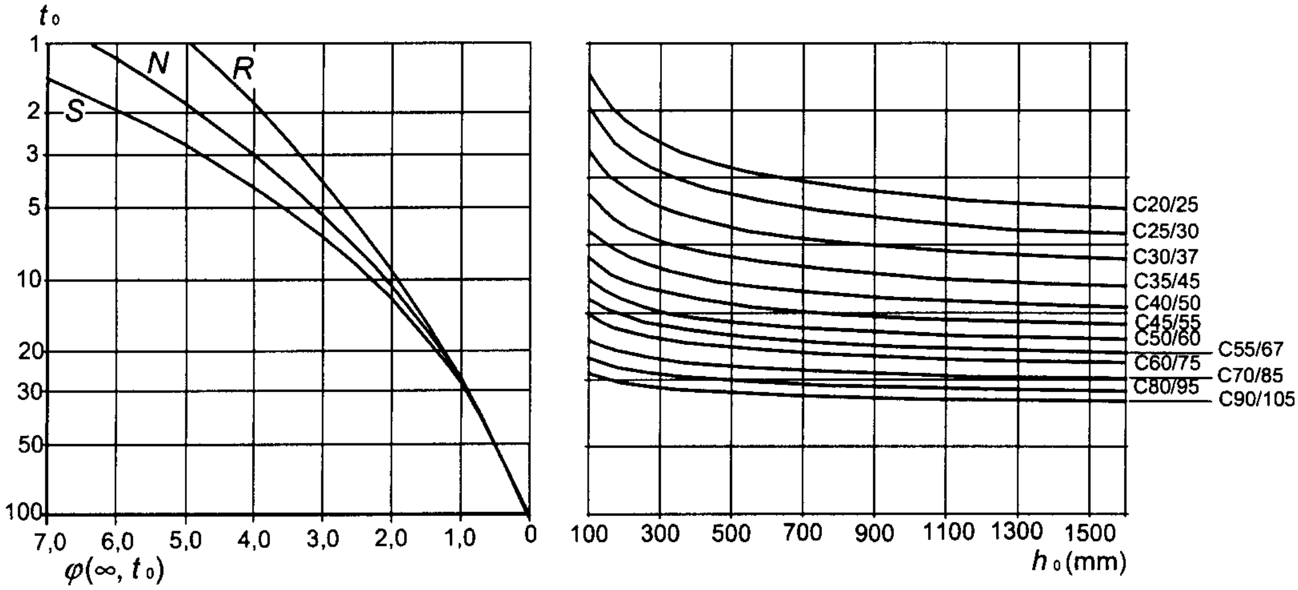
(4) When the compressive stress of concrete at an age t_0 exceeds the value 0,45 $f_{ck}(t_0)$ then creep non-linearity should be considered. Such a high stress can occur as a result of pretensioning, e.g. in precast concrete members at tendon level. In such cases the non-linear notional creep coefficient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

$$\varphi_k(\infty, t_0) = \varphi(\infty, t_0) \exp(1,5\ (k_\sigma - 0,45)) \tag{3.7}$$

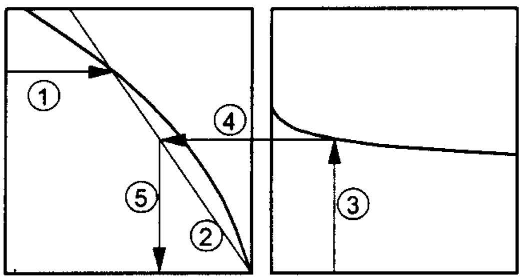
where:

$\varphi_k(\infty, t_0)$ is the non-linear notional creep coefficient, which replaces $\varphi(\infty, t_0)$

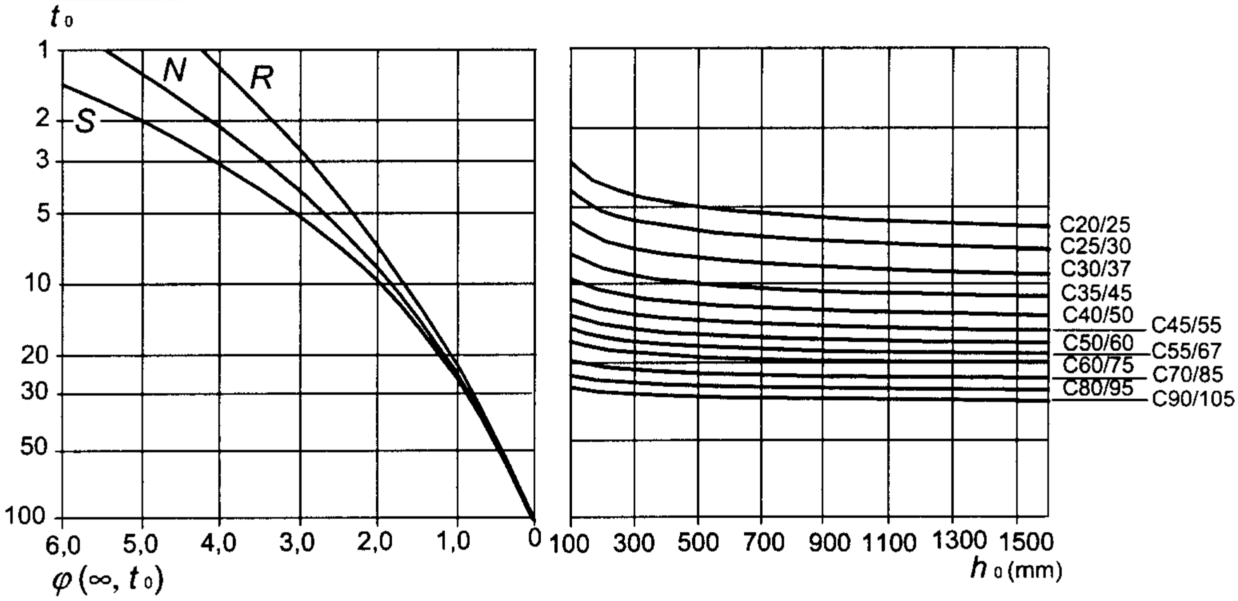
k_σ is the stress-strength ratio $\sigma_c/f_{cm}(t_0)$, where σ_c is the compressive stress and $f_{cm}(t_0)$ is the mean concrete compressive strength at the time of loading.



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°C and a mean relative humidity between RH = 40% and 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 first 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 for slow hardening cements
N is for normal and rapid hardening cements
R is for rapid hardening cements

(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

$$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca} \tag{3.8}$$

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 $\varepsilon_{cd,\infty} = k_h \cdot \varepsilon_{cd,0}$. $\varepsilon_{cd,0}$ may be taken from Table 3.2 (expected mean values, with a coefficient of variation of about 30%).

Note: The background expressions for the values in Table 3.2 are given in Appendix B).

Table 3.2 Nominal unrestrained drying shrinkage values $\varepsilon_{cd,0}$ (in $^0/_{00}$) for concrete

$f_{ck}/f_{ck,cube}$ (MPa)	Final unrestrained drying shrinkage (in $^0/_{00}$)					
	Relative Humidity (in $^0/_0$)					
	20	**40**	**60**	**80**	**90**	**100**
20/25	0.64	0.60	0.50	0.31	0.17	0
40/50	0.51	0.48	0.40	0.25	0.14	0
60/75	0.41	0.38	0.32	0.20	0.11	0
80/95	0.33	0.31	0.26	0.16	0.09	0
90/105	0.30	0.28	0.23	0.15	0.05	0

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

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

Note: $\varepsilon_{cd,0}$ is defined in Annex B.

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 Eq. (3.9)

h_0	k_h
100	1.0
200	0.85
300	0.75
≥ 500	0.70

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

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 which is exposed to drying

The autogenous shrinkage strain follows from:

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

where:

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

and

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

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 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):

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

where:

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

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

$k = 1{,}1\ E_{cm} \times |\varepsilon_{c1}|/f_{cm}$ (f_{cm} 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.

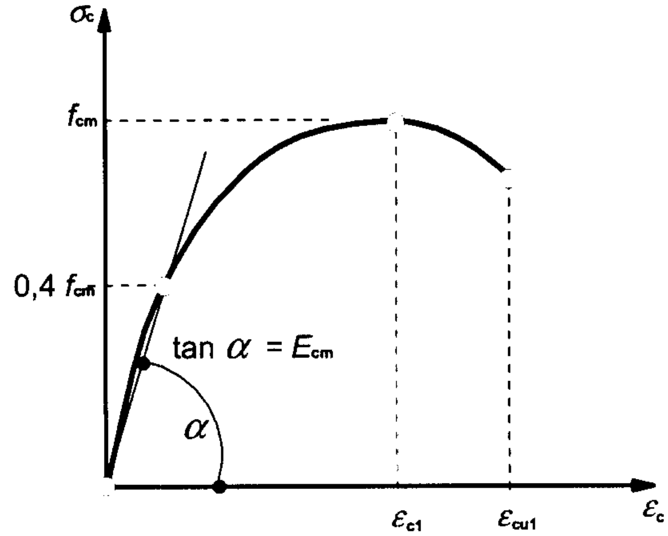


Figure 3.2 Schematic representation of the stress-strain relation for structural analysis.

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 2.4.1.4, 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 2.4.1.4, 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.

3.1.7 Stress-strain relations for the design of sections

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

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

$$\sigma_c = f_{cd} \quad \textit{for} \quad \varepsilon_{c2} \le \varepsilon_c \le \varepsilon_{cu2} \tag{3.18}$$

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

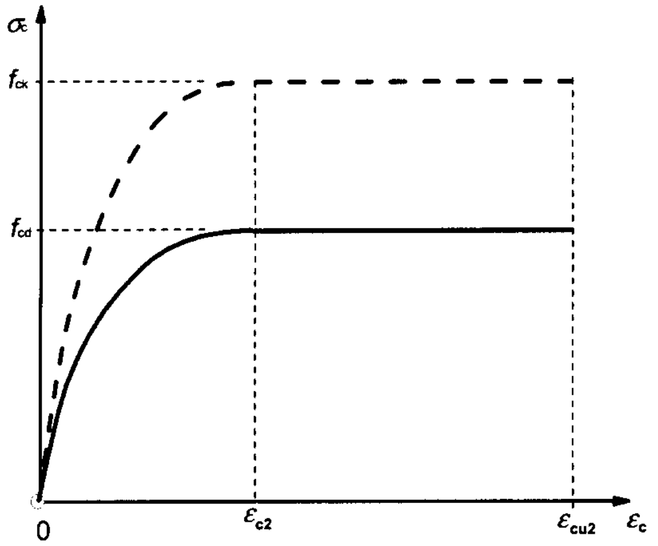


Figure 3.3. Parabola-rectangle diagram for concrete under compression.

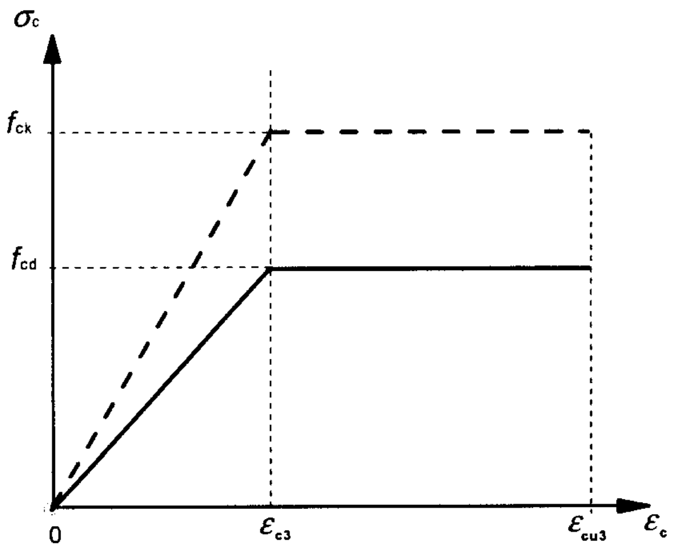


Figure 3.4. Bi-linear stress-strain relation.

(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.

(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:

$$\lambda = 0{,}8 \quad \text{for } f_{ck} \leq 50 \text{ MPa} \tag{3.19}$$
$$\lambda = 0{,}8 - (f_{ck} - 50)/400 \quad \text{for } 50 < f_{ck} \leq 90 \text{ MPa} \tag{3.20}$$

and

$$\eta = 1{,}0 \quad \text{for } f_{ck} \leq 50 \text{ MPa} \tag{3.21}$$
$$\eta = 1{,}0 - (f_{ck} - 50)/200 \quad \text{for } 50 < f_{ck} \leq 90 \text{ MPa} \tag{3.22}$$

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%.

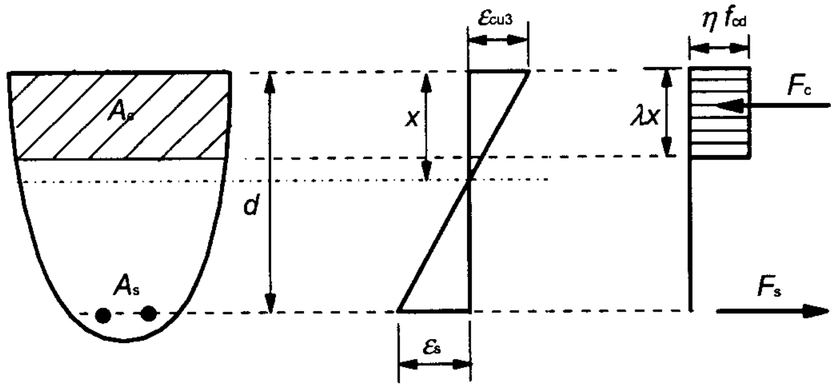


Figure 3.5. Rectangular stress distribution

3.1.8 Flexural tensile strength

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

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

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:

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

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

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

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

where σ_2 (= σ_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 links or cross-ties, which reach the plastic condition due to lateral extension of the concrete.

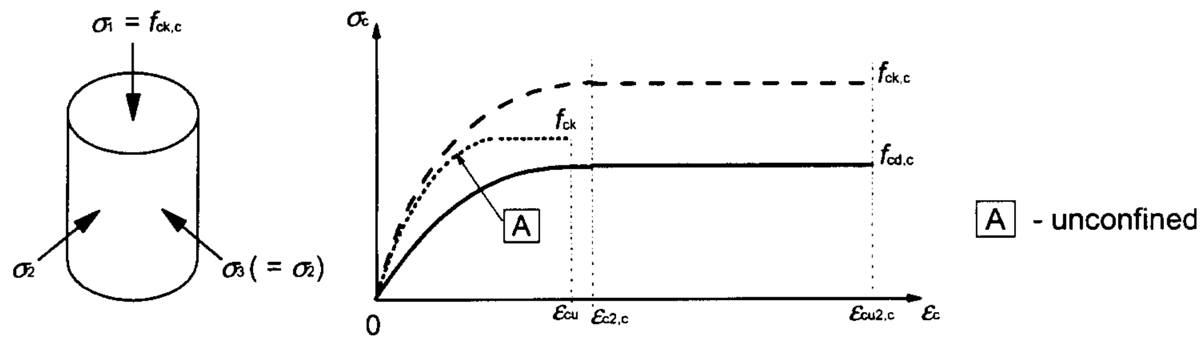


Figure 3.6: Stress-strain relationship for confined concrete

3.2 Reinforcing steel

3.2.1 General

(1)P The application rules of this Eurocode apply to reinforcement which is in the form of bars, de-coiled rods and welded fabric. 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 this Eurocode.

(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)P The application rules relating to lattice girders apply only to those made with ribbed bars. Lattice girders made with other types of reinforcement shall be in accordance with the appropriate European Technical Approval.

3.2.2 Properties

(1)P The behaviour of reinforcing steel is specified by the following properties:
- yield stress (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 EN 10080 for definition)
- 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. 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 Normative 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 up to a specified yield strength, f_{yk} = 600 MPa.

(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 the specification of relative rib area, f_R, in accordance with EN 10080.

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

3.2.3 Strength

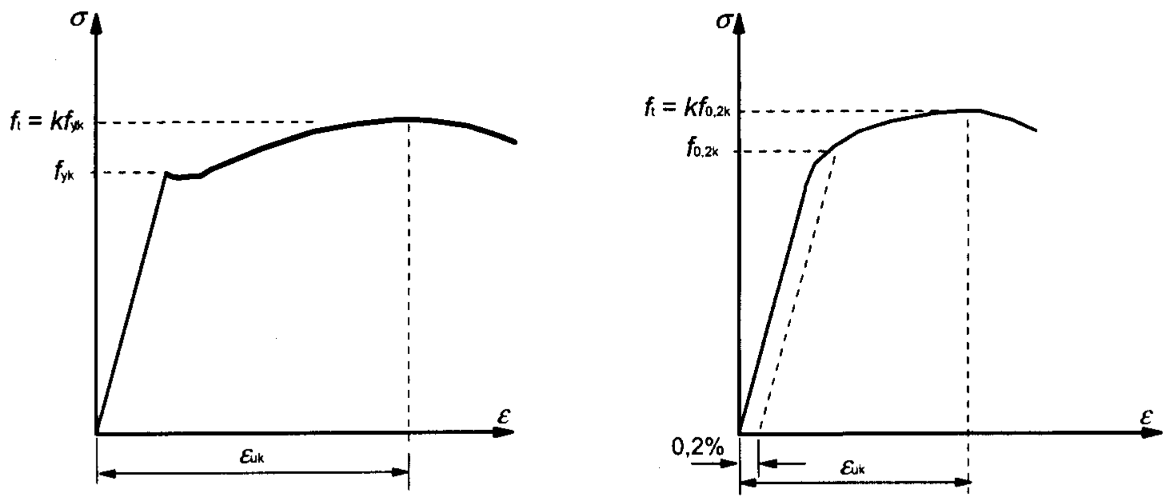
(1)P The yield stress f_{yk} (or the 0,2% proof stress, $f_{0,2k}$) and the tensile strength f_{tk} are defined respectively as the characteristic value of the yield load, and of the characteristic maximum load in direct axial tension, each divided by the nominal cross sectional area.

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.

Note: Values of $(f_t/f_y)_k$ and ε_{uk} for Class A, B and C are given in Normative Annex C.



a) Hot rolled steel **b) Cold worked steel**

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 steels 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	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[2]	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 (with one-point welding machine)	lap joint[4] cruciform joint[2, 4]	
Not predominantly static	flash-welding	butt joint	
	manual metal arc welding	-	butt joint with $\phi \geq 14$mm
	metal arc active welding[2]	-	butt joint with $\phi \geq 14$mm

Notes:

1. Only bars with approximately the same nominal diameter may be welded together.
2. Permitted ratio of mixed diameter bars ≥ 0,57
3. For bearing joints $\phi \leq 16$ mm
4. For bearing joints $\phi \leq 28$ mm

(2)P All welding shall be carried out in accordance with EN ISO 17760.

(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 30% 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 Normative Annex C.

3.2.7 Design assumptions

(1) Design shouldl 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.

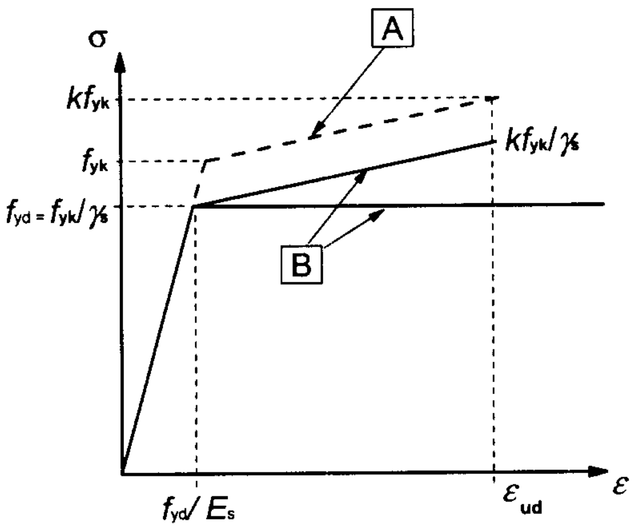
(2) For normal design, either of the following assumptions may be made (see Figure 3.8):

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

b) a horizontal top branch without 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 Normative Annex C.



$k = (f_t/f_y)_k$

A Idealised

B Design

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 a European Technical Approval.

(4)P The requirements for the properties of the prestressing tendons are for the materials as placed in their final position. Where the methods of production, testing and attestation of conformity for prestressing tendons are in accordance with EN 10138 or a 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)P Where other steels are used, which are not in accordance with EN 10138, the properties shall be in accordance with a 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 a 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 or 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) Srength, denoting the value of the 0,1% proof stress ($f_{p0,1k}$) and the value of the ratio of tensile strength to proof strength ($f_{pk}/f_{p0,1k}$) 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 appropriate European Technical Approval.

