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Eurocode 2: Bemessung und Konstruktion von Stahlbeton- und Spannbetontragwerken — Teil 1-1: Allgemeine Regeln und Regeln für Hochbauten, Brücken und Ingenieurbauwerke

Eurocode 2: Calcul des structures en béton — Partie 1-1: Règles générales et règles pour les bâtiments, les ponts et les ouvrages de génie civil

ICS:

Content

European foreword

This document (FprEN 1992-1-1:2022) has been prepared by Technical Committee CEN/TC 250 “Structural Eurocodes”, the secretariat of which is held by BSI. CEN/TC 250 is responsible for all Structural Eurocodes and has been assigned responsibility for structural and geotechnical design matters by CEN.

This document is currently submitted to the Formal Vote.

This document will supersede EN 1992-1-1:2004, EN 1992-2:2005 and EN 1992-3:2006 and their amendments and corrigenda.

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

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

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

0 Introduction

0.1 Introduction to the Eurocodes

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

— EN 1990, Eurocode: Basis of structural and geotechnical design

— EN 1991, Eurocode 1: Actions on structures

— EN 1992, Eurocode 2: Design of concrete structures

— EN 1993, Eurocode 3: Design of steel structures

— EN 1994, Eurocode 4: Design of composite steel and concrete structures

— EN 1995, Eurocode 5: Design of timber structures

— EN 1996, Eurocode 6: Design of masonry structures

— EN 1997, Eurocode 7: Geotechnical design

— EN 1998, Eurocode 8: Design of structures for earthquake resistance

— EN 1999, Eurocode 9: Design of aluminium structures

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

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

0.2 Introduction to EN 1992 Eurocode 2

(1) EN 1992 applies to the design of buildings, bridges and civil engineering structures 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 and geotechnical design*.

(2) EN 1992 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) EN 1992 is subdivided into the following parts:

— Part 1-1: General rules — Rules for buildings, bridges and civil engineering structures,

— Part 1-2: Structural fire design,

— Part 4: Design of fastenings for use in concrete.

0.3 Introduction to FprEN 1992-1-1

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

(2) FprEN 1992-1-1 also serves as a reference document for other CEN TCs concerning structural matters.

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

0.4 Verbal forms used in the Eurocodes

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

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

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

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

0.5 National Annex for FprEN 1992-1-1

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

The national standard implementing EN 1992-1-1 can have a National Annex containing all national choices to be used for the design of buildings, bridges and civil engineering works to be constructed in the relevant country.

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

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

National choice is allowed in FprEN 1992-1-1 through notes to the following:

4.2.1.5(3)	4.3.1(1)	4.3.2(1)	4.3.2(2)
4.3.3(1)	4.3.3(4)	5.1.3(3)	5.1.4(2)
5.1.5(4)	5.1.6(1)	5.1.6(2)	5.2.1(5)
5.2.2(1)	5.3.1(3)	5.3.2(1)	5.4.1(1)
6.3(3)	6.3(5)	6.4(1)	6.5.2.1(2)
6.5.2.2(1)	6.5.2.2(2)	6.5.2.2(3)	6.5.2.2(4)
6.5.2.2(5)	6.5.2.2(6)	6.5.2.2(9)	6.5.3(1)
7.3.2(5)	8.2.1(3)	8.2.2(5)	8.4.2(1)
8.4.4(4)	8.4.4(5)	8.4.4(6)	9.2.1(6)
9.2.3(2)	11.4.2(2)	11.4.2(3)	11.5.2(2)
11.6.3(2)	12.3.1(1)	12.4.1(1)	12.6(1)
12.7(2)	12.9.3(1)	14.2(1)	14.4.5.2(1)

A.3(1)	A.3(3)	A.3(6)	B.3(1)
B.6(1)	C.6(1)	C.7(1)	C.8(2)
E.4.2(1)	F.5.2(1)	F.7(2)	F.7(8)
H.4.2(4)	I.4.2.1(2)	I.5.2.1(3)	I.5.2.2(1)
I.8.3.1(1)	I.9.1(2)	J.4(1)	J.5.1(2)
K.5(2)	K.6(2)	K.6(3)	K.6(4)
K.7(2)	K.7(3)	K.8(2)	K.8(3)
K.8(4)	K.8(5)	K.8(6)	K.9(2)
K.9(3)	K.10(2)	K.11(2)	K.11(3)
K.11(4)	K.11(5)	K.12.2(1)	K.13(2)
K.13(4)	K.15(1)	L.4(1)	L.5.1(1)
L.5.5.2(1)	L.6(4)	L.7(1)	L.11.2(1)
L.12.3.1(2)	Q.3(1)	Q.4(2)	R.4(1)
R.5.1(2)			

National choice is allowed in FprEN 1992-1-1 on the application of the following informative annexes:

Annex A	Annex D	Annex F	Annex H
Annex I	Annex J	Annex L	Annex N
Annex O	Annex P	Annex R	Annex S

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

1 Scope

1.1 Scope of Part 1-1 of EN 1992

(1) This document gives the general basis for the design of structures in plain, reinforced and prestressed concrete made with normal weight, lightweight and heavyweight aggregates. It gives specific rules for buildings, bridges and civil engineering structures, including temporary structuress, additional requirements specific to bridges are given in Annex K. The rules are valid under temperature conditions between −40 °C and +100 °C generally. This document complies with the principles and requirements for the safety, serviceability, durability and robustness of structures, the basis of their design and verification that are given in EN 1990.

(2) FprEN 1992 is only concerned with the requirements for resistance, serviceability, durability, robustness and fire resistance of concrete structures. Other requirements, e.g. concerning thermal or sound insulation, are not considered.

(3) Specific rules or information are covered in Annexes:

— Interface to product standards in Annex C with values of product properties which are assumed to be met for design to this Eurocode;

— for Assessment of Existing Structures in Annex I;

— for Strengthening of Existing Concrete Structures with CFRP in Annex J;

— requirements specific to bridges are given in Annex K

— for Steel Fibres Reinforced Concrete in Annex L;

— for Stainless Reinforcing Steel in Annex Q;

— for embedded FRP reinforcement in Annex R.

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

— resistance to fire (see EN 1992-1-2);

— fastenings in concrete (see EN 1992-4);

— seismic design (see EN 1998 (all parts));

— particular aspects of special types of civil engineering works (such as dams, pressure vessels);

— structures made with no-fines concrete, aerated or cellular concrete, lightweight aggregate concrete with open structure components;

— structures containing steel sections considered in design (see EN 1994 (all parts)) for composite steel and concrete structures;

— structural parts made of concrete with a smallest value of the upper sieve aggregate size $D_{\text{lower}} < 8$ mm (or if known $D_{\max} < 8$ mm) unless otherwise stated in this Eurocode.

1.2 Assumptions

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

— Structures are designed by appropriately qualified and experienced personnel;

— requirements for execution and workmanship given in EN 13670 are complied with.

2 Normative references

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

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

EN 197-1, *Cement: Composition, specification and conformity criteria for common cements*

EN 206, *Concrete: Specification, performance, production and conformity*

EN 1504-4, *Products and systems for the protection and repair of concrete structures — Definitions, requirements, quality control and evaluation of conformity — Part 4: Structural bonding*

EN 1542, *Products and systems for the protection and repair of concrete structures — Test methods — Measurement of bond strength by pull-off*

EN 1990, *Basis of structural and geotechnical design*

EN 1991 (all parts), *Actions on structures*

FprEN 1992-1-2:2022, *Eurocode 2: Design of concrete structures — Part 1-2: General rules — Structural fire design*

EN 1992-4, *Eurocode 2: Design of concrete structures — Part 4: Design of fastenings for use in concrete*

EN 1993 -1-9, *Eurocode 3: Design of steel structures — Part 1-9: Fatigue*

EN 1997 (all parts), *Geotechnical design*

EN 13670, *Execution of concrete structures*

EN 13791:2019, *Assessment of in-situ compressive strength in structures and pre-cast concrete components*

EN 14651, *Test method for metallic fibre concrete — Measuring the flexural tensile strength (limit or proportionality (LOP), residual)*

EN ISO 14130, *Fibre reinforced plastic composites — Determination of apparent interlaminar shear strength by short beam-method*

EN ISO 17660, *Welding — Welding of reinforcing steel*

ISO 10406 (all parts), *Fibre-reinforced polymer (FRP) reinforcement of concrete — Test methods*

3 Terms, definitions and symbols

3.1 Terms and definitions

For the purposes of this document, the terms and definitions given in EN 1990 and the following apply.

3.1.1

anchoring mortar

mortar based on organic or inorganic binder, or a mixture of these, installed at a fluid or paste consistency with the aim to anchor reinforcing steel bars in a drilled hole in concrete structures and to transfer the axial forces in the reinforcing steel bar to the concrete structure

3.1.2

beam

linear member subject primarily to flexure and shear with cross-section width not exceeding 4 times its cross-section depth (otherwise it should be considered as a slab) and an effective span of not less than 3 times the cross-section depth

3.1.3

biaxial bending

simultaneous bending about two principal axes

3.1.4

braced members or systems

structural members or subsystems, which in analysis and design are assumed to be stabilised by bracing members and hence, not contributing to the overall horizontal stability of a structure

Note 1 to entry: Unbraced members are not stabilised by bracing members.

3.1.5

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

3.1.6

buckling

failure due to instability of a member or structure under under compression buckling or flexural buckling

3.1.7

buckling load

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

3.1.8

carbon steel

weldable non-alloy reinforcing steel reinforcement

3.1.9

chord

compression or tension part of a member idealised as having a narrow width and which interacts with adjacent membrane elements through longitudinal shear

3.1.10

column

linear member subjected primarily to axial compression forces, with cross-section width not exceeding 4 times its cross-section depth (otherwise it should be considered as a wall) and the length is at least 3 times the cross-section depth

3.1.11

compression field

region of a stress field where concrete is subjected to uniaxial compressive stresses

3.1.12

confinement reinforcement

reinforcement which can increase the uniaxial concrete compressive strength and the deformation capacity through the favourable effect of transverse compressive stresses or can reduce the required anchorage length by preventing cover spalling

Note 1 to entry: It can consist of stirrups, links, U-bars, headed bars or hoops placed perpendicular or at an angle to the axis of the member.

Note 2 to entry: Confinement reinforcement can reduce the design anchorage length if it is anchored into the body of the section.

3.1.13

couplers

steel reinforcement products used for the mechanical splicing of steel reinforcing bars

3.1.14

cover, concrete

distance between the surface of a reinforcement bar or tendon (including links and stirrups and surface reinforcement where relevant) and the nearest concrete surface

3.1.15

cover, minimum

minimum value of the concrete cover provided in order to ensure (i) safe transmission of bond forces, (ii) protection of the steel against corrosion (durability)

3.1.16

cover, nominal

specified value of the concrete cover defined as a minimum cover plus an allowance in design for deviation

3.1.17

crack formation phase

phase of the cracking process which occurs when stresses exceed the cracking resistance and in which the full crack pattern is not yet developed, typically it is the type of cracking due to imposed deformations with large crack spacings, and for which an increase in the imposed strain will not increase crack width but instead it will form new cracks

3.1.18

crack width, calculated

calculated crack width at surface of member

3.1.19

creep, basic

creep occurring in concrete when there is no moisture transfer with the surrounding environment

3.1.20

creep, drying

creep, additional to basic creep, occurring in concrete when there is moisture transfer with the surrounding environment, the total creep is the sum of basic and drying creep

3.1.21

deep beam

beam for which the effective span l_{eff} is less than 3 times the overall cross-section depth h

3.1.22

deformation capacity

ability of a member or part of it or a structure to deform while maintaining its resistance

3.1.23

damp patch

area which, when touched, might leave a light film of moisture on the hand but no droplets of water (i.e. beading)

3.1.24

diaphragm

planar member able to resist in-plane forces

3.1.25

effective tension area

concrete area in tension around reinforcement within which the crack opening is effectively controlled by the reinforcement (area of concrete that needs to be subject to tension up to the tensile resistance of concrete to produce a new crack)

3.1.26

effective depth

in a cross-section, distance from the extreme compression fibre to the centroid of the resultant force of the longitudinal tension reinforcement

3.1.27

effective length

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 axial force, having the same cross-section and buckling load as the actual member

3.1.28

European Technical Product Specification

— a European Product Standard (EN),

— or a European Technical Assessment (ETA) based on a European Assessment Document (EAD),

— or a product documentation based on a transparent and reproducible assessment that complies with all requirements of the relevant EAD

3.1.29

execution specification

document covering all drawings, technical data and requirements necessary for the execution of a particular project

3.1.30

exposure resistance classes

classes for defining the resistance of concrete against corrosion induced by carbonation (XRC) or by chlorides (XRDS) and damage caused by freeze/thaw attack (XRF)

3.1.31

external tendon

tendon external to the concrete, either within the depth of the cross-section or on the surface of the cross-section

3.1.32

first order effects

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

3.1.33

flat slab

slab supported directly by columns, can be solid, ribbed or waffle

3.1.34

general anchorage zone

zone in which the tendon force is dispersed over the member cross-section until a linear stress distribution may be assumed

3.1.35

headed bar

reinforcing bar with head attached at one or both ends

3.1.36

hook

end part of a reinforcement, bent at not less than 135°

3.1.37

hoop

closed reinforcement or spiral reinforcement enclosing longitudinal reinforcement in compression members

3.1.38

indented reinforcement

reinforcing steel with at least two rows of indentations, which are uniformly distributed over the entire length

3.1.39

internal force

resultant of stresses in cross-section of a member (axial force, shear force, bending moment, torsion)

3.1.40

internal tendon

tendon which is placed inside the concrete either with bond or without bond to the concrete

3.1.41

isolated member

member for which no load redistribution to adjacent members is possible

3.1.42

isolated precast element

precast element for which no load redistribution to adjacent members is possible

3.1.43

lattice girder

two dimensional or three dimensional metallic structure comprising an upper chord, one or more lower chords and continuous or discontinuous diagonals which are welded or mechanically connected to the chords

3.1.44

lightweight aggregate concrete

concrete having a closed structure and an oven-dry density of not less than 800 kg/m^3 and not more than 2 000 kg/m^3, consisting of or containing a proportion of artificial or natural lightweight aggregates having a particle dry density of less than 2 000 kg/m^3

3.1.45

linear member

structural element, straight or curved, with one dimension significantly larger than the others (such as beams and columns)

3.1.46

link

reinforcement bent to form single or multiple legs that surrounds longitudinal reinforcement, if provided in the form of links these can be closed or open with sufficient anchorage at their ends

Note 1 to entry: See also “Stirrup” which has a similar definition as “Link”, but does not include single leg Z- or C-shaped reinforcement.

3.1.47

local anchorage zone

zone in the immediate vicinity of the tendon anchorage or coupling device in which the tendon force is transmitted from the anchorage or coupling device to the concrete

3.1.48

loop

U-shaped reinforcement where both legs transmit their forces to other reinforcement or to concrete through bond

3.1.49

main reinforcement

in one-way slabs, the bending reinforcement placed in the direction perpendicular to the supports, in other members the longitudinal reinforcement with the largest overall capacity

3.1.50

membrane

planar member subjected primarily to in plane forces

3.1.51

nodal region

region of a stress field where the force is transferred amongst concurrent compression fields and/or ties

3.1.52

node

point of intersection of struts and/or ties transferring forces amongst them

3.1.53

nonlinear analysis

analysis method using models that account for mechanical and geometrical non-linear behaviour

3.1.54

ordinary reinforcement

reinforcement which is not prestressed, where not specified otherwise, it is made of reinforcing steel

3.1.55

plain reinforcement

reinforcement with a smooth surface

3.1.56

plain or lightly reinforced concrete members

structural concrete members having no reinforcement (plain concrete) or less reinforcement than the minimum amounts as defined in Clause 12

3.1.57

planar member

structural member with a dimension in one direction (depth) significantly smaller than those in the other directions (width) with width / depth > 4 (such as slabs, walls and shells)

3.1.58

pocket (or socket) foundation

member (precast, cast-insitu or partly precast) forming a tight pit for embedding the bottom of a precast column, fixed with infilled cast-insitu concrete

3.1.59

post-installed reinforcing steel system

deformed straight reinforcing steel bar and anchoring mortar installed using tools for drilling and preparing the hole (e.g. roughening and cleaning) as well as for injection of the mortar (e.g. dispenser, nozzles, piston plug, if applicable)

3.1.60

post-tensioning

prestress technique which consists in applying the prestress to tendons positioned in a hardened concrete member within a complete assembly of anchorages, sheathing with coating (for unbonded applications) or ducts to be grouted (for bonded applications)

3.1.61

precast concrete element

factory produced or site manufactured element cast and cured in a place other than its final location in the structure

3.1.62

precast concrete product

concrete element manufactured in accordance with a product standard by an industrial process under a factory production control system and protected from weather conditions during production

3.1.63

precast structure

structure assembled from precast concrete elements, connected to ensure the required structural integrity

3.1.64

prestress

effect of prestressing process, namely, internal forces in the sections and the deformations of the structure

3.1.65

prestressing process

process of prestressing consists in applying forces to the concrete structure and the tendons

3.1.66

prestressed reinforcement

reinforcement made of strands, wires or bars subjected to a prestressing process, where not specified otherwise, it is made of prestressing steel

3.1.67

pre-tensioning

process by which tendons are stressed before and remain stressed during their embedment in cast concrete

3.1.68

pre-tensioning tendon

tendon in which the prestressed reinforcement is embedded in and bonded directly to concrete

3.1.69

reinforcement

assembly of bars and/or tendons, prestressed (prestressed reinforcement) or not (ordinary reinforcement), embedded in or connected to concrete members. Where not specified otherwise, it is made of steel and is bonded to the concrete

3.1.70

ribbed reinforcement

reinforcing bars with at least two rows of ribs uniformly distributed over the entire length

3.1.71

ribbed slab

slab with narrow ribs spanning in one direction

3.1.72

second order effects

additional action effects caused by structural deformations

3.1.73

secondary reinforcement

in one-way slabs, the bending reinforcement placed in the direction parallel to the supports

3.1.74

shear reinforcement, shear assemblies

stirrups, links, headed bars or bent-up bars specifically placed to resist action effects caused by shear and torsion

3.1.75

shell

planar member, either plane or curved, that carries both in-plane and out of plane forces. Cylindrical shells are simply curved, spherical shells are double curved

3.1.76

shrinkage, basic

shrinkage occurring in concrete when there is no humidity transfer with the surrounding environment. Also known as autogenous shrinkage

3.1.77

shrinkage, drying

shrinkage, additional to basic shrinkage, occurring in concrete when there is humidity transfer with the surrounding environment. The total shrinkage is the sum of basic and drying shrinkage

3.1.78

slab

planar member loaded primarily perpendicularly to its plane for which the minimum panel dimension is not less than 4 times the overall thickness, possibly acting also as diaphragm

3.1.79

slab, solid

slab without voids or ribs

3.1.80

spiral reinforcement

continuously wound reinforcement in form of a helix, cylindrical or prismatic

3.1.81

stabilized cracking

phase of the cracking process in which the crack pattern is fully developed. An increase in the actions will normally result in an increase in the crack opening. This type of cracking is typically associated with applied external loads, when such loads are sensibly above the cracking loads

3.1.82

stainless steel

stainless reinforcing steel in accordance with prEN 10370

3.1.83

stirrup

reinforcement bent to form double or multiple legs that surrounds longitudinal reinforcement. Stirrups can be closed or open with sufficient anchorage at their ends

3.1.84

stress field

stress state in a structure equilibrating the external actions

3.1.85

strut

resultant of a compression field, part of a strut-and-tie model

3.1.86

strut-and-tie model

model composed of the resultant forces of a stress field with struts for the compression fields and ties for the tension reinforcement

3.1.87

support, direct

bearing by contact forces pushing against the member

3.1.88

support, indirect

support with local tensile stresses in the supporting member caused by applied loads

3.1.89

technical documentation of post-tensioning system

documentation containing all information relevant for design and construction of post-tensioned structures in accordance with this Eurocode.

3.1.90

tendon

in post-tensioned applications, the tendon is a complete assembly consisting of anchorages, prestressing steel (strand, wire, bar), and sheathing with coating for unbonded applications or ducts with grout for bonded applications. In pre-tensioned applications, the tendon is an individual element of prestressed reinforcement

3.1.91

tendon protection level

designation of a class or level of corrosion protection provided to tendons

3.1.92

tie

tension member as part of a strut-and-tie model representing concentrated or distributed reinforcements

3.1.93

transverse reinforcement

reinforcement arranged perpendicular to the bar considered

Note 1 to entry: In linear members it can consists of stirrups, links or hoops enclosing the longitudinal reinforcement considered; in planar member it consists of straight reinforcement parallel to the free surface.

3.1.94

unbonded tendon

tendon for post-tensioned members where bond of the prestressed reinforcement to the member is permanently prevented by encasing it in sheathing with soft filler or by placing the tendon outside the concrete section (see external tendon)

3.1.95

waffle slabs

slab with narrow ribs spanning in both directions

3.1.96

wall

planar member subjected primarily to in plane forces, with cross-section width exceeding 4 times its thickness (otherwise it should be considered as a column) and the height is at least 3 times the section thickness

Terms and definitions in Annex I

3.1.97

corrosion penetration depth

loss in cross-sectional radius of a bar due to homogeneous corrosion (not pitting corrosion or localized zones)

3.1.98

pitting corrosion

form of localised corrosion that leads to the creation of cavities or holes in the metal

Terms and definitions in Annex J

3.1.99

adhesive

material that possesses enough adhesive strength to join CFRP reinforcement to a concrete surface

3.1.100

adhesively bonded CFRP reinforcement

externally or Near Surface Mounted CFRP reinforcement bonded to concrete using adhesive to provide a longitudinal shear connection

3.1.101

CFRP bar

thermally hardened, unidirectional CFRP reinforcement industrially manufactured in various shapes used as NSM reinforcement

3.1.102

Carbon Fibre Reinforced Polymer

CFRP

fibre-polymer composite material comprising industrially manufactured carbon fibres embedded in a polymer matrix

3.1.103

Carbon Fibre Reinforced Polymer (CFRP) system

composite comprising carbon fibres with an accompanying adhesive material that is bonded to an adequately prepared concrete strata for the purpose of strengthening a structural concrete component

3.1.104

concrete cover separation

failure mode occurring at the end of adhesively bonded reinforcement, where a shift in tensile force may detach the concrete cover and the entire adhesively bonded reinforcement

3.1.105

externally bonded reinforcement

EBR

adhesively bonded CFRP reinforcement installed externally to a concrete surface

3.1.106

externally bonded stirrups

externally bonded CFRP system, embracing the member in closed or U-shaped form

3.1.107

Near Surface Mounted (NSM) reinforcement

adhesively bonded CFRP bar installed in slots cut into the existing concrete cover zone

3.1.108

sheets

textile surface structure comprising dry parallel fibre bundles arranged in one or more directions

3.1.109

slot

small recess cut into the concrete cover zone with predetermined dimensions along the member filled with adhesive in which adhesively bonded CFRP strips or bars (NSM) are embedded

3.1.110

strip

thermally hardened, unidirectional CFRP reinforcement industrially prefabricated in various rectangular flat shapes used as NSM or EBR reinforcement

Terms and definitions in Annex L

3.1.111

Steel Fibre Reinforced Concrete (SFRC)

concrete to which steel fibres are included into the concrete matrix to achieve post cracking residual strength

3.1.112

residual flexural strength

stress at the outer most tension layer of a SFRC cross-section in bending corresponding to a certain crack width determined using linear elastic material behaviour and the assumption that plane sections remain plane during bending

3.1.113

residual tensile strength

uniaxial tensile stress corresponding to a certain crack opening derived from the residual flexural strength using design rules provided in Annex L

3.1.114

residual strength class

classification that defines the response of a SFRC beyond the cracking strain of concrete. This class defines the strength of the SFRC concrete without additional reinforcing bars or prestressing

3.1.115

ductility class

classification that is defined by the ratio between the residual flexural strengths at $CMOD_1$ and $CMOD_3$

Terms and definitions in Annex R

3.1.116

Fibre Reinforced Polymer (FRP)

fibre-polymer composite material comprising industrially manufactured fibres embedded in a polymer matrix

3.1.117

fibre reinforced polymer (FRP) reinforcement

assembly of profiled or roughened fibre reinforced polymer reinforcement bars, embedded in or connected to concrete members

3.2 Symbols and abbreviations

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

3.2.1 Latin upper case letters

A	Correction factor	Table 5.2
A_{c}	Cross-sectional area of concrete	8.2.2(5)
A_{cc}	Compressive area	8.2.3(7), 14.4.3(3)
$A_{c,eff}$	Effective concrete area	9.2.1(7), 9.2.3(3)
A_{cc}	Area of confinement core of partially loaded area	8.6(4)
A_{c0}	Loaded area in partially loaded area	8.6(2), Figure 8.32
A_{c1}	Contributing area in partially loaded area	8.6(2), Figure 8.32
A_{e}	Equivalent area of reinforcement	10.3(2)
A_{gt}	Elongation at maximum force	C.4.1.(2), Table C.2

A_k	Area enclosed by centrelines of connecting walls of cross section	8.3.2(2), Figure 8.16
A_p	Cross-sectional area of prestressed reinforcement	8.1.1(6), Figure 8.1, 9.2.2(3),
$A_{p,red}$	Reduced cross-sectional area of prestressing steel	K.12.2(1)
A_s	Cross-sectional area of ordinary reinforcement	8.1.1(6), Figure 8.1
A_{sc}	Cross-sectional area of longitudinal reinforcement in the compression chord; confinement reinforcement	8.2.3(7); 11.4.2(7)
$A_{s,conf}$	Cross-sectional area of one leg of confinement reinforcement	8.1.4(3)
$A_{s,confx}$, $A_{s,confy}$	Value of $A_{s,conf}$ in the x and y-directions, respectively	8.1.4(3)
A_{sf}	Cross-sectional area of the transverse reinforcement in a flange	8.2.5(4), Figure 8.13
A_{si}	Cross-sectional area of bonded reinforcement across interface	8.2.6(5), Figure 8.15
A_{sint}	Robustness reinforcement in flat slabs	12.5.2(1)
A_{sl}	Effective area of tensile reinforcement	8.2.2(2), Figure 8.7
$A_{s,min}$	Minimum cross-sectional area of reinforcement	12.1(4), 12.2(1), 12.2(2)
$A_{s,min,h}$	Minimum amount of horizontal reinforcement	12.7(2)
$A_{s,min,v}$	Minimum amount of vertical reinforcement	12.7(2)
$A_{s,min,w1}$	Area of minimum reinforcement to be placed at the most tensioned face of the section part under consideration to control cracking	9.2.2(2), S.3(1)
$A_{s,min,w2}$	Area of minimum reinforcement to be placed at the least tensioned face of the section part under consideration to control cracking	9.2.2(2), S.3(1)
$A_{s,web}$	Reinforcement area to be provided in the web over a height limited by the neutral axis and the centroid of reinforcement with a spacing not exceeding 300 mm, to control cracking	9.2.2(4)
A_{st}	Cross-sectional area of longitudinal reinforcement in the tension chord; transverse reinforcement	8.2.3(7), 8.2.5(5); 11.4.2(7), 11.5.2(10)
A_{std}	Tie down reinforcement in laps of headed bars	11.5.5(7)
$A_{st,min}$	Minimum transverse reinforcement	8.2.5(2), 12.3.1(1), Table 12.1
$A_{s,v}$	Amount of vertical reinforcement	12.7(2)
A_{sw}	Cross-sectional area of shear reinforcement	8.2.3(5), 8.3.4(2)
$A_{sw,min}$	Minimum cross-sectional area of shear reinforcement	12.2(4)
$A_{s,req,span}$	Amount of required flexural non-prestressed reinforcement	12.3.1
C	Coefficient for fatigue strength of concrete under compression	E.5.3(1)
C_m	Coefficient used to obtain an equivalent constant moment to calculate second order effects in elements with differing end moments	O.7.2(2)
D_h	Diameter of a circular hoop or spiral reinforcement (defined by the bar's axis)	8.2.3(9), Figure 8.10
D_{lower}	Smallest value of the upper sieve size D in an aggregate for the coarsest fraction of aggregates in the concrete permitted by the specification of concrete [EN 206]	5.1.2
D_{max}	Declared value of the upper sieve size D of the coarsest fraction of aggregates actually used in the concrete [EN 206]	1.1(4), 8.2.1(4)