(4)P In this Eurocode, three classes of relaxation are defined:
- Class 1: wire or strand - ordinary prestressing tendons
- Class 2: wire or strand - low relaxation
- Class 3: hot rolled and processed bars

(5) The design calculations of 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 losses, 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 (3.30) and (3.31) apply for wires or strands for ordinary prestressing and low relaxation tendons respectively, whereas Expression (3.32) applies for hot rolled and processed bars.

Class 1
$$\frac{\Delta\sigma_{pr}}{\sigma_{pi}} = 5{,}39\ \rho_{1000}\ e^{6{,}7\mu}\left(\frac{t}{1000}\right)^{0{,}75\,(1-\mu)} 10^{-3} \qquad (3.30)$$

Class 2
$$\frac{\Delta\sigma_{pr}}{\sigma_{pi}} = 0{,}66\ \rho_{1000}\ e^{9{,}1\mu}\left(\frac{t}{1000}\right)^{0{,}75\,(1-\mu)} 10^{-3} \qquad (3.31)$$

Class 3
$$\frac{\Delta\sigma_{pr}}{\sigma_{pi}} = 1{,}98\ \rho_{1000}\ e^{8\mu}\left(\frac{t}{1000}\right)^{0{,}75\,(1-\mu)} 10^{-3} \qquad (3.32)$$

Where

$\Delta\sigma_{pr}$ is absolute value of the relaxation losses of the prestress for post-tensioning 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)

μ = σ_{pm0}/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 500000 hours (i.e. around 57 years).

(9) Relaxation losses are very sensitive to the temperature of the steel. Where heat treatment is applied (e.g. by steam), 10.3.2.2 applies. Otherwise where this temperature is greater than 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.

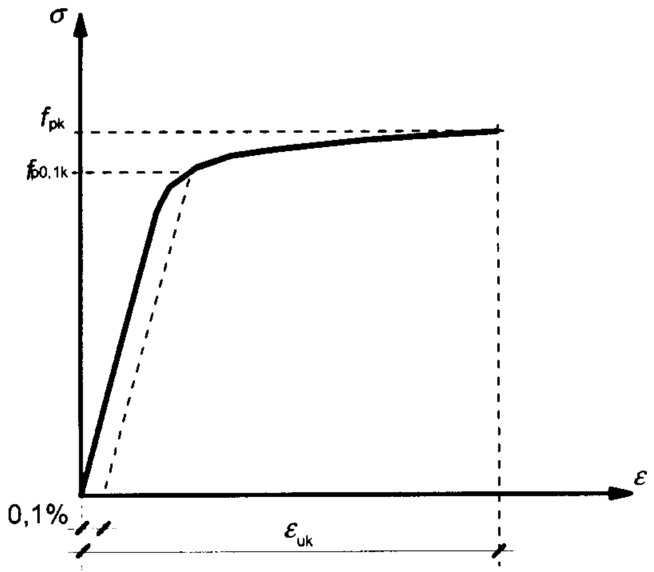


Figure 3.9: Stress-strain diagram for typical prestressing steel (absolute values are shown for tensile stress and strain)

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 the relevant standards.

(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 if the prestressing tendons obtain the specified value of $f_{p0.1}/f_{pk}$ given in EN 10138

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 European Technical Approval.

(3) For the fatigue design requirements of prestressing steel, reference should be made to 6.8.

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.

(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/m^3

(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 $0,9\varepsilon_{uk}$. If more accurate values are not known the recommended values are ε_{ud} = 0,02 and $f_{p0,1k}/f_{pk}$ = 0,9.

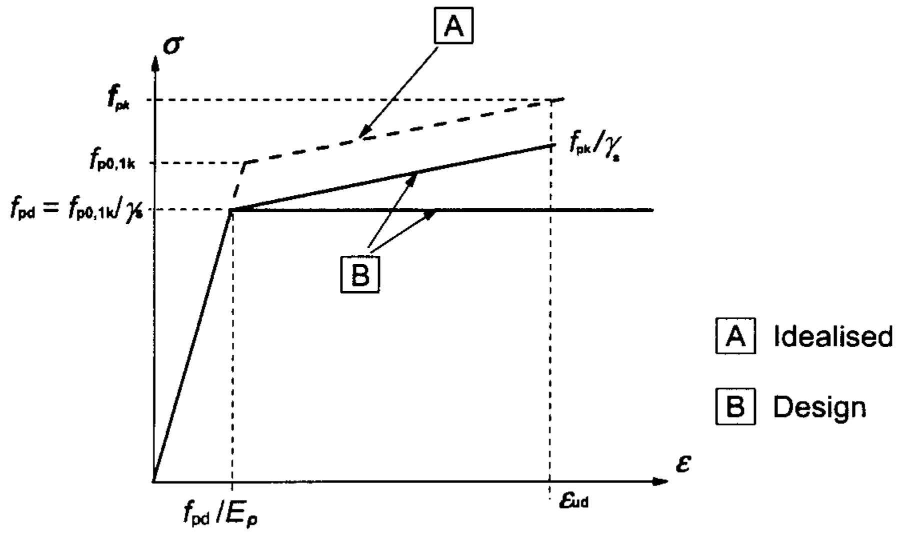


Figure 3.10: Idealised and design stress-strain diagrams for prestressing steel (absolute values are shown for tensile stress and strain)

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 This clause 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 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 ≥ 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 conform to 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 intended working life, without significant loss of utility or excessive unforeseen maintenance.

(2)P The required protection of the structure shall be established by considering its intended use, service 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 5.1.5).

(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) and may be related to a minimum strength class of concrete.

Note: Further information is given in Annex E.

(5) Exposed permanent metal fastenings may be of coated material if it can be inspected and replaced. If not, corrosion resistant material should be used.

(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 in general.

(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, ISO 9690)
- chlorides contained in the concrete (EN 206)
- alkali-aggregate reactions (EN 206, National Standards)

physical attack, arising from e.g.
- temperature change
- abrasion
- water penetration (EN 206).

Table 4.1: Exposure classes related to environmental conditions in accordance with EN 206

Class designation	Description of the environment	Informative examples where exposure classes may occur
1 No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there is freeze/thaw, abrasion or chemical attack For concrete with reinforcement or embedded metal: very dry	Concrete inside buildings with very low air humidity
2 Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2
3 Corrosion induced by chlorides		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools Concrete components exposed to industrial waters containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides Pavements Car park slabs
4 Corrosion induced by chlorides from sea water		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spray zones	Parts of marine structures
5. Freeze/Thaw Attack		
XF1	Moderate water saturation, without de-icing agent	Vertical concrete surfaces exposed to rain and freezing
XF2	Moderate water saturation, with de-icing agent	Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents
XF3	High water saturation, without de-icing agents	Horizontal concrete surfaces exposed to rain and freezing
XF4	High water saturation with de-icing agents or sea water	Road and bridge decks exposed to de-icing agents Concrete surfaces exposed to direct spray containing de-icing agents and freezing Splash zone of marine structures exposed to freezing
6. Chemical attack		
XA1	Slightly aggressive chemical environment according to EN 206, Table 2	Natural soils and ground water
XA2	Moderately aggressive chemical environment according to EN 206, Table 2	Natural soils and ground water
XA3	Highly aggressive chemical environment according to EN 206, Table 2	Natural soils and ground water

Note: The composition of the concrete affects both the protection of the reinforcement and the resistance of the concrete to attack. Annex C gives indicative classes for the particular environmental exposures. This may lead to the choice of higher strength than required for the structural design. In such cases the value of f_{ctm} should be associated with the higher strength in the calculation of minimum reinforcement and crack width control (see 7.3.2 -7.3.4).

4.3 Requirements for durability

(1)P In order to achieve the required working life of the structure, adequate measures shall be taken to protect each structural element against the relevant environmental actions.

(2)P The requirements for durability shall be included when considering the following:
Structural conception,
Material selection,
Construction details,
Execution,
Quality Control,
Inspection,
Verifications,
Special measures (e.g. use of stainless steel, coatings, cathodic protection).

4.4 Methods of verification

4.4.1 Concrete cover

4.4.1.1 General

(1)P The concrete cover is the distance between the surface of the reinforcement closest to the nearest concrete surface (including links and stirrups and surface reinforcement where relevant) and the nearest concrete surface.

(2)P The nominal cover shall be specified on the drawings. It is defined as a minimum cover, c_{min} (see 4.4.1.2), plus an allowance in design for tolerance, Δc_{tol} (see 4.4.1.3):

$$c_{nom} = c_{min} + \Delta c_{tol} \tag{4.1}$$

4.4.1.2 Minimum cover, c_{min}

(1)P Minimum concretecover, c_{min}, shall be provided in order to ensure:
- the safe transmission of bond forces (see also Sections 7 and 8)
- the protection of the steel against corrosion (durability)
- an adequate fire resistance (see EN 1992-1-2)

(2)P The greater value for c_{min} satisfying the requirements for both bond and environmental conditions shall be used for design.

$$c_{min} = \max \{c_{min,b};\ c_{min,dur} + \Delta c_{dur,\gamma} - \Delta c_{dur,st} - \Delta c_{dur,add};\ 10 \text{ mm}\} \tag{4.2}$$

where:

$c_{min,b}$ minimum cover due to bond requirement, see (3)
$c_{min,dur}$ minimum cover due to environmental conditions, see (5)
$\Delta c_{dur,\gamma}$ additive safety element, see (6)
$\Delta c_{dur,st}$ reduction of minimum cover for use of stainless steel, see (7)
$\Delta c_{dur,add}$ reduction of minimum cover for use of additional protection, see (8)

(3) In order to transmit bond forces safely and to ensure adequate compaction, the minimum cover should not be less than $c_{min,b}$ given in table 4.2.

Table 4.2: Minimum cover, $c_{min,b}$, requirements with regard to bond

Bond Requirement	
Type of steel	Minimum cover $c_{min,b}$*
Ordinary	Diameter of bar
Bundled	Equivalent diameter (ϕ_n)(see 8.9.1)
Post-tensioned	Circular duct for bonded tendons: diameter of the duct. There is no requirement for more than 80 mm for either type of duct.
*: If the nominal maximum aggregate size is greater than 32 mm, $c_{min,b}$ should be increased by 5 mm.	

Note: The values of $c_{min,b}$ for post-tensioned rectangular ducts for bonded tendons, and pre-tensioned tendons for use in a Country may be found in its National Annex. The recommended values are:
for post-tensioned rectangular ducts: greater of the smaller dimension or half the greater dimension
for pre-tensioned tendon: 2,0 x diameter of strand or wire
3,0 x diameter of indented wire.

(4) For prestressing tendons, the minimum cover of the anchorage should be provided in accordance with the appropriate European Technical Approval.

(5) The minimum cover values, $c_{min,dur}$, for ordinary carbon steel in normal weight concrete take account of the exposure classes and the structure classes and are given in Table 4.4 and 4.5.

Note: Structure classification and values of $c_{min,dur}$ for use in a Country may be found in its National Annex. The recommended Structural Class (service life of 50 years) is 4 for the indicative concrete strengths given in Annex C, recommended modifications to the structure class is given in Table 4.3N. The recommended value of $c_{min,dur}$ is given in Table 4.4 (reinforcing steel) and Table 4.5 (prestressing steel). The recommended minimum structure class is 1.

Table 4.3N: Recommended structure classification

Structural Class							
Criterion	**Exposure Class according to Table 4.1**						
	X0	XC1	XC2 / XC3	XC4	XD1	XD2 / XS1	XD3 / XS2 / XS3
Service Life of 100 years	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2
Strength Class 1) 2)	≥ C30/37 reduce class by 1	≥ C30/37 reduce class by 1	≥ C35/45 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C45/55 reduce class by 1
Member with slab geometry (not walkable during construction)	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1
Special Quality Control ensured	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1

Notes to Table 4.3N
1. The strength class and w/c ratio are considered to be related values. Relationship is subject to a national code. A special composition (type of cement, w/c value, fine fillers) with the intent to produce low permeability may be considered.
2. The limit may be reduced by one strength class if air entrainment of more than 4% is applied.

Table 4.4: Values of minimum cover, $c_{min,dur}$, requirements with regard to durability for reinforcement steel

Environmental Requirement for c_{min} (mm)							
Structural Class	**Exposure Class according to Table 4.1**						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
1	10	10	10	15	20	25	30
2	10	10	15	20	25	30	35
3	10	10	20	25	30	35	40
4	10	15	25	30	35	40	45
5	15	20	30	35	40	45	50
6	20	25	35	40	45	50	55

Table 4.5: Values of minimum cover, $c_{min,dur}$, requirements with regard to durability for prestressing steel

Environmental Requirement for c_{min} (mm)							
Structural Class	**Exposure Class according to Table 4.1**						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
1	10	15	20	25	30	35	40
2	10	15	25	30	35	40	45
3	10	20	30	35	40	45	50
4	10	25	35	40	45	50	55
5	15	30	40	45	50	55	60
6	20	35	45	50	55	60	65

(6) The concrete cover may be increased by the additive safety element $\Delta c_{dur,\gamma}$.

Note: The value of $\Delta c_{dur,\gamma}$ for use in a Country may be found in its National Annex. The recommended value is 0 mm.

(7) Where stainless steel is used or where other special measures have been taken, the minimum cover may be reduced by $\Delta_{cdur,st}$. For such situations the effects on all relevant material properties should be considered, including bond.

Note: The value of $\Delta c_{dur,st}$ for use in a Country may be found in its National Annex. The recommended value, without further specification, is 0 mm.

(8) For concrete with additional protection (e.g. coating) the minimum cover may be reduced by $\Delta c_{dur,add}$.

Note: The value of $\Delta c_{dur,add}$ for use in a Country may be found in its National Annex. The recommended value, without further specification, is 0 mm.

(9) Where in-situ concrete is placed against other concrete elements (precast or in-situ) the minimum concrete cover of the reinforcement to the interface may be reduced to a value corresponding to the requirement for bond (see (3) above) provided that:
- the concrete class is at least C25/30,
- the exposure time of the concrete surface to an outdoor environment is short (< 28 days),
- the interface has been roughened.

(10) For unbonded tendons the cover should be provided in accordance with the European Technical Approval.

(11) For uneven surfaces (e.g. exposed aggregate) the minimum cover should be increased by

at least 5 mm.

(12) Where freeze/thaw or chemical attack on concrete (Classes XF and XA) is expected special attention should be given to the concrete composition (see EN 206-1 Section 6). Cover in accordance with Tables 4.2 and 4.3 will normally be sufficient for such situations.

4.4.1.3 Allowance in design for tolerance

(1)P An addition to the minimum cover shall be made in design to allow for the tolerance (Δc_{tol}). The required minimum cover shall be increased by the accepted negative deviation given in the standard for execution. This may depend on the type of structure.

(2) For Buildings, EN 13670-1 gives the acceptable deviation and, hence, the allowance for tolerance. This is normally also sufficient for other types of structures. It should be considered when choosing the value of nominal cover for design. The nominal value of cover for design should be used in the calculations and stated on the drawings, unless a value other than the nominal cover is specified (e.g. minimum value).

Note: The value of Δc_{tol} for use in a Country may be found in its National Annex. The recommended value is 10 mm.

(3) In certain situations, the accepted deviation and hence allowance, Δc_{tol}, may be reduced.

Note: The reduction in Δc_{tol} in such circumstances for use in a Country may be found in its National Annex. The recommended values are:

- where fabrication is subjected to a quality assurance system, in which the monitoring includes measurements of the concrete cover, the allowance in design for tolerance Δc_{tol} may be reduced:

$$10 \text{ mm} \geq \Delta c_{tol} \geq 5 \text{ mm} \qquad (4.3N)$$

- where it can be assured that a very sensitive measurement device is used for monitoring and non conforming members are rejected (e.g. precast elements), the allowance in design for tolerance Δc_{tol} may be reduced:

$$10 \text{ mm} \geq \Delta c_{tol} \geq 0 \text{ mm} \qquad (4.4N)$$

(4) For concrete cast against uneven surfaces, the minimum cover should generally be increased by allowing larger tolerances in design. The increase should comply with the difference caused by the unevenness, but the cover should be at least 40 mm for concrete cast against prepared ground (including blinding) and 75 mm for concrete cast directly against soil. The cover to the reinforcement for any surface feature, such as ribbed finishes or exposed aggregate, should also be increased to take account of the uneven surface.

SECTION 5 STRUCTURAL ANALYSIS

5.1 General provisions

(1)P The purpose of structural analysis is to establish the distribution of either internal forces and moments, or stresses, strains and displacements, over the whole or part of a structure. Additional local analysis shall be carried out where necessary.

Note: In most normal cases analysis will be used to establish the distribution of internal forces and moments, and the complete verification or demonstration of resistance of cross sections is based on these action effects; however, for certain particular elements, the methods of analysis used (e.g. finite element analysis) give stresses, strains and displacements rather than internal forces and moments. Special methods are required to use these results to obtain appropriate verification.

(2) Local analyses may be necessary where the assumption of linear strain distribution is not valid, e.g.:
- in the vicinity of supports
- local to concentrated loads
- in beam-column intersections
- in anchorage zones
- at changes in cross section.

(3) For in-plane stress fields a simplified method for determining reinforcement may be used.

Note: A simplified method is given in Annex F.

(4)P Analyses shall be carried out using idealisations of both the geometry and the behaviour of the structure. The idealisations selected shall be appropriate to the problem being considered.

(5) The geometry is commonly idealised by considering the structure to be made up of linear elements, plane two dimensional elements and, occasionally, shells. Geometric idealisations are considered in 5.3.

(6)P The effect of the geometry and properties of the structure on its behaviour at each stage of construction shall be considered in the design

(7) Common idealisations of the behaviour used for analysis are:
- linear elastic behaviour (see 5.4)
- linear elastic behaviour with limited redistribution (see 5.5)
- plastic behaviour (see 5.6), including strut and tie models (see 5.6.4)
- non-linear behaviour (see 5.7)

(8) In buildings, the effects of shear and longitudinal forces on the deformations of linear elements and slabs may be ignored where these are likely to be less than 10% of those due to bending.

5.1.1 Special 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.

Note: For more information concerning the analysis of shallow foundations see Annex G.

(2) For the design of spread foundations, appropriately simplified models for the description of the soil-structure interaction may be used.

Note: For simple pad footings and pile caps the effects of soil-structure interaction may usually be ignored.

(3) For the strength design of individual piles the actions should be determined taking into account the interaction between the piles, the pile cap and the supporting soil.

(4) Where the piles are located in several rows, the action on each pile should be evaluated by considering the interaction between the piles.

(5) This interaction may be ignored when the clear distance between the piles is greater than two times the pile diameter.

5.1.2 Load cases and combinations

(1)P In considering the combinations of actions, see EN 1990 Section 6, the relevant cases shall be considered to enable the critical design conditions to be established at all sections, within the structure or part of the structure considered.

Note: Where a simplification in the number of load arrangements for use in a Country is required, reference is made to its National Annex. The following simplified load arrangements are recommended for buildings:

(a) alternate spans carrying the design variable and permanent load ($\gamma_Q Q_k + \gamma_G G_k + P_m$), other spans carrying only the design permanent load, $\gamma_G G_k + P_m$ and

(b) any two adjacent spans carrying the design variable and permanent loads ($\gamma_Q Q_k + \gamma_G G_k + P_m$). All other spans carrying only the design permanent load, $\gamma_G G_k + P_m$.

5.1.3 Second order effects

(1)P Second order effects (see EN 1990 Section 1),, shall be taken into account where they are likely to affect the overall stability of a structure significantly and for the attainment of the ultimate limit state at critical sections.

(2) Second order effects should be taken into account according to 5.8.

(3) For buildings, second order effects below certain limits may be ignored (see 5.8.2 (6)).

5.1.4 Deformations of concrete

(1)P Time dependent deformations of concrete from creep and shrinkage shall be taken into account where significant.

Note: For further guidance see 2.3.2.2.

(2)P The consequences of deformation due to temperature, creep and shrinkage shall be considered in design.

(3) The influence of the effects temperature, creep and shrinkage are 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.

5.1.5 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, for example in the verification of stability where second order effects are of importance. In other cases they need not be considered, provided that the ductility and rotation capacity of the members 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.

5.1.7 Uneven settlements

(1) Uneven settlements 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 between individual foundations or part of foundations, $d_{set,i}$ (i denotes the number of the individual foundation or part of foundation).

(2) The effects of uneven 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, for example where second order effects are of importance. 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 uneven settlements are taken into account they should be applied with a partial safety factor.

Note: The relevant partial safety factor is defined in the relevant annex of EN1990.

5.2 Geometric imperfections

(1)P The unfavourable effects of possible deviations in the geometry of the structure and the position of loads shall be taken into account in the analysis of members and structures.

Note: Deviations in cross section dimensions are normally taken into account in the material safety factors. A minimum eccentricity for cross section design is given in 6.1 (4). These should not be included in structural analysis.

(2)P Imperfections shall be taken into account in ultimate limit states in persistent and accidental design situations.

(3) Imperfections need not be considered for serviceability limit states.

(4) The following provisions apply for members with axial compression and structures with vertical load, mainly in buildings. Numerical values are related to normal execution tolerances (Class 1 in ENV 13670). With the use of other tolerances (e.g. Class 2), values should be adjusted accordingly.

(5) Imperfections may be represented by an inclination

$$\theta_i = \theta_0 \cdot \alpha_h \cdot \alpha_m \tag{5.1}$$

where

θ_0 is the basic value:

α_h is the reduction factor for height: $\alpha_h = 2/\sqrt{l}$; $2/3 \leq \alpha_h \leq 1$

α_m is the reduction factor for number of members: $\alpha_m = \sqrt{0,5(1+1/m)}$

l is the length or height [m], see (4)

m is the number of vertical members contributing to the total effect, see (4)

Note: The value of θ_0 for use in a Country may be found in its National Annex. The recommended value is 1/200

(6) In Expression (5.1), the definition of l and m depends on the effect considered, for which three main cases can be distinguished (see also Figure 5.1):

- Effect on isolated member: l = actual length of member, m =1.
- Effect on bracing system: l = height of building, m = number of vertical members contributing to the horizontal force on the bracing system.
- Effect on floor or roof diaphragms distributing the horizontal loads: l = storey height, m = number of vertical elements in the storey(s) contributing to the total horizontal force on the floor.

(7) For isolated members (see 5.8.1), the effect of imperfections may be taken into account in two alternative ways a) and b):

a) as an eccentricity e_i:
$e_i = \theta_i\, l_0 / 2$ where l_0 is the effective length, see 5.8.3.2 (5.2)

For walls and isolated columns in braced systems, $e_i = l_0/400$ can always be used as a simplification, corresponding to $\alpha_h = 1$.

b) as a transverse force H_i in the position that gives maximum moment:

for unbraced members (see Figure 5.1 a1):

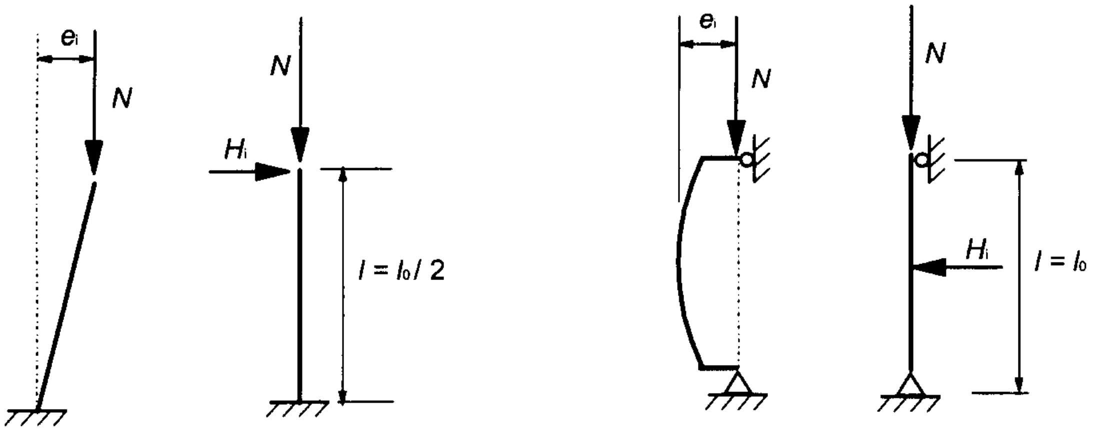
$$H_i = \theta_i N \tag{5.3a}$$

for braced members (see Figure 5.1 a2):

$$H_i = 2\theta_i N \tag{5.3b}$$

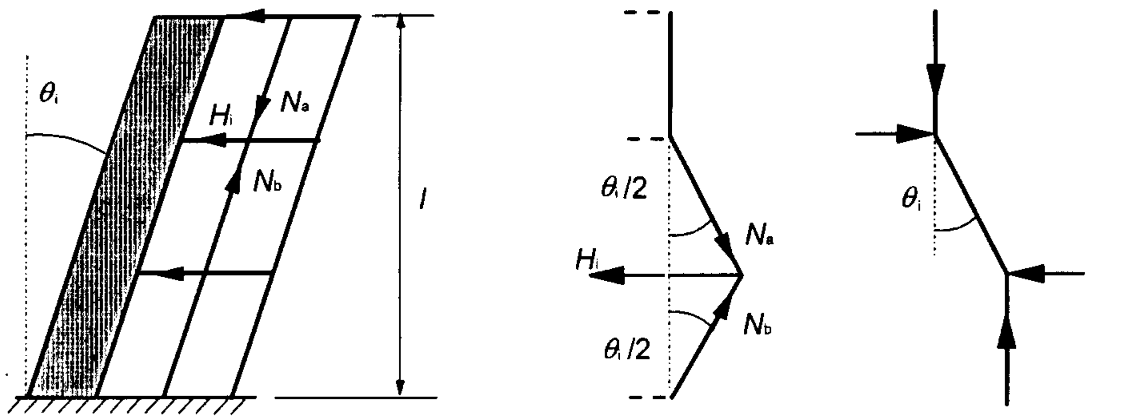
where N is the axial load

Note: Eccentricity is suitable for statically determinate members, whereas transverse load can be used for both determinate and indeterminate members. The force H_i can be substituted by some other equivalent transverse action.



a1) Unbraced **a2) Braced**

a) Isolated members with eccentric axial force or lateral force



b) Bracing system **c1) Floor diaphragm** **c2) Roof diaphragm**

Figure 5.1: Examples of the effect of geometric imperfections

(8) For structures, the effect of the inclination θ_i may be represented by transverse forces, to be included in the analysis together with other actions.

Effect on bracing system, (see Figure 5.1 b):

$$H_i = \theta_i \,(N_b - N_a) \qquad (5.4)$$

Effect on floor diaphragm, (see Figure 5.1 c1):

$$H_i = \theta_i (N_b + N_a) / 2 \qquad (5.5)$$

Effect on roof diaphragm, (see Figure 5.1 c2):

$$H_i = \theta_i \cdot N_a \qquad (5.6)$$

where N_a and N_b are vertical forces contributing to H_i.

(9) As a simplified alternative for walls and isolated columns in braced systems, an eccentricity $e_i = l_0/400$ may be used to cover imperfections related to normal tolerances (see 5.2 (4)).

5.3 Idealisation of the structure

5.3.1 Structural models for overall analysis

(1)P The elements of a structure are classified, by consideration of their nature and function, as beams, columns, slabs, walls, plates, arches, shells etc. Rules are provided for the analysis of the commoner of these elements and of structures consisting of combinations of these elements.

(2) For buildings the following provisions (3) to (7) are applicable:

(3) A beam is a member for which the span is not less than 3 times the overall section depth. Otherwise it should be considered as a deep beam.

(4) A slab is a member for which the minimum panel dimension is not less than 5 times the overall slab thickness.

(5) A slab subjected to dominantly uniformly distributed loads may be considered to be one-way spanning if either:

- it possesses two free (unsupported) and sensibly parallel edges, or
- it is the central part of a sensibly rectangular slab supported on four edges with a ratio of the longer to shorter span greater than 2.

(6) Ribbed or waffle slabs need not be treated as discrete elements for the purposes of analysis, provided that the flange or structural topping and transverse ribs have sufficient torsional stiffness. This may be assumed provided that:

- the rib spacing does not exceed 1500 mm
- the depth of the rib below the flange does not exceed 4 times its width.
- the depth of the flange is at least 1/10 of the clear distance between ribs or 50 mm, whichever is the greater.
- transverse ribs are provided at a clear spacing not exceeding 10 times the overall depth of the slab.

The minimum flange thickness of 50 mm may be reduced to 40 mm where permanent blocks are incorporated between the ribs.

(7) A column is a member for which the section depth does not exceed 4 times its width and the height is at least 3 times the section depth. Otherwise it should be considered as a wall.

5.3.2 Geometric data

5.3.2.1 Effective width of flanges (all limit states)

(1)P In T beams the effective flange width, over which uniform conditions of stress can be assumed, depends on the web and flange dimensions, the type of loading, the span, the support conditions and the transverse reinforcement.

(2) The effective width of flange should be based on the distance l_0 between points of zero moment, which may be obtained from Figure 5.2.

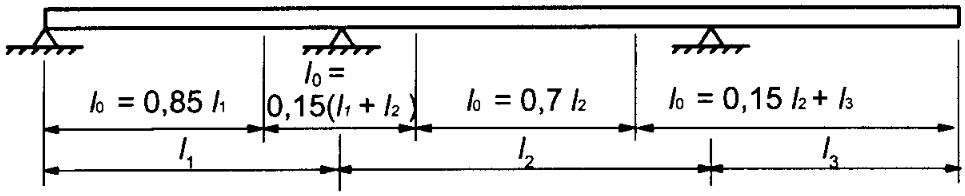


Figure 5.2: Definition of l_0, for calculation of flange width

Note: The length of the cantilever should be less than half the adjacent span and the ratio of adjacent spans should lie between 2/3 and 1,5.

(3) The effective flange width b_{eff} for a T beam or *L* beam may be derived as:

$$b_{eff} = \sum b_{eff,i} + b_w \leq b \tag{5.7}$$

where

$$b_{eff,i} = 0,2b_i + 0,1l_0 \leq 0,2l_0 \tag{5.7a}$$

and

$$b_{eff,i} \leq b_i \tag{5.7b}$$

(for the notations see Figures 5.2 above and 5.3 below).

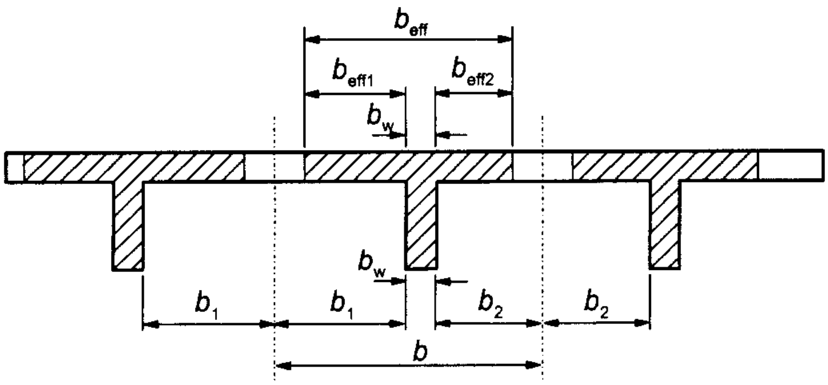


Figure 5.3: Effective flange width parameters

(4) For structural analysis, where a great accuracy is not required, a constant width may be assumed over the whole span. The value applicable to the span section should be adopted.

5.3.2.2 Effective span of beams and slabs in buildings

Note: The following provisions are provided mainly for member analysis. For frame analysis some of these simplifications may be used where appropriate.

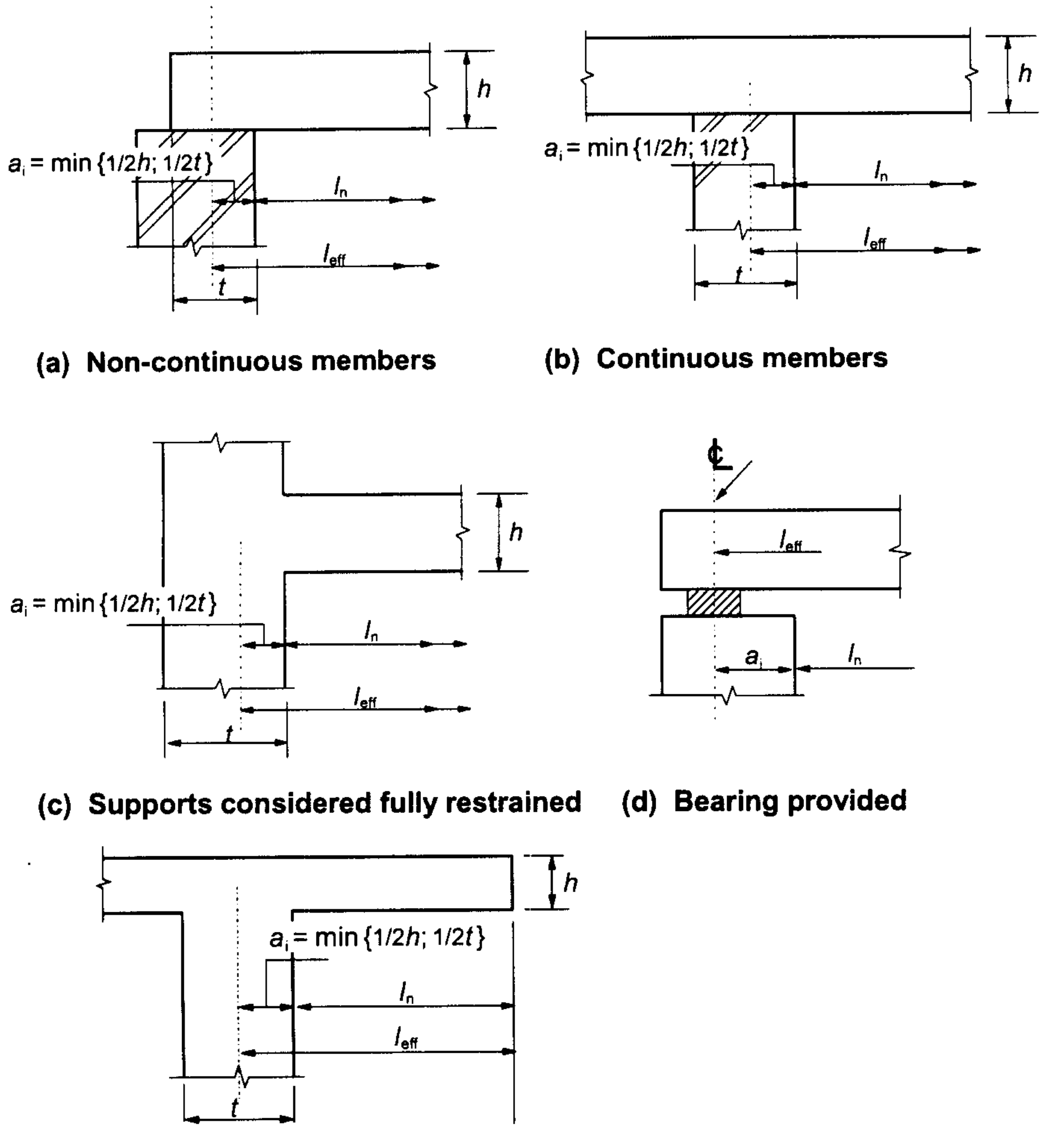
(1) The effective span, l_{eff}, of a member should be calculated as follows:

$$l_{eff} = l_n + a_1 + a_2 \tag{5.8}$$

where:

l_n is the clear distance between the faces of the supports;

values for a_1 and a_2 , at each end of the span, may be determined from the appropriate a_i values in Figure 5.4 where t is the width of the supporting element as shown.



(a) Non-continuous members

(b) Continuous members

(c) Supports considered fully restrained

(d) Bearing provided

(e) Cantilever

Figure 5.4: Effective span (l_{eff}) for different support conditions

(2) Continuous slabs and beams may generally be analysed on the assumption that the supports provide no rotational restraint.

(3) Where a beam or slab is monolithic with its supports, the critical design moment at the support should be taken as that at the face of the support. The design moment and reaction transferred to the supporting element (e.g. column, wall, etc.) should be taken as the greater of the elastic or redistributed values.

Note: The moment at the face of the support should not be less than 0.65 that of the full fixed end moment.

(3) Regardless of the method of analysis used, where a beam or slab is continuous over a support which may be considered to provide no restraint to rotation (e.g. over walls), the design support moment, calculated on the basis of a span equal to the centre-to-centre distance between supports, may be reduced by an amount ΔM_{Ed} as follows:

$$\Delta M_{Ed} = F_{Ed,sup}\, t / 8 \tag{5.9}$$

where:

$F_{Ed,sup}$ is the design support reaction
t is the breadth of the support (see Figure 5.4 b))

5.4 Linear elastic analysis

(1) Linear analysis of elements based on the theory of elasticity may be used for both the serviceability and ultimate limit states.

(2) For the determination of the action effects, linear analysis may be carried out assuming:
i) uncracked cross sections,
ii) linear stress-strain relationships and
iii) mean values of the elastic modulus.

(3) For thermal deformation, settlement and shrinkage effects at the ultimate limit state (ULS), a reduced stiffness corresponding to the cracked sections, neglecting tension stiffening but including the effects of creep, may be assumed. For the serviceability limit state (SLS) a gradual evolution of cracking should be considered.

5.5 Linear elastic analysis with limited redistribution

(1)P The influence of any redistribution of the moments on all aspects of the design shall be considered.

(2) Linear analysis with limited redistribution may be applied to the analysis of structural members for the verification of ULS.

(3 The moments at ULS calculated using a linear elastic analysis may be redistributed, provided that the resulting distribution of moments remains in equilibrium with the applied loads.

(4) In continuous beams or slabs which:
a) are predominantly subject to flexure and
b) have the ratio of the lengths of adjacent slabs in the range of 0,5 to 2,
redistribution of bending moments may be carried out without explicit check on the rotation capacity provided that:

$$\delta \geq k_1 + k_2 x_u/d \quad \text{for } f_{ck} \leq 50 \text{ MPa} \tag{5.10a}$$

$$\delta \geq k_3 + k_4 x_u/d \quad \text{for } f_{ck} > 50 \text{ MPa} \tag{5.10b}$$

$\geq 0{,}70$ where Class B and Class C reinforcement is used
$\geq 0{,}80$ where Class A reinforcement is used

Where:

δ is the ratio of the redistributed moment to the elastic bending moment
x_u is the depth of the neutral axis at the ultimate limit state after redistribution

d is the effective depth of the section

ε_{cu1} is the ultimate strain for the section in accordance with Table 3.1

Note: The values of k_1, k_2, k_3 and k_4 for use in a Country may be found in its National Annex. The recommended value for k_1 is 0,44, for k_2 is 1,25(0,6+0,0014/ε_{cu}), for k_3 = 0,54 and for k_4 = 1,25(0,6+0,0014/ε_{cu})

(5) Redistribution should not be carried out in circumstances where the rotation capacity cannot be defined with confidence (e.g. in the corners of prestressed frames).

(6) For the design of columns the elastic moments from frame action should be used without any redistribution.

5.6 Plastic methods of analysis

5.6.1 General

(1)P Methods based on plastic analysis shall only be used for the check at ULS.

(2)P The ductility of the critical sections shall be sufficient for the envisaged mechanism to be formed.

(3)P The plastic analysis should be based either on the lower bound (static) method or on the upper bound (kinematic) method.

(4) The effects of previous applications of loading may generally be ignored, and a monotonic increase of the intensity of actions may be assumed.

5.6.2 Plastic analysis for beams, frames and slabs

(1)P Plastic analysis without any direct check of rotation capacity may be used for the ultimate limit state if the conditions of 5.6.1 (2)P are met.

(2) The required ductility may be deemed to be satisfied if all the following are fulfilled:

i) the area of tensile reinforcement is limited such that, at any section
$x_u/d \leq 0,25$ for concrete strength classes $\leq$ C50/60
$\leq 0,15$ for concrete strength classes $\geq$ C55/67

ii) reinforcing steel is either Class B or C

iii) the ratio of the moments at intermediate supports to the moments in the span shall be between 0,5 and 2.

(3) Columns should be checked for the maximum plastic moments which can be transmitted by connecting members. For connections to flat slabs this moment should be included in the punching shear calculation.

(4) When plastic analysis of slabs is carried out account should be taken of any non-uniform reinforcement, corner tie down forces, and torsion at free edges.

(5) Plastic methods may be extended to non-solid slabs (ribbed, hollow, waffle slabs) if their response is similar to that of a solid slab, particularly with regard to the torsional effects.

5.6.3 Rotation capacity

(1) The simplified procedure for continuous beams and continuous one way spanning slabs is based on the rotation capacity of beam/slab zones over a length of approximately 1,2 times the

depth of section. It is assumed that these zones undergo a plastic deformation (formation of yield hinges) under the relevant combination of actions. The verification of the plastic rotation in the ultimate limit state is considered to be fulfilled, if it is shown that under the relevant action the calculated rotation, θ_s, is less than or equal to the allowable plastic rotation, $\theta_{pl,d}$ (see Figure 5.5).