D_{upper}	Largest value of the upper sieve size D in an aggregate for the coarsest fraction of aggregates in the concrete permitted by the specification of concrete [EN 206]	5.1.2, 11.2(2)
E	Effect of actions	EN 1990
E_0	Force or stress at the end of construction with no consideration for creep	7.3.1(6), (7)
E_{cd}	Design value of modulus of elasticity of concrete	7.4.3.3(3)
$E_{c,eff}$	Effective modulus of elasticity of concrete accounting for creep deformations	9.1(4)
E_{cm}	Secant modulus of elasticity of concrete	5.1.4(2)
$E_{c,28}$	Secant modulus of elasticity of concrete at age of 28 days	5.1.5(1)
E_p	Design value of modulus of elasticity of prestressing steel	5.3.3(3)
E_s	Design value of modulus of elasticity of ordinary reinforcing steel	5.2.4(3)
EI	Bending stiffness	O.3(1)
EI_{eff}	Effective bending stiffness	7.4.2.3
E_t	Action effect (force or stress) at time t	7.3.1(6)
$E_{t=0}$	Internal force or stress at time t=0	7.3.1(6),(7)
E_{wc}	Force or stress assuming the structure was built without changes in the support conditions	7.3.1(7)
F	Action	EN 1990
F_{cd}	Design value of the compression force in a compression chord or in a strut (compression positive)	8.2.3(8), 8.5.2(2)
F_d	Design value of an action	Figure 8.13, F.3(5), 10.2(1)
ΔF_d	Design value of the change of the axial force in the flange over the length Δx	8.2.5(1)
F_{Ed}	Design value of actions	13.7.1(9)
$F_{Ed,2}$	Fictitious magnified horizontal force to account for global second order effects	O.8.2(2)
$F_{Ed,sup}$	Design support reaction due to the loads applied on the beam or the slab	7.2.3(7)
F_{fat}	Relevant fatigue action (e.g. traffic load or other cyclic load)	10.2(1)
$F_{H,0Ed}$	First order horizontal force due to wind, imperfections etc.	O.3(2), O.8.2(2)
$F_{H,1Ed}$	Fictitious horizontal force	O.3(2)
$F_{H,i}$	Transverse force representing a geometrical imperfection	7.2.1.2(5)
F_{td}	Design value of the tension force in a tension chord or tie or in the transverse reinforcement	8.2.3(7), 8.5.5(2), 8.5.3(1), 8.5.5(3)
F_{Vi}	Vertical load to calculate $F_{H,i}$	7.2.1.2(5)
F_{VB}	Buckling load of the bracing structure	7.4.1(3), O.3(1)
F_{VBB}	Flexural buckling load of a cantilever, restricted by the floors, with base rotation	O.3(1)
F_{VBS}	Buckling load due to localised lateral storey deformations	O.3(1)
F_{VEd}	Total design vertical load on the bracing structure and the members braced by it	7.4.1(3)
F_{Rd}	Design value of the resistance of a tie or of a tension chord	8.5.3(1)

G_{cd}	Design value of the elastic shear modulus	O.3(1)
G_k	Characteristic value of a permanent action	10.2(1)
H	Distance between the points of application of two aligned forces	8.5.5(2)
I	Second moment of area of concrete section	13.5.5(2)
I_{cr}	Second moment of area of cracked concrete section	9.3.3(1)
I_g	Second moment of area of the gross concrete cross-section	9.3.3(1)
$J(t,t_0)$	Creep function or creep compliance, representing the total stress-dependent strain per unit stress	B.8(1)
L	Total height of the building above the base	O.3(1)
L_x, L_y	Spans of slab in x- and y-directions	8.4.3(3)
M	Bending moment in linear members	
M_{01}, M_{02}	First order end moments, including effect of imperfections such that $\|M_{02}\| \geq \|M_{01}\|$	14.4.5.2(1), O.6(1)
M_{0Ed}	Maximum first order moment due to the fundamental load combination, including the effect of imperfections	7.4.2(2), O.6(1)
M_{0Eqp}	Maximum first order moment due to the quasi-permanent load combination	7.4.2(2), 14.4.5.2(1)
M_2	Nominal 2nd order moment	O.7.2(1),O.7.2(3)
M_{cr}	Cracking moment of the section in presence of the simultaneous axial force $N_{Ed,}$ which may be calculated on the basis of the concrete tensile strength f_{ctm} assuming linear stress distribution and neglecting any contribution from reinforcement	7.3.2(4), 9.3.4(3), 12.2(2)
M_{Ed}	Design value of the applied internal bending moment	8.1.1, 8.2.1(8), Figure 8.4
M_{Edy}	Design moment about y-axis, including second order moment, where relevant	7.4.4(4)
M_{Edz}	Design moment about z-axis, including second order moment, where relevant	7.4.4(4)
M_{Rd}	Moment capacity	12.2(3)
$M_{Rdy,N}$	Moment resistance about y-axis for the given axial force	8.1.1(8)
$M_{Rdz,N}$	Moment resistance about z-axis for the given axial force	8.1.1(8)
M_{rep}	Cracking moment with extreme fibre tension reaching the relevant tensile strength for sections without prestressing	K.12.2(1)
$M_{R,min}$	Bending strength of the section with $A_{s,min}$ in presence of the simultaneous axial force N_{Ed}	12.2(2)
M_y	Moment when strain in tension reinforcement equals ε_{yd}	7.3.2(5)
ΔM_{Ed}	Reduction in the design support moment for a beam or slab continuous over a support that can be considered to provide no restraint to rotation; additional moment to calculate chord forces	7.2.3(7),8.2.3(12)
N	Axial force in linear members ; number of load cycles	7.2.1.2(6); E.4.2(1)
N^*	Number of load cycles corresponding to $\Delta\sigma_{Rsk}$	E.4.2(1), Figure E.1
N_a	Axial force in column above floor or diaphragm	7.2.1.2(6)
N_B	Elastic buckling load (*Euler*)	O.4(1)
N_b	Axial force in column below floor or diaphragm	7.2.1.2(6)

N_{Ed}	Design value of the applied axial force	7.4.4(4), Figure 7.5, 8.1.1, Figure 8.4
N_{Edw}	Design value of the axial force in the web	8.2.3(11)
N_{obs}	Number of lorries per year	K.10.2.2(2)
N_{Rd}	Design value of axial resistance	14.4.2(3), 14.4.5.2(1)
$N_{Rd,0}$	Design value of axial resistance without accompanying moments	8.1.1(8)
N_{Vd}	Design value of the sum of the additional axial forces in the tension and in the compression chords due to shear in a cross-section	8.2.3(8), 8.2.3(13), Figure 8.9
N_{years}	Design service life of bridge	K.10.2.2(3)
P	Prestressing force	EN 1990
P_d	Design value of the prestressing force	8.2.1(8), Figure 8.4
P_k	Characteristic value of the prestressing force	10.2(1)
$\bar{Q}$	Factor for traffic type	K.10.2.2(2), Table K.1
$Q_{k,i}$	Characteristic variable action	10.2
$Q(\mathrm{t})$	Total amount of hydration heat	D.4.2(1)
R	Resistance	EN 1990
R_{ax}	Restraint factor	9.2.3(4)
$R_{ax,1}$	Restraint factor corresponding to the boundary conditions present after concreting	D.5(4), D.6(3)
$R_{ax,2}$	Restraint factor corresponding to the boundary conditions present when the maximum temperature drop is expected to occur	D.5(4), D.6(3)
$R_{ax,3}$	Restraint factor corresponding to the boundary conditions prevalent during the development of drying shrinkage	D.5(4), D.6(3)
R_{cr}	Cracking risk	D.3(1)
R_d	Design value of the resistance	F.3.(5)
R_e, $R_{p0,2}$	Tensile yield strength of reinforcing steel	C.4.1(2), Table C.1
$R_{ea,cr}$	Project parameter related to the admissible risk of cracking	D.5(2)
RH	Relative humidity of the ambient environment in %	B.5(3), B.6(3)
RH_{eq}	Internal relative humidity of concrete at equilibrium, accounting for self-desiccation in high performance concrete	B.6(3)
R_m	Structural resistance based on a non-linear verification performed using the mean values of the material properties and the nominal geometrical dimensions; tensile strength of reinforcing or prestressing steel	F.5.1(1); C.4.1(2), Table C.2, C.5.1, Tables C.5 to C.7
R_{min}	Minimum radius of curvature of tendons	11.6.3(2)
$R_{p0,1}$	Tensile yield strength of prestressing steel	C.5.1, Tables C.5 to C.7
S	First moment of area above and about the centroidal axis	13.5.5(2)
S_{Ed}	Individual design actions for interaction formula	8.3.6(1)
S_{Rd}	Individual design resistances for interaction formula	8.3.6(1)
S_s	First order moment of area of the required tension and compression reinforcements with respect to the centroid of the gross cross-section	9.3.3(1)
T	Torsional moment; tension force in lapped headed bars	8.3;11.5.5(1), Figure 11.15

T_0	Temperature of the restraining structure	D.3(2)
T_1, T_2	Tension forces in legs of U-bar loops	11.5.4(1), Figure 11.14
T_{ci}	Temperature of fresh concrete	D.3(2)
$T_{c,max}$	Maximum temperature in concrete due to hydration heat	D.3(2)
T_{Ed}	Design value of the applied torsional moment	8.3.2(2)
T_{col}, T_i, T_p, T_v	Tensile force in horizontal ties to columns, internal, peripheral and vertical ties	12.9.2
T_{max}	Maximum temperature during heat treatment	13.4.2.1(1)
ΔT_{min}	Long term maximum temperature drop	D.3(2)
$T_{Rd,c}$	Design value of axial resistance related to concrete failure in laps using U-bar loops	11.5.4(2)
TS_{Mu}	Tension stiffening effect at ultimate limit state	7.3.2(5)
TS_{My}	Tension stiffening effect at yield	7.3.2(5)
V	Shear force in linear members	
V_{bcd}	Design shear force carried by the bottom chord	8.2.1(7), Figure 8.4
V_{Ed}	Design shear force in the section considered	8.2.1(3), Figure 8.4
V_{Edi}	Shear force acting parallel to the interface	8.2.6(4)
Vol	Volume of traffic (tonnes/year/track)	K.10.3.2(5)
ΔV_{Ed}	Portion of shear force which may be subtracted from V_{Ed} due to favorable circumstances	8.2.2(8)
V^*_R	Coefficient of variation of structural resistance	F.5.2(1)
$V_{Rd,hog}$	Resistance provided by hogging reinforcement in flat slabs	12.5.2(3)
$V_{Rd,int}$	Resistance of flat slabs without shear reinforcement for robustness	12.5.2(1)
$V_{Rd,w,int}$	Resistance of flat slabs with shear reinforcement for robustness	12.5.2(2)
V_{tcd}	Design shear force carried by the top chord	8.2.1(7), Figure 8.4

3.2.2 Latin lower case letters

a	Distance; geometrical data; distance from concrete surface to the centre of the outside layer of reinforcement (Figure 9.3)	8.2.3(12), 8.5.5(2)
a_{cs}	Effective shear span with respect to the control section	8.2.2(3)
$a_{cs,0}$	a_{cs} without considering effect of prestressing or external load	8.2.2(5)
a_F	Projection of a foundation from the column face	14.6.3(1)
a_i	Amplitude of buckling shape to be considered as an imperfection for bucking analysis of arches	7.2.1.2(4)
a_l	Distance by which moment curve is shifted to account for shear effect	12.3.2(1)
a_N	Exponent governing the shape of the simplified skew bending interaction diagram	8.1.1(8)
a_p	Distances between the centre of the support area and the point of contraflexure in slabs under concentrated loads	8.4.3(2)

a_{pd}	Parameter for calculating the punching shear resistance based on a_p	8.4.3(2)
$a_{p,x}$, $a_{p,y}$	Maximum distances from the centre of the support area to the two points (on the x- and on the y-axis, respectively) where the bending moments $m_{Ed,x}$, respectively $m_{Ed,y}$, are zero	8.4.3(2)
a_q	Distance between concentrated forces pushing against each other	8.2.1(11); 8.2.2(9), Figure 8.5
$a_{s,min}$	Minimum interface reinforcement along edges of composite slabs	8.2.6(9)
a_v	Mechanical shear span	8.2.2(3)
$a_{v,0}$	a_v without considering effect of prestressing or external load	8.2.2(5)
a_x	Distance from concrete surface to the centre of the outer reinforcement layer in the x direction	9.2.3(3), Figure 9.3
a_y	Distance from concrete surface to the centre of the outer reinforcement layer in the y direction	9.2.3(2)(3), Figure 9.1, Figure 9.3, 11.4.7(1)
a, b	Dimensions of load introduction block	8.6(2), Figure 8.32
a_0, b_0	Dimensions of loaded area A_{c0}	8.6(2), Figure 8.32
$a_{0,red}$, $b_{0,red}$	Reduced dimensions of loaded area due to eccentrically applied load	8.6(2), Figure 8.32
a_1, b_1	Dimensions of contributing area A_{c1}	8.6(2), Figure 8.32
a_h, b_h	Dimensions of rectangular head of headed bars	11.4.7(1), Figure 11.9
a_δ	Deformation parameter considered which may be, for example, a strain, a curvature, or a rotation or even a deflection	9.3.4(3)
a_I, a_{II}	Value of α_δ calculated for uncracked and fully cracked conditions, respectively	9.3.4(3)
b	Overall width of a cross-section, or actual flange width in a T or L beam	6.5.2.3, 7.2.3(3), 7.5(3)
b_0	Length of control perimeter at face of supporting area	8.4.3(1), Figure 8.18
$b_{0,5}$	Length of control perimeter at a distance of $0{,}5d_v$ from column edge	8.4.2(2), Figure 8.18
$b_{0,5,s}$	Control perimeter placed at the location of the shear reinforcement perimeter	8.4.4(4), Figure 8.24
$b_{0,5out}$	b_0 for the verification outside the shear reinforced area	8.4.4(7), Figure 8.24
b_b	Geometric mean of the minimum and maximum overall widths of the control perimeter	8.4.2(6), Figure 8.21
b_c	Width of a strut	8.5.2(2)
b_{cs}	Maximum width of the confined concrete core at confinement reinforcement	8.1.4(3)
b_{csx}, b_{csy}	Value of b_{cs} in the x and y-directions, respectively	8.1.4(3)
b_e	Width of element	13.6.1(4)
b_{eff}	Effective width of flange in T, L or box sections	7.2.3(3)
$b_{ef,hog}$	Effective width for consideration of hogging reinforcement in flat slabs	12.5.2(3)
b_i	Distance between longitudinal reinforcement bars fixed by confinement reinforcement	8.1.4(3), Figure 8.3
b_s	Width of the support strip	8.4.3(1), Figure 8.22, 12.8(2)
b_w	Minimum width of the cross-section between tension and compression chords and neutral axis	Figure 7.3, Figure 8.10

$b_{w,nom}$	Nominal web width due to the disturbance of ducts	8.2.3(10)
c	Concrete cover of reinforcement (to the surface of the bar, $\geq c_{nom}$). In 9.2 it refers to the bar which is closest to the concrete surface	6.5, 9.2.3(6)
$c_{1/r}$	Factor considering curvature distribution	O.7.2(2)
c_{act}	Actual concrete cover	Table 9.2 (NDP)
c_{d}	Nominal value of the concrete cover for designing the anchorage length	11.4.2(3), Figure 11.3 c)
$c_{d,conf}$	Nominal value of c_{d} in presence of confinement	11.4.2(5)
c_{min}	Minimum concrete cover c provided to ensure sufficient bond strength and protection against corrosion	6.5.2.1(1)
$c_{min,b}$	Minimum concrete cover c due to bond requirement	6.5.2.1(1), 6.5.2.3(1)
$c_{min,dur}$	Minimum concrete cover c due to durability requirement	6.5.2.1(1), 6.5.2.2(1)
c_{nom}	Nominal value of the concrete cover c which is specified in drawings and is basis for calculating fire resistance	6.5.1(1)
Δc_{abr}	Additional minimum cover for abrasion	6.5.2.2(6)
Δc_{dev}	Allowance in design for deviation of the concrete cover	6.5.1(1), 6.5.3
$\Delta c_{dur,red1}$, $\Delta c_{dur,red1}$	Reduction of minimum cover for use of additional concrete protection	6.5.2.2(5), 6.5.2.2(9)
Δc	Additional reduction or addition to cover	6.5.2.1(1)
$\Delta c_{min,30}$	Reduction of minimum cover for structures with design life of 30 years	6.5.2.2(2)
$\Delta c_{min,exc}$	Reduction of minimum cover for superior compaction or curing	6.5.2.2(3)
$\Delta c_{min,p}$	Additional minimum cover for prestressing tendons	6.5.2.2(4)
c_{s}	Clear distance between parallel reinforcement bars	11.2(2), Figure 11.3 c)
c_{u}	Effective width of concrete area carrying tensile forces due to the deviation of curved chords	11.7(3)
c_{v}	Parameter to determine $d_{v,out}$	8.4.4(4), Figure 8.24
c_{v1}, c_{v2}	Coefficients for the shear resistance at interfaces	8.2.6(5), Table 8.1, 8.2.6(7)
$c_{v1,fat}$	Coefficients for the shear resistance at interfaces under fatigue action	10.7(2)
c_{x}, c_{y}, c_{yb}	Concrete covers to reinforcement measured in x and y direction and at bends, respectively	Figure 11.3 c)
d	Effective depth of a cross-section	7.3.2, 8.1.1
d_{d}	Design value of the effective depth	4.3.3(2), 8.2.1(4), 8.4.2(1), A.3(6)
d_{dg}	Size parameter describing the crack and the failure zone roughness taking account of concrete type and its aggregate properties	8.2.1(4)
d_{key}	Height of shear key	8.2.6(6), Figure 8.15
d_{nom}	Nominal value of the effective depth determined on the basis of c_{nom}	8.2.2(2), A.3(6)
d_{p}	Effective depth of the prestressed reinforcement	8.2.2(6)
d_{s}	Effective depth of the ordinary reinforcement	8.2.2(6)

d_v	Shear-resisting effective depth of the reinforcement at the first and second shear reinforcement perimeter	8.4.2(1), Figure 8.17, 8.4.2(5)
$d_{v,out}$	d_v outside of the second reinforced perimeter	8.4.4(4), Figure 8.24
d_{vx}, d_{vy}	Shear-resisting effective depth of the reinforcement in x and y direction, respectively	8.4.2(1)Figure 8.17
d_x, d_y	Effective depth of the reinforcement in x and y direction, respectively	8.2.1(5)
e	Eccentricity	14.4.2, Figure 14.1
e_a, e_b	Eccentricity of applied load in partially loaded area	8.6(2), Figure 8.33
e_0	First order eccentricity	14.4.5.2(1)
e_2	Second order eccentricity	O.7.2(3)
e_b	Eccentricity of the resultant of shear forces with respect to the centroid of the control perimeter	8.4.2(6), Figure 8.21
$e_{b,x}$, $e_{b,y}$	Components of e_b in x and y direction, respectively	8.4.2(6), Table 8.3, Figure 8.21
$e_{d,min}$	Minimum eccentricity due to uncertainties related to modelling and analysis	8.1.1(5)
e_i	Additional eccentricity covering the effects of geometrical imperfections	7.2.1.2(5), O.7.2(1)
e_p	Eccentricity of the axial forces related to the centroid of the section at control section, positive when the eccentricity is on the side of the flexural reinforcement in tension	8.2.1(8), Figure 8.4, 8.2.2(5)
e_{tot}	Total eccentricity	14.4.5.2(1)
e_y', e_z'	Dimensionless eccentricity along the y-axis and z-axis, respectively	7.4.4(4)
f_{bcd}	Design resistance of bedding material	12.10(7)
$f_{bm,req}$	Required minimum mean bond strength of post-installed bar	C.8(2), Table C.9
f_c	Compressive strength of concrete	
f_{cd}	Design value of concrete compressive strength	5.1.6(1)
$f_{cd,c}$	Design value of strength of confined concrete	8.1.4(4)
$f_{cd,fat}$	Design value of concrete fatigue strength	10.5(1), E.4.3(1)
$f_{cd,pl}$	Design value of plain concrete compressive strength	14.2(1)
Δf_{cd}	Design value of strength increase due to transverse compressive stress or confinement	8.1.4(2)
f_{ck}	Characteristic concrete cylinder compressive strength at age t_{ref}	5.1.3(1), Table 5.1
$f_{ck,ref}$	Value of f_{ck} at reference age t_{ref}	5.1.6(1)
$f_{ck,28}$	Value of f_{ck} at reference age of 28 days	5.1.5
f_{cm}	Mean concrete cylinder compressive strength at age t_{ref}	Table (5.1)
$f_{cm}(t)$	Mean concrete cylinder compressive strength at age t	B.4(1)
f_{cmp}	Mean compressive strength of concrete after heat curing	13.3.1(1)
f_{ct}	Tensile strength, highest stress reached under concentric tensile loading	
f_{ctd}	Design value of the tensile strength of concrete	5.1.6(2)
$f_{ctd,pl}$	Design value of plain concrete tensile strength	14.2(1)

$f_{ct,eff}$	Mean value of the tensile strength of the concrete effective at the time when cracking may first be expected to occur;	9.1(3), D.3(1)
$f_{ctk;0,05}$	Characteristic axial tensile strength of concrete (5 % fractile)	5.1.3(1), Table (5.1)
$f_{ctk;0,95}$	Characteristic axial tensile strength of concrete (95 % fractile)	5.1.3(1), Table (5.1)
f_{ctm}	Mean axial tensile strength of concrete at age t_{ref}	5.1.3(1), Table (5.1),
$f_{ctm,fl}$	Mean flexural tensile strength of concrete	9.3.4(4)
f_p	Actual tensile strength of prestressing steel	5.3.2(2), B.9
f_{pd}	Design yield strength of prestressing steel	5.3.3(1)
f_{pk}	Characteristic tensile strength of prestressing steel	5.3.2(2), Table 5.6
$f_{p0,1k}$	Characteristic 0,1 % proof-stress of prestressing steel	5.3.2(2), Table 5.6
f_r	Relative flexibility of rotational restraints at the ends of a support	O.3(1), O.5(2)
f_R	Minimum relative rib area of reinforcement	C.4.1(2), Table C.1
f_{Rd}	Design value of bearing strength	12.10(7)
$f_{s,eff}$	Tensile stress in reinforcement when M_{Rd} is reached	7.3.2(5)
f_t	Tensile strength of reinforcement	5.2.2(2)
f_{tdx}, f_{tdy}	Tensile strength provided by reinforcement in membrane elements in x- and y-directions, respectively	G.3(2)
f_y	Yield strength of reinforcement	5.2.2(2)
f_{yd}	Design yield strength of reinforcement	5.2.4(1)
f_{yk}	Characteristic value of yield strength of reinforcement or, if yield phenomenon is not present, the characteristic value of 0,2 % proof strength	5.2.2(2), Table 5.4,
f_{ywd}	Design yield strength of shear reinforcement	8.2.3(5), 8.4.4(1)
$f_{0,2k}$	Characteristic 0,2 % proof-stress of reinforcement	5.2.2(2), Q.3
h	Overall depth of a cross-section or of a part of a cross-section	7.4.4(4), Figure 7.5
h_{col}	Largest side length of column	13.8.3(2)
$h_{c,eff}$	Height of the effective concrete area around reinforcement	9.2.3(3), Figure 9.3
h_D	Hydrostatic head	H.4.2(2)
h_F	Foundation depth;	14.6.3(1)
h_f	Thickness of a flange at the junction with the web	8.2.5(1), Figure 8.13
h_n	Notional size of concrete member	Table 5.2, Table 5.3 (NDP), B.5(3)
i	Radius of gyration	O.5(1)
i_s	Radius of gyration of total group of reinforcement	O.7.3(2)
i_y, i_z	Radius of gyration with respect to y-axis and z-axis, respectively	7.4.4(4)
k	Coefficient; Factor	5.1.6(3)
k	Ratio related to strain hardening of reinforcement	Table 5.5
k_b, $k_{b,simpl}$	Coefficient for bond conditions	9.2.3(6); S.4(1)
k_{bend}	Parameter accounting for the bend angle α_{bend}	11.3(4)
$k_{b,pi}$	Bond efficiency factor	11.4.8(4), C.8(2)

k_c	Coefficient reflecting the extent of cracking and the effect of non-linear material properties in the bracing system	O.3(1)
k_{cip}	Factor considering increased uncertainty and variability in γ_C for concrete geotechnical members	4.3.3(4)
$k_{conf,b}$, $k_{conf,s}$	Effectiveness factor for confinement	8.1.4(4), Table 8.1
k_{cp}	Coefficient accounting for casting effect on bond conditions	11.4.2(3), Figure 11.4
$k_{c,pl}$	Coefficient to determine the design compressive strength of plain concrete	14.2(1)
k_{dc}	Coefficient for moment resistance depending on ductility class of reinforcement	12.2(3)
k_{dowel}	Factor depending on the roughness of the interface	8.2.6(5), 8.2.6(7)
k_{duct}	Coefficient for calculating the nominal web width due to the disturbance of ducts	8.2.3(10)
k_E	an adjusting factor for the modulus of elasticity of concrete considering the type of aggregates.	5.1.4(2)
k_{f1}, k_{f2}	Stess exponent in *S-N* curves	E.4.2(1), Figure E.1, Table E.1(NDP), Table E.2(NDP)
k_{fl}, $k_{fl,simpl}$	Coefficient for type of loading (axial versus flexure)	9.2.3(6), S.4(1)
k_h	Coefficient which allows for the effect of non-uniform self -equilibrating stresses, which lead to a reduction of the apparent tensile strength	9.2.2(2)
k_{int}	Coefficient to calculate resistance for robustness	12.5.2(1)
k_l	Coefficient to accounting for the effect of cracking, for tension stiffening and for the fact that creep deformations are less than proportional to the creep coefficient in cracked sections	9.3.3(1)
k_{lb}	Factor for calculating the design anchorage length	11.4.2(3)
k_{ls}	Factor to increase anchorage to lap length	11.5.2(2)
k_N	Coefficient to consider beneficial effect of eccentric tendons on punching shear resistance in prestressed slabs	8.4.3(4)
k_p	Exponent to model the evolution of relaxation with time	B.9(3)
k_{pb}	Shear gradient enhancement coefficient for punching	8.4.3(1)
k_{pp}	Coefficient accounting for the influence of axial forces on the shear slenderness of slabs submitted to concentrated forces	8.4.3(4)
k_r	In the context of second order effects, correction coefficient for equilibrium curvature depending on axial force	O.7.3(1), O.7.3(3)
k_s	Coefficient accounting for the effect of cracking on the shrinkage deflection	9.3.3(1)
k_{st}	Resistance factor of the transverse reinforcement in laps using U-bar loops and headed bars	11.5.4(2), 11.5.5(4)
k_{surf}	Factor considering effect or ratio of actual cover / minimum cover	9.2.1(6), Table 9.2(NDP)
k_t	Coefficient accounting for the effect of the nature and duration of the load on tension stiffening effects for cracking	9.2.3(3)
k_{tc}	Coefficient considering the effect of high sustained loads on concrete compressive strength	5.1.6(1)
k_{Temp}	Coefficient accounting for the reduction in temperature from t_1 to t_2	D.5(4)