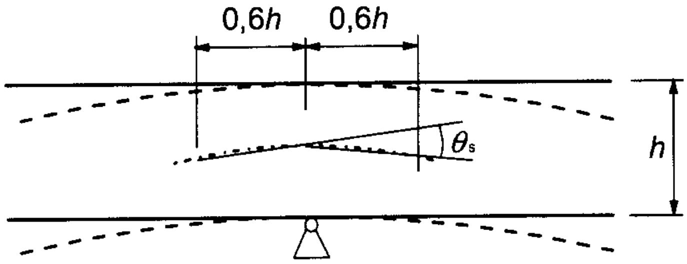


Figure 5.5: Plastic rotation θ_s of reinforced concrete sections for continuous beams and continuous one way spanning slabs.

(2) In regions of yield hinges, x_u/d shall not exceed the value 0,45 for concrete strength classes less than or equal to C50/60, and 0,35 for concrete strength classes greater than or equal to C55/67.

(3) The rotation θ_s should be determined on the basis of the design values for actions and materials and on the basis of mean values for prestressing at the relevant time.

(4) In the simplified procedure, the allowable plastic rotation may be determined by multiplying the basic value of allowable rotation by a correction factor k_λ that depends on the shear slenderness.

Note: Values of $\theta_{pl,d}$ for use in a Country may be found in its National Annex. The recommended values for steel Classes B and C (the use of Class A steel is not recommended for plastic analysis) and concrete strength classes less than or equal to C50/60 and C90/105 are given in Figure 5.6N.

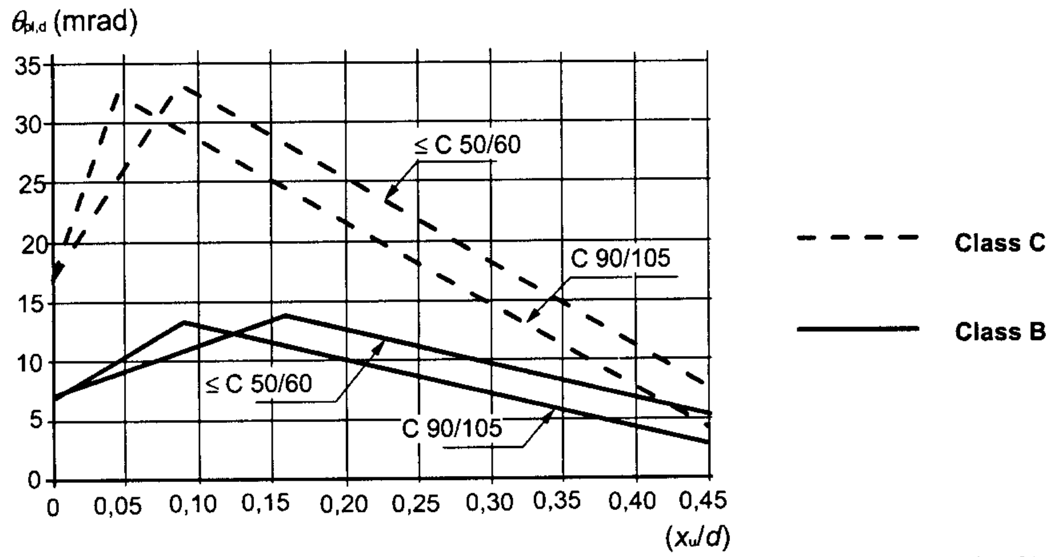


Figure 5.6N: Allowable plastic rotation, $\theta_{pl,d}$, of reinforced concrete sections for Class B and C reinforcement. The values apply for a shear slenderness $\lambda = 3{,}0$

The values for concrete strength classes C 55/67 to C 90/105 may be interpolated accordingly. The values apply for a shear slenderness λ = 3,0. For different values of shear slenderness $\theta_{pl,d}$ should be multiplied by k_λ:

$$k_\lambda = \sqrt{\lambda / 3} \tag{5.11N}$$

Where λ is the ratio of the distance between point of zero and maximum moment after redistribution and effective depth, d.

As a simplification λ may be calculated for the concordant design values of the bending moment and shear :

$$\lambda = M_{Sd} / (V_{Sd} \cdot d) \tag{5.12N}$$

5.6.4 Analysis with struts and ties

(1) Strut and tie models may be used for design in ULS of continuity regions (cracked state of beams and slabs, see 6.1 - 6.4) and for the design in ULS and detailing of discontinuity regions (see 6.5). In general these extend up to a distance h (section depth of member) from the discontinuity. Strut and tie models may also be used for members where a linear distribution within the cross section is assumed, e.g. plane strain.

(2) Verifications in SLS may also be carried out using strut-and-tie models, e.g. verification of steel stresses and crack width control, if approximate compatibility for strut-and-tie models is ensured (in particular the position and direction of important struts should be oriented according to linear elasticity theory)

(3) Strut-and-tie models consist of struts representing compressive stress fields, of ties representing the reinforcement, and of the connecting nodes. The forces in the elements of a strut-and-tie model shall be determined by maintaining the equilibrium with the applied loads in the ultimate limit state. The elements of strut-and-tie models should be dimensioned according to the rules given in 6.5.1 and 6.5.2.

(4) The ties of a strut-and-tie model should coincide in position and direction with the corresponding reinforcement.

(5) Possible means for developing suitable strut-and-tie models include the adoption of stress trajectories and distributions from linear-elastic theory or the load path method. All strut-and-tie models may be optimised by energy criteria.

5.7 Non-linear analysis

(1)P Non-linear methods of analysis may be used for both ULS and SLS, provided that equilibrium and compatibility are satisfied and an adequate non-linear behaviour for materials is assumed. The analysis may be first or second order.

(2)P At the ultimate limit state, the ability of local critical sections to withstand any inelastic deformations implied by the analysis shall be checked, taking appropriate account of uncertainties.

(3) For structures predominantly subjected to static loads, the effects of previous applications of loading may generally be ignored, and a monotonic increase of the intensity of the actions may be assumed.

(4)P The use of material characteristics which represent the stiffness in a realistic way but take account of the uncertainties of failure shall be used when using non-linear analysis. Only those design formats which are valid within the restricted fields of application shall be used.

(5) For slender structures, in which second order effects cannot be ignored, the design method given in 5.8.6 may be used.

5.8 Second order effects with axial load

5.8.1 Definitions

Biaxial bending: simultaneous bending about two principal axes

Braced members or systems: structural members or subsystems, which in analysis and design are assumed *not* to contribute to the overall horizontal stability of a structure

Bracing members or systems: structural members or subsystems, which in analysis and design are assumed to contribute to the overall horizontal stability of a structure

Buckling: failure due to instability of a member or structure under perfectly axial compression and without transverse load

Note. "Pure buckling" as defined above is not a relevant limit state in real structures, due to imperfections and transverse loads, but a nominal buckling load can be used as a parameter in some methods for second order analysis.

Buckling load: the load at which buckling occurs; for isolated elastic members it is synonymous with the Euler load

Effective length: a length used to account for the shape of the deflection curve; it can also be defined as buckling length, i.e. the length of a pin-ended column with constant normal force, having the same cross section and buckling load as the actual member

First order effects: action effects calculated without consideration of the effect of structural deformations, but including geometric imperfections

Isolated members: members that *are* isolated, or members in a structure that for design purposes may be treated as being isolated; examples of isolated members with different boundary conditions are shown in Figure 5.7.

Nominal second order moment: a second order moment used in certain design methods, giving a total moment compatible with the ultimate cross section resistance; 5.8.5 (2)

Second order effects: additional action effects caused by structural deformations

5.8.2 General

(1)P This section deals with members and structures in which the structural behaviour is significantly influenced by second order effects (e.g. columns, walls, piles, arches and shells). Global second order effects are likely to occur in structures with a flexible bracing system.

(2)P Where second order effects are taken into account, see (6), equilibrium and resistance shall be verified in the deformed state. Deformations shall be calculated taking into account the relevant effects of cracking, non-linear material properties and creep.

Note. In an analysis assuming linear material properties, this can be taken into account by means of reduced stiffness values, see 5.8.7.

(3)P Where relevant, analysis shall include the effect of flexibility of adjacent members and foundations (soil-structure interaction).

(4)P The structural behaviour shall be considered in the direction in which deformations can occur, and biaxial bending shall be taken into account when necessary.

(5)P Uncertainties in geometry and position of axial loads shall be taken into account as additional first order effects based on geometric imperfections, see 5.2.

(6) Second order effects may be ignored if they are less than 10 % of the corresponding first order effects. Simplified criteria are given for isolated members in 5.8.3.1 and for structures in 5.8.3.3.

5.8.3 Simplified criteria for second order effects

5.8.3.1 Slenderness criterion for isolated members

(1) As an alternative to 5.8.2 (6), second order effects may be ignored if the slenderness λ is below a certain value λ_{lim}. The following may be used:

$$\lambda_{lim} = 20 \cdot A \cdot B \cdot C / \sqrt{n} \tag{5.13}$$

where:

λ is the slenderness ratio as defined in 5.8.3.2
A $= 1 / (1+0,2\varphi_{ef})$ (if φ_{ef} is not known, A = 0,7 may be used)
B $= \sqrt{1+2\omega}$ (if ω is not known, B = 1,1 may be used)
C $= 1,7 - r_m$ (if r_m is not known, C = 0,7 may be used)
φ_{ef} effective creep ratio; see 5.8.4;
ω $= A_s f_{yd} / (A_c f_{cd})$; mechanical reinforcement ratio;
A_s is the total area of longitudinal reinforcement
n $= N_{Ed} / (A_c f_{cd})$; relative normal force
r_m $= M_{01}/M_{02}$; moment ratio
M_{01}, M_{02} are the first order end moments, $|M_{02}| \geq |M_{01}|$

(2) If the end moments M_{01} and M_{02} give tension on the same side, r_m should be taken positive (i.e. $C \leq 1,7$), otherwise negative (i.e. $C > 1,7$).

In the following cases, r_m should be taken as 1,0 (i.e. C = 0,7):
- for braced members with first order moments only or predominantly due to imperfections or transverse loading
- for unbraced members in general

(3) In cases with biaxial bending, the slenderness criterion may be checked separately for each direction. Depending on the outcome of this check, second order effects (a) may be

ignored in both directions, (b) should be taken into account in one direction, or (c) should be taken into account in both directions.

5.8.3.2 Slenderness and effective length of isolated members

(1) The slenderness ratio is defined as follows:

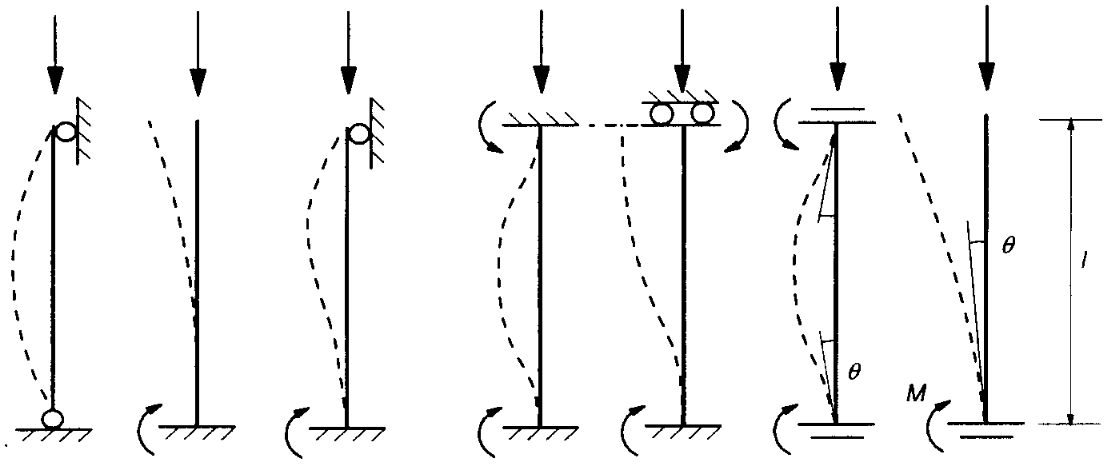
$$\lambda = l_0 / i \tag{5.14}$$

where:

l_0 is the effective length, see (2) to (7) below

i is the radius of gyration of the uncracked concrete section

(2) For a general definition of the effective length, see 5.8.1. Examples of effective length for isolated members with constant cross section are given in Figure 5.7.



a) $l_0 = l$ b) $l_0 = 2l$ c) $l_0 = 0,7l$ d) $l_0 = l / 2$ e) $l_0 = l$ f) $l/2 < l_0 < l$ g) $l_0 > 2l$

Figure 5.7: Examples of different buckling modes and corresponding effective lengths for isolated members

(3) For compression members in regular frames, the slenderness criterion (Expression (5.13)) should be checked with an effective length l_0 determined in the following way.

Braced members (see Figure 5.7 (f)):

$$l_0 = 0{,}5l \cdot \sqrt{\left(1 + \frac{k_1}{0{,}45 + k_1}\right) \cdot \left(1 + \frac{k_2}{0{,}45 + k_2}\right)} \tag{5.15}$$

Unbraced members (see Figure 5.7 (g)):

$$l_0 = l \cdot \max\left\{ \sqrt{1 + 10 \cdot \frac{k_1 \cdot k_2}{k_1 + k_2}}\ ;\ \left(1 + \frac{k_1}{1 + k_1}\right) \cdot \left(1 + \frac{k_2}{1 + k_2}\right) \right\} \tag{5.16}$$

where:

k_1, k_2 are the relative flexibilities of rotational restraints at ends 1 and 2 respectively:

$k = (\theta / M) \cdot (EI / l)$

θ is the rotation of restraining members for bending moment M; see also Figure 5.7 (f) and (g)
EI is the bending stiffness of compression member, see also (4) and (5)
l is the clear height of compression member between end restraints

Note: $k = 0$ is the theoretical limit for rigid rotational restraint, and $k = \infty$ represents the limit for no restraint at all. Since fully rigid restraint is rare in practise, a minimum value of 0,1 is recommended for k_1 and k_2.

(4) If an adjacent compression member (column) in a node is likely to contribute to the rotation at buckling, then (EI/l) in the definition of k should be replaced by $[(EI/l)_a+(EI/l)_b]$, a and b representing the compression member (column) above and below the node.

(5) In the definition of effective lengths, the stiffness of restraining members should include the effect of cracking, unless they can be shown to be uncracked in ULS.

(6) For other cases than those in (2) and (3), e.g. members with varying normal force and/or cross section, the criterion in 5.8.3.1 should be checked with an effective length based on the buckling load (calculated e.g. by a numerical method):

$$l_0 = \pi\sqrt{EI / N_B} \qquad (5.17)$$

where:
EI is a representative bending stiffness
N_B is buckling load expressed in terms of this EI
(in Expression (5.14), i should also correspond to this EI)

(7) The restraining effect of transverse walls may be allowed for in the calculation of the effective length of walls by the factor β given in 12.6.5.1. In Expression (12.9) and Table 12.1, l_w is then substituted by l_0 determined according to 5.8.3.2.

5.8.3.3 Global second order effects in buildings

(1) As an alternative to 5.8.2 (6), global second order effects in buildings may be ignored if

$$F_{V,Ed} \le 0,31 \cdot \frac{n_s}{n_s + 1,6} \cdot \frac{\sum E_{cd} I_c}{L^2} \qquad (5.18)$$

where:
$F_{V,Ed}$ is the total vertical load (on braced *and* bracing members)
n_s is the number of storeys
L is the total height of building above level of moment restraint
E_{cd} is the design value of the modulus of elasticity of concrete, see 5.8.6 (3)
I_c is the second moment of area (uncracked concrete section) of bracing member(s)

Expression (5.18) is valid only if all the following conditions are met:
- torsional instability is not governing, i.e. structure is reasonably symmetrical
- global shear deformations are negligible (as in a bracing system mainly consisting of shear walls without large openings)
- bracing members are rigidly fixed at the base, i.e. rotations are negligible
- the stiffness of bracing members is reasonably constant along the height

- the total vertical load increases by approximately the same amount per storey

(2) The constant 0,31 in Expression (5.18) may be replaced by 0,62 if it can be verified that bracing members are uncracked in ultimate limit state.

Note. For cases where the bracing system has significant global shear deformations and/or end rotations, see Informative Annex H (which also gives the background to the above rules).

5.8.4 Creep

(1)P The effect of creep shall be taken into account in second order analysis, with due consideration of both the general conditions for creep (see 3.1.3) and the duration of different loads in the load combination considered.

(2) The duration of loads may be taken into account in a simplified way by means of an effective creep ratio, φ_{ef}, which, used together with the design load, gives a creep deformation (curvature) corresponding to the quasi-permanent load:

$$\varphi_{ef} = \varphi_{(\infty,t0)} \cdot M_{0Eqp} / M_{0Ed} \tag{5.19}$$

where:

$\varphi_{(\infty,t0)}$ is the final creep coefficient according to 3.1.4
M_{0Eqp} is the first order bending moment in quasi-permanent load combination (SLS)
M_{0Ed} is the first order bending moment in design load combination (ULS)

Note. It is also possible to base φ_{ef} on total bending moments M_{Eqp} and M_{Ed}, but this requires iteration and a verification of stability under quasi-permanent load with $\varphi_{ef} = \varphi_{(\infty,t0)}$.

(3) If M_{0Eqp} / M_{0Ed} varies in a member or structure, the ratio may be calculated for the section with maximum moment, or a representative mean value may be used.

(4) The effect of creep may be ignored, i.e. $\varphi_{ef} = 0$ may be assumed, if the following three conditions are met:

- $\varphi_{(\infty,t0)} \leq 2$
- $\lambda \leq 75$
- $M_{0Ed}/N_{Ed} \geq h$

Here M_{0Ed} is the first order moment and h is the cross section depth in the corresponding direction.

Note. If the conditions for neglecting second order effects according to 5.8.2 (6) or 5.8.3.3 are only just achieved, it may be too unconservative to neglect both second order effects and creep, unless the mechanical reinforcement ratio (ω, see 5.8.3.1 (1)) is at least 0,25.

5.8.5 Methods of analysis

(1) The methods of analysis include a general method, based on non-linear second order analysis, see 5.8.6 and the following two simplified methods:

(a) Second order analysis based on nominal stiffness, see (2) below
(b) Method based on estimation of curvature, see (2) below

Note: The selection of Simplified Method (a) and (b) to be used in a Country may be found in its National Annex.

(2) Nominal second order moments provided by the simplified methods (a) and (b) are sometimes greater than those corresponding to instability. This is to ensure that the total moment is compatible with the cross section resistance.

(3) Method (a) may be used for both isolated members and whole structures, if nominal stiffness values are estimated appropriately; see 5.8.7.

(4) Method (b) is mainly suitable for isolated members; see 5.8.8. However, with realistic assumptions concerning the distribution of curvature, the method in 5.8.8 can also be used for structures.

5.8.6 General method

(1)P The general method is based on non-linear analysis, including geometric non-linearity i.e. second order effects. The general rules for non-linear analysis given in 5.7 apply.

(2)P Stress-strain curves for concrete and steel suitable for overall analysis shall be used. The effect of creep shall be taken into account.

(3) Stress-strain relationships for concrete and steel given in 3.1.5, Expression (3.14) and 3.2.3 (Figure 3.8) may be used. With stress-strain diagrams based on design values, a design value of the ultimate load is obtained directly from the analysis. In Expression (3.14), and in the *k*-value, f_{cm} is then substituted by the design compressive strength f_{cd} and E_{cm} is substituted by

$$E_{cd} = E_{cm} / \gamma_{cE} \qquad (5.20)$$

Note: The value of γ_{cE} for use in a Country may be found in its National Annex. The recommended value is 1,2.

(4) In the absence of more refined models, creep may be taken into account by multiplying all strain values in the concrete stress-strain diagram according to (3) with a factor $(1 + \varphi_{ef})$, where φ_{ef} is the effective creep ratio according to 5.8.4.

(5) The favourable effect of tension stiffening may be taken into account.

Note: This effect is favourable, and may always be ignored, for simplicity.

(6) Normally, conditions of equilibrium and strain compatibility are satisfied in a number of cross sections. A simplified alternative is to consider only the critical cross section(s), and to assume a relevant variation of the curvature in between, e.g. similar to the first order moment or simplified in another appropriate way.

5.8.7 Second order analysis based on nominal stiffness

5.8.7.1 General

(1) In a second order analysis based on stiffness, nominal values of the flexural stiffness should be used, taking into account the effects of cracking, material non-linearity and creep on the overall behaviour. This also applies to adjacent members involved in the analysis, e.g. beams, slabs or foundations. Where relevant, soil-structure interaction should be taken into account.

(2) The nominal stiffness should be defined in such a way that total bending moments resulting from the analysis can be used for design of cross sections to their resistance for bending moment and axial force, cf 5.8.5 (2).

5.8.7.2 Nominal stiffness

(1) The following model may be used to estimate the nominal stiffness of slender compression members with arbitrary cross section:

$$EI = K_c E_{cd} I_c + K_s E_s I_s \qquad (5.21)$$

where:

- E_{cd} is the design value of the modulus of elasticity of concrete, see 5.8.6 (3)
- I_c is the moment of inertia of concrete cross section
- E_s is the design value of the modulus of elasticity of reinforcement, 5.8.6 (3)
- I_s is the second moment of area of reinforcement, about the centre of area of the concrete
- K_c is a factor for effects of cracking, creep etc, see (2) below
- K_s is a factor for contribution of reinforcement, see (2) below

(2) The following factors may be used in Expression (5.21), provided $\rho \geq 0{,}002$:

$$K_s = 1$$
$$K_c = k_1 k_2 / (1 + \varphi_{ef}) \qquad (5.22)$$

where:

- ρ is the geometric reinforcement ratio, A_s/A_c
- A_s is the total area of reinforcement
- A_c is the area of concrete section
- φ_{ef} is the effective creep ratio, see 5.8.4
- k_1 is a factor which depends on concrete strength class, Expression (5.23)
- k_2 is a factor which depends on axial force and slenderness, Expression (5.24)

$$k_1 = \sqrt{f_{ck} / 20} \text{ (MPa)} \qquad (5.23)$$

$$k_2 = n \cdot \frac{\lambda}{170} \leq 0{,}20 \qquad (5.24)$$

where:

- n is the relative axial force, $N_{Ed} / (A_c f_{cd})$
- λ is the slenderness ratio, see 5.8.3

If the slenderness ratio λ is not defined, k_2 may be taken as

$$k_2 = n \cdot 0{,}30 \leq 0{,}20 \qquad (5.25)$$

(3) As a simplified alternative, provided $\rho \geq 0{,}01$, the following factors may be used in Expression (5.21):

$$K_s = 0$$
$$K_c = 0{,}3 / (1 + 0{,}5\varphi_{ef}) \qquad (5.26)$$

Note. The simplified alternative may be suitable as a preliminary step, followed by a more accurate calculation according to (2).

(4) In statically indeterminate structures, unfavourable effects of cracking in adjacent members should be taken into account. Expressions (5.21-5.26) are not generally applicable to such members. Partial cracking and tension stiffening may be taken into account e.g. according to 7.4.3. However, as a simplification, fully cracked sections may be assumed. The stiffness should be based on an effective concrete modulus:

$$E_{cd,eff} = E_{cd}/(1+\varphi_{ef}) \tag{5.27}$$

where:
E_{cd} is the design value according to 5.8.6 (3)
φ_{ef} is the effective creep ratio; same value as for columns may be used

5.8.7.3 Practical methods of analysis

(1) The total design moment, including second order moment, may be expressed as a magnification of the bending moments resulting from a linear analysis, namely:

$$M_{Ed} = M_{0Ed}\left[1+\frac{\beta}{(N_B/N_{Ed})-1}\right] \tag{5.28}$$

where:
M_{0Ed} is the first order moment; see also 5.8.8.2 (2)
β is a factor which depends on distribution of 1st and 2nd order moments, see (2)-(3) below
N_{Ed} is the design value of axial load
N_B is the buckling load based on nominal stiffness

(2) For isolated members with constant cross section and axial load, the second order moment may normally be assumed to have a sine-shaped distribution. Then

$$\beta = \pi^2 / c_0 \tag{5.29}$$

where:
c_0 is a coefficient which depends on the distribution of first order moment (for instance, $c_0 = 8$ for a constant first order moment, $c_0 = 9{,}6$ for a parabolic and 12 for a symmetric triangular distribution etc.).

(3) For members without transverse load, differing first order end moments M_{01} and M_{02} may be replaced by an equivalent constant first order moment M_{0e} according to 5.8.8.2 (2). Consistent with the assumption of a constant first order moment, $c_0 = 8$ should be used.

Note: The value of $c_0 = 8$ also applies to members bent in double curvature. It should be noted that in some cases, depending on slenderness and axial force, the end moments(s) can be greater than the magnified equivalent moment

(4) Where (2) or (3) is not applicable, $\beta = 1$ is normally a reasonable simplification. Expression (5.28) can then be reduced to:

$$M_{\mathrm{Ed}} = \frac{M_{0\mathrm{Ed}}}{1-(N_{\mathrm{Ed}}/N_{\mathrm{B}})} \tag{5.30}$$

Note: (4) is also applicable to the global analysis of certain types of structures, e.g. structures braced by shear walls and similar, where the principal action effect is bending moment in bracing units. For other types of structures, a more general approach is given in Informative Annex H, Clause H.2.

5.8.8 Method based on nominal curvature

5.8.8.1 General

(1) This method is primarily suitable for isolated members with constant normal force and a defined effective length l_0 (see 5.8.3.2). The method gives a nominal second order moment based on a deflection, which in turn is based on the effective length and an estimated maximum curvature (see also 5.8.5(4)).

(2) The resulting design moment is used for the design of cross sections with respect to bending moment and axial force according to 6.1, cf. 5.8.6 (2).

5.8.8.2 Bending moments

(1) The design moment is:

$$M_{\mathrm{Ed}} = M_{0\mathrm{Ed}} + M_2 \tag{5.31}$$

where:
$M_{0\mathrm{Ed}}$ is the 1st order moment, including the effect of imperfections, see also (2)
M_2 is the nominal 2nd order moment, see (3)

The maximum value of M_{Ed} is given by the distributions of $M_{0\mathrm{Ed}}$ and M_2; the latter may be taken as parabolic of sine shaped over the effective length.

Note: For statically indeterminate members, $M_{0\mathrm{Ed}}$ is determined for the actual boundary conditions, whereas M_2 will depend on boundary conditions via the effective length, cf. 5.8.8.1 (1).

(2) Differing first order end moments M_{01} and M_{02} may be replaced by an equivalent first order end moment M_{0e}:

$$M_{0e} = 0{,}6\, M_{02} + 0{,}4\, M_{01} \quad \geq 0{,}4\, M_{02} \tag{5.32}$$

M_{01} and M_{02} should have the same sign if they give tension on the same side, otherwise opposite signs. Furthermore, $|M_{02}| \geq |M_{01}|$.

(3) The nominal second order moment M_2 in Expression (5.29) is

$$M_2 = N_{\mathrm{Ed}}\, e_2 \tag{5.33}$$

where:
N_{Ed} is the design value of axial force
e_2 is the deflection = $(1/r)\, l_o^2 / c$
$1/r$ is the curvature, see 5.8.8.3
l_o is the effective length, see 5.8.3.2
c is a factor depending on the curvature distribution, see (4)

(4) For constant cross section, $c = 10$ ($\approx \pi^2$) is normally used. If the first order moment is constant, a lower value should be considered (8 is a lower limit, corresponding to constant *total* moment).

Note. The value π^2 corresponds to a sinusoidal curvature distribution. The value for constant curvature is 8. Note that c depends on the distribution of the *total* curvature, whereas c_0 in 5.8.7.3 (2) depends on the curvature corresponding to the first order moment only.

5.8.8.3 Curvature

(1) For members with constant symmetrical cross sections (incl. reinforcement), the following may be used:

$$1/r = K_r \cdot K_\varphi \cdot 1/r_0 \tag{5.34}$$

where:

K_r is a correction factor depending on axial load, see (3)
K_φ is a factor for taking account of creep, see (4)
$1/r_0 = \varepsilon_{yd} / (0{,}45\ d)$
$\varepsilon_{yd} = f_{yd} / E_s$
d is the effective depth; see also (2)

(2) If all reinforcement is not concentrated on opposite sides, but part of it is distributed parallel to the plane of bending, d is defined as

$$d = (h/2) + i_s \tag{5.35}$$

where i_s is the radius of gyration of the total reinforcement area

(3) K_r in Expression (5.34) should be taken as:

$$K_r = (n_u - n) / (n_u - n_{bal}) \leq 1 \tag{5.36}$$

where:

n $= N_{Ed} / (A_c\ f_{cd})$, relative axial force
N_{Ed} is the design value of axial force
n_u $= 1 + \omega$
n_{bal} is the value of n at maximum moment resistance; the value 0,4 may be used
ω $= A_s\ f_{yd} / (A_c\ f_{cd})$
A_s is the total area of reinforcement
A_c is the area of concrete cross section

(4) The effect of creep should be taken into account by the following factor:

$$K_\varphi = 1 + \beta \varphi_{ef} \geq 1 \tag{5.37}$$

where:

φ_{ef} is the effective creep ratio, see 5.8.4
β $= 0{,}35 + f_{ck}/200 - \lambda/150$
λ is the slenderness ratio, see 5.8.3.1

5.8.9 Biaxial bending

(1) The general method described in 5.8.6 may also be used for biaxial bending. The following provisions apply when simplified methods are used. Special care should be taken to identify the section along the member with the critical combination of moments.

(2) Separate design in each principal direction, disregarding biaxial bending, may be made as a first step. Imperfections need to be taken into account only in the direction where they will have the most unfavourable effect.

(3) No further check is necessary if the slenderness ratios satisfy the following two conditions

$$\lambda_y/\lambda_z \leq 2 \quad \text{and} \quad \lambda_z/\lambda_y \leq 2 \qquad (5.38a)$$

and if the relative eccentricities e_z/h and e_y/b satisfy one the following conditions (see Figure 5.7:

$$\frac{e_y/h}{e_z/b} \leq 0{,}2 \quad \text{or} \quad \frac{e_z/b}{e_y/h} \leq 0{,}2 \qquad (5.38b)$$

where:

b, h are the width and depth for section
$b = i_y \cdot \sqrt{12}$ and $h = i_z \cdot \sqrt{12}$ for an arbitrary section
λ_y, λ_z are the slenderness ratios l_0/i with respect to y- and z-axis respectively
i_y, i_z are the radii of gyration with respect to y- and z-axis respectively
e_z $= M_{Edy} / N_{Ed}$; eccentricity along z-axis
e_y $= M_{Edz} / N_{Ed}$; eccentricity along y-axis
M_{Edy} is the design moment about y-axis, including second order moment
M_{Edz} is the design moment about z-axis, including second order moment
N_{Ed} is the design value of axial load in the respective load combination

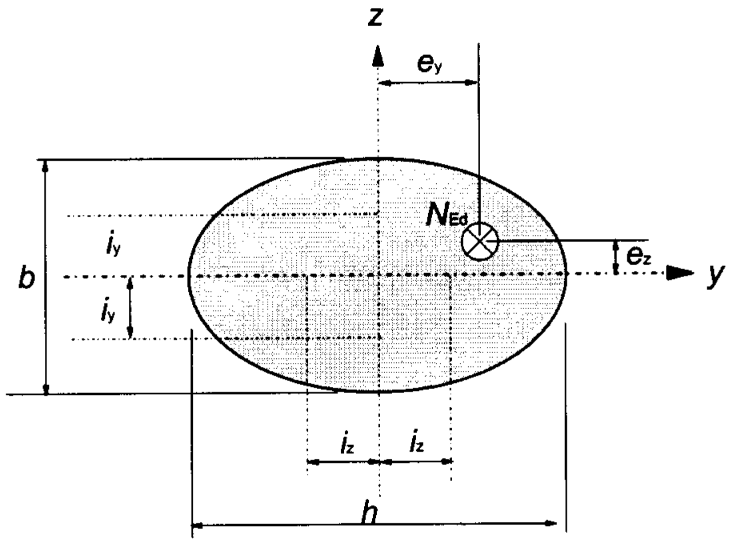


Figure 5.8. Definition of eccentricities e_y and e_z.

(4) If the condition of Expression (5.38) is not fulfilled, biaxial bending should be taken into account including the 2nd order effects in each direction (unless they may be ignored according to 5.8.2 (6) or 5.8.3). In the absence of an accurate cross section design for biaxial bending, the following simplified criterion may be used:

$$\left(\frac{M_{Edx}}{M_{Rdx}}\right)^a + \left(\frac{M_{Edy}}{M_{Rdy}}\right)^a \leq 1{,}0 \qquad (5.39)$$

where:

$M_{Edx/y}$ is the design moment around the respective axis, including nominal 2nd order moments.

$M_{Rdx/y}$ is the moment resistance in the respective direction

a is the exponent;

for circular and elliptical cross sections: $a = 2$

for rectangular cross sections:	N_{Ed}/N_{Rd}	0,1	0,7	1,0
	$a =$	1,0	1,5	2,0

with linear interpolation for intermediate values

N_{Ed} is the design value of axial force

N_{Rd} $= A_c f_{cd} + A_s f_{yd}$, design axial resistance of section.

where:

A_c is the gross area of the concrete section

A_s is the area of longitudinal reinforcement

5.9 Lateral instability of slender beams

(1)P Lateral instability of slender beams shall be taken into account where necessary, e.g. for precast beams during transport and erection, for beams without sufficient lateral bracing in the finished structure etc. Geometric imperfections shall be taken into account.

(2) A lateral deflection of l / 300 should be assumed as a geometric imperfection in the verification of beams in unbraced conditions, with l = total length of beam. In finished structures, bracing from connected members may be taken into account

(3) Second order effects in connection with lateral instability may be ignored if the following conditions are fulfilled:

- persistent situations: $\dfrac{l_{0f}}{b} \leq \dfrac{50}{(h/b)^{1/3}}$ and $h/b \leq 2{,}5$ (5.40a)

- transient situations: $\dfrac{l_{0f}}{b} \leq \dfrac{70}{(h/b)^{1/3}}$ and $h/b \leq 3{,}5$ (5.40b)

where:

l_{0f} is the distance between torsional restraints

h is the total depth of beam in central part of l_{0f}

b is the width of compression flange

(4) Torsion associated with lateral instability should be taken into account in the design of supporting structures.

5.10 Prestressed members and structures

5.10.1 General

(1)P The prestress considered in this Standard is that applied to the concrete by stressed tendons.

(2) The effects of prestressing may be considered as an action or a resistance caused by prestrain and precurvature. The bearing capacity should be calculated accordingly.

(3) In general prestress is introduced in the action combinations defined in EN 1990 as part of the loading cases and its effects should be included in the applied internal moment and axial force.

(4) Following the assumptions of (3) above, the contribution of the prestressing tendons to the resistance of the section should be limited to their additional strength beyond prestressing. This may be calculated assuming that the origin of the stress/strain relationship of the tendons is displaced by the effects of prestressing.

(5)P Brittle failure of the member caused by failure of prestressing tendons shall be avoided.

(6) Brittle failure should be avoided by one or more of the following methods:

Method A: Provide minimum reinforcement in accordance with 9.2.1.

Method B: Provide pretensioned bonded tendons.

Method C: Provide easy access to prestressed concrete members in order to check and control the condition of tendons by non-destructive methods or by monitoring.

Method D: Provide satisfactory evidence concerning the reliability of the tendons.

Method E: Ensure that if failure were to occur due to either an increase of load or a reduction of prestress under the frequent combination of actions, cracking would occur before the ultimate capacity would be exceeded, taking account of moment redistribution due to cracking effects.

Note: The selection of Methods to be used in a Country may be found in its National Annex.

5.10.2 Prestressing force during tensioning

5.10.2.1 Maximum stressing force

(1)P The force applied to a tendon, P_{max} (i.e. the force at the active end during tensioning) shall not exceed the following value.

$$P_{max}=A_p \cdot \sigma_{p,max} \tag{5.41}$$

where:

A_p is the cross-sectional area of the tendon

$\sigma_{p,max}$ is the maximum stress applied to the tendon $= \min \{ k_1 \cdot f_{pk} ; f_{p0,1k} \}$

Note: The values of k_1 and k_2 for use in a Country may be found in its National Annex. The recommended values are $k_1 = 0,8$ and $k_2 = 0,9$

(2) Overstressing is permitted if the force in the jack can be measured to an accuracy of ± 5 % of the final value of the prestressing force. In such cases the maximum prestressing force P_{max} may be increased to $k_3 \cdot f_{p0,1k}$ (e.g. for the occurrence of an unexpected high friction in long-line pretensioning).

Note: The values of k_3 for use in a Country may be found in its National Annex. The recommended value is 0,95.

5.10.2.2 Limitation of concrete stress

(1)P Local concrete crushing or splitting at the end of pre- and post-tensioned members shall be avoided.

(2)P Local concrete crushing or splitting behind post-tensioning anchors shall be limited in accordance with the relevant European Technical Approval.

(3)P The strength of concrete at application of or transfer of prestress shall not be less than the minimum value defined in the relevant European Technical Approval.

(4) If prestress in an individual tendon is applied in steps, the required concrete strength may be reduced. The minimum strength $f_{cm}(t)$ at the time t should be k_4 [%] of the required concrete strength for full prestressing given in the European Technical Approval. Between the minimum strength and the required concrete strength for full prestressing, the prestress may be interpolated between k_5 [%] and 100% of the full prestressing.

Note: The values of k_4 and k_5 are subject to a National Annex. The recommended value for k_4 is 50 and for k_5 is 30.

(5) The concrete compressive stress in the structure resulting from the prestressing force and other loads acting at the time of tensioning or release of prestress, should be limited to:

$$\sigma_c \leq 0,6\, f_{ck}(t) \tag{5.42}$$

where $f_{ck}(t)$ is the characteristic compressive strength of the concrete at time t when it is subjected to the prestressing force.

For pretensioned elements the stress at the time of transfer of prestress may be increased to $k_6 \cdot f_{ck}(t)$., if it can be justified by tests or experience that longitudinal cracking is prevented.

Note: The value of k_6 for use in a Country may be found in its National Annex. The recommended value is 0,7.

If the compressive stress permanently exceeds 0,45 $f_{ck}(t)$ the non-linearity of creep should be taken into account.

5.10.2.3 Measurements

(1)P In post-tensioning the prestressing force and the related elongation of the tendon shall be checked by measurements and the actual losses due to friction shall be controlled.

5.10.3 Prestressing force

(1)P At a given time t and distance x (or arc length) from the active end of the tendon the mean prestress force $P_{m,t}(x)$ is equal to the maximum force P_{max} imposed at the active end, minus the immediate losses and the time dependent losses (see below). Absolute values are considered for all the losses.

(2) The value of the initial prestress force $P_{m0}(x)$ (at time $t = t0$) applied to the concrete immediately after tensioning and anchoring (post-tensioning) or after transfer of prestressing (pre-tensioning) is obtained by subtracting from the force at tensioning P_{max} the immediate losses $\Delta P_i(x)$ which should not exceed the following value:

$$P_{m0}(x) = A_p \cdot \sigma_{pm0}(x) \tag{5.43}$$

where:

$\sigma_{pm0}(x)$ is the stress in the tendon immediately after tensioning or transfer
$= \min \{ k_7 \cdot f_{pk} \,;\, k_8 f_{p0,1k} \}$

Note: The values of k_7 and k_8 for use in a Country may be found in its National Annex. The recommended values are $k_7 = 0,75$ and $k_8 = 0,85$

(3) At a given time t and distance x (or arc length) from the active end of the tendon the prestressing force $P(x,t)$ is equal to the maximum force P_0 imposed at the active end, less the losses.

(4) When determining the immediate losses $\Delta P_i(x)$ the following immediate influences should be considered for pre-tensioning and post-tensioning where relevant (see 5.10.4 and 5.10.5):

- losses due to elastic deformation of concrete ΔP_{el}
- losses due to short term relaxation ΔP_r
- losses due to friction $\Delta P_{\mu(x)}$
- losses due to anchorage slip ΔP_{sl}

(5) The mean value of the prestress force $P_{m,t}(x)$ at the time $t > t0$ should be determined with respect to the prestressing method. In addition to the immediate losses given in (3) the time-dependent losses of prestress $\Delta P_{c+s+r}(x)$ (see 5.10.6) as a result of creep and shrinkage of the concrete and the long term relaxation of the prestressing steel should be considered and $P_{m,t}(x) = P_{m0}(x) - \Delta P_{c+s+r}(x)$.

5.10.4 Immediate losses of prestress for pre-tensioning

(1) The following losses occurring during pre-tensioning should be considered:

(i) during the stressing process: loss due to friction at the bends (in the case of curved wires of strands) and losses due to wedge draw-in of the anchorage devices.

(ii) before the transfer of prestress to concrete: loss due to relaxation of the pretensioning tendons during the period which elapses between the tensioning of the tendons and prestressing of the concrete.

Note: In case of heat curing, losses due to shrinkage and relaxation are modified and should be assessed accordingly; direct thermal effect should also be considered (see Annex D)

(iii) at the transfer of prestress to concrete: loss due to elastic deformation of concrete as the result of the action of pre-tensioned tendons when they are released from the anchorages.

5.10.5 Immediate losses of prestress for post-tensioning

5.10.5.1 Losses due to the instantaneous deformation of concrete

(1) Account should be taken of the loss in tendon force corresponding to the deformation of concrete, taking account the order in which the tendons are stressed.