$k_{t,pl}$	Coefficient to determine the design tensile strength of plain concrete	14.2(1)
k_{tt}	Coefficient considering the effect of high sustained loads on concrete tensile strength	5.1.6(2)
k_v	Factor depending on roughness of interface	8.2.6(7), Table 8.2
k_{vp}	Coefficient considering effect of axial force	8.2.2(4)
k_w	Factor converting the mean crack width into a design crack width	9.2.3(2)
k_1	Factor considering effect of axial force and eccentricity	8.2.2(5)
$k_{1/r}$ $k_{1/r,simpl}$	Coefficient accounting for increase of crack width due to curvature	9.2.3(2), S.3(1)
k_θ	Sum of rotational restraint stiffnesses at the base of the bracing members	O.3(1)
k_μ	Unintentional angular displacement for internal tendons (per unit length)	7.6.3.2(1)
k_σ	Ratio of concrete stress to $0{,}4f_{cm}$, used to account for non-linear creep	B.5(6)
$k_{\sigma 1}$, $k_{\sigma 2}$	Coefficient for determining $\sigma_{s,lim}$	S.3(1)
k_φ	In the context of second order effects, correction coefficient for equilibrium curvature accounting for creep	O.7.3(1), O.7.3(4)
l (or L)	Length; span	
l_0	Effective length of the member	7.2.1.2(5), 14.4.5.1(1), O.4(1)
l_{0b}	Distance l_{0b} between points of zero moment	7.2.3(3)
l_{0t}	Distance between torsional restraints	7.5(3)
l_{aw}	Half wavelength of the buckling mode with the lowest buckling load	7.2.1.2(4)
l_{bd}	Design value of anchorage length of reinforcing steel	11.4.2(3)
l_{bdn}	Part of the anchorage length in the nodal region	8.5.4.3(4), Figure 8.28 c),
$l_{bd,pi}$	Design anchorage length of post-installed reinforcing steel	11.4.8(4)
$l_{bd,tot}$	Design anchorage length measured along the centre line of bars with bends and hooks in tension	11.4.4(2)
l_{bpd}	Anchorage length of pretensioning tendon	13.5.2, Figure 13.3
l_{disp}	Dispersion length of effect of pretensioning tendon	13.5.2, Figure 13.2
l_{pt}	Transmission length of pretensioning tendon	13.5.2, Figure 13.2
l_s	Actual lap length	11.7(4),
l_{sd}	Design value of lap length of reinforcing steel	11.5.2(2)
l_w	Clear height of the member	14.4.5.1(1)
$l_{w,p}$	Length of passing crack	H.4.2(7)
m	Number of load bearing members in one storey that bear a significant part of the vertical load	7.2.1.2(2)
m_{Ed}	Design value of the applied internal bending moment per unit width in planar members	8.1.3(1), 8.2.4(2), Figure 8.12
m_{Edx}, m_{Edy}	Value of m_{Ed} in the x and y-directions, respectively	8.1.3
m_{Edxy}	Design value of the applied internal torsion moment per unit width in planar members	8.1.3(1)

m_{Rd}	Design value of the flexural strength per unit width of a planar member resisting positive moments	8.1.3(1), 8.2.4(2), Figure 8.12
m_{Rd}'	Design value of the flexural strength per unit width of a planar member resisting negative moments (value positive)	8.1.3(1)
m_{Rdx}, m_{Rdy}	Value of m_{Rd} in the x and y-directions, respectively (without the influence of torsion moments)	8.1.3(1)
m_{Rdx}', m_{Rdy}'	Value of m_{Rd}' in the x and y-directions, respectively (without the influence of torsion moments)	8.1.3(1)
n	Non-dimensional axial force	O.7.3(3)
n_b	Number of anchored bars or paris of lapped bars in the potential splitting failure surface	11.4.2(5)
n_c	Number of legs of confinement reinforcement crossing the potential splitting failure surface	11.4.2(5)
n_i	Number of acting stress cycles for stress level i	E.5.1(1)
n_s	Number of storeys	O.3(1)
n_{st}	Proportion of traffic crossing bridge simultaneously	K.10.2.3(5)
n_{trans}	Number of transverse bars in the bend	11.3(5)
n_1	Number of layers with bars anchored at the same point in the member	11.4.2(7)
n_2	Number of bars anchored in each layer	11.4.2(7)
n_σ	Exponent to consider effect of steel stress on anchorage length	11.4.2(3)
p	Water pressure	H.4.2(3)
p_{Rd}	Maximum transverse bearing stress on tendon	11.6.3(2)
Δp	Water pressure difference between the ends of a passing crack	H.4.2(7)
q	Leakage rate through cracks	H.4.2(7)
q_d	Distributed load	8.2.3(12)
q_{ed}	Design value of variable load	13.6.1(4)
r	Radius of curvature	11.7(3), Figure 11.17
r_{inf}	Factor to account for variation in prestress in serviceability and fatigue verifications when prestressing is favourable	4.2.1.5(3)
r_m	Ratio of end moments $\|r_m\| \leq 1{,}0$	O.6(1)
r_{sup}	Factor to account for variation in prestress in serviceability and fatigue verifications when prestressing is unfavourable	4.2.1.5(3)
$1/r$	Curvature at a particular section	9.3.3(1), O.7.2, O.7.3
s	Spacing of the shear reinforcement or confinement reinforcement measured along the longitudinal axis of the member	8.1.4(3), 8.2.3(5), Figure 8.9, 8.3.4(2),
s_c	Spacing of the confinement reinforcement along the bar to be anchored	11.4.2(5)
s_f	Spacing of transverse reinforcement	8.2.5(2); Figure 8.13
s_0	Distance from the column face to the axis of the first perimeter of punching reinforcement	8.4.4(1), Figure 8.23
s_1	Spacing between shear links in radial direction	8.4.4(1), Figure 8.23
s_C	Coefficient for different early strength development of concrete and concrete strength	B.4(1), Table B.2

s_L	Spacing of longitudinal ribs	13.6.1(7)
s_T	Spacing of transverse ribs	13.6.1(7)
s_l	Centre-to-centre spacing of longitudinal bars	9.2.3(1)
$s_{l,surf,max}$	Maximum spacing of surface reinforcement in beams with downstand	12.3.1(1)
$s_{bu,max}$	Maximum longitudinal spacing of bent-up bars	12.3.1(1)
$s_{max,col}$	Maximum spacing of transverse reinforcement along the column	12.6(1)
$s_{l,max}$	Maximum longitudinal spacing of shear assemblies/stirrups	12.3.1(1), 12.4.1(1)
$s_{slab,max}$	Maximum spacing of bars for slabs	12.4.1(1)
$s_{stir,max}$	Maximum spacing for torsion assemblies / stirrups	12.3.1(1)
$s_{tr,max}$	Maximum transverse spacing of shear legs	12.3.1(1), 12.4.1(1)
s_r	Spacing of shear links in the radial direction	8.4.4(1)
$s_{r,m,cal}$	Calculated mean crack spacing when all cracks have formed or where not all cracks have formed, the maximum length along which there is slip between concrete and steel adjacent to a crack	9.2.3(2), 9.2.3(6)
$s_{r,m,cal,x}$, $s_{r,m,cal,y}$	Calculated mean crack spacing in the *x* direction and in the *y* direction, respectively	G.5
s_t	Average spacing of shear links in the tangential direction at the investigated control perimeter; spacing of transverse reinforcement along the bar to be anchored	8.4.4(1); 11.4.2(5)
s_x	Bar spacing in a group of headed bars	11.4.7(1), Figure 11.9
t	Thickness of a strut; length over which the support reaction is distributed	8.5.2(2); 7.2.3(7)
t	Time being considered, age of the concrete	5.1.3
Δt	Time interval	13.4.2.1(1), B.5(5)
t_{eff}	Effective wall thickness	8.3.2(2), Figure 8.16
t_{eq}	Equivalent time to consider effect of heat treatment loss of prestress	13.4.2.1(2)
t_{fac}	Tensile force in horizontal ties to walls	12.9.2, 12.9.3
t_p	Duration of heat curing	13.3.1
t_0	Age of concrete when the event under consideration occurs (prestressing, settlement, start of drying, loading age)	5.1.5, Table 5.2, B.5
$t_{0,adj}$	Concrete age at loading adjusted for strength class of cement and temperature	B.5(3)
$t_{0,T}$	Temperature-adjusted concrete age at loading	B.5(4)
t_1	Time when the maximum concrete temperature due to heat hydration is reached	D.3(4), Figure D1, Figure D.2
t_2	Time when concrete starts to develop tensile stresses	D.3(4), Figure D1, Figure D.2
t_c	Age of concrete when support conditions change	7.3.1(7)
t_{crit}	Critical time for early-age cracking. Time at which thermal equilibrium with the restraining structure is achieved (within 2 °C) and the greater part of basic shrinkage has already developed	D.3(3), Figure D1, Figure D.2
t_{dor}	Dormant time, e.g. time from concreting until stresses begin to develop	D.4.1(1), Figure D.1, Figure D.2

t_{ref}	Age of concrete at which the concrete strength is determined in days	5.1.3(2)
t_s	Age of concrete at the beginning of drying	B.6(1)
t_T	Temperature-adjusted concrete age in days	B.5(5)
u	Perimeter of concrete cross-section, having area A_c	Table 5.2, 8.3.2(2), B.5(3)
v_{Ed}	Principal out of plane shear force per unit width in planar members	8.2.1(3), 8.2.1(5)
$v_{Ed,x}$, $v_{Ed,y}$	Out of plane shear force in planar members on the cross-sections perpendicular to the x and y direction, respectively	8.2.1(5)
$w_{k,cal}$	Calculated crack width	9.2.3(2)
$w_{k,cal,1}$	Crack width at end of a through-crack, where the crack is wider	H.4.2(7)
$w_{k,cal,2}$	Crack width at end of a through-crack, where the crack is thinner	H.4.2(7)
$w_{k,cal,e}$	Equivalent width of a passing crack, of variable width	H.4.2(7)
$w_{k,lim,1}$	Limiting crack width for water-tightness	H.4.2(1)
$w_{lim,cal}$	Limiting crack width to be compared with the calculated crack width $w_{k,cal}$	9.2.1(6), Table 9.1 (NDP), Table 9.2 (NDP)
x	Depth of the neutral axis at serviceability limit state	7.3.2(5), 9.2.2(4), 9.2.3(2)
Δx	Length under consideration for shear transfer	8.2.5(1), Figure 8.13
x_{cs}	Distance between the neutral axis and the axis of confinement reinforcement	8.1.4(3), Figure 8.3
x_g	Depth of neutral axis of uncracked section	9.2.3(6)
x, y, z	Coordinates	
x_{min}	Minimum depth of the compression zone to guarantee water-tightness in elements subjected to flexure	H.4.2(4)
x_{sb}	Depth of the compression zone assuming a stress block	Figure 8.2, 8.2.3(9), Figure 8.10
x_u	Depth of the neutral axis at ultimate limit state	7.3.2(3), 8.1.2
z	Inner lever arm of internal forces for shear design	8.2.1(3), Figure 8.9
z_{cp}	Distance between the centroid of the concrete section and the tendons	7.6.4(2)
z_{red}	Reduced depth of section at segment joint due to joint opening	K.13(4)

3.2.3 Greek letters

α	Angle; ratio	
α	Inclination of reinforcement across interface	8.2.6(5), Figure 8.15
α_{bend}	Bend angle	11.3(4), 11.4.4, Figure 11.6
α_{bs}	Coefficient accounting for the effect of the strength class of cement on basic shrinkage	B.6(2), Table B.3
α_c	Function to determine tangent modulus of elasticity of concrete	B.5(7)
$\alpha_{c,th}$	Coefficient of thermal expansion of concrete	5.1.6(6)

α_{ds}	Coefficients accounting for the effect of the strength class of cement on drying shrinkage	B.6(3)
α_{e}	Modular ratio, $\alpha_{e} = E_{s}/E_{c}$	9.2.3(3)
$\alpha_{e,ef}$	Modular ratio, $\alpha_{e} = E_{s}/E_{c,eff}$	9.3.3(1)
α_{fcm}	Coefficient accounting for the effect of concrete strength on time evolution of drying creep	B.5(3)
α_{h}	Reduction coefficient for length or height	7.2.1.2(2)
α_{lb}	Factor accounting for cracks along post-installed bar	11.4.8(4)
α_{m}	Reduction coefficient for number of members	7.2.1.2(2)
$\alpha_{NDP,b}$, $\alpha_{NDP,d}$	Coefficient to determine basic shrinkage and drying shrinkage, respectively	B.6(1)
α_{R}	Sensitivity factor for the reliability of the resistance	F.5.2(1)
α_{RA}	Substitution rate of recycled concrete aggregates	N.3(3)
α_{SC}	Exponent accounting for the strength class of cement on the adjusted loading age	B.5(4)
$\alpha_{s,th}$	Coefficient of thermal expansion of reinforcement	5.2.4(5), 5.3.3(5)
α_{v}	Angle between the principal shear force and the x-axis	8.2.1(5)
α_{w}	Angle between shear reinforcement and the member axis perpendicular to the shear force	Figure 8.11 b); 8.2.3(13); 8.4.4(2)
α_{1}, α_{2}	Coefficients for determination of transmission length	13.5.3(1)
α_{3}	Coefficient for determination of anchorage length considering type of verification (fatigue or other)	13.5.4(2)
α_{μ}	Sum of the absolute values of angular displacements over a distance for the calculation of prestressing losses due to friction	7.6.3.2(1)
β_{tgt}	Target value of reliability index	F.5.2(1)
β_{new}	Ratio of longitudinal force in new concrete due to composite action	8.2.6(4)
β_{incl}	Angle of inclined cross-sections for determining the shear resistance in case of direct strutting in deep beams or in presence of concentrated loads near to the support	8.2.3(12), Figure 8.11 a)
$\beta_{bc,fcm}$	Coefficient accounting for the effect of concrete strength on the basic creep coefficient	B.5(2)
$\beta_{bc,t-t0}$	Coefficient describing the evolution with time of basic creep and accounting for age of loading	B.5(2)
$\beta_{bs,t}$	Coefficient describing the evolution with time of basic shrinkage	B.6(1)
β_{c}	Coefficient which depends on the distribution of 1st and 2nd order moments	O.8.2(1)
$\beta_{cc}(t)$	Coefficient for determining the compressive concrete strength which depends on the age of the concrete t	B.4(1), D.5(3)
$\beta_{dc,fcm}$	Coefficient accounting for the effect of concrete strength on the drying creep coefficient	B.5(3)
$\beta_{dc,RH}$	Coefficient accounting for the effect of relative humidity on the drying creep coefficient	B.5(3)
$\beta_{dc,to}$	Coefficient accounting for the effect of age of loading on the drying creep coefficient	B.5(3)

$\beta_{dc,t-t0}$	Coefficient describing the evolution with time of drying creep and accounting for the effect of notional size and age at loading	B.5(3)
$\beta_{ds,t-ts}$	Coefficient describing the evolution with time of drying shrinkage and accounting for the effect of notional size	B.6(1)
β_{e}	Coefficient accounting for concentrations of the shear forces along a control perimeter	8.4.2(6), Table 8.3
β_{Eul}	Euler coefficient	14.4.5.1(1)
β_{fck}	Coefficient accounting for concrete strength and slenderness, used in the nominal curvature method	O.7.3(4)
β_{h}	Coefficient accounting for the effect of notional size and concrete strength on the time development of drying creep	B.5(3)
β_{p}	Angle between the tendon and the axis of the member, for the sign, the angle indicated in Figure 8.4 is positive	8.2.1(8), Figure 8.4
β_{RH}	Coefficient accounting for the effect of relative humidity on drying shrinkage	B.6(1)
β_{t}	Coefficient to account for duration of loading or of repeated loading on average strain	9.3.4(3)
γ	Partial factor (safety and serviceability)	EN 1990
$\gamma(t_{0,adj})$	Exponent accounting for the influence of age of loading in the time development of drying creep	B.5(3)
γ_{C}	Partial factor for concrete	4.3.3(1), Table 4.3 (NDP)
γ_{CE}	Partial factor for the modulus of elasticity of concrete	4.3.3(1), Table 4.3 (NDP), 7.4.3.3(3)
γ_{F}	Partial factor for actions, also accounting for model uncertainties and dimensional variations	EN 1990
γ_{Ff}	Partial factor for fatigue actions	E.4.2(1)
γ_{m}	Partial factor for a material property	EN 1990
γ_{P}	Partial factor for prestressing actions P	4.3.2(1), Table 4.2 (NDP)
$\gamma_{P,fav}$, $\gamma_{P,unfav}$	Partial factor for favourable and unfavourable prestress effects	4.3.2(1), Table 4.2 (NDP)
$\gamma_{\Delta P,sup}$, $\gamma_{\Delta P,inf}$	Partial factors for change in stress in unbonded prestressing tendons associated with deformation of the member	4.3.2(2), Table 4.2 (NDP)
γ_{Q}	Partial factor for variable actions Q; also accounting for model uncertainties and dimensional variations	EN 1990
γ_{R*}	Global resistance factor accounting for the uncertainties related to the material properties, geometrical dimensions and the resisting model	F.5.1(1)
γ_{Rd}	Partial factor associated with the uncertainty of the resistance model	F.4.1(1)
γ_{S}	Partial factor for reinforcing or prestressing steel	4.3.3(1), Table 4.3 (NDP)
γ_{sh}	Partial factor for shrinkage action	4.3.1(1)
γ_{V}	Partial factor for shear and punching resistance without shear reinforcement	4.3.3(1), Table 4.3 (NDP)

γ_{water}	Specific weight of water	H.4.2(3)
γ_0	Partial factor for model uncertainty	7.3.2(5)
δ	Deflection	9.3.3(1)
$\delta_{0,Eqp}$	Maximum short-term horizontal deflection due to the quasi permanent load combination determined assuming uncracked cross-sections	7.4.2(2)
δ_{Ed}	Maximum short-term horizontal deflection due to the fundamental load combinations from a first order analysis determined assuming uncracked cross-sections	7.4.2(2)
δ_{loads}	Linear elastic deflection due to the relevant combination of actions	9.3.3(1)
δ_M	Ratio of the redistributed moment to the elastic bending moment	7.3.2(3)
$\delta_{\varepsilon cs}$	Linear elastic deflection due to shrinkage	9.3.3(1)
ε_c	Compressive strain in the concrete	Figure 5.1, 5.1.6(3), 8.1.2(1)
ε_{c1}	Strain at maximum stress	Figure 5.1, 5.1.6(3)
ε_{c2}	Compressive strain in the concrete at the peak stress f_c	8.1.2(1)
$\varepsilon_{c2,c}$	Value of ε_{c2} in case of confined concrete	8.1.4(5)
ε_{cbs}	Basic shrinkage strain	B.6(1)
$\varepsilon_{cbs,fcm}$	Notional basic shrinkage coefficient, accounting for the effect of concrete strength and strength class of cement on basic shrinkage	B.6(1)
$\varepsilon_{ci}(t_0)$	Elastic strain due to a constant stress $\sigma_c(t_0)$ applied at time t_0	B.8(1)
$\varepsilon_{cc}(t,t_0)$	Creep strain due to a constant stress $\sigma_c(t_0)$ applied at time t_0	5.1.5(1), B.8(1)
$\varepsilon_{cds,fcm}$	Notional drying shrinkage coefficient, accounting for the effect of concrete strength and strength class of cement on drying shrinkage	B.6(1)
ε_{cm}	Mean strain in the concrete between cracks at the same level of ε_{sm}	9.2.3(2)
ε_{cs}	Shrinkage strain	Table 5.3 (NDP), B.6(1)
ε_{cu}	Ultimate compressive strain in the concrete	7.3.2, 8.1.2(1), Figure 8.2
$\varepsilon_{cu,c}$	Value of ε_{cu} in case of confined concrete	8.1.4(5)
ε_{cds}	Drying shrinkage strain	B.6(1)
$\varepsilon_{c\sigma}(t,t_0)$	Time-dependent strain due to a constant stress $\sigma_c(t_0)$ applied at time t_0	B.8(1)
$\varepsilon_{c\sigma}(t,\sigma_c)$	Time-dependent strain due to a stress history $\sigma_c(t)$	B.8(1)
ε_{free}	Imposed strain	9.2.3(4), D.6(3)
ε_{imp}	Imposed strain in element	9.2.3(4)
ε_p	Strain in the prestressing steel	Figure 5.3, Figure 8.1
$\varepsilon_{p(0)}$	Strain difference between prestressing steel and surrounding concrete	Figure 8.1
$\Delta\varepsilon_p$	Strain increase in the prestressing steel	Figure 8.1
ε_{restr}	Strain developing in restrained element	9.2.3(4)
ε_s	Strain in reinforcing steel	Figure 5.2, Figure 8.1

ε_{sm}	Mean strain in the reinforcement closest to the most tensioned concrete surface under the relevant combination of actions, including the effect of imposed deformations and taking into account the effects of tension stiffening. Only the additional tensile strain beyond the state of zero strain of the concrete at the same level is considered	9.2.3(2)
ε_{ud}	Design strain of reinforcing and prestressing steel at maximum load	5.2.4(2), 5.3.3(2)
ε_{uk}	Characteristic strain of reinforcement or prestressing steel at maximum load	5.2.2(2), Table 5.5, 5.3.2, Table 5.6
ε_{yd}	Design yield strain of reinforcement	7.3.2(5), O.7.3(1)
ε_{x}	Average strain in x-direction, of the flexural chords under tension and compression	8.2.3(7)
ε_{xt}, ε_{xc}	Strain in flexural chords under tension and compression	8.2.3(7)
ε_{1}	Value of the maximum principal tensile strain in concrete	8.5.2(5), G.3(6)
ε_{2}	Value of maximum principal compressive strain in membrane element	G.5(3)
ζ	Distribution coefficient allowing for tension stiffening at a section	9.3.4(3)
ζ_{f}	Reduction factor for fatigue strength of bent bar	E.4.2(1), Table E.1(NDP)
η	Ratio of strains used to define stress strain model	5.1.6(3)
η_{1}	Coefficient for determination of transmission length of pretensioning tendon accounting for position during concreting	13.5.3(1)
η_{c}	Strength reduction coefficient for shear resistance $\tau_{Rd,c}$	8.4.4(1)
η_{cc}	Factor to account for the difference between the undisturbed compressive strength of a cylinder and the effective compressive strength that can be developed in the structural component	5.1.6(1)
$\eta_{cc,fat}$	Value of η_{cc} for fatigue actions	10.5(1)
$\eta_{lw,Ec}$	Coefficient related to f_{c} in lightweight concrete	M.3(1), Table M.1
$\eta_{lw,fc}$	Coefficient related to f_{c} in lightweight concrete	M.3(1), Table M.1
$\eta_{lw,fct}$	Coefficient related to f_{ct} in lightweight concrete	M.3(1), Table M.1
η_{pb}	Coefficient accounting for the influence of membrane forces due to restrained deformations on the shear slenderness of slabs submitted to concentrated forces for new structures	8.4.4(6)
η_{pm}	Coefficient accounting for the influence of membrane forces due to restrained deformations on the shear slenderness of slabs submitted to concentrated forces for existing structures	8.4.4(5)
η_{s}	Strength reduction coefficient for the contribution of the shear reinforcement	8.4.4(1)
η_{sys}	Coefficient accounting for the performance of punching shear reinforcing systems	8.4.4(5)
η_{visc}	Dynamic viscosity	H.4.2(7)
θ	Angle between the compression field and the member axis; rotation under bending moment	Figure 8.9, Figure 8.11, 8.2.3(4); O.3(1)
θ_{cf}	Spreading angle of a concentrated force	8.5.5(2), Figure 8.30, 8.5.5(3), Figure 8.31

θ_{cs}	Angle between the compression field and a tie	8.5.2(4), Figure 8.26, Figure 8.28, Figure 8.29
θ_{f}	Angle between the compression field in a flange and the longitudinal axis	8.2.5(3), Figure 8.13
θ_{fat}	Angle between the compression field and the member axis under fatigue actions	10.3(4)
θ_{i}	Inclination representing a geometrical imperfection	7.2.1.2(2)
θ_{min}	Minimum allowed value of θ	8.2.3(4), 8.2.3(5)
Θ_{Rd}	Rotation capacity	7.3.2(5)
Θ_{Ed}	Rotation demand	7.3.2(5)
λ	Slenderness ratio l_0/i	14.4.5.1(5), O.5(1)
λ_{c}	Correction factor to calculate upper and lower stresses of damage equivalent stress spectrum caused by LM71	K.10.3.2(1)
$\lambda_{c,0}$, $\lambda_{c,1}$, $\lambda_{c,2,3}$, $\lambda_{c,4}$	Factor accounting for permanent stress, member type, traffic volume and design service life, number of loaded tracks	K.10.3.2(2)
λ_{lim}	Limiting slenderness for isolated members below which second order effects may be neglected	O.6(1)
λ_{s}	Damage equivalent factor for fatigue	K.10.2.1(2)
$\lambda_{s,1}$, $\lambda_{s,2}$, $\lambda_{s,3}$, $\lambda_{s,4}$	Factor accounting for member type, traffic volume, design service life, number of loaded lanes/tracks	K.10.2.1(2)
λ_{y}	Slenderness ratio, l_0/i_y with respect to the y-axis	7.4.4(4)
λ_{z}	Slenderness ratio, l_0/i_z with respect to the z-axis	7.4.4(4)
μ	Coefficient of friction between the tendons and their ducts	7.6.3.2(1), Table 7.2
μ_{v}	Coefficient of friction at concrete interfaces	8.2.6(5), Table 8.1, 8.2.6(7)
$\mu_{v,fat}$	Coefficient of friction at concrete interfaces for fatigue action	10.7(2)
ν	Strength reduction factor for concrete cracked due shear or other actions	8.2.3(6) (7), 8.2.5(4), 8.3.4(3), 8.5.2(4), G.3(6),
ν_{part}	Confinement factor of partially loaded area; factor for capacity of headed bars	8.6(2); 11.4.7(2)
ξ	Ratio of bond strength of prestressing and reinforcing steel	9.2.2(3), 10.3(2), Table 10.1
ξ_{1}	Adjusted ratio of bond strength taking into account the different diameters of prestressing and reinforcing steel	9.2.2(3)
ξ_{bc1}	Adjustment parameter for basic creep to account for test results	B.7(5)
ξ_{bc2}	Adjustment parameter for time development function of basic creep to account for test results	B.7(5)
ξ_{bs1}	Adjustment parameter for basic shrinkage to account for test results	B.7(5)
ξ_{bs2}	Adjustment parameter for time development function of basic shrinkage to account for test results	B.7(5)
ξ_{dc1}	Adjustment parameter for drying creep to account for test results	B.7(5)
ξ_{dc2}	Adjustment parameter for time development function of drying creep to account for test results	B.7(5)

ξ_{ds1}	Adjustment parameter for drying shrinkage to account for test results	B.7(5)
ξ_{ds2}	Adjustment parameter for time development function of drying shrinkage to account for test results	B.7(5)
ξ_{v}	Effective damping ratio (vibrations)	9.4(4)
$\xi_{v,st}$	Structural component of effective damping ratio (vibrations)	9.4(4)
ρ	Reinforcement ratio	
ρ_{c}	Oven-dry density of concrete in kg/m³	M.3(1), Table M.1
ρ_{conf}	Ratio of the reinforcement providing confinement referred to the diameter of the bar to be anchored or spliced	11.4.2(5)
ρ_{i}	Ratio of bonded reinforcement across interface	8.2.6(5)
ρ_{l}	Reinforcement ratio for bonded longitudinal reinforcement in the tensile zone due to bending referred to the nominal concrete area $d \cdot b_{w}$	8.2.2(2), 8.2.2(6), 8.2.2(7), 8.4.3(1)
$\rho_{l,x}$, $\rho_{l,y}$	Value of ρ_{l} in x- and y-directions, respectively	8.2.2(7)
ρ_{min}	Minimum reinforcement ratio	
ρ_{p}	Tensile reinforcement ratio accounting for the different bond properties of reinforcing bars and prestressing tendons	S.4(1)
$\rho_{p,eff}$	Tensile reinforcement ratio accounting for the different bond properties of reinforcing bars and prestressing tendons, referred to the effective concrete area	9.2.3(3)
ρ_{w}	Shear reinforcement ratio	8.2.3(5), 8.4.4(1)
$\rho_{w,min}$	Minimum shear reinforcement ratio	12.2(4)
$\rho_{w,stir}$	Minimum ratio of shear and torsion reinforcement in the form of stirrups	12.3.1(1), Table 12.1 (NDP)
ρ_{x}, ρ_{y}	Reinforcement ratio in x- and y-directions, respectively	G.3(2)
ρ_{1000}	Value of relaxation loss (in %), at 1 000 hours after tensioning and at a mean temperature of 20 °C	B.9(1) (2), Table B.4
σ_{1Ed}	Design value of principal tensile stress in uncracked concrete in pretensioned member	13.5.5
σ_{a}	Stress range (2 σ_{a})	C.4.1, C.4.2
σ_{c}	Compressive stress in the concrete	5.1.5(1), 5.1.6(3)
σ_{cable}	Tensile stress in stay and extradosed cable	K.12.4(3)
σ_{cd}	Design value of compressive stress in the concrete	8.1.2(1), 8.2.3(5), 8.2.3(13), 8.2.5(4), 8.5.2(2), 10.5(1)
$\sigma_{cd,max,equ}$, $\sigma_{cd,min,equ}$	Upper and lower stress of damage equivalent stress amplitude for N=10⁶cycles, respectively	E.4.3(1)
$\sigma_{cd,max,i}$, $\sigma_{cd,min,i}$	Maximum and minimum compressive stress in stress level i	E.5.3(1)
σ_{cp}	Axial stress	8.2.2(5), 14.4.3(3)
$\sigma_{cp,QP}$	Stress in the concrete adjacent to the tendons, due to self-weight and initial prestress and other quasi-permanent actions where relevant.	7.6.4(2)
σ_{ct}	Tensile stress in concrete	D.3(1)

σ_{c2d}	Design value of the transverse stress in concrete due to confinement or minimum transverse compression stress due to external actions (compression positive)	8.1.4(2), 8.1.4(3)
$\sigma_{c,lim}$	Limited compressive stress for shear strength in plain concrete	14.4.3(3)
σ_{ccd}	Design value of the mean compression stress perpendicular to a free surface near bars to be anchored or spliced	11.4.2(5)
σ_{d}	Design value of the average stress, tension positive	8.4.3(4)
σ_{Edx}, σ_{Edy}, τ_{Edxy}	Membrane stresses	G.3(1), Figure G.1
σ_{gd}	Design value of ground pressure	14.6.3(1)
σ_{n}	Compressive stress across interface	8.2.6(5), Figure 8.15
σ_{p}	Stress in prestressing steel	5.3.3, Figure 5.3
$\Delta\sigma_{p}$	Stress variation in prestressing tendon from state of zero strain	9.2.2(3)
$\sigma_{p,max}$	Maximum prestressing stress imposed at the active end by the jack	7.6.2(1), Table 7.1, 7.6.3.2(1)
σ_{pd}	Design value of the stress in the tendon	7.6.5(2), 11.6.3
σ_{pi}	Initial stress in prestressing steel	B.9(2) (3)
$\sigma_{pk,inf}$	Lower characteristic value of prestress	4.2.1.5(3)
$\sigma_{pk,sup}$	Upper characteristic value of prestress	4.2.1.5(3)
$\sigma_{p,m}$ (x,t)	Mean value of the prestressing stress after accounting for the immediate losses and the time-dependent losses at time *t* and a distance *x* from the active end	4.2.1.5(3), 7.6.2, Table 7.1, 7.6.2(3)
σ_{pm0}	Tendon stress immediately after release	13.5.3(1)
$\sigma_{pm,\infty}$	Long-term stress level in prestressing tendons at the state of zero (elastic) strain of the concrete at the same level	7.3.2(3)
σ_{Rdu}	Design resistance of partially loaded area	8.6(2)
σ_{s}	Serviceability value of steel stress, determined on the basis of a cracked section	5.2.4, Figure 5.2, 9.2.3(3)
σ_{sd}	Design value of the reinforcing steel stress at the cross-section	Figure 11.2, 11.4, 11.5,
σ_{sd}'	Maximum tensile stress in the reinforcing steel of headed bars developed by the head	11.4.7(2), Figure 11.9
$\sigma_{s,lim}$	Limiting value of the serviceability steel stress in order to comply with a given limiting crack width	S.3(1)
σ_{sr}	Stress in the tension reinforcement calculated on the basis of a cracked section under the loading conditions causing first cracking	9.3.4(3)
σ_{swd}	Design value of the stress in the shear reinforcement	8.2.3(12), 8.2.3(13)
σ_{uk}	Characteristic breaking strength of stay and extradosed cable	K.12.4(3)
$\Delta\sigma_{freq}$	Variation of tensile stress in stay or extradosed cable under frequent traffic loads	K.12.4(3)
$\Delta\sigma_{p,c+s+r}$	Time dependent losses of prestress	7.6.2(3), 7.6.4(2)
$\Delta\sigma_{pd}$	Design value of stress increase in tendon; design value of stress range in prestressing steel under fatigue load combination	7.6.5(4); 10.4(1)
$\Delta\sigma_{pr}$	Absolute value of the variation of stress in the tendons at location *x*, at time *t*, due to the relaxation of the prestressing steel	7.6.3.1(1), 7.6.4(2), B.9