(2) This loss, ΔP_{el}, may be assumed as a mean loss in each tendon as follows:

$$\Delta P_c = A_p \cdot E_p \cdot \sum \left[\frac{j \cdot \Delta\sigma_c(t)}{E_{cm}(t)} \right] \tag{5.44}$$

where:

$\Delta\sigma_c(t)$ is the variation of stress at the centre of gravity of the tendons applied at time t

j is a coefficient equal to

- $(n-1)/2n$ where n is the number of identical tendons successively prestressed. As an approximation this may be taken as 1/2
- 1 for the variations of permanent actions applied after prestressing.

5.10.5.2 Losses due to friction

(1) The losses due to friction $\Delta P_\mu(x)$ in post-tensioned tendons may be estimated from:

$$\Delta P_\mu(x) = P_{max}\,(1 - e^{-\mu(\theta + kx)}) \tag{5.45}$$

where:

θ is the sum of the angular displacements over a distance x (irrespective of direction or sign)

μ is the coefficient of friction between the tendon and its duct

k is an unintentional angular displacement for internal tendons (per unit length)

x is the distance along the tendon from the point where the prestressing force is equal to P_{max} (the force at the active end during tensioning)

The values μ and k are given in the relevant European Technical Approval. The value μ depends on the surface characteristics of the tendons and the duct, on the presence of rust, on the elongation of the tendon and on the tendon profile.

The value k for unintentional angular displacement depends on the quality of workmanship, on the distance between tendon supports, on the type of duct or sheath employed, and on the degree of vibration used in placing the concrete.

(2) In the absence of more exact data given in a European Technical Approval the values for μ given in Table 5.1 may be assumed, when using Expression (5.45).

(3) In the absence of more exact data in a European Technical Approval, values for unintended regular displacements for internal tendons will generally be in the range $0{,}005 < k < 0{,}01$ per metre may be used.

(4) For external tendons, the losses of prestress due to unintentional angles may be ignored.

Table 5.1: Coefficients of friction μ of post tensioned tendons and external unbonded tendons

	Internal tendons [1]	External unbonded tendons			
		Steel duct/ non lubricated	HDPE duct/ non lubricated	Steel duct/ lubricated	HDPE duct/ lubricated
Cold drawn wire	0,17	0,25	0,14	0,18	0,12
Strand	0,19	0,24	0,12	0,16	0,10
Deformed bar	0,65	-	-	-	-
Smooth round bar	0,33	-	-	-	-
[1] for tendons which fill about half of the duct					

Note: HPDE - High density polyethylene

5.10.5.3 Losses at anchorage

(1) Account should be taken of the losses due to wedge draw-in of the anchorage devices, during the operation of anchoring after tensioning, and due to the deformation of the anchorage itself.

(2) Values of the wedge draw-in are given in the European Technical Approval.

5.10.6 Time dependent losses of prestress for pre- and post-tensioning

(1) The time dependent losses may be calculated by considering the following two reductions of stress:

(a) due to the reduction of strain, caused by the deformation of concrete due to creep and shrinkage, under the permanent loads:

(b) the reduction of stress in the steel due to the relaxation under tension.

Note: The relaxation of steel depends on the reduction of strain due to creep and shrinkage of concrete. This interaction can generally and approximately be taken into account by a reduction factor 0,8.

(2) A simplified method to evaluate time dependent losses at location x under the permanent loads is given by Expression (5.46).

$$\Delta P_{c+s+r} = A_p \Delta\sigma_{p,c+s+r} = A_p \frac{\varepsilon_{cs} E_p + 0{,}8\Delta\sigma_{pr} + \frac{E_p}{E_{cm}}\varphi(t,t_0).\sigma_{c,QP}}{1 + \frac{E_p}{E_{cm}}\frac{A_p}{A_c}\left(1 + \frac{A_c}{I_c} z_{cp}^2\right)\left[1 + 0{,}8\,\varphi(t,t_0)\right]} \qquad (5.46)$$

where:

$\Delta\sigma_{p,c+s+r}$ is the absolute value of the variation of stress in the tendons due to creep, shrinkage and relaxation at location x, at time t

ε_{cs} is the estimated shrinkage strain according to 3.1.4(6) in absolute value

E_p is the modulus of elasticity for the prestressing steel, see 3.3.3 (9)

E_{cm} is the modulus of elasticity for the concrete (Table 3.1)

$\Delta\sigma_{pr}$ is the absolute value of the variation of stress in the tendons at location x, at time t, due to the relaxation of the prestressing steel. It is determined for a stress of $\sigma_p = \sigma_p(G+P_{m0}+\psi_2 Q)$

where $\sigma_p = \sigma_p(G+P_{m0}+\psi_2 Q)$ is the initial stress in the tendons due to initial prestress and quasi-permanent actions.

$\varphi(t,t_0)$ is the creep coefficient at a time t and load application at time t_0

$\sigma_{c,QP}$ is the stress in the concrete adjacent to the tendons, due to self-weight and initial prestress and other quasi-permanent actions where relevant. The value of $\sigma_{c,QP}$ may be the effect of part of self-weight and initial prestress or the effect of a full quasi-permanent combination of action ($\sigma_c(G+P_{m0}+\psi_2 Q)$), depending on the stage of construction considered.

A_p is the area of all the prestressing tendons at the level being considered.

A_c is the area of the concrete section.

I_c is the second moment of area of the concrete section.

z_{cp} is the distance between the centre of gravity of the concrete section and the tendons

Compressive stresses and the corresponding strains given in Expression (5.46) should be used with a positive sign.

(3) Expression (5.46) applies for bonded tendons when local values of stresses are used and for unbonded tendons when mean values of stresses are used. The mean values should be calculated between straight sections limited by the idealised deviation points for external tendons or along the entire length in case of internal tendons.

5.10.7 Consideration of prestress in analysis

(1) Second order moments can arise from prestressing with external tendons.

(2) Moments from secondary effects of prestressing arise only in statically indeterminate structures.

(3) For linear analysis both the primary and secondary effects of prestressing should be applied before any redistribution of forces and moments is considered (see 5.5).

(4) In plastic and non-linear analysis the secondary effect of prestress may be treated as additional plastic rotations which should then be included in the check of rotation capacity.

(5) Rigid bond between steel and concrete may be assumed after grouting of bonded tendons. However before grouting the tendons should be considered as unbonded.

(6) External tendons may be assumed to be straight between deviators.

5.10.8 Effects of prestressing at ultimate limit state

(1) In general, the design value of the prestressing force may be determined by $P_{d,t}(x) = \gamma_P P_{m,}(x)_t$(see 5.10.3 (4) for the definition of $P_{m,t}(x)$).

(2) For prestressed members with permanently unbonded tendons, it is generally necessary to take the deformation of the whole member into account when calculating the increase of the stress in the prestressing steel. If no detailed calculation is made, it may be assumed that the increase of the stress from the effective prestress to the stress in the ultimate limit state is $\Delta\sigma_{p,ULS}$.

Note: The value of $\Delta\sigma_{p,ULS}$ for use in a Country may be found in its National Annex. The recommended value is 100MPa.

(3) If the stress increase is calculated using the deformation state of the whole member the mean values of the material properties should be used. The design value of the stress increase $\Delta\sigma_{pd} = \Delta\sigma_p \cdot \gamma_{\Delta P}$ should be determined by applying partial safety factors $\gamma_{\Delta P,sup}$ and $\gamma_{\Delta P,inf}$ respectively.

Note: The values of $\gamma_{\Delta P,sup}$ and $\gamma_{\Delta P,inf}$ for use in a Country may be found in its National Annex. The recommended values for $\gamma_{\Delta P,sup}$ and $\gamma_{\Delta P,inf}$ are 1,2 and 0,8 respectively. If linear analysis with uncracked sections is applied, a lower limit of deformations may be assumed and the recommended value for both $\gamma_{\Delta P,sup}$ and $\gamma_{\Delta P,inf}$ is 1,0.

5.10.9 Effects of prestressing at serviceability limit state and limit state of fatigue

(1)P For serviceability calculations, allowance shall be made for possible variations in prestress. Two characteristic values of the prestressing force at the serviceability limit state are estimated from:

$$P_{k.sup} = r_{sup}\, P_{m,t}\,(x) \tag{5.47}$$

$$P_{k.inf} = r_{inf}\, P_{m,t}(x) \tag{5.48}$$

where:

$P_{k.sup}$ is the upper characteristic value
$P_{k.inf}$ is the lower characteristic value

Note: The values of r_{sup} and r_{inf} for use in a Country may be found in its National Annex. The recommended values are:
- for pre-tensioning or unbonded tendons: r_{sup} = 1,05 and r_{inf} = 0,95
- for post-tensioning with bonded tendons: r_{sup} = 1,10 and r_{inf} = 0,90
- when appropriate measures (e.g. direct measurements of pretensioning under serviceability conditions) are taken: $r_{sup} = r_{inf}$ = 1,0.

5.11 Analysis for some particular elements

5.11.1 Flat Slabs

5.11.1.1 Definition

(1)P Slabs supported on columns are defined as flat slabs.

(2) For the purpose of this section flat slabs may be of uniform thickness or they may incorporate drops (thickenings over columns).

(3) Flat slabs should be analysed using a proven method of analysis, such as grillage (in which the plate is idealised as a set of interconnected discrete members), finite element, yield line or equivalent frame. Appropriate geometric and material properties should be employed.

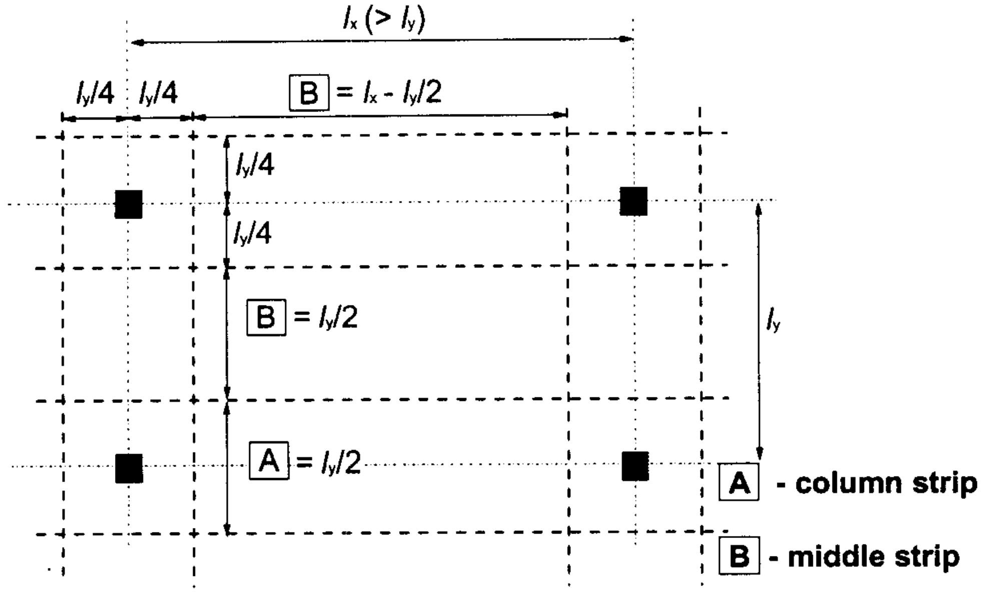
5.11.1.2 Equivalent frame analysis

(1) The structure should be divided longitudinally and transversely into frames consisting of columns and sections of slabs contained between the centre lines of adjacent panels (area bounded by four adjacent supports). The stiffness of members may be calculated from their

gross cross-sections. For vertical loading the stiffness may be based on the full width of the panels. For horizontal loading 40% of this value should be used to reflect the increased flexibility of the column/slab joints in flat slab structures compared to that of column/beam joints. Total load on the panel should be used for the analysis in each direction.

(2) The total bending moments obtained from analysis should be distributed across the width of the slab. In elastic analysis negative moments tend to concentrate towards the centre lines of the columns.

(3) The panels should be assumed to be divided into column and middle strips (see Figure 5.9) and the bending moments should be apportioned as given in Table 5.2.



Note: When drops of width > (l_y/3) are used the column strips may be taken to be the width of drops. The width of middle strips should then be adjusted accordingly.

Figure 5.9: Division of panels in flat slabs

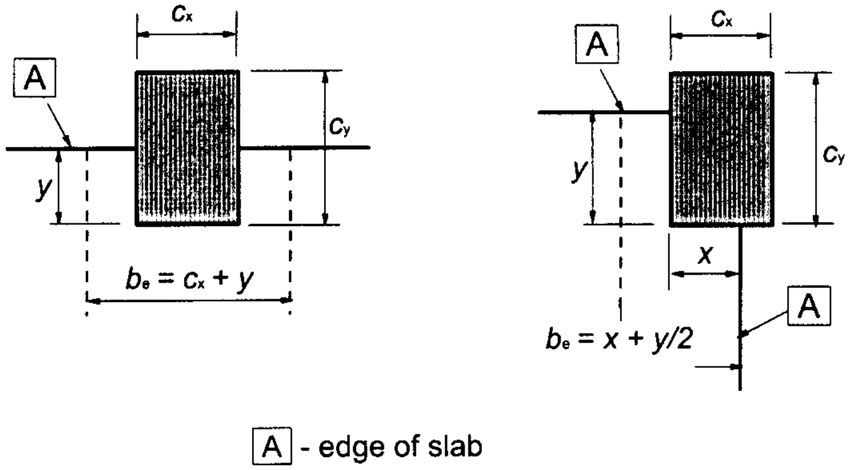
Table 5.2 Simplified apportionment of bending moment for a flat slab

	Negative moments	Positive moments
Column Strip	60 - 80%	50 - 70%
Middle Strip	40 - 20%	50 - 30%
Note: Total negative and positive moments to be resisted by the column and middle strips together should always add up to 100%.		

(4) Where the width of the column strip is different from $0{,}5l_x$ as shown in Figure 5.9 (e.g.) equal to width of drops, then the design moments to be resisted by the column and middle strips should be adjusted in proportion to their revised widths.

(5) Unless there are perimeter beams, which are adequately designed for torsion, moments transferred to edge or corner columns should be limited to the moment of resistance of a

rectangular section equal to 0,17 $b_e d^2 f_{ck}$ (see Figure 5.10 for the definition of b_e). The positive moment in the end span should be adjusted accordingly.



Note: y can be > c_y

a) Edge column

Note: x can be > c_x and y can be > c_y

b) Corner column

Note: y is the distance from the edge of the slab to the innermost face of the column.

Figure 5.10: Definition of effective breadth, b_e

5.11.1.3 Irregular column layout

(1) Where, due to the irregular layout of columns, a flat slab can not be sensibly analysed using the equivalent frame method, a grillage or other elastic method may be used. In such a case the following simplified approach will normally be sufficient:

i) analyse the slab with the full load, $\gamma_Q Q_k + \gamma_G G_k$, on all bays

ii) the midspan and column moments should then be increased to allow for the effects of pattern loads. This may be achieved by loading a critical bay (or bays) with $\gamma_Q Q_k + \gamma_G G_k$ and the rest of the slab with $\gamma_G G_k$. Where there may be significant variation in the permanent load between bays, γ_G should be taken as 1 for the unloaded bays.

iii) the effects of this particular loading may then be applied to other critical bays and supports in a similar way.

(2) The restrictions with regard to the transfer of moments to edge columns given in 5.11.2 should be applied.

5.11.2 Shear Walls

(1)P Shear walls are plain or reinforced concrete walls which contribute to the lateral stability of the structure.

(2)P Lateral load resisted by each shear wall in a structure shall be obtained from a global analysis of the structure, taking into account the applied loads, the eccentricities of the loads with respect to the shear centre of the structure and the interaction between the different structural walls.

(3)P The effects of asymmetry of wind loading shall be considered (see EN 1991-1-4).

(4)P The combined effects of axial loading and shear shall be taken into account.

(5) In addition to other serviceability criteria in this code, the effect of sway of shear walls on the occupants of the structure should also be considered, (see EN 1990).

(6) In the case of building structures not exceeding 25 storeys, where the plan layout of the walls is reasonably symmetrical, and the walls do not have openings causing significant global shear deformations, the lateral load resisted by a shear wall may be obtained as follows:

$$P_n = \frac{P(EI)_n}{\Sigma(EI)} \pm \frac{(Pe)y_n(EI)_n}{\Sigma(EI)y_n^2} \quad (5.49)$$

where:

P_n is the lateral load on wall n

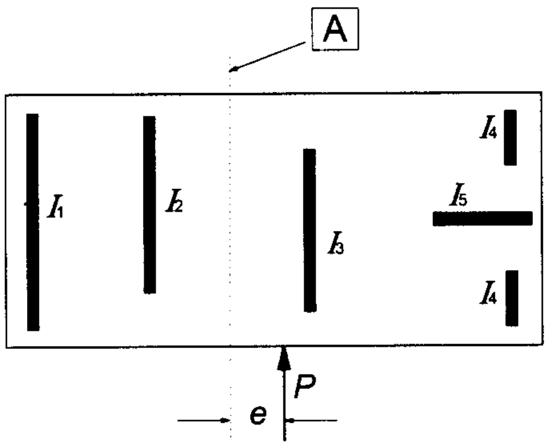
$(EI)_n$ is the stiffness of wall n

P is the applied load

e is the eccentricity of P with respect to the centroid of the stiffnesses (see Figure 5.11)

y_n is the distance of wall n from the centroid of stiffnesses.

(7) If members with and without significant shear deformations are combined in the bracing system, the analysis should take into account both shear and flexural deformation.



A - Centroid of shear wall group

Figure 5.11: Eccentricity of load from centroid of shear walls

SECTION 6 ULTIMATE LIMIT STATES

6.1 Bending with or without axial force

(1)P This section applies to undisturbed regions of beams, slabs and similar types of members for which sections remain approximately plane before and after loading. The discontinuity regions of beams and other members in which plane sections do not remain plane may be designed and detailed according to 6.5.

(2)P When determining the ultimate moment resistance of reinforced or prestressed concrete cross-sections, the following assumptions are made:

- plane sections remain plane.
- the strain in bonded reinforcement or bonded prestressing tendon, whether in tension or in compression, is the same as that in the surrounding concrete.
- the tensile strength of the concrete is ignored.
- the stresses in the concrete in compression are derived from the design stress/strain relationship given in 3.1.7.
- the stresses in the reinforcing or prestressing steel are derived from the design curves in 3.2 (Figure 3.8) and 3.3 (Figure 3.10).
- the initial strain in prestressing tendons is taken into account when assessing the stresses in the tendons.

(3)P The compressive strain in the concrete shall be limited to ε_{cu2}, or ε_{cu3}, depending on the stress-strain diagram used, see 3.1.7 and Table 3.1. The strains in the reinforcing steel and the prestressing steel shall be limited to ε_{ud} (where applicable); see 3.2.7 (2) and 3.3.6 (7) respectively.

(4) For reinforced concrete cross-sections subjected to a combination of bending moment and compression, the design value of the bending moment should be at least $M_{Ed} = e_0 \cdot N_{Ed}$ where $e_0 = h/30$ but not less than 20 mm where h is the depth of the section.

(5) In parts of cross-sections which are subjected to approximately concentric loading (e/h < 0,1), such as compression flanges of box girders, the limiting compressive strain should be assumed to be ε_{c2} (or ε_{c3} if the bilinear relation of Figure 3.4 is used) over the full depth of the part considered.

(6) The possible range of strain distributions is shown in Figure 6.1.

(7) For prestressed members with permanently unbonded tendons see 5.10.8.

(8) For external prestressing tendons the strain in the prestressing steel between two subsequent contact points (anchors or deviation saddles) is assumed to be constant. The strain in the prestressing steel is then equal to the initial strain, realised just after completion of the prestressing operation, increased by the strain resulting from the structural deformation between the contact areas considered. See also 5.10.

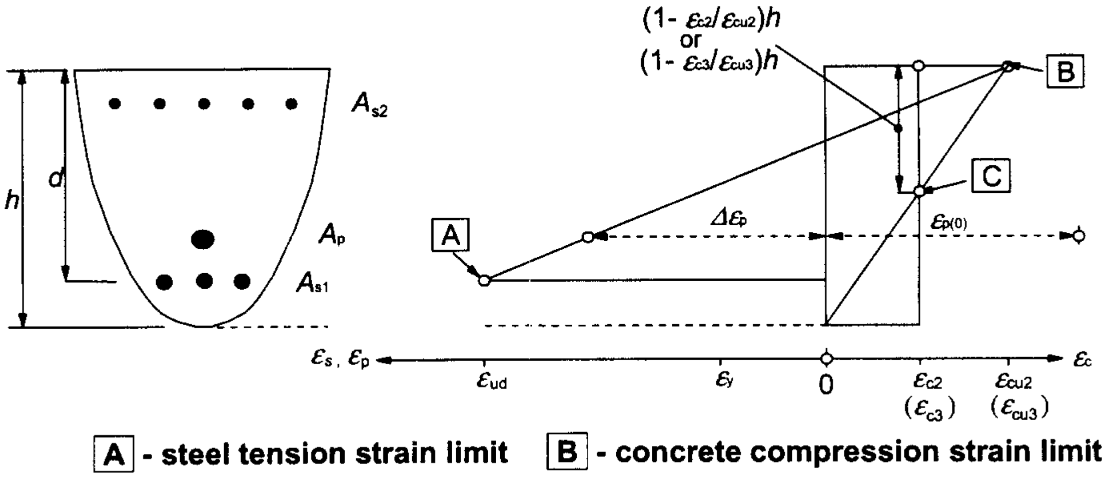


Figure 6.1: Possible strain distributions in the ultimate limit state

6.2 Shear

6.2.1 General verification procedure

(1)P For the verification of shear resistance the following symbols are defined:

$V_{Rd,c}$ is the design shear resistance of the member without shear reinforcement.
$V_{Rd,s}$ is the design value of the shear force which can be sustained by the yielding shear reinforcement.
$V_{Rd,max}$ is the design value of the maximum shear force which can be sustained by the member, limited by crushing of the compression struts.