$\Delta\sigma_{p,ULS}$	Increase of the stress from the effective prestress to the stress in the ultimate limit state for prestressed members with permanently unbonded tendons	7.6.5(3)
$\Delta\sigma_{p,\mu}$	Prestressing losses due to friction	7.6.3.2(1),
$\Delta\sigma_{Rsk}$	Stress range resistance at N^* cycles from relevant S-N curve	10.7(4), E.4.2(1), Table E.1 (NDP), Table E.2 (NDP)
$\Delta\sigma_{sd}$	Design value of stress range in reinforcing steel under fatigue load combination	10.4(1)
$\Delta\sigma_{s,max}$	Maximum reinforcing steel stress range under relevant load combination	E.4.2(1)
$\Delta\sigma_{s,equ}$	Damage equivalent stress range for reinforcement	E.4.2(1)
$\Delta\sigma_{\theta}$	Thermal loss induced by heat treatment	13.4.2.2(1)
τ_{cp}	Shear stress in the concrete from acting shear force	14.4.3(3)
τ_{Ed}	Average acting shear stress over a cross-section	8.2.1(3), 8.4.2(6)
$\tau_{Ed,i}$	Design value of the shear stress at interfaces	8.2.6(3), 8.2.6(4)
τ_{Rd}	Shear resistance governed either by yielding of shear reinforcement or crushing of concrete	8.2.1(1), 8.2.3(5) 8.2.4(1)
$\tau_{Rd,c}$	Shear stress resistance of members without shear reinforcement (average shear stress over a cross-section)	8.2.2(2), 8.2.2(5), 8.4.3(1)
$\tau_{Rdc,min}$	Minimum shear stress resistance allowing to avoid a detailed verification for shear (average shear stress over a cross-section)	8.2.1(4)
$\tau_{Rd,cs}$	Shear stress resistance of planar members with shear reinforcement subjected to concentrated forces	8.4.1(2), 8.4.4(1)
$\tau_{Rd,i}$	Shear stress resistance at interfaces	8.2.6(3), 8.2.6(5)
τ_{Rdm}	Shear resistance stress reduced by influence of transverse bending	8.2.4(2)
$\tau_{Rd,max}$	Maximum shear stress resistance of planar members with shear reinforcement subjected to concentrated forces	8.2.2(5), 8.4.4(5)
$\tau_{Rd,pl}$	Design strength of plain concrete in shear	14.4.3(3)
$\tau_{Rd,sy}$	Shear stress resistance governed by yielding of shear reinforcement	8.2.3(5)
$\tau_{t,i}$	Torsional shear stress in wall i	8.3.2(2), Figure 8.16
$\tau_{t,Rd}$	Torsional shear stress resistance	8.3.4(1)
$\tau_{t,Rd,sw}$, $\tau_{t,Rd,sl}$, $\tau_{t,Rd,max}$	Torsional shear stress resistance governed by yielding of shear reinforcement, by yielding of longitudinal reinforcement or by crushing of the concrete in the compression field	8.3.4(2)
ϕ	Diameter of a reinforcing bar	5.2.1(2)
ϕ_b	Equivalent diameter of a bundle of reinforcing bars	11.4.3(1)
Φ_c	Diameter of confinement reinforcement	11.4.2(5)
ϕ_{duct}	Outer diameter of a post-tensioning duct	8.2.3(10)
ϕ_{eq}	Equivalent bar diameter for bond calculations when tensile reinforcement is composed by bars of different diameters	9.2.3(6)
ϕ_h	Diameter of circular head of headed bar	11.4.7(1), Figure 11.9
ϕ_{mand}	Mandrel diameter for bent reinforcement bars	11.3

$\phi_{\mathrm{mand,min}}$	Minimum value of ϕ_{mand}	11.3
ϕ_{p}	Nominal diameter of the pretensioning tendon	9.2.2(3), 13.5.1(2), Figure 13.1
$\phi_{\mathrm{p,eq}}$	Equivalent diameter of tendons	10.3(2)
ϕ_{t}	Diameter of transverse reinforcement between the bar to be anchored and the free surface; diameter of welded transverse bar	11.4.2(5); 11.4.5(1)
ϕ_{trans}	Diameter of transverse bars within bend	11.3(5)
ϕ_{w}	Diameter of of punching shear reinforcement	8.4.4(1)
$\phi_{\mathrm{w,max}}$	Maximum diameter of punching shear reinforcement	12.5.1(3)
Φ	Factor taking into account eccentricity, including second order effects; dynamic factor for railway bridges	14.4.5.2(1); K.10.2.1(2)
φ	Damage equivalent impact factor φ_{fat} or Φ for road and railway bridges	K.10.2.1(2)
$\varphi(t,t_0)$	Creep coefficient, defining creep between times t and t_0, related to elastic deformation at 28 days	5.1.5(1), B.5(1)
$\varphi(50\mathrm{y},t_0)$	Creep coefficient after 50 years of loading	Table 5.2
$\varphi_{0,05}$, $\varphi_{\mathrm{k};0,05}$	Lower-bound value and characteristic value of creep coefficient corresponding to a 5 % cut-off, based on a normal distribution	B.5(8)
$\varphi_{0,10}$, $\varphi_{\mathrm{k};0,10}$	Lower-bound value and characteristic value of creep coefficient corresponding to a 10 % cut-off, based on a normal distribution	B.5(8)
$\varphi_{0,90}$, $\varphi_{\mathrm{k};0,90}$	Upper-bound value and characteristic value of creep coefficient corresponding to a 90 % cut-off, based on a normal distribution	B.5(8)
$\varphi_{0,95}$, $\varphi_{\mathrm{k};0,95}$	Upper-bound value and characteristic value of creep coefficient corresponding to a 95 % cut-off, based on a normal distribution	B.5(8)
$\varphi_{\mathrm{bc}}(t,t_0)$	Basic creep coefficient	B.5(1), B.5(2)
$\varphi_{\mathrm{dc}}(t,t_0)$	Drying creep coefficient	B.5(1), B.5(3)
$\varphi_{\mathrm{eff,b}}$	Effective creep ratio for local second order effects	7.4.2(2)
$\varphi_{\mathrm{eff,s}}$	Effective creep ratio for global second order effects	7.4.2(2)
$\varphi_{\sigma}(t,t_0)$	Creep coefficient, adjusted for non-linearity due to concrete stresses above $0{,}4f_{\mathrm{cm}}$	B.5(6)
χ	Aging coefficient which may be taken equal to 0,8 for long term calculations	7.3.1(6)
ω	Mechanical reinforcement ratio	O.6(1)
ω_{r}	Required mechanical tension reinforcement ratio at mid-span to resist the moment due to the design loads (at support for cantilevers)	9.3.2(1), Table 9.3

3.3 Symbols in Annex A

3.3.1 Latin upper case letters

V_i	Coefficient of variation of the variable i	A.3(2), Table A.3
V_{RM}	Coefficient of variation of the resistance	A.3(3)

3.3.2 Latin lower case letters

$f_{\mathrm{c,ais}}$	Actual insitu concrete compressive strength in the structure	A.3(2), Table A.3

$f_{c,is}$	Compressive strength of a core taken at a test location within a structural element or precast concrete component expressed in terms of the strength of a 2:1 core of diameter ≥ 75 mm	A.3(2), Table A.3
f_{ym}	Mean value of yield strength of reinforcing steel or, if yield phenomenon is not present, the characteristic value of 0,2 % proof strength	Annex A.3(2), Table A.3

3.3.3 Greek lower case letters

α_R	Sensitivity factor for the reliability of the resistance	A.3(3)
β_{tgt}	Target value of reliability index	A.3(3)
γ_M	Partial material factor	A.3(3)
μ_i	Bias factor of the variable i defined as ratio $X_{i,m}/X_{i,k}$	A.3(2), Table A.3
η_{is}	Insitu factor of the concrete compressive strength defined as ratio $f_{c,ais}$ / f_c	A.3(2), Table A.3
μ_{RM}	Bias factor of resistance	A.3(3)

3.4 Symbols in Annex I

3.4.1 Latin upper case letters

AAR	Alkali-aggregate reaction	I.4.1.2
A_{sq}	Cross-sectional area of a square plain bar	I.11.4.1(4)
D_{max}	Declared value of the upper sieve size of the coarsest fraction of aggregates used in concrete	I.5.2.1(4)
DEF	Delayed ettringite formation	I.4.1.2
P_x	Corrosion penetration depth	I.8.1
$V_{fc,is}$	Coefficient of variation of $f_{c,is}$	I.4.2.1(2)
$V_{fc,is,lim}$	Limit value of $V_{fc,is}$ for the adjustment of the partial factors for concrete	I.4.2.1(2)
V_{fy}	Coefficient of variation of yield strength of reinforcement	I.4.2.1(2)
$V_{fy,lim}$	Limit value of V_{fy} for the adjustment of the partial factor for reinforcement	I.4.2.1(2)
V_x	Coefficient of variation of the material property X	I.5.3
X_k	Characteristic value of the material property	I.5.3

3.4.2 Latin lower case letters

$b_{w,i}$, $b_{w,i-1}$, $b_{w,i+1}$	Coefficients to evaluate b_w for shear resistance in case of shear reinforcement not fulfilling the maximum longitudinal spacing of shear assemblies/stirrups or bent-up bars given in Clause 12	I.8.3.2(3)
c_f	Additional distance with respect to concentrated loads of reaction forces acting on compression flanges	I.8.3.2(4)
$c_{min,xy}$	Minimum value of concrete cover between c_x and c_y for designing the anchorage in case of low concrete cover	I.11.4.1(7)
d_{sys}	Relevant depth considered for determination of η_{sys}	I.8.5.2(2)
$f_{ck,is}$	Characteristic in-situ compressive strength of concrete cores expressed in terms of the strength of a 2:1 core of diameter ≥ 75 mm (5 % fractile)	I.5.2.1(3)
$f_{ck,ft}$	Residual characteristic compressive cylinder strength of concrete	I.11.4.1(11)

$f_{\text{ctk},is;0,05}$	Characteristic measured insitu axial tensile strength of concrete (5% fractile)	I.11.4.1(11)
$k_{\text{lbs,c}}$	Coefficient for bond calculation in case of low concrete cover	I.11.4.1(7), Table I.8
k_n	Characteristic fractile factor for a sample size n	I.5.3(2), Table I.4
k_{ns}	Coefficient for shear stress resistance of members with shear reinforcement	I.8.3.2(2)
k_{part}	Coefficient for determining σ_{Rdt}	I.8.6(1)
k_{vd}	Coefficient for shear stress resistance of members without shear reinforcement	I.8.3.1(1)
$k_{\mu\text{fc}}$	Parameter to be used to evaluate f_{ck} from $f_{\text{ck},is}$ and that account for the representativeness of the insitu compressive concrete strength	I.5.2.1(3)
m_x	Mean value of the variable X from n sample results	I.5.3(2)
n	Number of test results	I.4.2.1(2), I.5.3(2)
s_X	Estimated value of the standard deviation of the variable X from n sample results	I.5.3(2)
x_i	Basic variable i	I.5.3(2)

3.4.3 Greek lower case letters

γ_{def}	Partial factor covering uncertainties related to calculation of deformations	I.8.3.1(1)
δ_1, δ_2	Coefficients for the design value of the reinforcement stress at the cross-section to be anchored by bond over l_{bd} in case of bends and hooks	I.11.4.2(1)
$\eta_1 \eta_2\ \eta_3\ \eta_4$	Coefficients for the design anchorage length l_{bd} for plain bars	I.11.4.1(3)
σ'_{sd}	Design value of the reinforcement stress at the cross-section to be anchored by bond over l_{bd} in case of bends and hooks	I.11.4.2(1)
$\sigma_{\text{Rd,t}}$	Maximum design stress applied to partially loaded area not requiring transverse reinforcement	I.8.6(1)
$\Delta\sigma_{\text{sd}}$	Design value of the reinforcement stress at the cross-section developed by bends and hooks	I.11.4.2(1)
$\phi_{\text{sq,eq}}$	Equivalent bar diameter for bond calculation of square cross-section bars	I.11.4.1(4)
ψ	Maximum rotation of slab around supporting area	I.8.5.1(2)

3.5 Symbols in Annex J

3.5.1 Latin upper case letters

A_{f}	Cross-sectional area of a CFRP system	J.5.1(3)
D	Diameter of circular column section	J.8.1.2(2)
D_{eq}	Equivalent diameter of member with rectangular cross section	J.8.1.2(2)
E_{f}	Mean modulus of elasticity in longitudinal direction of ABR CFRP	J.5.1(3)
F_{bfRd}	Design bond force resistance of the ABR CFRP	J.11.1.3(1)
F_{bsm}	Bond strength per unit length	J.11.1.2.3(4)
$F_{\text{fEd,cr}}$	Force in CFRP at first crack in the strengthened area	J.10.1(1)
$F_{\text{f,NSM,max}}$	The maximum force in the NSM CFRP system, taking the shift of the tension envelope into account	J.10.3(1)

$\Delta F_{\mathrm{f,min}}$	Minimum value of ΔF_{f} under the fatigue load combination	J.10.2(1)
$\Delta F_{\mathrm{f,max}}$	Maximum value of ΔF_{f} under the fatigue load combination	J.10.2(1)
ΔF_{f}	Force change in CFRP under the fatigue load combination	J.10.2(1)
ΔF_{fEd}	Difference in FRP tension force between cracks	J.10.1(1)
$\Delta F_{\mathrm{fEd,fat}}$	Design difference in the change in force in the CFRP system between cracks	J.10.2(1)
$\Delta F_{\mathrm{fRd,fat1}}$	Fatigue resistance limited by an elastic response in the bond of the CFRP to the concrete surface	J.10.1(1)
$\Delta F_{\mathrm{fRd,fat2}}$	Fatigue resistance of the CFRP system subject to N* stress cycles	J.10.2(1)
ΔF_{fRd}	Bond resistance between cracks	J.10.2(1)
$\Delta F_{\mathrm{fE,equ}}$	Maximum difference in CFRP stress under the relevant load combination between cracks	J.10.1(1)
N^*	Number of stress cycles	J.10.2(1)
$V_{\mathrm{Rd,cfE}}$	Design resistance against concrete cover separation	J.11.1.2.4(1)

3.5.2 Latin lower-case letters

a_{fE}	Distance from end of CFRP flexural strengthening to adjacent point of zero bending moment	J.11.1.2.4(1), Figure J.6
a_{r}	Distance of adhesively bonded CFRP reinforcement from free edge	J.11.1.3(1)
b_{f}	Width of the adhesively bonded CFRP reinforcement sheets or strips or square bars	J.5.1(3)
b_{slot}	Width of slot for NSM CFRP reinforcement	J.12.2(2)
c_{fat}	Reduction factor taking into account the stress cycles	J.10.2(1)
f_{bfRd}	Design bond strength of the anchorage	J.10.1(1), J.11
f_{bsm}	Mean bond stress of reinforcing steel	J.11.1.2.3(4)
$f_{\mathrm{ctm,surf}}$	Mean surface tensile strength of concrete	J.5.3(5), J.11.1.1.1(2)
f_{Ack}	Characteristic compressive strength of the adhesive	J.5.2(5)
f_{Atk}	Characteristic tensile strength of the adhesive	J.5.1(3), J.5.2(5)
f_{bfRd}	Limiting design strength of the bond	J.8.2.3(6)
f_{fud}	Ultimate design short-term tensile strength of the ABR CFRP	J.5.3(1), Figure J.1
f_{fuk}	Characteristic short-term tensile strength of the ABR CFRP	J.5.2(1), Figure J.1
f_{fwd}	Design shear strength of the CFRP system	J.8.2.3(4)
Δf_{fED}, $\Delta f_{\mathrm{fED,max}}$	Difference and maximum difference in CFRP stress between cracks, respectively	J.10.1(1), J.10.2(1)
$\Delta f_{\mathrm{fk,B}}$	Basic value of adhesive bond strength between cracks	J.11.1.2.3(5)
$\Delta f_{\mathrm{fk,C}}$	Increase of bond strength between cracks resulting from clamping from curvature of the beam	J.11.1.2.3(5)
$\Delta f_{\mathrm{fk,F}}$	Increase of bond strength between cracks resulting from bond friction	J.11.1.2.3(5)
Δf_{fRd}	Bond resistance between adjacent cracks	J.11.1.2.3(5)

h_{f}	Height of CFRP shear reinforcement crossing shear crack	J.8.2.3(6), (7), Figure J.4
k_{cc}	Confinement factor for columns strengthened with CFRP	J.8.1.2(4)
$k_{c,surf}$	Coefficient considering the concreting position for estimation of surface tensile strength	J.11.1.1.1(2)
k_{e}	Confinement effectiveness factor for rectangular columns	J.8.1.2(4)
k_{f}	Coefficient for determining the effective thickness for a number of layers	J.5.2(4)
k_{h}	Confinement effectiveness factor for helical wrapping	J.8.1.2(4)
k_{f3}	Exponent for determining factor for stress cycles	J.10.2(1)
k_{r}	Factor considering corner radius	J.8.1.2(4)
$k_{sys,b1}$, $k_{sys,b2}$, $k_{sys,b3}$	Product-specific system factor	J.11.1.1.3(1)
l_{bf}	Bond length of the adhesively bonded CFRP reinforcement	J.8.2.3(6)
$l_{bf,max,k}$	Characteristic maximum value of effective bond length of ABR CFRP	J.8.2.3(6), J.11.1.1.2(1)
n_{f}	Number of CFRP layers	J.5.2(4)
r_{c}	Corner radius	J.8.1.2(2)
s_{f}	Centre to centre spacing of FRP strips	J.8.1.2(4)
s_{f0k}	Maximum bond slip	J.11.1.1.3(1)
$s_{cr,min}$	Minimum spacing of bending cracks	J.11.1.2.3(4)
t_{f}	Nominal thickness of the adhesively bonded reinforcement	J.5.2(2)
t_{slot}	Depth of slot for NSM CFRP reinforcement bar or strip	J.12.2(2)

3.5.3 Greek lower-case letters

α_{bc}	Product-specific system factor for long-term behaviour of concrete	J.11.1.3(1)
α_{bA}	Product-specific system factor for long-term behaviour of the adhesive	J.11.1.3(1)
α_{f}	Angle formed between the CFRP system and longitudinal axis of a member in bending	J.8.2.3(4)
α_{fat2}	Reduction factor for fatigue	J.10.2(1)
β_{1}	Reduction factor for bond capacity taking account of anchorage length	J.11.1.1.2(1)
β_{f}	Angle formed between the CFRP system and the transverse axis of a column strengthened by CFRP confinement	J.8.1.2(4)
γ_{BA}	Partial factor for adhesively bonded CFRP reinforcement for bond	J.4(1)
γ γ_{f}	Partial factor for tensile strength of adhesively bonded CFRP reinforcement	J.4(1)
ε_{fud}	Long-term design strain of adhesively bonded CFRP reinforcement	J.5.3(3), Figure J.1
ε_{fuk}	Characteristic ultimate strain for the adhesively bonded CFRP reinforcement	J.5.2(1), Figure J.1
η_{f}	Reduction factor applied to the tensile stress of the EBR CFRP sheet or strip	J.5.2(1)
$\Delta\sigma_{f}$	Stress range of an NSM CFRP reinforcement subjected to fatigue	J.10.3(1)
τ_{bck}	Bond strength of concrete with NSM CFRP reinforcement strips	J.11.1.3(1)

τ_{bAk}	Bond strength of adhesive with NSM CFRP reinforcement strips	J.11.1.3(1)
τ_{bAd}	Design value of the shear stress resistance of adhesive	J.11.1.3(1)
$\tau_{Ed,f}$	Design shear stress in adhesively bonded CFRP stirrups for shear induced crack separation	J.11.1.2.5(1)
τ_{f1k}	Maximum bond strength of adhesively bonded CFRP reinforcement	J.11.1.1.3(1)
τ_{Rd}	Design shear resistance of the member without shear strengthening	J.8.2.3(4)
$\tau_{Rd,CFRP}$	Design shear resistance of a section with CFRP shear strengthening	J.8.2.3(4)
$\tau_{Rd,f}$	Contribution of ABR CFRP shear strengthening to design shear resistance	J.8.2.3(4)
ϕ_f	Diameter of NSM CFRP bars	J.12.2(2)

3.6 Symbols in Annex L

3.6.1 Latin upper case letters

$CMOD_1$	= 0,5 mm is the crack width (Crack Mouth Opening Displacement) for which the characteristic residual flexural strength, $f_{R,1k}$, is determined (defined in EN 14651).	L.15(1)
$CMOD_3$	= 2,5 mm is the crack width (Crack Mouth Opening Displacement) for which the characteristic residual flexural strength, $f_{R,3k}$, is determined (defined in EN 14651).	L.15(1)
A_{ct}	Area of the tension zone (in m^2) of the cross-section involved in the failure of an equilibrium system	L.5.5.1(7)

3.6.2 Latin lower case letters

$f_{R,1k}$	Characteristic residual flexural strength at $CMOD_1$ = 0,5 mm representing the residual strength class	L.5.1(1), Table L.2
$f_{R,3k}$	Characteristic residual flexural strength at $CMOD_3$ = 2,5 mm representing the performance class	L.5.1(1), Table L.2
$f_{Fts,ef}$	Effective residual tensile strength for crack widths at the serviceability limit state accounting for fibre orientation	L.5.5.1(2)
$f_{Ft1,ef}$	Effective residual tensile strength for crack width = 0,5 mm accounting for fibre orientation to be used in the constitutive law for bi-linear stress distribution	L.5.5.2(1)
f_{Ftsd}	Design residual tensile strength for crack widths at the serviceability limit state accounting for fibre orientation	L.5.5.1(2)
f_{Ft1d}	Design residual tensile strength for crack width = 0,5 mm accounting for fibre orientation to be used for bi-linear stress distribution	L.8.1(3)
$f_{Ft3,ef}$	Effective residual tensile strength for crack width = 2,5 mm accounting for fibre orientation to be used in the constitutive law for bi-linear analysis	L.5.5.2(1)
f_{Ftud}	Design value of the residual tensile strength accounting for fibre orientation	L.5.5.1(2)
$f_{Ftu,ef}$	Effective residual tensile strength of SFRC for given crack width accounting for fibre orientation and volume effect	L.5.5.1(2)
f_{Ft3d}	Design residual tensile strength for crack width = 2,5 mm accounting for fibre orientation to be used for bi-linear stress distribution	L.8.1(3)
k_{AS}	Parameter that limits the replacement of minimum longitudinal reinforcement by fibres	L.12.3.1(2)

k_{dur}	Coefficient to determine the distance to which the residual tensile strength of SFRC shall be disregarded	L.6(4)
k_F	Coefficient to determine clear bar spacing as a function of the fibre length	L.11.2(1)
k_{SFRC}	Factor to determine ε_{cu1} for SFRC	L.5.5.2(2)
l_{cs}	Structural length used to convert the stress-crack width relationship of SFRC to a stress-strain relationship compatible with concrete design	L.5.5.2(1)
$s_{r,m,cal,F}$	Mean crack spacing of SFRC members subject to bending	L.9.3(1)
w_u	Maximum crack opening accepted in the structural design	L.5.5.2(1)

3.6.3 Greek letters

α_{duct}	Coefficient to determine required A_s for which plastic analysis can be used without direct check of rotation capacity	L.7(1)
α_f	Coefficient for determining $s_{r,m,cal,F}$	L.9.3(1)
γ_{SF}	Partial factor for SFRC in tension	L.4(1)
η_{cF}, η_{sw}, η_F	Parameters expressing that the shear capacity contributions from steel fibres and ordinary reinforced concrete are not 100 % additive	L.8.2.2(1), L.8.2.3(1)
ε_{Ftu}	Ultimate tensile strain for SFRC	L.5.5.2(1), Figure L.2
ε_{Ftud}	Design tensile strain limit used for SFRC cross-sections with or without axial force according to 5.1.7 and 8.1.5	L.8.1(2)
$\varepsilon_{F,0}$	Equivalent strain value used to define the post-cracking constitutive law for non-linear analysis	L.5.5.2(1), Figure L.2
κ_G	Factor taking into account the size of the tensile zone involved in the failure state	L.5.5.1(7)
$\kappa_{k,max}$	Factor defining the upper limit of the ratio between characteristic and mean residual flexural strengths	L.5.5.1(7), L.13.1(1), L.15(5)
κ_O	Factor taking into account the orientation of the steel fibres in the concrete matrix in relation to the orientation of the principal tensile stress arising from the action effects	L.5.5.1(4) (5) (6)
$\rho_{Fw,min}$	Minimum shear reinforcement ratio	L.12.1(3)
$\tau_{Rd,cF}$	Design value of the shear strength of SFRC	L.8.2.2(1)
$\tau_{Rd,sF}$	Design value of the shear strength of SFRC with shear reinforcement	L.8.2.3(1)
$\tau_{Rd,swF}$	Design value of the torsional resistance in the transversal direction	L.8.3(1)
$\tau_{Rd,slF}$	Design value of the torsional resistance in the longitudinal direction	L.8.3(1)

3.7 Symbols in Annex R

3.7.1 Latin upper case letters

A_{fl}	Cross-sectional area of longitudinal FRP reinforcement	R.8.2(3)
$A_{f,surf}$	FRP surface reinforcement	
C_c, C_e, C_t,	Long term strength reduction factor to account for temperature, for creep and for environmental conditions, respectively	R.5.3(2)
E_{fR}	Design value of modulus of elasticity of FRP-reinforcement	R.5.1(3), Figure R.1
E_{fwR}	Design value of modulus of elasticity of FRP shear reinforcement	R.8.2(3)

3.7.2 Latin lower case letters

f_{ftd}	Design tensile strength of FRP reinforcement	R.5.3(2), Figure R.1
f_{ftk0}	Characteristic tensile strength of FRP reinforcement at the rupture strain ε_{ftk0}	R.5.2(1), Figure R.1
$f_{\mathrm{ftk},100a}$	Characteristic long term tensile strength of FRP reinforcement	R.5.1(3), Figure R.1
f_{bd100a}	Long term bond strength of FRP reinforcement	R.11.4(3)
$f_{\mathrm{fwk,100a}}$	Characteristic long term tensile strength of FRP shear reinforcement	R.8.2(3)
f_{fwRd}	Design tensile strength of FRP shear reinforcement	R.5.2(2), R.8.2(3)

3.7.3 Greek letters

$\alpha_{\mathrm{FRP,th}}$	Coefficient of thermal expansion of FRP reinforcement	R.5.3(5)
γ_{FRP}	Partial factor for FRP reinforcing material	R.4(1)
$\varepsilon_{\mathrm{fRd}}$	Strain of FRP reinforcement at design tensile strength f_{ftd}	R.5.3(3), Figure R.1
$\varepsilon_{\mathrm{ftk,100a}}$	Long term rupture strain of FRP reinforcement	R.5.3(3), Figure R.1
$\varepsilon_{\mathrm{ftk0}}$	Rupture strain of FRP reinforcement	R.5.3(3), Figure R.1
$\varepsilon_{\mathrm{fwRd}}$	Strain of FRP shear reinforcement at design tensile strength f_{fwRd}	R.5.2(2)
$\tau_{\mathrm{Rd.f}}$	Shear resistance of a member with FRP reinforcement	R.8.2(3)
ρ_{lf}	Longitudinal reinforcement ratio for FRP reinforcement	R.5.1(3)
σ_{f}	Serviceability value of stress in the FRP reinforcement, determined on the basis of a cracked section	R.9.2(3)
$\sigma_{\mathrm{f,lim}}$	Limiting value of the serviceability stress in the FRP reinforcement in order to comply with a given limiting crack width	R.9.2(4)
σ_{ftd}	Design value of stress in the FRP reinforcement at the cross-section	R.11.4(3)
ϕ_{f}	Diameter of a FRP reinforcement bar	R.11.4(4)

3.8 Abbreviations

AVCP	Assessment and Verification of Constancy of Performance.	
CS, CN, CR	Classes of Concrete with Slow/Normal/Rapid strength development	
CFRP	Carbon Fibre Reinforced Polymer reinforcement adhesively bonded to the concrete surface	
ERC	Exposure Resistance Class	
FPC	Factory Production Control	
FRP	profiled or roughened glass or Carbon Fibre Reinforced Polymer reinforcement	
GFRP	Glass Fibre Reinforced Polymer reinforcement	
lg	Logarithm with basis 10	
LWAC	Ligthweight Aggregate Concrete	
PE	Polyethylene	
n. a.	not applicable	
SCM	Supplementary cementitious materials	B.3
SFRC	Steel Fibre Reinforced Concrete	
SLS	Serviceability Limit State	

SSRC	Stainless Steel Resistance Class
ULS	Ultimate Limit State

3.9 Units

Angle	Degrees/Radians
E-Modulus	For unit dependent formulae, MPa shall be used.
Geometric data	For unit dependent formulae, mm shall be used.
Relative humidity	%
Stresses and material strengths	For unit dependent formulae, MPa shall be used.
Temperature	°C, K
Time	Days unless otherwise stated

3.10 Sign conventions

(1) In general, forces, stresses and strains which result in an elongation of material have a positive and those which result in shortening have a negative sign. When compressive or tensile forces are indicated in a figure by a vector they have a positive sign when they are acting in the direction described by the vector.

(2) All tensile and compressive material strengths shall be used with a positive sign.

(3) Shortening due to shrinkage is considered positive.

4 Basis of design

4.1 General rules

4.1.1 Basic requirements

(1) The basis of design for concrete structures shall be in accordance with the general rules given in EN 1990, supplemented by the provisions for basis of design for concrete structures given in this document.

(2) The basic requirements of EN 1990, Clause 4 should be 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 (all parts) and EN 1997 (all parts);

— combination of actions in accordance with EN 1990; and

— resistances, robustness, durability and serviceability in accordance with all relevant parts of EN 1992.

4.1.2 Structural reliability and quality management

(1) The rules for structural reliability and quality management given in EN 1990 shall be followed.

4.1.3 Design service life

(1) The design service life of structures or members of structures shall be specified.

NOTE For values of design service life, see EN 1990, Annex A.

(2) Structures or members of structures shall be designed consistently with respect to all time-dependent effects including durability, serviceability and fatigue.

4.2 Basic variables

4.2.1 Actions and time-dependent effects

4.2.1.1 General

(1) Actions to be used in design shall be obtained from the relevant parts of EN 1991 (all parts) or EN 1997 (all parts). Where relevant, other actions not covered by EN 1991 (all parts) or EN 1997 (all parts) shall be in accordance with EN 1990 and as specified by the relevant authority or agreed for a specific project by the relevant parties.

NOTE Actions specific to this Eurocode (such as prestress, creep and shrinkage) are given in the relevant clauses.

4.2.1.2 Time-dependent effects

(1) Time dependent effects, including relaxation of the prestressing steel, shrinkage and creep of the concrete, should be accounted for in design, where relevant.

(2) Where creep is taken into account its design effects should be evaluated under the quasi-permanent combination of actions, and applied in all relevant combinations of actions.

4.2.1.3 Effects resulting from restrained, imposed deformations

(1) Effects resulting from restrained, imposed deformations should be quantified and considered when verifying serviceability limit states and fatigue.

NOTE 1 Effects resulting from restrained, imposed deformations can be reduced, when necessary, using various methods such as varying the composition of the concrete mix (guidance is given in D.3) and adjusting the stiffness of integral structural restraints. The use of bearings and joints can also reduce these effects.

(2) The effects of restrained, imposed deformations may be neglected at ultimate limit states where it can be demonstrated or has been shown by experience with similar structures that:

(i) there is sufficient deformation capacity to allow the respective movements to occur and fulfil the ultimate limit state; and

(ii) the structures behaviour is not sensitive to second order effects caused by large displacements.

In all other cases, the effects of restrained imposed deformations should be considered.

NOTE 2 For a detailed analysis, see Annex D.

4.2.1.4 Ground-structure interaction

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

(2) Where differential settlements/movements of the structure due to ground subsidence are taken into account, predicted values should be estimated in accordance with EN 1997-1 and limiting values for foundation movement set in accordance with EN 1990.

4.2.1.5 Prestress

(1) The design prestress action shall be determined.

NOTE 1 The prestress considered in this Eurocode is applied by tendons made of high-strength steel (wires, strands or bars).

NOTE 2 Tendons can be external to the structure with points of contact at possible deviators, at anchorages, or with continuous contact on curved surfaces.

(2) When considered in accordance with 7.6.1(1) b), the design prestress action at ultimate limit states should be taken as the mean value of the prestressing stress (as calculated in 7.6.2, 7.6.3 and 7.6.4) multiplied by the partial factor for prestress.

(3) For serviceability and fatigue verifications, allowance shall be made for possible variations in prestress. Upper and lower characteristic values of the prestressing stress at the serviceability limit state and in fatigue design shall be estimated from the mean value $\sigma_{pm}(x,t)$ according to Formulae (4.1) and (4.2).

$$\sigma_{pk,sup}(x,t) = r_{sup} \cdot \sigma_{pm}(x,t) \tag{4.1}$$

$$\sigma_{pk,inf}(x,t) = r_{inf} \cdot \sigma_{pm}(x,t) \tag{4.2}$$

NOTE 3 The values of r_{sup} and r_{inf} given in Table 4.1 (NDP) apply unless the National Annex gives different values.

Table 4.1 (NDP) — Factors for calculating the upper and lower characteristic values of the prestress action

Type of prestress	r_{sup}	r_{inf}
Pre-tensioning	1,05	0,95
Post-tensioning with unbonded tendons	1,05	0,95
Post-tensioning with bonded tendons	1,10	0,90

4.2.1.6 Effect of water or gas pressure

(1) In structures exposed to high fluid or gas pressure the effect of potential pressure build up in pores and cracks shall be accounted for in the design where it increases the action effects or reduces the resistance by more than 10 %.

4.2.2 Geometric data

(1) Geometric tolerances shall comply with EN 13670, Tolerance Class 1, or where other tolerances are permitted they shall be specified in the execution specification and suitable allowances shall be made in the design.

NOTE Examples of such members include cast-in-place bored piles where the steel casing is pulled, or concrete piles driven through rock. This standard offers no guidance on what allowance is adequate, but engineering practice in the various countries could. Allowance can be made either by a reduced cross-section, an assumed deformation or a reduced resistance.

4.3 Verification by the partial factor method

4.3.1 Partial factor for shrinkage action

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

NOTE The value $\gamma_{SH} = 1,0$ applies unless the National Annex gives a different value.

4.3.2 Partial factors for prestress action

(1) The partial factors $\gamma_{P,fav}$ or $\gamma_{P,unfav}$ shall be applied to the prestress for ultimate limit state verifications when considered in accordance with 7.6.1(1) b).

NOTE 1 The values of $\gamma_{P,fav}$ or $\gamma_{P,unfav}$ given in Table 4.2 (NDP) apply unless the National Annex gives different values.

Table 4.2 (NDP) — Partial factors for prestress action for ultimate limit states

Factor for prestress	Value	Applied to	ULS verification type
$\gamma_{P,fav}$	1,00	Prestress force for bonded and unbonded tendons	Verifications where an increase in prestress would be favourable
$\gamma_{P,unfav}$	1,20		Verifications where an increase in prestress would be unfavourable
$\gamma_{\Delta P,sup}$	0,80	Change in stress in unbonded tendons	Verifications where increase in stress would be favourable
$\gamma_{\Delta P,inf}$	1,20		Verifications where increase in stress would be unfavourable
$\gamma_{\Delta P,sup}$ $\gamma_{\Delta P,inf}$	1,0		Verifications where linear analysis with uncracked sections, i.e. assuming a lower limit of deformations, is applied

(2) Partial factors $\gamma_{\Delta P,sup}$ or $\gamma_{\Delta P,inf}$ shall be applied to the change in stress in unbonded prestressing tendons associated with the deformation of the member for ultimate limit state verifications (see 7.6.5(4)).

NOTE 2 The values of $\gamma_{\Delta P,sup}$ and $\gamma_{\Delta P,inf}$ given in Table 4.2 (NDP) apply unless the National Annex gives different values.

4.3.3 Partial factors for materials

(1) Partial factors for materials γ_S, γ_C, γ_{CE} and γ_V shall be used.

NOTE 1 The values of γ_S, γ_C, γ_{CE} and γ_V in Table 4.3 (NDP) apply unless the National Annex gives different values.

NOTE 2 For fire design the partial factors are obtained from EN 1992-1-2. For seismic design the partial factors are obtained from EN 1998 (all parts).

Table 4.3 (NDP) — Partial factors for materials

Design situations — Limit states	γ_S for reinforcing and prestressing steel	γ_C and γ_{CE} for concrete	γ_V for shear and punching resistance without shear reinforcement
Persistent and transient design situation	1,15	1,50[a]	1,40
Fatigue design situation	1,15	1,50	1,40
Accidental design situation	1,00	1,15	1,15
Serviceability limit state	1,00	1,00	—
NOTE The partial factors for materials correspond to geometrical deviations of Tolerance Class 1 and Execution Class 2 in EN 13670.			
[a] The value for γ_{CE} applies when the indicative value for the elastic modulus according 5.1.4(2) is used. A value $\gamma_{CE} = 1{,}3$ applies when the elastic modulus is determined according to 5.1.4(1).			

(2) Lower values of partial factor γ_S and γ_V for the verification of the ULS in case of persistent, transient and accidental design situations may be used according to A.3 if a design value of the effective depth d_d is considered.

(3) Lower values of γ_S, γ_C, γ_{CE} and γ_V may be used if justified by measures reducing the uncertainty in the calculated resistance, as specified in Annex A.

(4) To allow for increased uncertainty and variability of concrete strength in cast-in-place concrete members, cast in the ground at significant depth without permanent casing, the partial factor γ_C should be multiplied by a factor k_{cip}.

NOTE 3 The value of $k_{cip} = 1{,}1$ applies in general and the value $k_{cip} = 1{,}0$ for cast-in-place concrete members built in accordance with EN 1536, EN 1538 or EN 14199 unless the National Annex gives different values.

4.4 Requirements for connection of elements to concrete members

(1) Reinforcement which is either cast-in or drilled-in and grouted into hardened concrete extending out of a member and/or connecting a member to an adjacent member, shall be properly anchored into the concrete member.

(2) The design of fastenings used to connect structural elements or non-structural elements to concrete members and anchor / transmit the actions into the concrete (local effects) shall be done in accordance with EN 1992-4. The transmission of these actions from the fastenings within the concrete member to its supports (global effects) shall be done in accordance with this standard.

NOTE For connections using post-installed reinforcing steel systems, see 11.4.8.

5 Materials

5.1 Concrete

5.1.1 General

(1) 5.1 gives provisions for normal weight and heavy weight concrete.

NOTE 1 Specific rules for steel fibre reinforced concrete are given in Informative Annex L.

NOTE 2 Specific rules for lightweight aggregate concrete are given in Informative Annex M.

NOTE 3 Specific rules for concrete using recycled aggregates are given in Informative Annex N.

(2) Concrete used for structures designed in accordance with this Eurocode shall comply with EN 206.

5.1.2 Properties and related conditions

(1) Specified properties and related conditions of concrete that are required for design according to this Eurocode should include at least:

— concrete strength class according to 5.1.3(3) and the strength development class according to B.4(1);

— aggregate sizes (D_{upper} and D_{lower}) in accordance with EN 206;

— exposure class related to environmental conditions and exposure resistance classes, respectively according to Clause 6;

— chloride class in accordance with EN 206;

— execution class in accordance with EN 13670.

(2) The following properties may either be derived in accordance with the provisions of 5.1 and Annex B, or may be determined by testing, or may be specified for special cases:

— tensile strength ($f_{ctm,}$ $f_{ctk0,05,}$ $f_{ctk0,95}$);

— modulus of elasticity (E_{cm});

— Poisson's ratio;

— coefficient of thermal expansion ($\alpha_{c,th}$);

— creep coefficient (φ);

— shrinkage value (ε_{cs});

— density of concrete.

(3) The design properties of concrete may be used for service temperatures in the range from −40 °C to +100 °C.

(4) Experimental verification should be used in cases where a member or a structure is sensitive to any of these properties and there is no previous experience or established practice showing that such verifications are not necessary.

NOTE For guidance to experimental determination of creep and shrinkage values, see Annex B.

5.1.3 Strength

(1) The compressive strength of concrete shall be denoted by concrete strength classes which relate to the characteristic (5 %) cylinder strength f_{ck} of the concrete in accordance with EN 206, determined at an age t_{ref}.

(2) The value for t_{ref}

(i) should be taken as 28 days in general; or

(ii) may be taken between 28 and 91 days when specified for a project.

(3) The compressive and tensile strength characteristics necessary for design should be taken from Table 5.1.

Table 5.1 — Compressive and tensile strength of concrete [MPa]

f	Strength classes for concrete [EN 206]															Governing formulae
	C12/15	**C16/20**	**C20/25**	**C25/30**	**C30/37**	**C35/45**	**C40/50**	**C45/55**	**C50/60**	**C55/67**	**C60/75**	**C70/85**	**C80/95**	**C90/105**	**C100/115**	
f_{ck}	12	16	20	25	30	35	40	45	50	55	60	70	80	90	100	—
f_{cm}	20	24	28	33	38	43	48	53	58	63	68	78	88	98	108	$f_{cm} = f_{ck} + 8$ MPa
f_{ctm}	1,6	1,9	2,2	2,6	2,9	3,2	3,5	3,8	4,1	4,2	4,3	4,5	4,7	4,9	5,1	$f_{ctm} = 0{,}3 f_{ck}^{2/3}$ ($f_{ck} \leq 50$ MPa) $f_{ctm} = 1{,}1 f_{ck}^{1/3}$ ($f_{ck} > 50$ MPa)
$f_{ctk;0,05}$	1,1	1,3	1,5	1,8	2,0	2,2	2,5	2,7	2,9	2,9	3,0	3,2	3,3	3,5	3,6	$f_{ctk;0,05} = 0{,}7 f_{ctm}$ (5 %-fractile)
$f_{ctk;0,95}$	2,0	2,5	2,9	3,3	3,8	4,2	4,6	4,9	5,3	5,4	5,6	5,9	6,2	6,4	6,6	$f_{ctk;0,95} = 1{,}3 f_{ctm}$ (95 %-fractile)

NOTE 1 All strength classes apply unless the National Annex excludes specific classes.

NOTE 2 Intermediate strength classes can be used, if included in the National Annex.

NOTE 3 The relationship between cube strength and cylinder strength is covered in EN 206.

NOTE 4 For restrictions on the scope of application of Table 5.1 to lightweight concrete or recycled aggregate concrete, refer to Annex M and Annex N, respectively.

NOTE 5 The design clauses of SLS and for minimum reinforcement make allowance for an over-strength of tensile strength in a class between $f_{ctk;0,05}$ and $f_{ctk;0,95}$.

(4) If required, the concrete compressive strength $f_{ck}(t)$, should be specified for times t that can be before or after t_{ref} for a number of stages (e.g. demoulding, removal of propping, transfer of prestress).

(5) Where concrete tensile strength is tested and documented at the same frequency as for compressive strength, a statistical analysis of test results may be used as a basis for the evaluation of the tensile strength $f_{ctk,0,05}$, $f_{ctk,0,95}$ and f_{ctm}, as an alternative to Table 5.1.

(6) Unless verified by testing, the development of concrete strength with time, temperature, curing conditions and binder composition should be estimated according to Annex B.

5.1.4 Elastic deformation

(1) The values of the elastic modulus of concrete should be specified and/or determined by testing if the structure is likely to be sensitive to deviations from the approximate indicative values given in (2).

NOTE 1 The elastic deformations of concrete largely depend on its composition (especially the aggregates).

(2) Approximate indicative values for the modulus of elasticity E_{cm} may be taken as:

$$E_{cm} = k_E \cdot f_{cm}^{1/3} \tag{5.1}$$

For concrete with quartzite aggregates $k_E = 9\,500$ may be assumed. For other types of aggregates k_E can vary between 5 000 and 13 000.

NOTE 2 The National Annex can specify values k_E to be used in the country.

NOTE 3 E_{cm} corresponds to the secant modulus between $\sigma_c = 0$ and $\sigma_c = 0{,}4f_{cm}$.

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

5.1.5 Creep and shrinkage

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

$$\varepsilon_{cc}(t,t_0) = \varphi(t,t_0) \cdot (\sigma_c/E_{c,28}) \tag{5.2}$$

where

$E_{c,28}$ is the tangent modulus of elasticity, which may be taken as $1{,}05E_{cm}$, or calculated more accurately from Formula (B.21) or determined by testing.

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

(2) If the compressive stress of concrete at an age t_0 does not exceed $0{,}40f_{cm}(t_0)$ under the quasi-permanent combination of actions and where great accuracy is not required, the values given in Table 5.2 may be considered as the creep coefficient $\varphi(50\text{y},t_0)$ for plain concrete. Where more accurate predictions of creep are needed or where the design service life of a structure is greater than 50 years, Annex B should be used.

NOTE 2 Annex B provides further information and guidance, including the development of the modulus of elasticity with time and the development of creep with time.

Table 5.2 — Creep coefficient $\varphi(50\text{y},t_0)$ of plain concrete after 50 years of loading

Age at loading t_0 [days] for strength development classes of concrete[a]			**Dry atmospheric conditions (RH = 50 %)**				**Humid atmospheric conditions (RH = 80 %)**			
			h_n[b] [mm]				h_n[b] [mm]			
CS	**CN**	**CR**	**100**	**200**	**500**	**1 000**	**100**	**200**	**500**	**1 000**
3	**1**	**1**	4,2	3,8	3,4	3,1	3,0	2,8	2,6	2,5
10	**7**	**3**	3,1	2,8	2,5	2,3	2,2	2,1	2,0	1,9
32	**28**	**23**	2,4	2,2	1,9	1,8	1,7	1,6	1,6	1,5
91	**91**	**91**	1,9	1,7	1,5	1,4	1,4	1,3	1,2	1,2
365	**365**	**365**	1,4	1,3	1,1	1,0	1,0	0,9	0,9	0,8
Correction exponent A			0,82	0,79	0,75	0,72	0,71	0,68	0,66	0,64

NOTE 1 For geometries outside the given range of notional size, Annex B should be used.

NOTE 2 The values of the creep coefficient apply to $f_{ck,28}$ = 35 MPa. For other strength in the range of 12 MPa $\leq f_{ck,28} \leq$ 100 MPa, the values should be multiplied by the factor $(35/f_{ck})^A$ where A is the correction exponent in the table, considering the effect of the concrete strength class.

NOTE 3 The creep coefficients are mean values with a coefficient of variation of about 30 %.

[a] Classes CS, CN and CR stand for slow, normal and rapid strength development of concrete, respectively, see B.3(1).

[b] h_n 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.

(3) When the compressive stress at age t_0 exceeds the value $0{,}40f_{cm}(t_0)$ under the quasi-permanent combination of actions then creep non-linearity should be considered. In such cases the non-linear notional creep coefficient should be obtained according to Annex B.

(4) Values for total shrinkage after 50 years are given in Table 5.3 (NDP) for plain concrete. Where a more accurate shrinkage prediction is required, Annex B should be used.

NOTE 3 The values in Table 5.3 (NDP) apply unless the National Annex gives different values.

Table 5.3 (NDP) — Nominal total shrinkage values $\varepsilon_{cs,50y}$ [‰] for concrete after a duration of drying of 50 years

Strength development of concrete[a]	$f_{ck,28}$	Dry atmospheric conditions ($RH = 50$ %) h_n [mm]				Humid atmospheric conditions ($RH = 80$ %) h_n [mm]			
		100	**200**	**500**	**1 000**	**100**	**200**	**500**	**1 000**
Class CS	**20**	0,57	0,56	0,48	0,36	0,33	0,32	0,28	0,21
	35	0,53	0,51	0,45	0,35	0,31	0,31	0,27	0,22
	50	0,49	0,48	0,43	0,35	0,30	0,29	0,27	0,23
Class CN	**20**	0,67	0,65	0,56	0,41	0,38	0,37	0,32	0,24
	35	0,60	0,59	0,51	0,39	0,34	0,34	0,30	0,24
	50	0,55	0,54	0,48	0,37	0,31	0,31	0,28	0,23
	80	0,48	0,48	0,43	0,36	0,30	0,30	0,28	0,25
Class CR	**35**	0,76	0,74	0,65	0,48	0,42	0,41	0,36	0,28
	50	0,67	0,66	0,58	0,44	0,36	0,35	0,32	0,26
	80	0,55	0,54	0,49	0,39	0,31	0,30	0,28	0,25

NOTE 1 The values are mean values with a coefficient of variation of about 30 %.

NOTE 2 The notional size h_n is defined in the notes to Table 5.2.

[a] Classes CS, CN and CR stand for slow, normal and rapid strength development of concrete, respectively, see B.4(1).

5.1.6 Design assumptions

(1) The value of the design compressive strength shall be taken as:

$$f_{cd} = \eta_{cc} \cdot k_{tc} \frac{f_{ck}}{\gamma_C} \tag{5.3}$$

where

η_{cc} is a factor to account for the difference between the undisturbed compressive strength of a cylinder and the effective compressive strength that can be developed in the structural member. It shall be taken as:

$$\eta_{cc} = \left(\frac{f_{ck,ref}}{f_{ck}}\right)^{\frac{1}{3}} \leq 1{,}0 \tag{5.4}$$

k_{tc} is a factor considering the effect of high sustained loads and of time of loading on concrete compressive strength.

NOTE 1 The following values apply unless the National Annex gives different values:

— $f_{ck,ref}$ = 40 MPa;

— k_{tc} = 1,00 for t_{ref} ≤ 28 days for concretes with classes CR and CN and t_{ref} ≤ 56 days for concretes with class CS where the design loading is not expected for at least 3 months after casting;

— k_{tc} = 0,85 for other cases including when f_{ck} is replaced by f_{ck}(t) in accordance with 5.1.3(4).

(2) The value of the design tensile strength f_{ctd} shall be taken as:

$$f_{ctd} = k_{tt}\frac{f_{ctk,0,05}}{\gamma_C} \tag{5.5}$$

where

k_{tt} is a factor considering the effect of high sustained loads and of time of loading on concrete tensile strength.

NOTE 2 The value is $k_{tt} = 0{,}80$ for $t_{ref} \leq 28$ days for concretes with classes CR an CN and $t_{ref} \leq 56$ days for concretes with class CS, and $k_{tt} = 0{,}70$ for other cases including when $f_{ck}(t)$ is determined in accordance with 5.1.3(4) unless the National Annex gives different values.

(3) The relation between compressive stress σ_c and strain ε_c shown in Figure 5.1 and described by the Formula (5.6) may be used to model the response of concrete to short term uniaxial compression.

$$\frac{\sigma_c}{f_{cm}} = \frac{k\eta - \eta^2}{1 + (k-2)\eta} \tag{5.6}$$

$$k = 1{,}05\ E_{cm} \cdot \varepsilon_{c1}/f_{cm} \tag{5.7}$$

$$\eta = \varepsilon_c/\varepsilon_{c1} \tag{5.8}$$

$$\varepsilon_{c1}\ [‰] = 0{,}7 f_{cm}^{1/3} \leq 2{,}8\ ‰ \tag{5.9}$$

$$\varepsilon_c < \varepsilon_{cu1}\ [‰] = 2{,}8 + 14 \cdot (1 - f_{cm}/108)^4 \leq 3{,}5\ ‰ \tag{5.10}$$

NOTE 3 Simplified stress distributions in cross-sections used to determine the resistance to axial and flexural effects at the ultimate limit state are provided in 8.1.2.

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

(5) Unless more precise values are available, the mean unit weight of normal weight reinforced concrete for the purposes of design may be taken as 25 kN/m^3, and for plain normal weight concrete as 24 kN/m^3.

(6) Unless more accurate information is available, the linear coefficient of thermal expansion may be taken as $\alpha_{c,th}$ = 10 · 10^{-6} °C^{-1}.

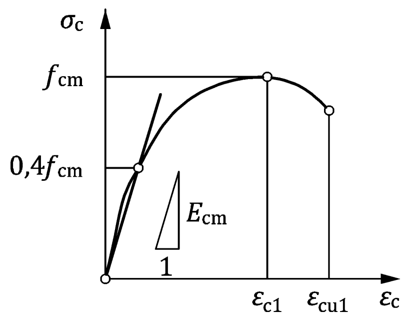


Figure 5.1 — Stress-strain relation for concrete in compression

5.2 Reinforcing steel

5.2.1 General

(1) 5.2 gives provisions for the following types of carbonreinforcing steels suitable for design of concrete structures in accordance with this Eurocode:

— weldable ribbed and indented reinforcing steel, in the form of bars including de-coiled bars;

— weldable ribbed and indented reinforcing steel in the form of welded fabric and lattice girders.

NOTE 1 Annex I gives information for assessment of existing structures with plain (smooth) bars.

NOTE 2 For additional or modified rules for stainless reinforcing steel see Annex Q.

(2) The rules in this Eurocode may be used for:

— ribbed reinforcement with $\phi \leq 50$ mm;

— indented reinforcement with $\phi \leq 14$ mm.

(3) The requirements for the properties of the reinforcing steel apply for the material as placed in the finished structure. If workshop processing or site operations can affect the properties of the reinforcing steel, then those properties shall be verified after such operations.

(4) The requirements for the properties of the reinforcing steel apply for service temperatures in the range of -40 °C to +100 °C unless otherwise specified by the relevant authority, or where not specified, as agreed for a specific project by the relevant parties.

(5) Reinforcing steels used for structures in accordance with this Eurocode shall comply with the relevant standards for reinforcing steel.

NOTE 3 The National Annex can specify relevant standards for reinforcing steel.

NOTE 4 The harmonised product standard EN10080 for reinforcing steel is currently under development .

5.2.2 Properties

(1) Specified properties of reinforcing steel that are required for design to this Eurocode shall include at least:

— strength class in accordance with Table 5.4;

— ductility class in accordance with Table 5.5;

— diameter or size.

(2) The following properties of reinforcing steel may be derived in accordance with the provisions of 5.2.2:

— yield strength f_{yk} or $f_{0,2k}$;

— stress ratio $(f_t/f_y)_k$;

— elongation at maximum load ε_{uk};

— *S-N* curves for fatigue, see Annex E.

(3) Reinforcing steel suitable for design of concrete structures in accordance with this Eurocode shall satisfy the requirements of C.4.

Table 5.4 — Strength classes of reinforcing steel

Properties for stress-strain-diagram (Fig. 5.2)	Reinforcing steel strength class					
	B400	**B450**	**B500**	**B550**	**B600**	**B700**
Characteristic value f_{yk} [MPa]	400	450	500	550	600	700
NOTE All strength classes apply unless the National Annex excludes specific classes. Intermediate strength classes can be used, if included in the National Annex.						

Table 5.5 — Ductility classes of reinforcing steel

Properties for stress-strain-diagram (Fig. 5.2)	Reinforcing steel ductility class		
	A	**B**	**C**
Characteristic value of $k = (f_t/f_y)_k$	1,05	1,08	1,15 to 1,35
Characteristic strain at maximum force ε_{uk}	2,5 %	5,0 %	7,5 %

5.2.3 Welding of reinforcing bars

(1) Welding of reinforcing bars shall be designed and detailed, and welds shall be specified in the execution specification.

(2) When the loading is predominantly static, the welding should be carried out in accordance with EN ISO 17660.

5.2.4 Design assumptions

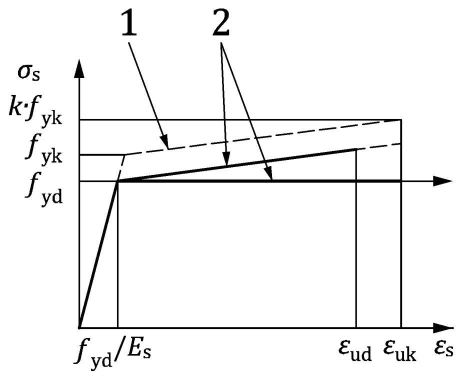
(1) Design shall be based on the nominal cross-section area of the reinforcing steel and the design values derived from the characteristic values given in 5.2.2 with:

$$f_{yd} = f_{yk}/\gamma_S \text{ and } f_{yd} = f_{0,2k}/\gamma_S\text{, respectively} \tag{5.11}$$

(2) For design either of the following assumptions may be made (see Figure 5.2):

a) an inclined post-elastic branch with a strain limit of $\varepsilon_{ud} \leq \varepsilon_{uk}/\gamma_S$ and a maximum stress of $k \cdot f_{yk}/\gamma_S$ at ε_{uk}, where $k = (f_t/f_y)_k$;

b) a horizontal post-elastic branch without strain limit.



Key

1 nominal diagram for reference

2 design diagrams

Figure 5.2 — Stress-strain diagrams for carbon reinforcing steel (for tension and compression)

(3) The design value of the modulus of elasticity E_s may be assumed to be 200 000 MPa for weldable reinforcing steel unless more precise values are known.

(4) The mean unit weight of reinforcing steel for the purposes of design may be taken as 78,5 kN/m^3.

(5) The coefficient of thermal expansion may be taken as $\alpha_{s,th} = 10 \cdot 10^{-6}$ °C^{-1} for weldable reinforcing steel unless more precise values are known.

5.2.5 Reinforcement bar couplers

(1) Couplers for splicing of reinforcing bars shall be capable of developing strength and ductility of the reinforcing bar as defined at C.6.

5.2.6 Headed bars for reinforcement

(1) Headed bars shall meet the performance requirements of C.7.

5.3 Prestressing steel

5.3.1 General

(1) 5.3 gives provisions for prestressing steel in the form of wires, strands and bars suitable for design of concrete structures in accordance with this Eurocode.

(2) The requirements for the properties of the prestressing steel apply for the material as placed in the finished structure. Design properties given in 5.3 may be used for service temperatures in the range of -40 °C to +100°C except for relaxation as noted in B.9(6) and unless otherwise specified by the relevant authority, or where not specified, as agreed for a specific project by the relevant parties.

(3) Prestressing steels used for structures designed in accordance with this Eurocode shall comply with the relevant standards for prestressing steel.

NOTE 1 The National Annex can specify relevant standards for prestressing steel.

NOTE 2 The harmonised product standard EN 10138 for prestressing steels is currently under development.

5.3.2 Properties

(1) Specified properties of prestressing steel that are required for design to this Eurocode shall include at least:

— strength class in accordance with Table 5.6 unless different values are given in the relevant standard for prestressing steel;

— product type wire, strand or bar;

— diameter or size.

(2) The following properties of prestressing steel may be derived in accordance with the provisions of 5.3, Annex B.9 and Annex E:

— 0,1 % proof stress $f_{p0,1k}$;

— tensile strength f_{pk};

— stress ratio $(f_p/f_{p0,1})_k$;

— elongation at maximum load ε_{uk};

— relaxation properties according to B.9;

— *S*-*N* curves for fatigue according to Annex E.