In members with inclined chords the following additional values are defined (see Figure 6.2):

V_{ccd} is the design value of the shear component of the force in the compression area, in the case of an inclined compression chord.
V_{td} is the design value of the shear component of the force in the tensile reinforcement, in the case of an inclined tensile chord.

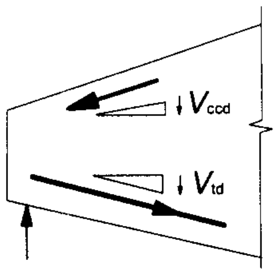


Figure 6.2: Shear component for members with inclined chords

(2) The shear resistance of a member with shear reinforcement is equal to:

$$V_{Rd} = V_{Rd,s} + V_{ccd} + V_{td} \quad (6.1)$$

(3) In regions of the member where $V_{Ed} < V_{Rd,c}$ no calculated shear reinforcement is necessary. V_{Ed} is the design shear force in the section considered resulting from external loading and prestressing (bonded or unbonded).

(4) When, on the basis of the design shear calculation, no shear reinforcement is required, minimum shear reinforcement should nevertheless be provided according to 9.2.2. The minimum shear reinforcement may be omitted in members such as slabs (solid, ribbed or hollow core slabs) where transverse redistribution of loads is possible. Minimum reinforcement may also be omitted in members of minor importance (e.g. lintels with span ≤ 2 m) which do not contribute significantly to the overall resistance and stability of the structure.

(5) In regions where $V_{Ed} > V_{Rd,c}$ according to Expressions (6.2.a) and (6.2.b) or (6.3), sufficient shear reinforcement should be provided in order that $V_{Ed} \leq V_{Rd}$ (see Expression (6.1)).

(6) The sum of the design shear force and the contributions of the flanges, $V_{Ed} - V_{ccd} - V_{td}$, should not exceed the permitted maximum value $V_{Rd,max}$ (see 6.2.3), anywhere in the member.

(7) The longitudinal tension reinforcement should be able to resist the additional tensile force caused by shear (see 6.2.3 (6)).

(8) For members subject to predominantly uniformly distributed loading the design shear force only needs to be checked at a distance d from the face of the support. Any shear reinforcement required should continue to the support. In addition it should be verified that the shear at the support does not exceed $V_{Rd,max}$ (see also 6.2.2 (6) and 6.2.3 (7)..

6.2.2 Members not requiring design shear reinforcement

(1) The design value for the shear resistance $V_{Rd,c}$ is given by:

$$V_{Rd,c} = [C_{Rd,c}k(100\,\rho_l f_{ck})^{1/3} + 0{,}15\,\sigma_{cp}]\,b_w d \qquad (6.2.a)$$

with a minimum of

$$V_{Rd,c} = (v_{min} + 0{,}15\sigma_{cp})\,b_w d \qquad (6.2.b)$$

where:

f_{ck} is in MPa

$k \quad = 1+\sqrt{\frac{200}{d}} \leq 2{,}0$ with d in mm

$\rho_l \quad = \frac{A_{sl}}{b_w d} \leq 0{,}02$

A_{sl} is the area of the tensile reinforcement, which extends ≥ ($l_{bd} + d$) beyond the section considered (see Figure 6.3).

b_w is the smallest width of the cross-section in the tensile area (mm)

σ_{cp} = $N_{Ed}/A_c < 0{,}2\,f_{cd}$ (MPa)

N_{Ed} is the axial force in the cross-section due to loading or prestressing in Newtons (N_{Ed}>0 for compression). The influence of imposed deformations on N_E may be ignored.

A_C is the area of concrete cross section (mm^2)

$V_{Rd,c}$ is in Newtons

Note: The values of $C_{Rd,c}$ and v_{min} for use in a Country may be found in its National Annex. The recommended value for $C_{Rd,c}$ is $0{,}18/\gamma_c$ and that for v_{min} is given by Expression (6.3N)

$$v_{min} = 0{,}035\, k^{3/2} \cdot f_{ck}^{1/2} \tag{6.3N}$$

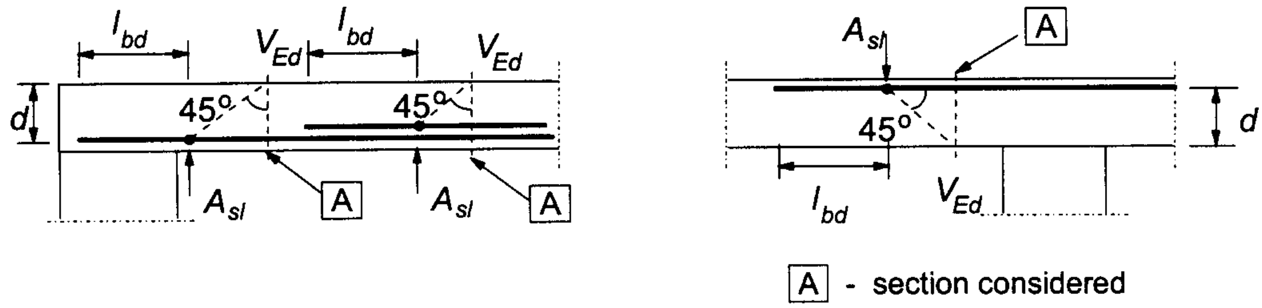


Figure 6.3: Definition of A_{sl} in Expression (6.2)

(2) In prestressed single span members without shear reinforcement, the shear resistance of the regions cracked in bending may be calculated using Expression (6.2a). In regions uncracked in bending (where the flexural tensile stress is smaller than $f_{ctk,0,05}/\gamma_c$) the shear resistance should be limited by the tensile strength of the concrete. In these regions the shear resistance is given by:

$$V_{Rd,c} = \frac{I \cdot b_w}{S} \sqrt{(f_{ctd})^2 + \alpha_l \sigma_{cp} f_{ctd}} \tag{6.4}$$

where

- I is the second moment of area
- b_w is the width of the cross-section at the centroidal axis, allowing for the presence of ducts in accordance with Expressions (6.16) and (6.17)
- S is the first moment of area above and about the centroidal axis
- α_l $= l_x/l_{pt2} \leq 1{,}0$ for pretensioned tendons
  $= 1{,}0$ for other types of prestressing
- l_x is the distance of section considered from the starting point of the transmission length
- l_{pt2} is the upper bound value of the transmission length of the prestressing element according to Expression (8.17).
- σ_{cp} is the concrete compressive stress at the centroidal axis due to axial loading or prestressing ($\sigma_{cp} = N_{Ed}/A_c$ in MPa, $N_{Ed} > 0$ in compression)

For cross-sections where the width varies over the height, the maximum principal stress may occur on an axis other than the centroidal axis. In such a case the minimum value of the shear resistance should be found by calculating $V_{Rd,c}$ at various axes in the cross-section.

(3) The calculation of the shear resistance according to Expression (6.4) need not be carried out for cross-sections that are nearer to the support than the point which is the intersection of the elastic centroidal axis and a line inclined from the inner edge of the support at an angle of 45°.

(4) For the general case of members subjected to a bending moment and an axial force, which can be shown to be uncracked in flexure at the ULS, reference is made to 12.6.3.

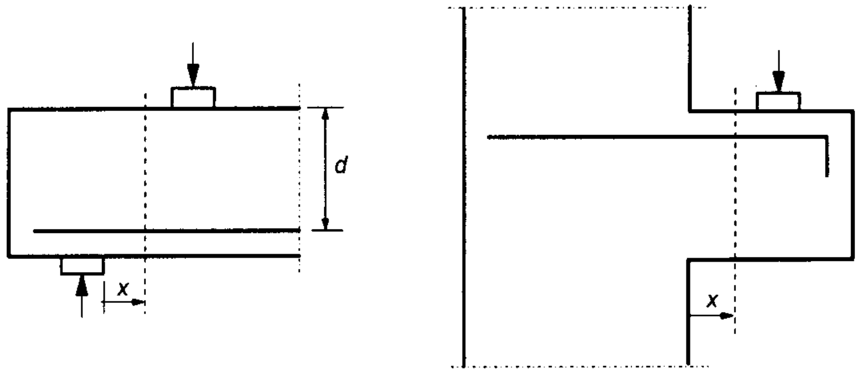
(5) For the design of the longitudinal reinforcement, in the region cracked in flexure, the M_d - line should be shifted over a distance $a_l = d$ in the unfavourable direction (see 9.2.1.3 (2)).

(6) At a distance $0{,}5d \le x < 2d$ from the edge of a support the shear resistance may be increased to:

$$V_{Rd,c} = \left[C_{Rd,c} k (100 \rho_l f_{ck})^{1/3} \left(\frac{2d}{x} \right) + 0{,}15 \sigma_{cp} \right] b_w d \le 0{,}5 b_w d \, \nu f_{cd} \tag{6.5}$$

$$\text{where } \nu = 0{,}6 \left[1 - \frac{f_{ck}}{250} \right] \; (f_{ck} \text{ and } \sigma_{cp} \text{ in MPa}) \tag{6.6}$$

This increase is only valid for loads applied at the upper side of the member (see Figure 6.4), where the longitudinal reinforcement is completely anchored at the node. For $x \le 0.5d$ the value for $x = 0.5d$ should be used. For $C_{Rd,ct}$ see 6.2.2.(1).



(a) Beam with direct support **(b) Corbel**

Figure 6.4: Loads near supports

(7) Beams with loads next to supports and corbels can alternatively be designed with strut and tie models. For this alternative, reference is made to 6.5.

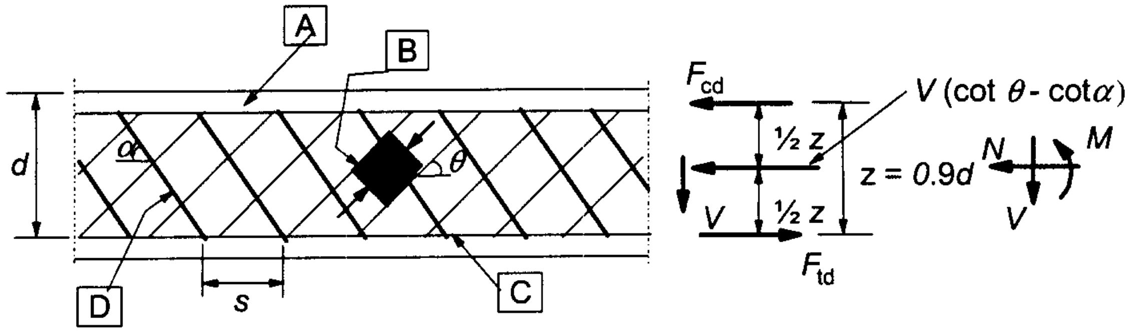
6.2.3 Members requiring design shear reinforcement

(1) The design of members with shear reinforcement is based on a truss model (Figure 6.5). Limiting values for the angle θ of the inclined struts in the web are given in 6.2.3 (2).

In Figure 6.5 the following notations are shown:

- α is the angle between shear reinforcement and the main tension chord (measured positive as shown)
- θ is the angle between concrete compression struts and the main tension chord
- F_{td} is the design value of the tensile force in the longitudinal reinforcement
- F_{cd} is the design value of the concrete compression force in the direction of the longitudinal member axis.
- b_w is the minimum width between tension and compression chords
- z is the inner lever arm, for a member with constant depth, corresponding to the maximum bending moment in the element under consideration. In the shear analysis, the approximate value $z = 0.9d$ may normally be used.

In elements with inclined prestressing tendons, longitudinal reinforcement at the tensile chord should be provided to carry the longitudinal tensile force due to shear defined by Expression (6.18).



$\boxed{A}$ - compression chord, $\boxed{B}$ - struts, $\boxed{C}$ - tensile chord, $\boxed{D}$ - shear reinforcement

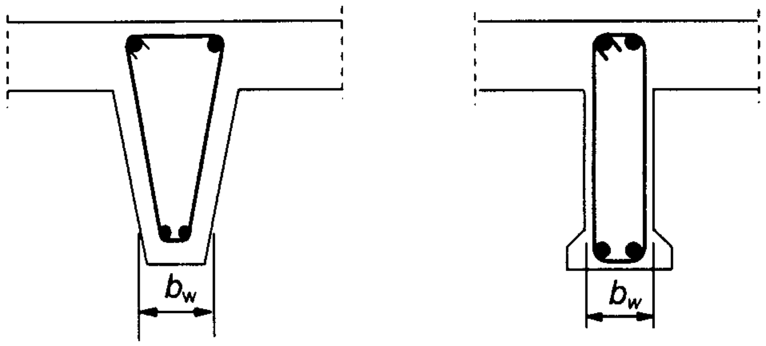


Figure 6.5: Truss model and notation for shear reinforced members

(2) The angle θ should be limited.

Note: The limiting values of $\cot\theta$ for use in a Country may be found in its National Annex. The recommended limits are given in Expression (6.7N).

$$1 \leq \cot\theta \leq 2{,}5 \qquad (6.7N)$$

(3) For members with vertical shear reinforcement, the shear resistance, $V_{Rd,s}$ should be taken as the lesser of:

$$V_{Rd,s} = \frac{A_{sw}}{s} z\, f_{ywd} \cot\theta \qquad (6.8)$$

and

$$V_{Rd,max} = \alpha_c\, b_w\, z\, \nu\, f_{cd}/(\cot\theta + \tan\theta) \qquad (6.9)$$

where:

A_{sw} is the cross-sectional area of the shear reinforcement
s is the spacing of the stirrups
f_{ywd} is the design yield strength of the shear reinforcement
ν follows from Expression (6.6)

For reinforced and prestressed members, if the design stress of the shear reinforcement is below 80% of the characteristic yield stress f_{yk}, ν may be taken as:

$\nu = 0.6$ for $f_{ck} \leq 60$ MPa (6.10.a)

$\nu = 0.9 - f_{ck}/200 > 0.5$ for $f_{ck} \geq 60$ MPa (6.10.b)

Note: The value of α_c for use in a Country may be found in its National Annex. The recommended value is as follows:

1 for non-prestressed structures

$(1 + \sigma_{cp}/f_{cd})$ for $0 < \sigma_{cp} \leq 0,25\ f_{cd}$ (6.11.aN)

1,25 for $0,25\ f_{cd} < \sigma_{cp} \leq 0,5\ f_{cd}$ (6.11.bN)

$2,5\ (1 - \sigma_{cp}/f_{cd})$ for $0,5\ f_{cd} < \sigma_{cp} < 1,0\ f_{cd}$ (6.11.cN)

where:

σ_{cp} is the mean compressive stress, measured positive, in the concrete due to the design axial force. This should be obtained by averaging it over the concrete section taking account of the reinforcement. The value of σ_{cp} need not be calculated at a distance less than 0.5d cot θ from the edge of the support.

The maximum effective cross-sectional area of the shear reinforcement $A_{sw,max}$ is given by:

$$\frac{A_{sw,max} f_{ywd}}{b_w s} \leq \tfrac{1}{2}\alpha_c \nu f_{cd} \qquad (6.12)$$

(4) For members with inclined shear reinforcement, the shear resistance is the smaller value of

$$V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} (\cot\theta + \cot\alpha)\sin\alpha \qquad (6.13)$$

and

$$V_{Rd,max} = \alpha_c\ b_w\ z \nu f_{cd}\ (\cot\theta + \cot\alpha)/(1 + \cot^2\theta) \qquad (6.14)$$

The maximum effective shear reinforcement, $A_{sw,max}$ follows from:

$$\frac{A_{sw,max}\ f_{ywd}}{b_w s} \leq \frac{\tfrac{1}{2}\alpha_c \nu f_{cd}\ \sin\alpha}{1 - \cos\alpha} \qquad (6.15)$$

(5) Where the web contains grouted ducts with a diameter $\phi > b_w/8$ the shear resistance $V_{Rd,max,comp}$ should be calculated on the basis of a nominal web thickness given by:

$$b_{w,nom} = b_w - 0,5\Sigma\phi \qquad (6.16)$$

where ϕ is the outer diameter of the duct and $\Sigma\phi$ is determined for the most unfavourable level.

For non-grouted ducts or unbonded tendons the nominal web thickness is:

$$b_{w,nom} = b_w - 1,2\ \Sigma\phi \qquad (6.17)$$

The value 1,2 in Expression (6.17) is introduced to take account of splitting of the concrete struts due to transverse tension. If adequate transverse reinforcement is provided this value may be reduced to 1,0.

(6) The additional tensile force, ΔF_{td}, in the longitudinal reinforcement due to shear V_{Ed} may be calculated from:

$$\Delta F_{td} = 0,5\ V_{Ed}\ (\cot\theta - \cot\alpha) \qquad (6.18)$$

$(M_{Ed}/z) + \Delta F_{td}$ should be taken not greater than $M_{Ed,max}/z$.

(7) At a distance $0,5d < x < 2,0\ d$ from the edge of a support the shear resistance may be increased to:

$$V_{Rd} = V_{Rd,ct} + A_{sw} \cdot f_{ywd} \sin\alpha \qquad (6.19)$$

Where $V_{Rd,ct}$ is calculated using Expression (6.5) for the most unfavourable value of x, and $A_{sw} \cdot f_{ywd}$ is the resistance of the shear reinforcement crossing the inclined shear crack between the loaded areas (see Figure 6.6). Only the shear reinforcement within the central $0,75\ a_v$ should be taken into account.

For $x < 0.5d$ the value $x = 0.5d$ should be used.

The value V_{Rd} from Expression (6.19) should not exceed the value $V_{Rd,max}$ given by Expression (6.9) or (6.14).

(8) Where a load is applied near the bottom of a section, sufficient vertical reinforcement to carry the load to the top of the section should be provided in addition to any reinforcement required to resist shear.

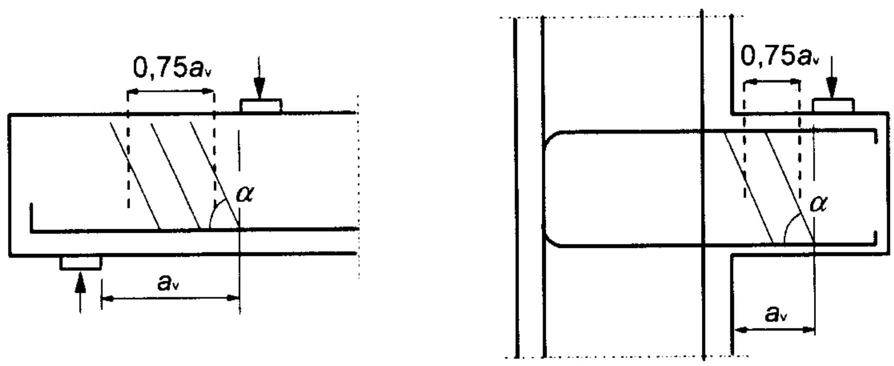


Figure 6.6: Shear reinforcement in short shear spans with direct strut action

6.2.4 Shear between web and flanges of T-sections

(1) The shear strength of the flange may be calculated by considering the flange as a system of compressive struts combined with ties in the form of tensile reinforcement.

(2) The ultimate limit state may be attained by compression in the struts or by tension in the ties which ensure the connection between flange and web. A minimum amount of reinforcement

should be provided, as specified in 9.2.1.