Table 5.6 — Strength classes of prestressing steel

Properties in stress-strain-diagram (Fig. 5.3) (characteristic values)	**(a) Wires**[a]			
	Y1570	**Y1670**	**Y1770**	**Y1860**
proof stress $f_{p0,1k}$ [MPa]	1 380	1 470	1 550	1 650
tensile strength f_{pk} [MPa]	1 570	1 670	1 770	1 860
	(b) Strands[a]			
	Y1770	**Y1860**	**Y1960**	**Y2060**
proof stress $f_{p0,1k}$ [MPa]	1 560	1 640	1 740	1 830
tensile strength f_{pk} [MPa]	1 770	1 860	1 960	2 060
	(c) Bars[a]			
	Y1030	**Y1050**	**Y1100**	**Y1230**
proof stress $f_{p0,1k}$ [MPa]	835	950	900	1 080
tensile strength f_{pk} [MPa]	1 030	1 050	1 100	1 230

NOTE 1 All strength classes apply unless the National Annex excludes specific classes. Intermediate strength classes can be used, if included in the National Annex.

NOTE 2 For requirements to classify steel products see C.5.

[a] In all strength classes, stress ratio of $k = (f_p/f_{p0,1})_k \geq 1{,}1$ and characteristic strain at maximum force $\varepsilon_{uk} = 3{,}5$ % apply.

(3) Prestressing steel suitable for design of concrete structures in accordance with this Eurocode shall satisfy the requirements of C.5.

(4) More accurate information based on production data of the prestressing steel may be used in design such as e.g. stress-strain diagrams, modulus of elasticity and isothermal stress relaxation.

5.3.3 Design assumptions

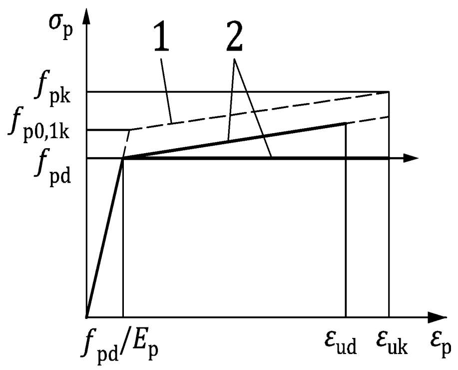
(1) Design shall be based on the nominal cross-section area of the prestressing steel and the design values derived from the characteristic values given in 5.3.2 with:

$$f_{pd} = f_{p0,1k}/\gamma_S \tag{5.12}$$

(2) For design, either of the following assumptions may be made (see Figure 5.3):

a) an inclined post-elastic branch, with a strain limit $\varepsilon_{ud} \leq \varepsilon_{uk}/\gamma_S$ and a maximum stress of f_{pk}/γ_S at ε_{uk};

b) or a horizontal post-elastic branch without strain limit.



Key

1 nominal diagram for reference

2 design diagrams

Figure 5.3 — Stress-strain diagrams for prestressing steel (only tension)

(3) The design value for the modulus of elasticity E_p may be assumed to be 200 000 MPa. If more accurate values are required, values based on testing should be used.

NOTE The actual value of the E-modulus for wires is 205 000 MPa. For strands it can range from 190 000 MPa to 200 000 MPa, depending on the manufacturing process and the geometrical configuration of the prestressing steel. For bars, the modulus is 205 000 MPa however, the secant modulus between 0 and $0,7f_{pk}$. can be as low as 170 000 MPa depending on the manufacturing process.

(4) The mean unit weight of prestressing steel for the purposes of design may be taken as 78,5 kN/m^3.

(5) The coefficient of thermal expansion may be taken as $\alpha_{s,th} = 10 \cdot 10^{-6}$ °C^{-1} for prestressing steel unless more precise values are known.

(6) The relaxation loss may be obtained either from Table B.4, from the relevant standard for prestressing steel or may be based on testing and adapted for the effects of initial stress and time in accordance with Annex B.9 where required.

5.4 Prestressing systems

5.4.1 General

(1) Prestressing systems including tendon anchorage assemblies and tendon coupler assemblies used for design of concrete structures in accordance with this Eurocode shall comply with the relevant standards for prestressing systems.

NOTE 1 The National Annex can specify relevant standards for prestressing systems.

NOTE 2 EAD 160004 is available for the assessment and determination of properties of prestressing systems required for design to this Eurocode.

(2) As a minimum the following information shall be provided in the technical documentation of the post-tensioning systems as basis for the design of concrete structures in accordance with this Eurocode:

— product type and properties of prestressing steel;

— types of applications of tendons;

— types and dimensions of ducts, anchorages and tendon couplers;

— arrangement and detailing of tendon supports;

— minimum spacing and edge distances of anchorages and tendon couplers with corresponding local anchorage zone reinforcement as a function of concrete strength;

— minimum radii of tendon curvature;

— friction coefficients;

— anchorage seating;

— details for corrosion protection as a function of the tendon protection level.

(3) Prestressing tendons shall be adequately and permanently protected against corrosion along the tendon length and at anchorages.

(4) Internal bonded post-tensioning tendons may be provided with different levels of corrosion protection:

— Protection Level 1: Tendon grouted within a metal duct;

— Protection Level 2: Tendon fully encapsulated within a grouted polymer duct and anchorage caps;

— Protection Level 3: Tendon fully encapsulated within a grouted polymer duct and anchorage caps and encapsulation monitorable with electrical resistance measurement or equivalent methods.

NOTE 3 Guidance for the assessment of tendon protection levels and polymer ducts can be found in *fib* Bulletin 75.

5.4.2 Anchorage zones

(1) The strength of the anchorage and coupling devices and anchorage zones shall be sufficient for the transfer of the design tendon force at ULS to the concrete. The formation of cracks in the anchorage zones shall not impair the function of the anchorage and coupling device. These requirements may be assumed to be complied with if:

— detailing of the local anchorage zones is in accordance with the technical documentation of a post-tensioning system which complies with 5.4.1(1);

— design and detailing of the general anchorage zones is in accordance with 11.6.4.

6 Durability and concrete cover

6.1 General

(1) A durable structure shall meet the requirements of serviceability, strength and stability throughout its design service life, without significant loss of utility, with anticipated maintenance but without major repair being necessary, in line with the general requirements of EN 1990.

(2) The required protection against deterioration of the structure shall be established by considering environmental conditions, intended use, design service life, maintenance programme and actions.

(3) Concrete structures complying with the provisions of this standard, EN 206 and EN 13670 are assumed to be sufficiently durable. Requirements for durability should be considered in addition to SLS and ULS.

NOTE 1 For the purpose of assessing durability, end of design service life is considered to have been reached when there is corrosion attack in the reinforcement and/or loss of concrete thickness or strength of the structural element, to an extent that impairs its performance.

NOTE 2 Corrosion attack can be judged by loss of bar diameter in the case of carbonation attack and pitting depth in the reinforcement in the case of chloride attack.

(4) The possible significance of direct and indirect actions (like temperature, shrinkage and creep), environmental conditions and consequential effects shall be considered.

(5) Corrosion protection of steel reinforcement should be provided by controlling quality of concrete, thickness and extent of cracking of the concrete in the cover zone.

(6) Where metal fastenings are not fully embedded in concrete with sufficient cover, they should be of corrosion resistant material. Where it is possible to inspect and replace or repair them, fastenings with protective coating may be used in exposed conditions.

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

6.2 Requirements for durability

(1) In order to achieve the required design service life of the structure, adequate measures shall be taken to protect each concrete member against the relevant environmental actions.

(2) The requirements for durability shall be considered at all stages, including:

— structural conception;

— material selection;

— construction details;

— execution;

— quality management;

— life-time maintenance.

(3) Special measures may be considered (e.g. coatings of concrete surface, cathodic protection, corrosion inhibitors or reinforcing steel with metallic or non-metallic coatings). For such situations, the effects on all relevant material properties and design parameters should be considered, including bond. For stainless steel, the provisions in Annex Q shall be used.

6.3 Environmental exposure conditions

(1) The environmental exposure conditions are those chemical, physical and biological conditions to which the structure is exposed (additional to the mechanical actions). The exposure conditions can be different on the various surfaces or elevations of a concrete member, and consequently while the concrete shall meet the relevant requirements, the requirements for cover to the reinforcement and the limitation of cracking may be different.

(2) All relevant forms of attack on the structure shall be taken into account. The forms of attack include:

— alkali-aggregate reaction (AAR);

— biological attacks arising from e.g.:

- algae;
- vegetation;

— chemical attacks arising from e.g. from soil, ground water or the use of the structure (storage of liquids, etc.):

- acid solutions;
- soft water;
- sulfates;
- other chemicals;

— delayed ettringite formation (DEF);

— physical attack, arising from e.g.:

- abrasion;
- temperature change (including freeze/thaw);
- water penetration;

— reinforcement corrosion due to carbonation or chlorides ingress;

— reinforcement corrosion that can be due to chlorides present in concrete before exposure;

— stress corrosion cracking.

(3) Table 6.1 defines exposure classes X for the most common environmental exposure conditions.

NOTE 1 Guidance for the selection of the exposure classes (e.g. the duration of the exposure, limits for RH in the XC classes or limits for the chloride contents in the XD and XS classes as well as classification of the freeze/thaw climate in the XF classes and any additional categories) can be found in the place of use of the concrete, based on local geographical and climatic conditions, the operational practices and levels of protection at national level.

NOTE 2 Table 6.1 provides examples where exposure classes occur unless the National Annex provides other examples or requirements for the selection of exposure classes.

Table 6.1 — Exposure classes related to environmental conditions

Class	Description of the environment	Informative examples where exposure classes can occur (NDP)
1. No risk of corrosion or attack		
For concrete without reinforcement or embedded metal:		
X0	All exposures except where there is freeze/thaw, abrasion or chemical attack.	Plain concrete members without any reinforcement.
2. Corrosion of embedded metal induced by carbonation		
Where concrete containing steel reinforcement or other embedded metal is exposed to air and moisture, the exposure should be classified as follows:		
XC1	Dry.	Concrete inside buildings with low air humidity, where the corrosion rate will be insignificant.
XC2	Wet or permanent high humidity, rarely dry.	Concrete surfaces subject to long-term water contact or permanently submerged in water or permanently exposed to high humidity; many foundations; water containments (not external). NOTE 1 Leaching could also cause corrosion (see (5), and (6), XA classes).
XC3	Moderate humidity.	Concrete inside buildings with moderate humidity and not permanent high humidity; External concrete sheltered from rain.
XC4	Cyclic wet and dry.	Concrete surfaces subject to cyclic water contact (e.g. external concrete not sheltered from rain as walls and facades).
3. Corrosion of embedded metal induced by chlorides, excluding sea water		
Where concrete containing steel reinforcement or other embedded metal is subject to contact with water containing chlorides, including de-icing salts, from sources other than from sea water, the exposure should be classified as follows:		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides.
XD2	Wet, rarely dry.	Swimming pools; Concrete components exposed to industrial waters containing chlorides. NOTE 2 If the chloride content of the water is sufficiently low then XD1 applies.
XD3	Cyclic wet and dry.	Parts of bridges exposed to water containing chlorides; Concrete roads, pavements and car park slabs in areas where de-icing agents are frequently used.

Class	Description of the environment	Informative examples where exposure classes can occur (NDP)
4. Corrosion of embedded metal induced by chlorides from sea water		
Where concrete containing steel reinforcement or other embedded metal is subject to contact with chlorides from sea water or air carrying salt originating from sea water, the exposure should be classified as follows:		
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 and structures in seawater.
XS3	Tidal, splash and spray zones.	Parts of marine structures and structures temporarily or permanently directly over sea water.
5. Freeze/Thaw Attack		
Where concrete is exposed to significant attack by freeze/thaw cycles whilst wet, the exposure should be classified as follows. A XF-classification is not necessary in cases where freeze/thaw cycles are rare.		
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 and freezing; concrete surfaces exposed to direct spray containing de-icing agents and freezing; splash zone of marine structures exposed to freezing.
6. Chemical attack		
Where concrete is exposed to chemical attack from natural soils and ground water, the exposure should be classified as follows:		
XA1	Slightly aggressive chemical environment.	Natural soils and ground water according to Table 6.2.
XA2	Moderately aggressive chemical environment.	Natural soils and ground water according to Table 6.2.
XA3	Highly aggressive chemical environment.	Natural soils and ground water according to Table 6.2.
7. Mechanical attack of concrete by abrasion		
Where concrete is exposed to mechanical abrasion, the exposure should be classified as follows:		
XM1	Moderate abrasion.	Members of industrial sites frequented by vehicles with pneumatic tyres.
XM2	Heavy abrasion.	Members of industrial sites frequented by fork lifts with pneumatic or solid rubber tyres.
XM3	Extreme abrasion.	Members of industrial sites frequented by fork lifts with elastomer or steel tyres or track vehicles.

(4) In addition to the exposure conditions in Table 6.1, the other particular forms of aggressive or indirect action according to (2) and (6) shall be considered.

(5) Classification of the exposure with respect to natural chemical attack from the soil and ground water should refer to Table 6.2, the validity of which is ensured in the case of natural soil and ground water at water/soil temperatures between 5 °C and 25 °C and a water velocity sufficiently slow to approximate to static conditions. The most onerous value for any single chemical characteristic determines the class. Where two or more aggressive characteristics lead to the same class, the environment should be classified into the next higher class unless a special study for this specific case proves that it is not necessary.

NOTE 3 Guidance on the selection of exposure classes for chemical attack from natural soil and ground water and any additional categories can be found in the place of use of the concrete based on local ground conditions and levels of protection at national level.

NOTE 4 The exposure classes of Table 6.2 apply unless the National Annex specifies additional exposure classes and limiting values for chemical attack.

(6) In the case of chemical attack, a special study may be needed to establish the relevant exposure conditions and appropriate protective measures, where there is:

— values outside the limiting values of Table 6.2;

— leaching, e.g. due to long-term contact to (soft) water or other liquids (see also XC2);

— other aggressive chemicals;

— chemically polluted ground or water;

— high water velocity in combination with the chemicals in Table 6.2.

Table 6.2 — Limiting values for exposure classes for chemical attack from natural soil and ground water

Chemical characteristic	Reference test method	XA1	XA2	XA3
Groundwater				
SO_4^{2-} [mg/l]	EN 196-2	≥ 200 and ≤ 600	> 600 and ≤ 3 000	> 3 000 and ≤ 6 000
pH	ISO 4316	≤ 6,5 and ≥ 5,5	< 5,5 and ≥ 4,5	< 4,5 and ≥ 4,0
CO_2 [mg/l]	EN 13577	≥ 15 and ≤ 40	> 40 and ≤ 100	> 100 up to saturation
NH_4^+ [mg/l]	ISO 7150-1	≥ 15 and ≤ 30	> 30 and ≤ 60	> 60 and ≤ 100
Mg^{2+} [mg/l]	EN ISO 7980	≥ 300 and ≤ 1 000	> 1 000 and ≤ 3 000	> 3 000 up to saturation
Soil				
SO_4^{2-} [mg/kg][a] total	EN 196-2[b]	≥ 2 000 and ≤ 3 000[c]	> 3 000[c] and ≤ 12 000	> 12 000 and ≤ 24 000
Acidity according to Baumann Gully [ml/kg]	EN 16502	> 200	Not encountered in practice	

a	Clay soils with a permeability below 10^{-5} m/s may be moved into a lower class.
b	The test method prescribes the extraction of SO_4^{2-} by hydrochloric acid; alternatively, water extraction may be used, if experience is available in the place of use of the concrete.
c	The 3 000 mg/kg limit should be reduced to 2 000 mg/kg, where there is a risk of accumulation of sulfate ions in the concrete due to drying and wetting cycles or capillary suction.

6.4 Exposure resistance classes

(1) Exposure resistance classes ERC should be used to classify concrete with respect to resistance against corrosion induced by carbonation (class XRC) or by chlorides (class XRDS) and damage caused by freeze/thaw attack (XRF). Selection of concrete to resist deterioration and protect against corrosion for all those exposure classes (EC) that are relevant, should be based on the exposure resistance classes given in EN 206.

NOTE 1 As specified in EN 206, complemented by the provisions valid in the place of use, the ERC can be satisfied by compliance with relevant limiting values and/or, for some ERCs, by proving the performance in meeting specification of relevant physical characteristics determined using standardized test methods. In the event EN 206 does not refer to ERC, the National Annex or National Application Document to EN 206 can provide the necessary advice on how to implement ERC rules in a country.

NOTE 2 An informative Annex P provides an alternative approach to design cover for durability without use of Exposure Resistance Classes (ERC) based on EN 1992-1-1:2004. The use of ERC according to 6.4 applies unless the National Annex specifies use of Annex P..

(2) The structural design should be based on a realistic concrete strength class consistent with the required exposure resistance class ERC.

NOTE 3 The composition of the concrete, e.g. the type of binder and the water-binder ratio, affects simultaneously the durability, physical and mechanical properties of the concrete. A choice based on the protection of reinforcement can lead to higher strength classes than required for the structural design and hence, can require higher minimum reinforcement and other measures.

(3) Adequate durability may be assumed against corrosion caused by carbonation or chloride ingress where cover to reinforcement is selected appropriate to the exposure class, exposure resistance class and the design service life and not less than the minimum cover for durability $c_{min,dur}$ given in Table 6.3 (NDP) and Table 6.4 (NDP).

(4) Concrete shall be specified with a maximum permitted chloride content in accordance with the chloride content classes (Cl) in EN 206.

(5) Concrete to resist freeze-thaw attack shall be specified using XRF classes according to EN 206.

(6) Adequate durability against chemical attack may be assumed when concrete with a composition documented to be resistant against the potential deterioration mechanisms for the intended design service life is used. Otherwise additional protective measures should be taken, such as linings or durable/replaceable coating.

NOTE 4 Rules on concrete composition to resist the chemical attacks detailed in Table 6.2 are given in EN 206 or in its national application document.

(7) Adequate durability against mechanical attack may be assumed when concrete with a composition documented to be resistant against the potential abrasion for the intended design service life is used. Otherwise additional protective measures should be taken, such as sacrificial layers according to 6.5.2.2(6).

NOTE 5 Rules on concrete composition to resist the mechanical attacks detailed in Table 6.1 can be given in EN 206 or in its national application document.

6.5 Concrete cover

6.5.1 Nominal cover

(1) The nominal cover shall be specified in the execution specification. It is defined as a minimum cover, c_{min} (see 6.5.2), plus an allowance in design for deviation, Δc_{dev} (see 6.5.3):

$$c_{\mathrm{nom}} = c_{\mathrm{min}} + \Delta c_{\mathrm{dev}} \tag{6.1}$$

(2) Sufficient concrete cover shall be provided in order to ensure:

— the safe transmission of bond forces;

— the protection of the steel against corrosion (durability);

— an adequate fire resistance (see EN 1992-1-2, where axis distance is used).

(3) For bored piles and for diaphragm walls the nominal cover values for durability and bond of this Eurocode apply. The cover values according to EN 1536 and EN 1538 considering the type and deviations of execution should be checked additionally. The largest value according to this Eurocode and EN 1536 or EN 1538 applies.

6.5.2 Minimum cover

6.5.2.1 General

(1) The value for c_{min} shall satisfy the requirements for both bond and durability:

$$c_{\mathrm{min}} = \max\{c_{\mathrm{min,dur}} + \Sigma\Delta c;\ c_{\mathrm{min,b}};\ 10\ \mathrm{mm}\} \tag{6.2}$$

where

$c_{\mathrm{min,dur}}$ minimum cover required for environmental conditions, see 6.5.2.2;

$\Sigma\Delta c$ sum of the following applicable reductions and additions:

$\Delta c_{\mathrm{min,30}}$ reduction of minimum cover for structures with design life of 30 years or less, see 6.5.2.2(2);

$\Delta c_{\mathrm{min,exc}}$ reduction of minimum cover for superior compaction or improved curing, see 6.5.2.2(3);

$\Delta c_{\mathrm{min,p}}$ additional minimum cover for prestressing tendons, see 6.5.2.2(4);

$\Delta c_{\mathrm{dur,red1}}$ and $\Delta c_{\mathrm{dur,\,red2}}$
reduction of minimum cover for use of additional concrete protection or use of special measures for protection of reinforcing steel, see 6.5.2.2(5) and 6.5.2.2(9);

$\Delta c_{\mathrm{dur,abr}}$ additional minimum cover for abrasion, see 6.5.2.2(6);

$c_{\mathrm{min,b}}$ minimum cover for bond requirement, see 6.5.2.3.

(2) For concrete cast directly against soil surface, the minimum cover should be increased by Δc_{min} considering the increased uncertainty and variability of concrete and the reduced compaction against soil.

NOTE The increase of the cover is Δc_{min} = +5 mm for casting against a vertical soil surface and Δc_{min} = 0 mm for a horizontal soil ground surface unless the National Annex gives other values.

(3) Minimum cover to post-installed reinforcing bars with respect to the transfer of bond forces and durability is given in 6.5.2 and with respect to drilling and installation in 11.4.8(2).

6.5.2.2 Minimum cover for durability

(1) The minimum concrete covers $c_{min,dur}$ dependent on design service life, exposure class and exposure resistance class (ERC) are given in Table 6.3 (NDP) and Table 6.4 (NDP).

NOTE 1 The recommended minimum covers apply unless the National Annex gives other values.

NOTE 2 Additional and intermediate or a selection of exposure resistance classes can be applied according to the National Annex with correspondingly adjusted minimum cover for durability, provided they are based on the same methodology and give protection against deterioration consistent with that inherent in Table 6.3 (NDP) and Table 6.4 (NDP).

Table 6.3 (NDP) — Minimum concrete cover $c_{min,dur}$ for carbon reinforcing steel — Carbonation

ERC	Exposure class (carbonation)							
	XC1		XC2		XC3		XC4	
	Design service life (years)							
	50	100	50	100	50	100	50	100
XRC 0,5	10	10	10	10	10	10	10	10
XRC 1	10	10	10	10	10	15	10	15
XRC 2	10	15	10	15	15	25	15	25
XRC 3	10	15	15	20	20	30	20	30
XRC 4	10	20	15	25	25	35	25	40
XRC 5	15	25	20	30	25	45	30	45
XRC 6	15	25	25	35	35	55	40	55
XRC 7	15	30	25	40	40	60	45	60

NOTE 1 XRC classes for resistance against corrosion induced by carbonation are derived from the carbonation depth [mm] (characteristic value 90 % fractile) assumed to be obtained after 50 years under reference conditions (400 ppm CO_2 in a constant 65 %-*RH* environment and at 20 °C). The designation value of XRC has the dimension of a carbonation rate [mm/$\sqrt{}$(years)].

NOTE 2 The recommended minimum concrete cover values $c_{min,dur}$ assume execution and curing according to EN 13670 with at least execution class 2 and curing class 2.

NOTE 3 The minimum covers can be increased by an additional safety element $\Delta c_{dur,\gamma}$ considering special requirements (e.g. more extreme environmental conditions).

Table 6.4 (NDP) — Minimum concrete cover $c_{min,dur}$ for carbon reinforcing steel — Chlorides

ERC	Exposure class (chlorides)											
	XS1		XS2		XS3		XD1		XD2		XD3	
	Design service life (years)						Design service life (years)					
	50	100	50	100	50	100	50	100	50	100	50	100
XRDS 0,5	20	20	20	30	30	40	20	20	20	30	30	40
XRDS 1	20	25	25	35	35	45	20	25	25	35	35	45
XRDS 1,5	25	30	30	40	40	50	25	30	30	40	40	50
XRDS 2	25	30	35	45	45	55	25	30	35	45	45	55
XRDS 3	30	35	40	50	55	65	30	35	40	50	55	65
XRDS 4	30	40	50	60	60	80	30	40	50	60	60	80
XRDS 5	35	45	60	70	70	—	35	45	60	70	70	—
XRDS 6	40	50	65	80	—	—	40	50	65	80	—	—
XRDS 8	45	55	75	—	—	—	45	55	75	—	—	—
XRDS 10	50	65	80	—	—	—	50	65	80	—	—	—

NOTE 1 XRDS classes for resistance against corrosion induced by chloride ingress are derived from the depth of chlorides penetration [mm] (characteristic value 90 % fractile), corresponding to a reference chlorides concentration (0,6 % by mass of binder (cement + type II additions)), assumed to be obtained after 50 years on a concrete exposed to one-sided penetration of reference seawater (30 g/l NaCl) at 20 °C. The designation value of XRDS has the dimension of a diffusion coefficient $[10^{-13}\ m^2/s]$.

NOTE 2 The recommended minimum concrete cover values $c_{min,dur}$ assume execution and curing according to EN 13670 with at least execution class 2 and curing class 2.

NOTE 3 The minimum covers can be increased by an additional safety element $\Delta c_{dur,\gamma}$ considering special requirements (e. g. more extreme environmental conditions).

(2) For temporary structures or for structures with a design service life of 30 years or less, $c_{min,dur}$ for a design service life of 50 years according to Table 6.3 (NDP) and Table 6.4 (NDP) may be reduced by $-\Delta c_{min,30}$.

NOTE 3 The reduction of the cover is $-\Delta c_{min,30} \leq 5$ mm unless the National Annex gives a different value.

(3) The values of $c_{min,dur}$ given in Table 6.3 (NDP) and Table 6.4 (NDP) may be reduced $-\Delta c_{min,exc}$ under the following execution conditions:

(i) enhanced compaction of concrete can be ensured by geometrical characteristics, placement and curing (e.g. members with slab geometry with positions of reinforcement not affected by construction process);

(ii) or curing complies with at least curing Class 3 of EN 13670.

NOTE 4 The reduction of the cover is $-\Delta c_{min,exc} \leq 5$ mm unless the National Annex gives a different value.

(4) For prestressing tendons, pre- or post-tensioned, the cover values in Table 6.3 (NDP) and Table 6.4 (NDP) should be increased by $\Delta c_{min,p}$, except where the internal bonded post-tensioning systems are provided with protection level 2 or 3 according to 5.4.1, and internal unbonded prestressing tendons are encased in corrosion resistant sheaths.

NOTE 5 The addition to the cover is $\Delta c_{\mathrm{min,p}}$ = + 10 mm unless the National Annex gives a different value

(5) Where the concrete is provided with an additional protection (e.g. surface coating) the minimum cover values in Table 6.3 (NDP) and Table 6.4 (NDP) may be reduced by $\Delta c_{\mathrm{dur,red1}}$. The value to be used shall be established based on experience or testing in line with provisions valid in the project.

NOTE 6 The reduction of the cover is $\Delta c_{\mathrm{dur,red1}} \leq 10$ mm unless the National Annex gives a different value.

(6) For concrete abrasion according to XM classes according to Table 6.1, special attention should be given on the requirements to concrete mixes in EN 206. Optionally concrete abrasion may be allowed for by increasing the concrete cover (sacrificial layer) with $\Delta c_{\mathrm{dur,abr}}$.

NOTE 7 The following values of $\Delta c_{\mathrm{dur,abr}}$ for sacrificial layer apply unless the National Annex gives different values:

— for XM1: $\Delta c_{\mathrm{dur,abr}} = +5$ mm;

— for XM2: $\Delta c_{\mathrm{dur,abr}} = +10$ mm;

— for XM3: $\Delta c_{\mathrm{dur,abr}} = +15$ mm.

(7) For concrete surfaces subjected to abrasion from moving objects like vehicles and wheels and not protected by asphalt, or other protective layers, the selection of XM class and optional increase of concrete cover for abrasion should be considered based on an assessment taking account of factors such as annual daily traffic load, type of traffic, use of studded tyres as well as the concrete composition.

(8) Where insitu concrete is placed against other concrete elements (precast or insitu) the minimum concrete cover of the reinforcement for durability is not required provided that:

— the concrete strength is at least $f_{\mathrm{ck}} \geq 25$ MPa;

— the interface is rough see 8.2.6(6).

(9) Where special measures according to 6.2(3) other than coating of the concrete surface are taken, the minimum cover may be reduced by $\Delta c_{\mathrm{dur,red2}}$. The value to be used shall be established based on experience or testing in line with provisions valid in the project.

NOTE 8 The reduction of the cover $\Delta c_{\mathrm{dur,red2}} = 0$ unless a National Annex gives a different value.

6.5.2.3 Minimum cover for bond

(1) In order to transmit bond forces safely and to ensure adequate compaction of the concrete, the minimum cover should not be less than $c_{\mathrm{min,b}}$ given in Tables 6.5 and 6.6.

NOTE 1 The minimum concrete cover values $c_{\mathrm{min,b}}$ assume execution and curing according to EN 13670 with at least execution class 2 and curing class 2.

NOTE 2 For minimum concrete cover values $c_{\mathrm{min,b}}$ of pre-tensioning tendons see 13.5.1(2), Table 13.1.

NOTE 3 For minimum concrete cover values $c_{\mathrm{min,b}}$ of bonded post-installed reinforcing steel see 11.4.8, Table 11.2.

Table 6.5 — Minimum cover $c_{min,b}$ for reinforcing steel

Steel type	$c_{min,b}$ [a]
Separated bars	Diameter of bar
Bundled bars	Equivalent diameter ϕ_b (see 11.4.3)

[a] Where the specified maximum aggregate size D_{upper} is > 32 mm, the minimum cover $c_{min,b}$ should be increased by 5 mm.

Table 6.6 — Minimum cover $c_{min,b}$ for post-tensioning ducts

Duct type	$c_{min,b}$	
	with transverse reinforcement[a]	without transverse reinforcement[a]
Circular duct	$0{,}5\phi_{duct} \leq 80$ mm	$1{,}0\phi_{duct} \leq 80$ mm
Rectangular duct	$\geq \max\{a; b/3\}^b \leq 80$ mm	$\geq \max\{a; b/2\}^b \leq 80$ mm

[a] The transverse reinforcement should comply with the minimum secondary reinforcement in Table 12.2 and should be placed in the cover outside of the ducts.

[b] Where $a \leq b$ for a duct with cross-sectional dimensions a and b.

6.5.3 Allowance in design for deviation in cover

(1) To calculate the nominal cover c_{nom}, an addition to the minimum cover c_{min} according to Formula (6.2) shall be made in design to allow for the deviation Δc_{dev} which shall be taken as the absolute value of the accepted negative deviation specified in the execution specification, e.g. given on the construction drawings (see EN 13670). Values for Δc_{dev} are given in Table 6.7 (NDP).

NOTE Cases and values of Table 6.7 (NDP) apply unless the National Annex gives different cases and values.

Table 6.7 (NDP) — Allowance for deviation Δc_{dev}

	Case	Δc_{dev}
1	In general: for execution in tolerance class 1 according to EN 13670	10 mm
2	For execution in tolerance class 2 according to EN 13670	5 mm
3	Where fabrication is subjected to a quality assurance system, in which the systematic monitoring includes measurements of the cover	5 mm
4	Where it can be assured that an accurate measurement device is used for systematic monitoring and non conforming members are rejected (e.g. precast elements)	0 mm
5	For concrete members in exposure class XC1, where the risk of corrosion is insignificant	5 mm
6	For concrete cast against surfaces with exposed aggregate (e.g. interfaces)	5 mm
7	For concrete cast against unevenness due to formwork or excavation sheeting (e.g. ribbed finishes or architectural textures)	10 mm + dimension of unevenness
8	Concrete cast against prepared ground (including uneven blinding layer)[a]	40 mm[a]
9	Concrete cast directly against unprepared soil[a]	75 mm[a]
10	Post-installed reinforcing bars	5 mm or according to project specification

[a] These allowances for deviation Δc_{dev} also apply for bored piles and for diaphragm walls designed according to this Eurocode, unless the National Annex gives other special values.

7 Structural analysis

7.1 General

(1) The purpose of structural analysis is to establish the distribution of either internal forces, or stresses, strains and displacements, over the whole or part of a structure.

(2) Analyses shall be carried out using idealisations of both the structure and its geometry (see 7.2) and the behaviour of the structure (see 7.3). The idealisations selected shall be appropriate to the problem being considered.

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

(4) Common idealisations in behaviour of the structure used for analysis are:

— linear elastic behaviour (see 7.3.1);

— linear elastic behaviour with redistribution (see 7.3.2);

— plastic behaviour (see 7.3.3), including strut-and-tie models and stress field models (see 7.3.3.3);

— non-linear behaviour (see 7.3.4).

(5) Imperfections and second order effects shall be considered where they are significant (see 7.4).

(6) Structural analysis shall be performed consistently with the design. A specific stiffness (e.g. torsional) or restraint at supports (e.g. moment) may be reduced or neglected in the analysis for both ultimate and serviceability limit states if the member is designed consistently with these assumptions and the minimum reinforcement provisions for crack control and robustness are fulfilled.

(7) Local analyses may be necessary where the assumption that plane sections remain plane (linear strain distribution) is not valid.

NOTE Examples of such cases are:

— in the vicinity of supports;

— in the vicinity of concentrated loads;

— in beam-column intersections;

— in anchorage zones;

— at changes in cross-section.

7.2 Structural modelling for analysis

7.2.1 Geometric imperfections

7.2.1.1 General

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

(2) Deviations in cross-section dimensions which comply with tolerance class 1 of EN 13670 are considered in the partial factors for materials and generally may be neglected in structural analysis.

(3) Maximum deviations from theoretical geometry may be specified by the relevant authority or agreed for a specific project by the relevant parties. Where project specific execution specifications define stricter maximum deviations (e.g. when carrying out the assessment of an existing structure) as described in 7.2.1.2(1), the effect of imperfections should be based on these maximum deviations multiplied by 1,2.

(4) For members with an axial force, other than internal prestressing, imperfections and their effects shall be considered in ultimate limit states to determine the first order effects where relevant.

(5) Imperfections may be neglected for serviceability limit states.

7.2.1.2 Representation of imperfections

(1) The following provisions apply for members and structures with an axial force and execution deviations according to EN 13670. If project-specific execution specifications for maximum deviations apply, these provisions should be modified in accordance with 7.2.1.1(3).

(2) Imperfections may be represented by an inclination θ_i (see Figure 7.1), given by Formula (7.1):

$$\theta_{\mathrm{i}} = \alpha_{\mathrm{h}} \cdot \alpha_{\mathrm{m}} \cdot \frac{1}{200} \tag{7.1}$$

where

α_h is the reduction coefficient for length or height:

$$0{,}4 \le \alpha_{\mathrm{h}} = \frac{2}{\sqrt{l}} \le 1{,}0; \tag{7.2}$$

α_m is the reduction coefficient for the number of members:

$$\alpha_{\mathrm{m}} = \sqrt{0{,}5\left(1 + \frac{1}{m}\right)}; \tag{7.3}$$

l is the length or height [m], see (3);

m is the number of load bearing members in one section that bear a significant part of the vertical load and that, due to their inclination, contribute to the effect considered.

(3) In Formulae (7.2) and (7.3), the values of l and m should be taken as follows, depending on the effect considered:

— effect on individual member (see Figure 7.1 a)): l = actual length of member, m =1;

— effect on bracing system (see Figure 7.1 b)): l = height of the structure, m = number of vertical members between two adjacent levels contributing to the horizontal force on the bracing system;

— effect on intermediate or end diaphragms distributing the horizontal loads (see Figure 7.1 c)): l = storey height, m = number of vertical braced elements between two adjacent levels contributing to the total horizontal force on the section.

(4) Alternatively (e.g. arches), the imperfection of the structural geometry may be determined from the governing buckling mode. Each mode shape may be idealized by a sinusoidal profile. The amplitude should be taken as

$$a_{\mathrm{i}} = \theta_{\mathrm{i}} \cdot l_{\mathrm{aw}}/2 \tag{7.4}$$

where

l_{aw} is half of the wavelength of the buckling mode with the lowest buckling load.

(5) For individual members, the effect of imperfections may be taken into account in two alternative ways:

a) For members in statically determinate structures, as an eccentricity e_i given by:

$$e_{\mathrm{i}} = \theta_{\mathrm{i}} \cdot \frac{l_0}{2} \tag{7.5}$$

where l_0 is the effective length;

For walls and individual columns in braced systems, $e_i = l_0/400$ may always be used as a simplification, corresponding to $\alpha_h = 1{,}0$.

b) For members in both statically determinate and indeterminate structures as a fictitious transverse force $F_{H,i}$ in the position that gives maximum moment:

- for unbraced members (see Figure 7.1 a1)):

$$F_{\mathrm{H,i}} = \theta_{\mathrm{i}} \cdot F_{\mathrm{V,i}} \tag{7.6}$$

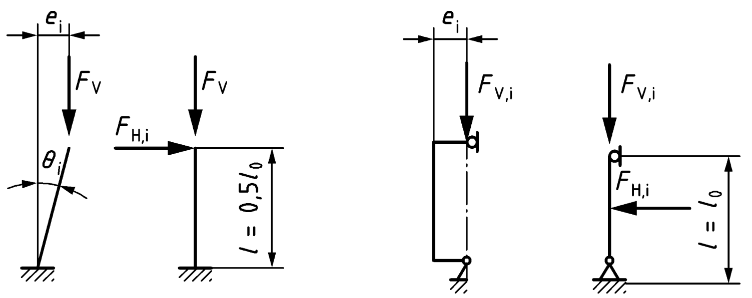
- for braced members (see Figure 7.1 a2)):

$$F_{\mathrm{H,i}} = 2 \cdot \theta_{\mathrm{i}} \cdot F_{\mathrm{V,i}} \tag{7.7}$$

where F_V is the vertical load.

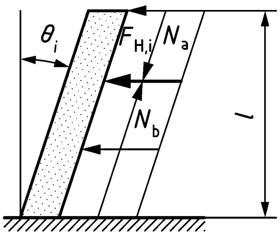
For walls and individual columns in braced systems $\theta_i = 1/200$ may always be used as a simplification, corresponding to $\alpha_h = 1$.

The force $F_{H,i}$ may be substituted by some other equivalent transverse action. Since the force $F_{H,i}$ is fictitious, it should not be added to the forces transmitted by the individual member to other members of the structure.

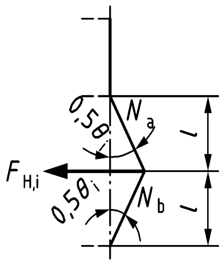


a1) unbraced a2) braced

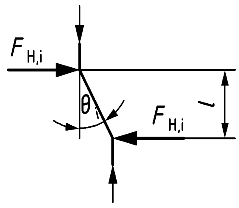
a) Individual members with eccentric axial force or lateral force



b) Bracing system
($m = 3$ i.e. 2 columns plus 1 bracing wall; l = height of structure)



c1) Intermediate diaphragm



c2) Top diaphragm

Key

l length of a member (length of structure in b))

Figure 7.1 — Examples of the effect of geometric imperfections

(6) For structures not part of a bridge, the effect of the inclination θ_i may be represented by the fictitious transverse forces, indicated in Formulae (7.8), (7.9) and (7.10), to be included in the analysis together with other actions. If the foundation is considered as a full diaphragm, equal and opposite forces should be applied at the level of foundations so that no reactions are transmitted to the foundations due to these fictitious forces.

Effect on bracing system (see Figure 7.1 b)):

$$F_{H,i} = \theta_i \cdot (N_b - N_a) \tag{7.8}$$

The overall effect of geometrical imperfections may be addressed by designing the structure to take account of equivalent horizontal loads acting at the centroid of the individual intermediate diaphragms. In this case, the load is determined according to Formula (7.8) by replacing ($N_b - N_a$) with the total vertical load acting on the actual intermediate diaphragm.

Effect on intermediate diaphragm (see Figure 7.1 c1)):

$$F_{H,i} = \theta_i \cdot (N_b + N_a)/2 \tag{7.9}$$

Effect on top diaphragm (see Figure 7.1 c2)):

$$F_{\mathrm{H,i}} = \theta_{\mathrm{i}} \cdot N_{\mathrm{a}} \tag{7.10}$$

where N_a and N_b are axial forces contributing to $F_{H,i}$.

In the determination of the first order internal forces due to the combination of actions in the persistent and transient or the accidental design situation the horizontal forces $F_{H,i}$ should be considered in the same load case as the corresponding axial force N.

7.2.2 Idealisation of the structure

(1) The structure should be idealized with suitable models considering static and geometrical boundary conditions as well as the transfer of support reactions.

(2) Significant asymmetry in geometry or loading should be considered either by a 3D-model or by adjusted planar models.

(3) Interaction of soil and structure should be considered appropriately and consistently with 4.2.1.4. Non-linear behaviour in the soil-structure interaction should be considered.

(4) Ribbed or waffle or void enclosing slabs may be treated as solid members 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 1 500 mm;

— the depth of the rib below the flange does not exceed 4 times its width, or the space between voids;

— transverse ribs are provided at a clear spacing not exceeding 10 times the overall depth of the slab. This applies also to the biggest horizontal void dimension; and

— the flange thickness is at least 1/10 of the clear distance between ribs, the smallest horizontal void dimension or 50 mm, whichever is the greater. The minimum flange thickness of 50 mm may be reduced to 40 mm where permanent blocks are incorporated between the ribs.

7.2.3 Geometric data

(1) The effect of non-uniform stress distribution across wide flanges in T-beams should be considered at ULS where brittle behaviour may be expected and at SLS where relevant (stress limitations, deflections).

(2) In the absence of a more detailed analysis, the effective width of the flange may be calculated based on the distance l_{0b} between points of zero moment. The length l_{0b} may be obtained from Figure 7.2 if all of the following conditions are fulfilled:

— loading is predominantly uniform;

— the cross section is constant;

— the length of the cantilever, l_3, is less than half the adjacent span; and

— the ratio of adjacent spans lies between 2/3 and 1,5.

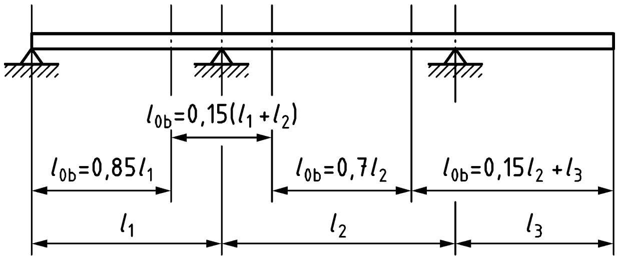


Figure 7.2 — Definition of l_{0b} for calculation of effective flange width

(3) In the absence of a more detailed analysis, the effective flange width b_{eff} for a T beam or L beam may be derived as:

$$b_{\mathrm{eff}} = \sum b_{\mathrm{eff,i}} + b_{\mathrm{w}} \leq b \tag{7.11}$$

where

$$b_{\mathrm{eff,i}} = \min\{0{,}2b_{\mathrm{i}} + 0{,}1l_{0\mathrm{b}};\ 0{,}2l_{0\mathrm{b}};\ b_{\mathrm{i}}\} \tag{7.12}$$

(for the notations see Figures 7.2 and 7.3)

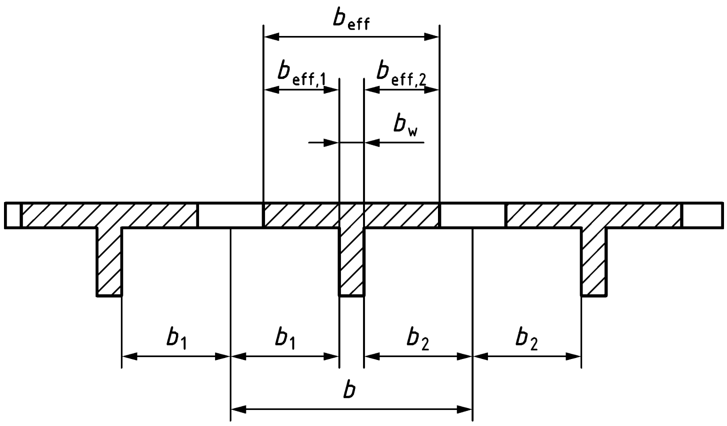


Figure 7.3 — Effective flange width parameters

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

(5) The span of a beam or slab should be the distance between centrelines of supporting members or bearings, in general. Reductions of this length may be permitted accounting for the support dimensions if the resulting eccentricities are accounted for in the design of supporting elements. Generally, the support reaction may be assumed to be distributed in a length equal to the minimum of the support width or the beam or slab height. For elastomeric bearings the stress should be assumed to be evenly distributed over the whole area of the support.

(6) Continuous beams and one-way slabs may be analysed assuming that the supports provide no rotational restraint.

(7) Where a supporting member is modelled as line or point support, the peak bending moment at the line or point support may be reduced based on the assumed distribution of the support reaction. For a uniformly distributed support reaction, the peak moment, may be reduced by an amount ΔM_{Ed} as follows:

$$\Delta M_{\mathrm{Ed}} = \frac{F_{\mathrm{Ed,sup}} \cdot t}{8} \tag{7.13}$$

where

$F_{\mathrm{Ed,sup}}$ is the design support reaction due to the loads applied on the beam or the slab;

t is the length according to (5) over which the reaction is distributed

7.3 Methods of analysis

7.3.1 Linear elastic analysis

(1) Linear analysis of structures and members based on the theory of elasticity may be used for the calculation of internal forces in both serviceability and ultimate limit states.

(2) Except when specified otherwise as e.g. in (3), (4) and (5), and for structures designed for earthquake resistance, linear elastic analysis may be carried out for the determination of the action effects assuming:

(i) uncracked cross-sections;

(ii) linear stress-strain relationships;

(iii) a mean value of the modulus of elasticity.

(3) A reduced stiffness may be considered in the analysis in member regions where cracking is expected under the relevant load combination. The assumed reduction of stiffness should be consistent with the amount of reinforcement provided in these regions. In case of continuous structures with prestressed and non-prestressed sections, the difference in stiffness between the prestressed and the non-prestressed areas should be considered in SLS.

(4) For the determination of the effect of imposed deformations at the serviceability limit state (SLS), cracking should be considered.

(5) When the effect of imposed deformations has to be considered at the ultimate limit state (ULS) (see 4.2.1.3(2)), a reduced stiffness due to cracking and creep and non-linear material behaviour may be assumed.

(6) Forces or stresses due to long-term differential settlements or shrinkage obtained by linear elastic analysis may be reduced to E_{t}, to account for creep relaxation:

$$E_{\mathrm{t}} = \frac{E_{t=0}}{1 + \chi\varphi(t,t_0)} \tag{7.14}$$

where

E_{t} is the internal force or stress at time t;

$E_{t=0}$ is the internal force or stress at the end of construction with no consideration for creep;

χ is the aging coefficient which may be taken equal to 0,8 for long term calculations;

t is the age of concrete when stresses are being evaluated;

t_0 is the age of concrete when the settlement occurred or when curing ended, as appropriate.

(7) Creep redistribution of internal forces or stresses due to a change in the support conditions (i.e. due to construction procedure) may be accounted for by Formula (7.15):

$$E_{\mathrm{t}} = E_{t=0} + (E_{\mathrm{wc}} - E_{t=0}) \frac{\varphi(t,t_0) - \varphi(t_{\mathrm{c}},t_0)}{1 + \chi\varphi(t,t_{\mathrm{c}})} \tag{7.15}$$

where

E_{t} and $E_{t=0}$ see (6);

E_{wc} is the internal force or stress assuming the structure was built with the final support conditions;

t_0 is the age of concrete when the loads producing the force or stress considered are applied;

t_{c} is the age of concrete when support conditions change.

7.3.2 Linear elastic analysis with redistribution

(1) Limited redistribution is allowed for braced slender structures. When redistribution of moments is applied its effects shall be considered on all aspects of design. The resulting distribution of internal forces and the reaction after redistribution shall remain in equilibrium with the applied loads.

(2) The amount of redistribution may be verified in two ways, without an explicit verification of the rotation capacity according (3) or with an explicit verification according (5).

(3) For elements in which second order effects are negligible, linear analysis with limited redistribution without explicit check on the rotation capacity may be applied to the analysis of structural members for the verification of ULS provided that the following conditions are fulfilled:

— all members are predominantly subjected to flexure (second order effects are negligible);

— in case of continuous beams or slabs, the ratio of lengths of adjacent spans is in the range of 0,5 to 2,0;

— the ratio δ_{M} of the moment after redistribution to the elastic bending moment complies with the value given by Formula (7.16):

$$\delta_{\mathrm{M}} \geq \frac{1}{1 + 0{,}7\varepsilon_{\mathrm{cu}} \cdot E_{\mathrm{s}}/f_{\mathrm{yd}}} + \frac{x_{\mathrm{u}}}{d} \tag{7.16}$$

$\geq$ 0,7 where Class B or Class C reinforcing steel or prestressing steel is used (see Table 5.5);

$\geq$ 0,8 where Class A reinforcing steel is used (see Table 5.5).

In case of prestressed members, f_{yd} in Formula (7.16) should be replaced by:

$$f = \frac{(f_{\mathrm{pd}} - \sigma_{\mathrm{pm},\infty})A_{\mathrm{p}} + f_{\mathrm{yd}}A_{\mathrm{s}}}{A_{\mathrm{p}} + A_{\mathrm{s}}} \tag{7.17}$$

where

$\sigma_{\mathrm{pm},\infty}$ is the long-term stress level in prestressing tendons at the state of zero (elastic) strain of the concrete at the same level.

All values refer to the section of the redistributed moment.

(4) The arrangement of reinforcement in flat slabs should reflect the behaviour under service conditions, normally leading to a concentration of reinforcement over the columns. The requirements for maximum spacing of flexural reinforcement for solid slabs in 12.4.1 apply.

(5) Linear analysis with redistribution with an explicit check on the rotation capacity may be applied for the verification of ULS provided that the rotation demand θ_{Ed} for the section with the plastic moment resistance is smaller than or equal to the rotation capacity θ_{Rd} of the section considered.

The design value of rotation demand follows from the integral of the curvatures after start of yielding in the section where cracking of concrete should be considered and the tensile strength and tension stiffening may be considered.

The rotation capacity may be derived from Formula (7.18).

$$\theta_{\mathrm{Rd}} = \frac{1{,}3d}{\gamma_{\theta}}\left(\left(\frac{1}{r}\right)_{\mathrm{u,m}} - TS_{\mathrm{My}}\frac{\varepsilon_{\mathrm{yd}}}{d-x}\right) \tag{7.18}$$

where

$$\left(\frac{1}{r}\right)_{\mathrm{u,m}} = TS_{\mathrm{Mu}} \cdot \min\left\{\frac{\varepsilon_{\mathrm{ud}}}{(d-x_{\mathrm{u}})};\frac{\varepsilon_{\mathrm{cu,d,\rho_w}}}{x_{\mathrm{u}}}\right\} \tag{7.19}$$

$$\varepsilon_{\mathrm{cu,d,\rho_w}} = 0{,}002 + \frac{1{,}35}{d} + 3\rho_w \leq 0{,}015 \tag{7.20}$$

$$TS_{\mathrm{Mu}} = 1 - \frac{1}{2\alpha}\cdot\left(1-\frac{\varepsilon_{\mathrm{yd}}}{\varepsilon_{\mathrm{ud,ef}}}\right) \text{ when } \alpha \geq 1 \tag{7.21}$$

$$TS_{\mathrm{Mu}} = \frac{\alpha}{2} + \frac{\varepsilon_{\mathrm{yd}}}{\varepsilon_{\mathrm{ud,ef}}}\left(1-\frac{\alpha}{2}+\left(\frac{f_{\mathrm{s,ef}}}{f_{\mathrm{yd}}}-1\right)\cdot\left(2-\frac{1}{\alpha}-\alpha\right)\right) \text{ when } \alpha < 1 \tag{7.22}$$

$$TS_{\mathrm{My}} = 1 - 0{,}6\left[\frac{M_{\mathrm{cr}}}{M_{\mathrm{y}}}\right] \geq 0{,}4 \tag{7.23}$$

$$\alpha = \frac{\left(f_{\mathrm{s,ef}} - f_{\mathrm{yd}}\right)}{0{,}6 f_{\mathrm{cm}}^{\frac{2}{3}}} \cdot \frac{\phi}{s_{\mathrm{r,m,cal}}} \tag{7.24}$$

where

$f_{s,ef}$	is the tensile stress in the reinforcement when M_{Rd} is reached, assuming an inclined post-elastic stress-strain-relation, see Figure 5.2;
$\varepsilon_{ud,ef}$	is the strain in the reinforcement with a stress equal to $f_{s,ef}$;
M_y	is the internal moment when the strain in the tension reinforcement equals ε_{yd};
ε_{yd}	$= f_{yd} / E_s$
γ_θ	is a partial factor for model uncertainty.

NOTE The value of γ_θ = 3,0 applies unless the National Annex gives a different value.

7.3.3 Plastic analysis

7.3.3.1 General

(1) Methods based on plastic analysis may be used for the check at ULS only, except for analyses with stress fields and strut-and-tie models (see 7.3.3.3) which may also be used in SLS in certain conditions (see 9.2.3(8)).

(2) Plastic analysis should in general be based on the lower bound theorem of limit analysis. Plastic methods based on the upper bound theorem of limit analysis may be used, if it is known by experience that the type of assumed mechanisms can develop.

(3) The plastic deformation capacity of the critical sections shall be sufficient for the envisaged mechanism to be formed.

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

(5) Plastic analysis shall only be used for regions in which, within the plastic hinge, reinforcing steel is of Class B or C. Prestressing steel may be considered as Class B steel.

(6) For plastic analysis, the horizontal branch of the stress-strain diagram for the reinforcement (see Figures 5.2 and 5.3) shall be used when determining the sectional capacity of the cross-sections.

7.3.3.2 Analysis for beams, frames and slabs without verification of rotation capacity

(1) Plastic analysis without any direct check of rotation capacity may be used for the ultimate limit state if all the following conditions are fulfilled:

(i) the area of tensile reinforcement is limited such that, at any section where plastic hinges are expected to occur $x_{\mathrm{u}}/d \leq 0{,}25$;

(ii) the ratio of the moments at intermediate supports to the moments in the span is between 0,5 and 2,0.

7.3.3.3 Analysis with stress fields and strut-and-tie models

(1) Stress fields and strut-and-tie models may be used to determine the action effects in structures and members. This includes deep beams, walls and zones of discontinuities.

(2) Internal forces in struts and ties may be calculated based on linear elastic analysis, non-linear analysis or plastic analysis.

7.3.4 Non-linear analysis

(1) Where non-linear methods of analysis are used for verification of ULS or SLS, equilibrium and compatibility shall be satisfied and realistic non-linear behaviour of materials should be considered.

(2) The general rules for non-linear analysis procedures given in EN 1990 shall be respected.

If non-linear methods of analysis are used for verification of ULS by numerical simulations, the results shall be confirmed by means of a comparison with simplified analytical methods. In case of significant differences with analysis according to Clause 8, the discrepancies should be justified.

NOTE For the verification of the ultimate limit states the specific provisions given in Annex F can be followed.

(3) The constitutive material models for concrete, reinforcement and their interaction should capture all relevant features of material behaviour for the specific problem to be considered. Time-dependent material properties of concrete (such as shrinkage and creep), should be considered if necessary.

(4) Non-linear material models and numerical procedures should be validated for each field of application by tests, analytical solutions and/or benchmark test results, including basic tests on materials, structural reference tests and mesh sensitivity studies.

(5) If material characteristics, such as tensile strength or fracture energy of concrete, have an important influence on the results, a sensitivity analysis of the structural behaviour with respect to such characteristics should be performed.

(6) When analysis and verification are combined, the resistance against bending and axial forces, as well as the resistance of planar elements against membrane forces and/or bending and twisting moments, shall not depend on the tensile strength of concrete.

(7) A proper description of multi-axial states of stress in concrete should be considered, particularly when tension and compression are combined in the same finite element response.

(8) Modelling of cracking should consider the direction of the reinforcement able to control the crack opening.

7.4 Second order structural analysis of members and systems with axial force

7.4.1 General

(1) The provisions of 7.4 should be applied to members or structures (e.g. those with a flexible bracing system) in which the structural behaviour is significantly influenced by second order effects.

NOTE 1 Annex O provides complementary guidance to simplified methods for second order structural analysis of members and structures.

(2) Where second order effects are considered, equilibrium and resistance shall be verified in the deformed state. Deformations shall be calculated considering the relevant effects of cracking, non-linear material properties and creep.

(3) Second order effects may be ignored if they are not more than 10 % of the corresponding first order effects.

For global second order effects this condition may be considered satisfied if Formula (7.25) is satisfied:

$$\frac{F_{\mathrm{VB}}}{F_{\mathrm{VEd}}} \geq 10 \tag{7.25}$$

where

F_{VEd} is the total design vertical load on the bracing structure and the members braced by it;

F_{VB} is the buckling load of the bracing structure determined from a set of forces proportional to F_{VEd}.

NOTE 2 F_{VB} can be determined according to Formula (O.1). For local second order effects (isolated members), simplified criteria are given in O.4.

(4) The design moment for ultimate limit state verification shall be the larger of the first order moment and the total design moment including second order effects.

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

(6) The structural behaviour shall be considered in the direction in which deformations can occur, and biaxial bending shall be considered when necessary.

7.4.2 Creep

(1) The effect of creep should be considered in second order analysis, with due regard to both the general conditions for creep (see 5.1.5 and B.5) and the duration of different loads in the load combination considered.

(2) The duration of loads may be considered in a simplified way by means of an effective creep coefficient, which, used together with the design load, gives a creep deformation corresponding to the

quasi-permanent load. For global second order effects, $\varphi_{\text{eff,s}}$ may be taken from Formula (7.26). For isolated members and local second order effects, $\varphi_{\text{eff,b}}$ may be taken from Formula (7.27).

$$\varphi_{\text{eff,s}} = \varphi(t_{\text{DL}}, t_0)\frac{\delta_{\text{0Eqp}}}{\delta_{\text{Ed}}} \tag{7.26}$$

$$\varphi_{\text{eff,b}} = \varphi(t_{\text{DL}}, t_0)\frac{M_{\text{0Eqp}}}{M_{\text{0Ed}}} \tag{7.27}$$

where

$\varphi(t_{\text{DL}},t_0)$ is the creep coefficient at design service life according to Table 5.2 or B.5, where relevant the average of the creep coefficients of the members considered may be used;

δ_{0Eqp} is the maximum short-term horizontal deflection due to the quasi permanent load combination determined assuming uncracked cross-sections;

δ_{Ed} is the maximum short-term horizontal deflection due to the relevant load combinations from a first order analysis determined assuming uncracked cross-sections. At least two combinations should be considered: the one corresponding to the combination with dominant horizontal load and the one corresponding to the combination with dominant vertical imposed load, the largest resulting value of $\varphi_{\text{ef,b}}$ should be used;

M_{0Eqp} is the maximum first order moment due to the quasi-permanent load combination including the effect of the imperfections as described in 7.2.1;

M_{0Ed} is the maximum first order moment due to the relevant load combination.

(3) As an approximation the ratio between the moments ($M_{\text{0Eqp}}/M_{\text{0Ed}}$) in Formula (7.27) may be replaced by the ratio of vertical loads in the quasi permanent and design situations.