(3) The longitudinal shear stress, v_{Ed}, at the junction between one side of a flange and the web is determined by the change of the normal (longitudinal) force in the part of the flange considered, according to:

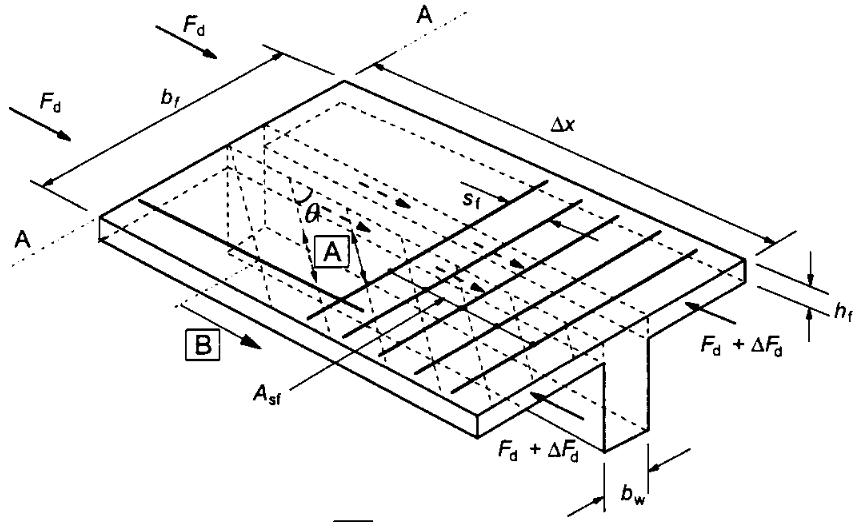
$$v_{Ed} = \Delta F_d/(h_f \cdot \Delta x) \qquad (6.20)$$

where:

h_f is the thickness of flange at the junctions

Δx is the length under consideration, see Figure 6.7

ΔF_d is the change of the normal force in the flange over the length Δx.



A - compressive struts   B - longitudinal bar anchored beyond this projected point

Figure 6.7: Notations for the connection between flange and web

The maximum value that may be assumed for Δx is half the distance between the section where the moment is 0 and the section where the moment is maximum. Where point loads are applied the length Δx should not exceed the distance between point loads.

(4) The transverse reinforcement per unit length A_{sf}/s_f may be determined as follows:

$$(A_{sf}f_{yd}/s_f) > v_{Ed} \cdot h_f/ \cot \theta_f \qquad (6.21)$$

To prevent crushing of the compression struts in the flange, the following condition should be satisfied:

$$v_{Ed} < \nu f_{cd} \sin\theta_f \cos\theta_f \qquad (6.22)$$

In the absence of a more rigorous calculation, the following values for θ_f may be used:

$1,0 \leq \cot \theta_f \leq 2,0$ for compression flanges ($45° \geq \theta_f \geq 26,5°$)

$1,0 \leq \cot \theta_f \leq 1,25$ for tension flanges ($45° \geq \theta_f \geq 38,6°$)

(5) In the case of combined shear between the flange and the web, and transverse bending, the area of steel should be the greater of that given by Expression (6.21) or half that given by Expression (6.21) plus that required for transverse bending.

(6) If v_{Ed} is less than or equal to 0,4 f_{ctd} no extra reinforcement above that for flexure is required.

(7) Longitudinal tension reinforcement in the flange should be anchored beyond the strut required to transmit the force back to the web at the section where this reinforcement is required (See Section (A - A) of Figure 6.7).

6.2.5 Shear at the interface between concretes cast at different times

(1) In addition to the requirements of 6.2.1- 6.2.4 the shear stress at the interface between concrete cast at different times should also satisfy the following:

$$v_{Edi} \leq v_{Rdi} \tag{6.23}$$

v_{Edi} is the design value of the shear stress in the interface and is given by:

$$v_{Edi} = \beta \, V_{Ed} \, / \, (z \, b_i) \tag{6.24}$$

where:

β is the ratio of the longitudinal force in the new concrete area and the total longitudinal force either in the compression or tension zone, both calculated for the section considered

V_{Ed} is the transverse shear force

z is the lever arm of composite section

b_i is the width of the interface (see Figure 6.8)

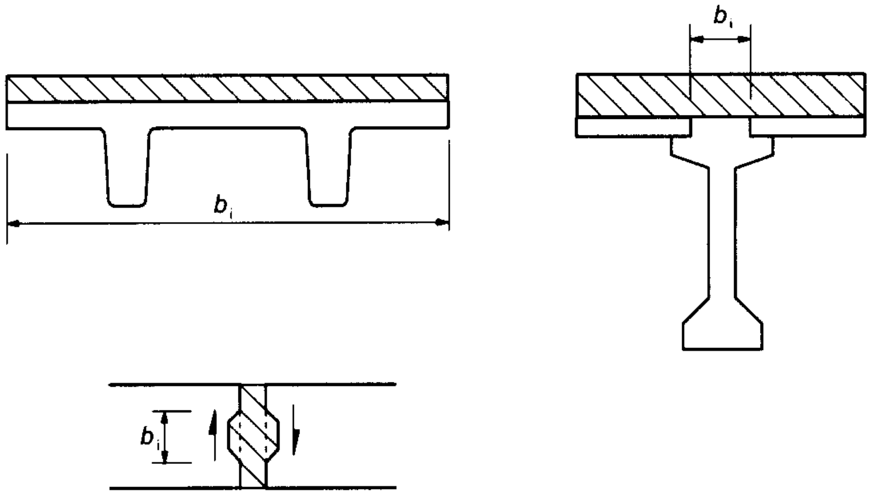


Figure 6.8: Examples of interfaces

v_{Rdi} is the design shear resistance at the interface and is given by:

$$v_{Rdi} = c \, f_{ctd} + \mu \, \sigma_n + \rho \, f_{yd} \, (\mu \sin \alpha + \cos \alpha) \leq 0{,}5 \, \nu \, f_{cd} \tag{6.25}$$

where:

c and μ are factors which depend on the roughness of the interface (see (2))

f_{ctd} is the design tensile strength of the concrete with the lowest strength with $f_{ctd} = f_{ctk,0,05}/\gamma_c$, where $f_{ctk,0,05}$ follows from Table 3.1

σ_n stress per unit area caused by the minimum external normal force across the interface that can act simultaneously with the shear force, positive for compression, such that $\sigma_n < 0,6\ f_{cd}$, and negative for tension. When σ_n is tensile $c\ f_{ctd}$ should be taken as 0.

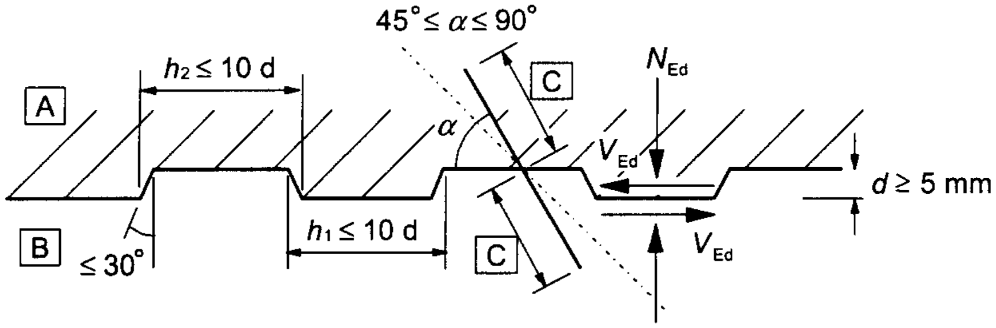
ρ $= A_s / A_i$

A_s area of reinforcement crossing the interface, including ordinary shear reinforcement (if any), with adequate anchorage at both sides of the interface.

A_i area of the joint

α defined in Figure 6.9, and should be limited by $45° \leq \alpha \leq 90°$

v effectivity factor according to Expression (6.6)



A - new concrete, B - old concrete, C - anchorage

Figure 6.9: Indented construction joint

(2) In the absence of more detailed information (e.g. that given in product standards) surfaces are classified as very smooth, smooth, rough or indented, with the following examples:

- Very smooth: a surface cast against steel, plastic or specially prepared wooden moulds: $c = 0,025$ and $\mu = 0,5$
- Smooth: a slipformed or extruded surface, or a free surface left without further treatment after vibration: $c = 0,35$ and $\mu = 0,6$
- Rough: a surface with at least 3 mm roughness at about 40 mm spacing, achieved by raking, exposing of aggregate or other methods giving an equivalent behaviour: $c = 0,45$ and $\mu = 0,7$
- Indented: a surface with indentations complying with Figure 6.9: $c = 0,50$ and $\mu = 0,9$

(3) A stepped distribution of the transverse reinforcement may be used, as indicated in Figure 6.10. Where the connection between the two different concretes is ensured by stiffeners (beams with lattice girders), the steel contribution to v_{Rdi} may be taken as the resultant of the forces taken from each of the diagonals provided that $45° \leq \alpha \leq 135°$.

(4) The longitudinal shear resistance of grouted joints between slab or wall elements may be calculated according to 6.2.5 (1). However in cases where the joint can be significantly cracked, c should be taken as 0 for smooth and rough joints and 0,5 for indented joints (see also 10.9.2 (12)).

(5) Under fatigue or dynamic loads, the values for c in 6.2.5 (1) should be halved.

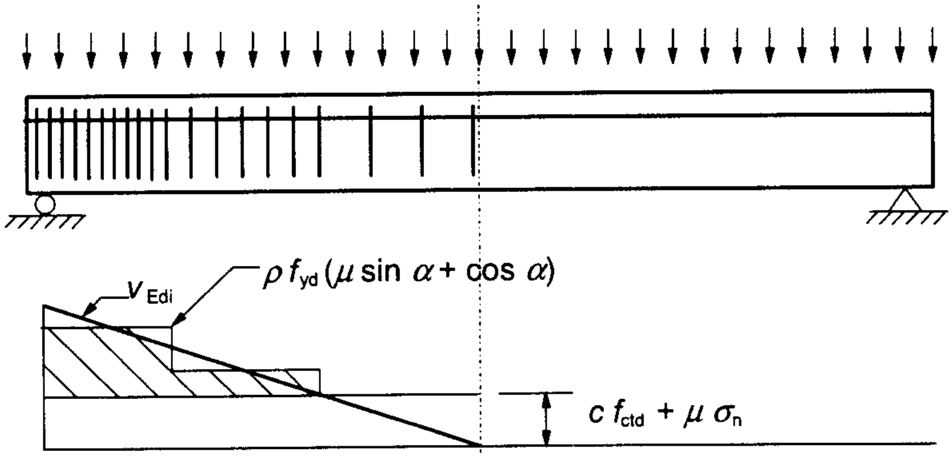


Figure 6.10: Shear diagram representing the required interface reinforcement

6.3 Torsion

6.3.1 General

(1)P Where the static equilibrium of a structure depends on the torsional resistance of elements of the structure, a full design covering both ultimate and serviceability limit states shall be made.

(2) Where, in statically indeterminate structures, torsion arises from consideration of compatibility only, and the structure is not dependent on torsional resistance for its stability, then it will normally be unnecessary to consider torsion at the ultimate limit state. In such cases a minimum reinforcement, given in Sections 7.3 and 9.2, in the form of stirrups and longitudinal bars should be provided in order to prevent excessive cracking.

(3) The torsional resistance of sections may be calculated on the basis of a thin-walled closed section, in which equilibrium is satisfied by a closed shear flow. Solid sections may be modelled by equivalent thin-walled sections. Complex shapes, such as T-sections, may be divided into a series of sub-sections, each of which is modelled as an equivalent thin-walled section, and the total torsional resistance taken as the sum of the capacities of the individual elements.

(4) The distribution of the acting torsional moments over the sub-sections should be in proportion to their uncracked torsional stiffnesses. For non-solid sections the equivalent wall thickness should not exceed the actual wall thickness.

(5) Each sub-section may be designed separately.

6.3.2 Design procedure

(1) The shear stress due to a pure torsional moment may be calculated from:

$$\tau_{t,i} t_{ef,i} = \frac{T_{Ed}}{2A_k} \tag{6.26}$$

The shear force $V_{Ed,i}$ in a wall i due to torsion is given by:

$$V_{Ed,i} = \tau_{t,i}\, t_{ef,i}\, z_i \qquad (6.27)$$

where
T_{Ed} applied design torsion (see Figure 6.11)

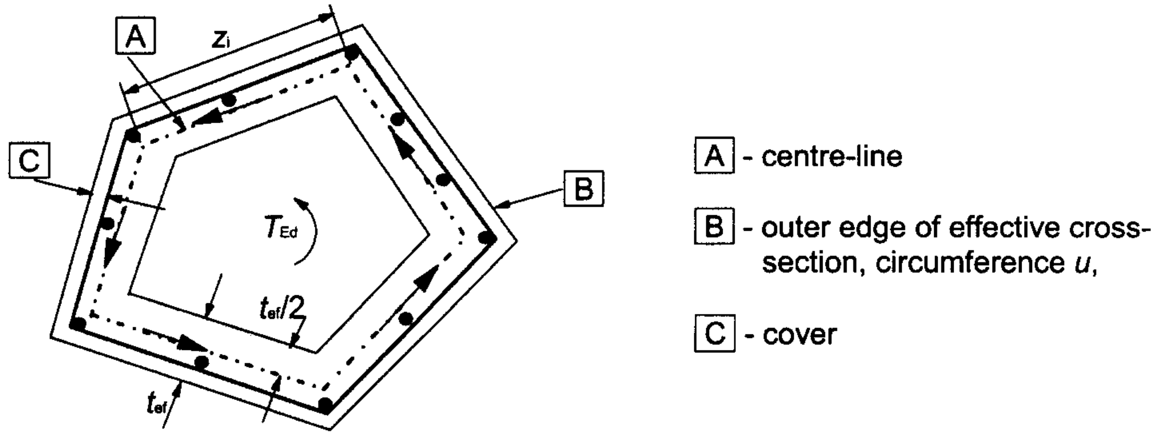


Figure 6.11: Notations and definitions used in Section 6.3

A_k area enclosed by the centre-lines of the connecting walls, including inner hollow areas.
$\tau_{t,i}$ torsional shear stress
$t_{ef,i}$ is the effective wall thickness. It may be taken as A/u, but need not be taken as less than twice the distance between edge and centre of the longitudinal reinforcement. For hollow sections the real thickness is an upper limit
A is the total area of the cross-section within the outer circumference, including inner hollow areas
u is the outer circumference of the cross-section
z_i is the side length of a wall defined by the distance between the intersection points with the adjacent walls

(2) The required transverse reinforcement for the effects of torsion (see Expression (6.27)) and shear for both hollow and solid members should be superimposed. The limits for θ given in 6.2.3 (2) are also fully applicable for the case of combined shear and torsion.

The maximum bearing capacity of a member loaded in shear and torsion follows from 6.3.2 (4).

(3) The required cross-sectional area of the longitudinal reinforcement for torsion ΣA_{sl} may be calculated from Expression (6.28):

$$\frac{\Sigma A_{sl} f_{yd}}{u_k} = \frac{T_{Ed}}{2A_k}\cot\theta \qquad (6.28)$$

where
u_k is the perimeter of the area A_k
f_{yd} is the design yield stress of the longitudinal reinforcement A_{sl}
θ is the angle of compression struts (see Figure 6.5).

In compressive chords, the longitudinal reinforcement may be reduced in proportion to the available compressive force. In tensile chords the longitudinal reinforcement for torsion should be added to the other reinforcement. The longitudinal reinforcement should generally be distributed over the length of side, z_i, but for smaller sections it may be concentrated at the ends of this length.

(4) The maximum resistance of a member subjected to torsion and shear is limited by the capacity of the compression struts. In order not to exceed this capacity the following condition should be satisfied:

- for solid cross-sections:

$$\left(\frac{T_{Ed}}{T_{Rd,max}}\right)^2 + \left(\frac{V_{Ed,w}}{V_{Rd,max}}\right)^2 \leq 1 \qquad (6.29)$$

- for hollow cross-sections:

$$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,w}}{V_{Rd,max}} \leq 1 \qquad (6.30)$$

where:

T_{Ed} is the design torsional moment
$V_{Ed,w}$ is the design transverse force
$T_{Rd,max}$ is the design torsional resistance moment according to

$$T_{Rd,max} = 2\nu\,\alpha_c\,f_{cd}\,A_k\,t_{ef,i}\,\sin\theta\cos\theta \qquad (6.31)$$

where ν follows from Expression (6.6) and α_c from Expression (6.9)

$V_{Rd,max}$ is the design shear resistance according to Expressions (6.9) or (6.12).

(5) For approximately rectangular solid sections only minimum reinforcement is required (see 9.2.1.1) provided that both the following conditions are satisfied:

$$T_{Ed} \leq \frac{V_{Ed}\,b_w}{4,5} \qquad (6.32)$$

$$V_{Ed}\left[1+\frac{4,5\,T_{Ed}}{V_{Ed}\,b_w}\right] \leq V_{Rd,ct} \text{ (see Expressions (6.2.a) and (6.2.b))} \qquad (6.33)$$

where b_w is the width of the cross section.

6.3.3 Warping torsion

(1) For closed thin-walled sections and solid sections, warping torsion may normally be ignored.

(2) In open thin walled members it may be necessary to consider warping torsion. For very slender cross-sections the calculation should be carried out on the basis of a beam-grid model and for other cases on the basis of a truss model. In all cases the design should be carried out according to the design rules for bending and longitudinal normal force, and for shear.

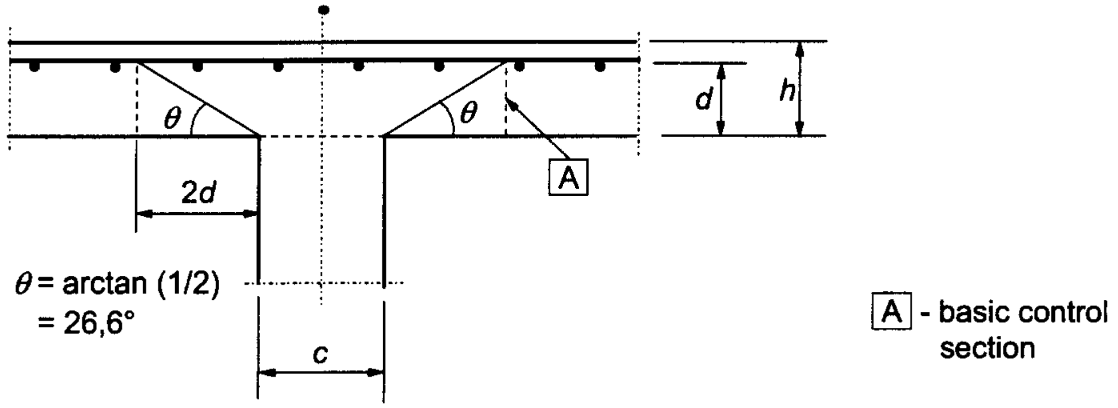
6.4 Punching

6.4.1 General

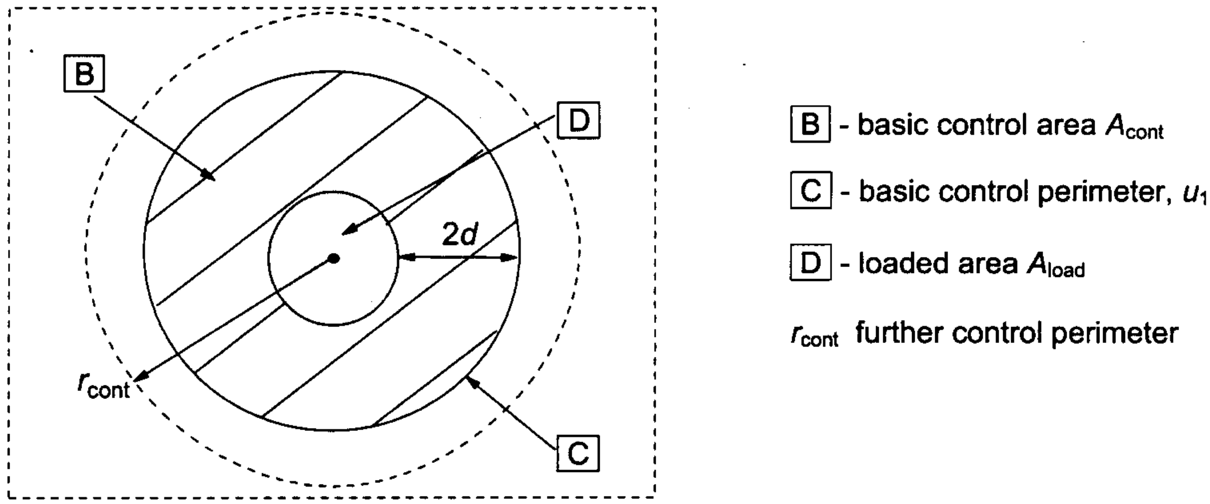
(1)P The rules in this Section complement those given in 6.2 and cover punching shear in solid slabs, waffle slabs with solid areas over columns, and foundations.

(2)P Punching shear can result from a concentrated load or reaction acting on a relatively small area, called the loaded area A_{load} of a slab or foundation.

(3) An appropriate verification model for checking punching failure at the ultimate limit state is shown in Figure 6.12.



a) Section



b) Plan

Figure 6.12: Verification model for punching shear at the ultimate limit state

(4) The shear resistance should be checked along defined control perimeters.