(4) Where global and local analysis are combined, φ_{eff} should be taken as the maximum of $\varphi_{\text{eff,s}}$ and $\varphi_{\text{eff,b}}$ unless it is demonstrated that local second order effects are not governing, in which case $\varphi_{\text{eff}} = \varphi_{\text{eff,s}}$.

7.4.3 Methods of analysis

7.4.3.1 General

(1) Either one of the following three methods of analysis may be used:

— a simplified method based on nominal curvature, which only accounts for local second order effects (see 7.4.3.2);

— a second order linear elastic analysis method based either on a reduced stiffness value (see 7.4.3.2), or on a moment magnification factor; and

— a general, fully non-linear analysis method (see 7.4.3.3).

NOTE An example of a method based on:

— nominal curvature is given in O.7;

— a reduced stiffness value is given in O.8.1;

— a moment magnification factor is given in O.8.2.

7.4.3.2 Simplified methods based on nominal curvature and second order linear analysis

(1) The methods based on the nominal curvature and on second order linear elastic analysis should use an effective stiffness. This stiffness may conservatively be taken as the stiffness corresponding to the situation where yielding occurs (see Figure 7.4 a)). For global effects, when yielding of the reinforcement

occurs successively at different locations, the stiffness corresponding to the situation for which the last plastic hinge develops may be conservatively used (see Figure 7.4 b)). Other less conservative estimates may also be used where justified.

a) local effects b) global effects

Figure 7.4 — Possible equivalent stiffness

7.4.3.3 General non-linear analysis method

(1) For the general method based on full non-linear analysis, including geometric non-linearity i.e. second order effects, the general rules for non-linear analysis given in 7.3.4 shall apply.

(2) Stress-strain curves for concrete and reinforcement suitable for overall analysis shall be used. The effect of creep shall be considered.

(3) Stress-strain relationships given in 5.1.6, (Formula (5.6)) for concrete and in 5.2.4 (Figure 5.2) for reinforcing steel and in 5.3.3 (Figure 5.3) for prestressing steel may be used. With stress-strain diagrams based on design values, a design value of the ultimate load is obtained directly from analysis.

When Formula (5.6) is used to determine the stress-strain relationships for concrete, f_{cm} shall be substituted by the design compressive strength f_{cd} (except in Formulae (5.9) for ε_{c1}) and ε_{cu1} shall be substituted by 0,0035 and E_{cm} shall be substituted by E_{cd} given in Formula (7.28):

$$E_{cd} = \frac{E_{cm}}{\gamma_{CE}} \tag{7.28}$$

NOTE Where alternative stress-strain relationships for concrete are used, the nature of the analysis to be used and the phenomenon included need to be considered.

(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 7.4.3.3(3) with a factor $(1 + \varphi_{eff})$, where φ_{eff} is the effective creep ratio according to 7.4.2(4).

(5) The favourable effect of tension stiffening may be considered in the determination of the stiffness provided that 7.3.4(6) is fulfilled.

(6) Normally, conditions of equilibrium and strain compatibility should be satisfied in a number of cross-sections. As a simplification, this condition may be imposed only at the critical cross-section(s), assuming a relevant variation of the curvature in between, e.g. similar to the first order moment or simplified in another appropriate way.

7.4.4 Compression member with biaxial bending

(1) The general method in 7.4.3.3 may be used for biaxial bending. If simplified methods are used, provisions (3) and (4) apply.

(2) The cross-section of the member with the critical combination of moments should be used for verifications.

(3) Separate design in each principal direction taking into account second order effects, disregarding biaxial bending interaction, may be used as a first step. Imperfections should be considered only in the direction where they will have the most unfavourable effect.

(4) Further checks may be omitted if the slenderness ratio satisfies the following condition:

$$0{,}5 \leq \frac{\lambda_y}{\lambda_z} \leq 2 \tag{7.29}$$

and if the ratio of the dimensionless eccentricities e'_y and e'_z (see Figure 7.5) satisfy one of the following conditions:

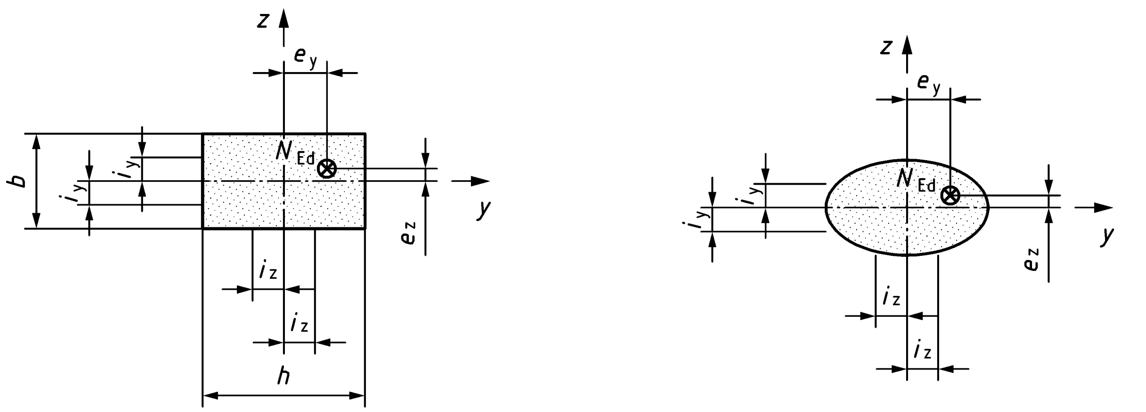
$$\frac{e'_y}{e'_z} \leq 0{,}2 \text{ or } \frac{e'_y}{e'_z} \geq 5 \tag{7.30}$$

where

$e'_z = M_{Edy}/\lvert N_{Ed} \cdot b\rvert$	is the dimensionless eccentricity along the z-axis;
$e'_y = M_{Edz}/\lvert N_{Ed} \cdot h\rvert$	is the dimensionless eccentricity along the y-axis.

For non-rectangular sections, b and h shall be replaced by $b_{eq} = i_y\sqrt{12}$ and $h_{eq} = i_z\sqrt{12}$.

(5) Unless all of the conditions of Formulae (7.29) and (7.30) are fulfilled, biaxial bending as described in 8.1.1(8) should be taken into account including second order effects in each direction.



a) rectangular b) general cross-section

Figure 7.5 — Definition of eccentricities e_y and e_z

7.5 Lateral instability of slender beams

(1) Lateral instability of slender beams shall be considered 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 considered in the calculation of effects due to lateral instability.

(2) In absence of project-specific execution specifications which define maximum deviations, 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 considered.

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

— persistent design situations:

$$\frac{l_{0t}}{b} \leq \frac{50}{(h/b)^{1/3}} \tag{7.31}$$

— transient design situations:

$$\frac{l_{0t}}{b} \leq \frac{70}{(h/b)^{1/3}} \tag{7.32}$$

where

l_{0t} is the distance between torsional restraints;

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

b is the width of the compression flange.

(4) Torsion associated with lateral instability should be considered in the design of members supporting slender beams.

7.6 Prestressed members and structures

7.6.1 General

(1) The effects of prestressing may be considered by one of the following equivalent approaches:

a) self-equilibrated state of stresses in the concrete and the tendons (approach also known as prestressing considered on the side of resistance). In this case, the internal forces due prestressing are due only to external restraints (statically indeterminate component of prestressing);

NOTE In this case the effect of prestressing can be modelled by imposing on the sections of the structure an axial strain equal to the prestressing force divided by the axial stiffness of the section and a curvature equal to the prestressing force times the eccentricity of prestressing with respect to the centroid divided by the flexural stiffness of the section.

b) set of self-equilibrated system of forces (anchorage, deviation and friction forces) exerted by the tendons on the concrete member (approach also known as prestressing considered as external action). In this case, the internal forces include both the statically determined and the statically indeterminate components of prestressing.

(2) Since the internal forces resulting from the two approaches are different, the resistance of sections and members shall be verified consistently, according to 7.6.5(1).

7.6.2 Prestressing force

(1) At a given time t and distance x (or arc length) from the active end of the tendon the mean prestress stress $\sigma_{p,m}(x,t)$ should be taken equal to the maximum stress $\sigma_{p,max}$ imposed at the active end, minus the immediate losses (see 7.6.3) and the time dependent losses (see 7.6.4), using absolute values for all losses.

(2) Short-term prestressing stresses should be limited to the values given in Table 7.1 or agreed for a specific project by the relevant parties.

Table 7.1 — Limits to short-term prestressing stresses

<table>
<tr><th>Stress being limited</th><th>Stress limit</th></tr>
<tr><td rowspan="2">Maximum stressing stress $\sigma_{p,max}$</td><td>$\leq 0{,}8 f_{pk}$</td></tr>
<tr><td>$\leq 0{,}9 f_{p0,1k}$</td></tr>
<tr><td>Overstressing[a] (e.g. for the occurrence of unexpectedly high friction)</td><td>$\leq 0{,}95 f_{p0,1k}$</td></tr>
<tr><td rowspan="2">Maximum stress after prestress transfer/anchoring[b] $\sigma_{p,m}(x,0)$</td><td>$\leq 0{,}75 f_{pk}$</td></tr>
<tr><td>$\leq 0{,}85 f_{p0,1k}$</td></tr>
<tr><td colspan="2">a Overstressing is permitted only if the force in the jack can be measured to an accuracy of ± 5 % of the final value of the prestressing force.
b The stress after transfer/anchoring is determined by subtracting the immediate losses (see 7.6.3) from the stress imposed at the active end.</td></tr>
</table>

(3) The mean value of the prestress stress $\sigma_{p,m}(x,t)$ at the time $t > t_0$ should be determined taking into account the prestressing method. In addition to the immediate losses $\Delta\sigma_{p,i}$ given in 7.6.3 time-dependent losses of prestress $\Delta\sigma_{p,c+s+r}(x)$ given in 7.6.4 as a result of creep and shrinkage of the concrete and long-term relaxation of the prestressing steel should be considered:

$$\sigma_{p,m}(x,t) = \sigma_{p,m}(x,t_0) - \Delta\sigma_{p,c+s+r}(x,t) \tag{7.33}$$

7.6.3 Immediate losses of prestress

7.6.3.1 General

(1) When determining the immediate losses $\Delta\sigma_{p,i}(x)$ the following influences should be considered for pre-tensioning and post-tensioning where relevant:

— during the stressing process: losses due to friction between the prestressing steel and duct or deviation devices $\Delta\sigma_{p,\mu}(x)$, see 7.6.3.2;

— during the stressing process: losses due to anchorage seating (e.g. wedge draw-in), see 7.6.3.3;

— at the transfer of prestress to concrete: losses due to instantaneous deformation of concrete, see 7.6.3.4;

— losses due to short-term relaxation of the pretensioning tendons during the period which elapses between the tensioning of the tendons and prestressing of the concrete $\Delta\sigma_{pr}$ (where relevant), see B.9. 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 13.4.2).

7.6.3.2 Losses due to friction

(1) The losses due to friction $\Delta\sigma_{p,\mu}(x)$ in prestressing tendons should be estimated from:

$$\Delta\sigma_{p,\mu}(x) = \sigma_{p,max}\left[1 - \exp\left(-\mu\left(\alpha_{\mu} + k_{\mu}x\right)\right)\right] \tag{7.34}$$

where

α_{μ} is the sum of the absolute values of angular deviations in radians over a distance x;

μ is the coefficient of friction between the tendon and its duct or deviation device;

k_{μ} is an unintentional angular deviation for internal post-tensioning tendons in grouted ducts and greased strands in radian per unit length- (curvature); and

x is the distance along the tendon from the point where the prestressing stress is equal to $\sigma_{p,max}$ (the force at the active end during tensioning).

NOTE 1 The values μ and k_{μ} can be found in the technical documentation of post-tensioning system.

NOTE 2 The value μ depends on the surface characteristics of the tendons and the duct or deviation device, on the presence of rust, on the elongation of the tendon and on the tendon profile.

NOTE 3 The value k_{μ} for unintentional angular deviation 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 precise data, the values for μ given in Table 7.2 may be used, in Formula (7.34).

(3) In the absence of more precise data, values for unintentional angular deviation for internal post-tensioning tendons will generally be in the range $0{,}005 < k_{\mu} < 0{,}01$ per metre. For pre-tensioning tendons and external tendons, the losses of prestress due to unintentional angular displacement may be ignored.

Table 7.2 — Coefficients of friction μ of internal post-tensioning tendons and external tendons to be used in the absence of more precise data

Type of prestressing steel	Internal tendons		Greased and sheathed strand	External tendons
	metal duct	polymer duct	PE duct	PE duct
Cold drawn wire	0,17	0,12	—	0,10
Strand	0,19	0,14	0,05	0,12
Deformed bar	0,65	—	—	—
Smooth round bar	0,33	—	—	—

7.6.3.3 Losses due to anchorage seating

(1) Account should be taken of the losses due to anchorage seating, during the operation of anchoring the prestressing steel after tensioning.

NOTE Values for anchorage seating are given in the technical documentation of post-tensioning system.

7.6.3.4 Losses due to the instantaneous deformation of concrete

(1) Where relevant, account should be taken of the loss in tendon stress corresponding to the deformation of concrete, considering the tensioning programme in which the tendons are stressed.

7.6.4 Time dependent losses of prestress

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

a) That due to the reduction of strain, caused by the deformation of concrete due to creep and shrinkage, under the quasi-permanent loads; and

b) that due to the reduction of stress in the steel due to the relaxation under tension (see B.9).

The interaction between the relaxation of prestressing steel and the concrete deformation due to creep and shrinkage may generally and approximately be considered by applying a reduction factor of 0,8 on the stress relaxation. More refined methods may be used for the calculation of relaxation losses, taking into account the variation of the tendon elongation due to creep and shrinkage of concrete.

(2) Time dependent losses of stress in the tendons due to creep, shrinkage and relaxation at location x, at time t under the quasi-permanent combination of actions may be evaluated by Formula (7.35).

$$\Delta\sigma_{\mathrm{p,c+s+r}} = \frac{\varepsilon_{\mathrm{cs}} E_{\mathrm{p}} + 0{,}8\Delta\sigma_{\mathrm{pr}} + \dfrac{E_{\mathrm{p}}}{E_{\mathrm{cm}}}\varphi(t,t_0)\sigma_{\mathrm{cp,QP}}}{1 + \dfrac{E_{\mathrm{p}}}{E_{\mathrm{cm}}}\dfrac{A_{\mathrm{p}}}{A_{\mathrm{c}}}\left(1 + \dfrac{A_{\mathrm{c}}}{I_{\mathrm{c}}} z_{\mathrm{cp}}^2\right)[1 + 0{,}8\varphi(t,t_0)]} \tag{7.35}$$

where

$\Delta\sigma_{\mathrm{pr}}$ 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 should be determined for the initial stress in the tendons due to initial prestress and quasi-permanent combination of actions $\sigma_{\mathrm{pr}} = \sigma_{\mathrm{pr}}(G + P_{\mathrm{m0}} + \psi_2 Q)$;

$\sigma_{\mathrm{cp,QP}}$ 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 $\sigma_{\mathrm{cp,Q}P}$ can be the effect of part of self-weight and initial prestress or the effect of a full quasi-permanent combination of actions $[\sigma_{\mathrm{cp}} \cdot (G + P_{\mathrm{m0}} + \psi_2 Q)]$, depending on the stage of construction considered (see B.4);

z_{cp} is the distance between the centroid of the concrete section and the tendons.

NOTE 1 It is conservative to adopt a value of 1,0 for the denominator of Formula (7.35).

NOTE 2 Unless specified for the project, it is not necessary to consider time dependent losses beyond the design service life of the structure.

(3) Formula (7.35) applies to bonded tendons when local values of stresses are used and to unbonded tendons when mean values of stresses are used. The mean values should be calculated between straight sections limited by the deviation points for external tendons or along the entire length in case of internal tendons.

7.6.5 Effects of prestressing at ultimate limit state

(1) The statically determinated component of prestressing may be considered either as an external action or as a resistance when verifying the ultimate limit state for flexural resistance (see 7.6.1(1)). In the first case, when determining the flexural capacity of the section, the resistance of the tendon shall be limited to $f_{\mathrm{pd}} - \sigma_{\mathrm{pd}}$. In the second case it shall be limited to f_{pd}.

(2) In general, the design value of the prestressing stress as external action may be determined as $\sigma_{\mathrm{pd}}(x,t) = \gamma_{\mathrm{P}}\sigma_{\mathrm{p,m}}(x,t)$.

(3) For prestressed members with unbonded tendons, the deformation of the whole member should generally be taken into account, when calculating the increase of the stress in the prestressing steel. If no detailed calculation is made and if the distance between fixed points does not exceed one span, it may be assumed that the increase of the stress at ultimate limit state due to the deformation of the member before reaching failure is $\Delta\sigma_{\mathrm{p,ULS}} = 100$ MPa.

(4) If the stress increase in unbonded tendons is calculated for prestressed members 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 $\gamma_{\Delta P,sup}$ or $\gamma_{\Delta P,inf}$, according to 4.3.2.

(5) For external prestressing tendons, the strain in the prestressing steel between two subsequent fixed points may be assumed to be constant. The strain in the prestressing steel should then be taken as equal to the initial strain, determined just after completion of the prestressing operation of each tendon, increased by the strain resulting from the structural deformation between the contact points considered, considering time-dependent losses. A deviator in an external tendon may be considered as a fixed point if the difference of tendon force between the two ends of the deviation point is smaller than the friction loss of the tendon in the deviation point.

8 Ultimate Limit States (ULS)

8.1 Bending with or without axial force

8.1.1 General

(1) 8.1 applies to undisturbed regions of beams, slabs and similar types of members for which plane 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 the general approach provided in 8.5.

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

— plane sections remain plane;

— the change in strain in bonded reinforcing steel or bonded prestressing tendons, whether in tension or in compression, is the same as the change in strain in the surrounding concrete;

— the tensile strength of concrete is ignored;

— the stresses in the concrete in compression are derived from the design stress distributions given in 8.1.2;

— the stresses in the reinforcing or prestressing steel are derived from the design stress-strain relationships in 5.2 (Figure 5.2) and 5.3 (Figure 5.3);

— the strain difference between prestressing steel and surrounding concrete is considered when assessing the stresses in the tendons with due regard to time-dependent losses at the time considered.

(3) The compressive strain in the concrete shall be limited to ε_{cu}, see 8.1.2(1) unless the concrete is confined, see 8.1.4.

(4) The strains in the reinforcing steel and the prestressing steel shall be limited to ε_{ud} (where applicable); see 5.2.4(2) and 5.3.3(2) respectively.

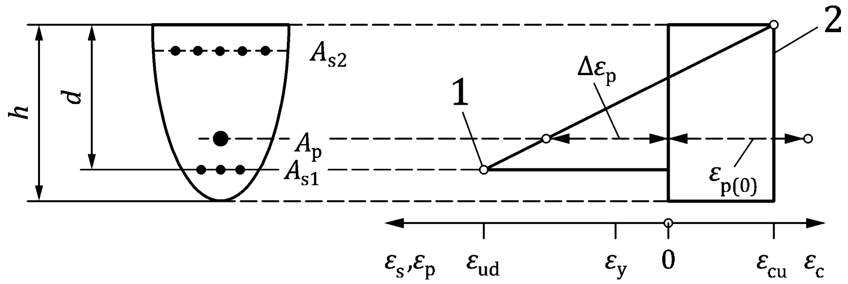
(5) Cross-sections loaded by an axial compression force N_{Ed} unless second order effects and the effects of geometric imperfections and of imposed deformations have been accounted for, should be designed for a minimum moment of at least

$$M_{Ed,min} = \pm N_{Ed} \cdot e_{d,min} \tag{8.1}$$

where

$$e_{d,min} = \max\{h/30;\ 20\text{ mm}\}$$

(6) Limiting strain distributions in a cross-section should be at least those shown in Figure 8.1.



Key

1 tensile strain limit of reinforcing steel

2 compressive strain limit of concrete

Figure 8.1 — Possible strain distributions in the ultimate limit state

(7) For prestressed members with permanently unbonded tendons, 7.6.5 applies.

(8) In the absence of an accurate cross-section design for biaxial bending, the following simplified criterion may be used:

$$\left(\frac{|M_{\mathrm{Edz}}|}{M_{\mathrm{Rdz,N}}}\right)^{a_{\mathrm{N}}} + \left(\frac{|M_{\mathrm{Edy}}|}{M_{\mathrm{Rdy,N}}}\right)^{a_{\mathrm{N}}} \leq 1{,}0 \tag{8.2}$$

where

$M_{\mathrm{Edz/y}}$ is the design moment about the respective axis, including a 2nd order moment;

$M_{\mathrm{Rdz/y,N}}$ is the moment resistance in the respective direction for the given axial compression force;

a_{N} is the exponent whose value is determined as follows:

— for circular and elliptical cross-sections: $a_{\mathrm{N}} = 2$;

— for rectangular cross-sections:

$\lvert N_{\mathrm{Ed}}\rvert / N_{\mathrm{Rd,0}}$	0,1	0,7	1,0
a_{N}	1,0	1,5	2,0

with linear interpolation for intermediate values;

$N_{\mathrm{Rd,0}}$ is the design value of axial resistance under compression without accompanying moments:

$$N_{\mathrm{Rd,0}} = A_{\mathrm{c}} f_{\mathrm{cd}} + A_{\mathrm{s}} f_{\mathrm{yd}} \tag{8.3}$$

where in case of members with confinement reinforcement, f_{cd} should be replaced by $f_{\mathrm{cd,c}}$ according to Formula (8.15).

8.1.2 Stress distribution in the compression zones

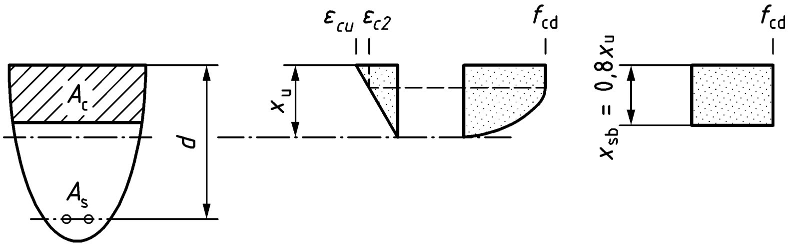
(1) For the design of cross-sections, the following stress distribution may be used, see Figure 8.2c) (compressive strain shown positive):

$$\sigma_{\mathrm{cd}} = \begin{cases} f_{\mathrm{cd}} \left[1 - \left(1 - \frac{\varepsilon_{\mathrm{c}}}{\varepsilon_{\mathrm{c2}}}\right)^2\right] & \text{for } 0 \leq \varepsilon_{\mathrm{c}} \leq \varepsilon_{\mathrm{c2}} \\ f_{\mathrm{cd}} & \text{for} \varepsilon_{\mathrm{c2}} \leq \varepsilon_{\mathrm{c}} \leq \varepsilon_{\mathrm{cu}} \end{cases} \tag{8.4}$$

where

$\varepsilon_{c2} = 0{,}002;$

$\varepsilon_{cu} = 0{,}0035.$



a) cross-section

b) assumed strain distribution

c) parabola-rectangle stress distribution

d) rectangular stress distribution

Figure 8.2 — Stress distributions within the compression zone

(2) Alternatively, a rectangular stress block distribution as given in Figure 8.2d) may be assumed.

8.1.3 Bending in slabs

(1) Orthogonally reinforced solid slab elements with bending and torsional moments where $|m_{Edxy}| \leq 0{,}07d^2 f_{cd}$ may be designed using the following formulae:

$$m_{Rdx} \geq m_{Edx} + |m_{Edxy}| \tag{8.5}$$

$$m_{Rdy} \geq m_{Edy} + |m_{Edxy}| \tag{8.6}$$

$$m_{Rdx}{}' \geq -m_{Edx} + |m_{Edxy}| \tag{8.7}$$

$$m_{Rdy}{}' \geq -m_{Edy} + |m_{Edxy}| \tag{8.8}$$

where x and y are local axes parallel to the reinforcement. Alternatively, G.4(5) may be used.

(2) In cases where $|m_{Edxy}| > 0{,}07d^2 f_{cd}$, G.4(1)-(2) may be used.

8.1.4 Confined concrete

(1) The concrete compressive design strength f_{cd} may be enhanced by the favourable effect of confinement reinforcement or of triaxial compressive stresses.

(2) The compressive strength increase of a concrete with $d_{dg} \geq 32$ mm due to a transverse compressive stress σ_{c2d} may be calculated according to:

$$\Delta f_{cd} = 4 \cdot \sigma_{c2d} \qquad \text{for } \sigma_{c2d} \leq 0{,}6 f_{cd} \tag{8.9}$$

$$\Delta f_{cd} = 3{,}5 \cdot \sigma_{c2d}^{3/4} \cdot f_{cd}^{1/4} \qquad \text{for } \sigma_{c2d} > 0{,}6 f_{cd} \tag{8.10}$$

In case of concrete with $d_{dg} < 32$ mm, the strength increase Δf_{cd} according to Formulae (8.9) and (8.10) shall be reduced by factor $d_{dg}/32$ mm.

where

d_{dg} is defined in 8.2.1(4);

σ_{c2d} is to the absolute value of the minimum principal transverse compressive stress.

NOTE Provided that the strength increase Δf_{cd} is reduced by factor $d_{dg}/32$ mm, Formulae (8.9) and (8.10) apply also for concrete and mortar with $d_{dg} < 24$ mm.

(3) The confinement stress σ_{c2d} resulting from confinement reinforcement in columns, compression zones due to bending and struts may be calculated as:

$$\sigma_{c2d} = \frac{2A_{s,conf} \cdot f_{yd}}{b_{cs} \cdot s} \quad (8.11)$$

for circular and square members in compression with single confinement reinforcement (Figure 8.3 a) and b))

$$\sigma_{c2d} = \frac{2A_{s,conf} \cdot f_{yd}}{\max\{b_{csx}; b_{csy}\} \cdot s} \quad (8.12)$$

for rectangular members in compression with single confinement reinforcement (b_{csx} and b_{csy} according to Figure 8.3 c))

$$\sigma_{c2d} = \min\left\{\frac{\Sigma A_{s,confx}}{b_{csy}}; \frac{\Sigma A_{s,confy}}{b_{csx}}\right\} \cdot \frac{f_{yd}}{s} \quad (8.13)$$

for members in compression with multiple confinement reinforcement (Figure 8.3 c) and d))

$$\sigma_{c2d} = \min\left\{\frac{\Sigma A_{s,confx}}{b_{csy}}; \frac{A_{s,confy}}{x_{cs}}\right\} \cdot \frac{f_{yd}}{s} \quad (8.14)$$

for compression zones (Figure 8.3 e))

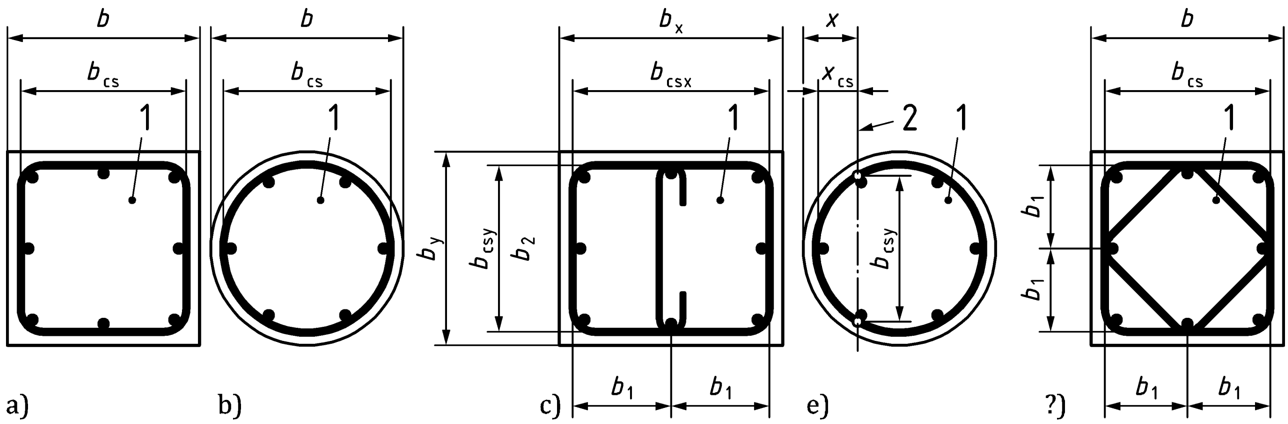
where

$A_{s,conf}$ is the cross-sectional area of one leg of confinement reinforcement;

b_{cs} is the width of the confinement core (to the centrelines of the confinement reinforcement, see Figure 8.3);

s is the spacing of confinement reinforcement.

For confining reinforcements not in the x, or y-axes, the corresponding components should be considered (in the instance of Figure 8.3 d)), the area of the inner confinement reinforcement may be multiplied by $\sqrt{2}$).



a) Square section

b) Circular section

c) Rectangular section

e) Section with axial force and flexure

?) Square section with multiple confinement reinforcement

Key

1 confined area $A_{c,conf}$

2 neutral axis

Figure 8.3 — Definition of dimensions of confinement reinforcement

(4) The concrete strength increase in the confined areas defined by the centrelines of the confinement reinforcement as shown in Figure 8.3 (where rounded corners may be neglected) may be smeared over the compression zone by considering the following average strength:

$$f_{cd,c} = f_{cd} + k_{conf,b} \cdot k_{conf,s} \cdot \Delta f_{cd} \tag{8.15}$$

where the effectiveness factors $k_{conf,b}$ and $k_{conf,s}$ are defined in Table 8.1.

Table 8.1 — Effectiveness factors $k_{conf,b}$ and $k_{conf,s}$ for confinement reinforcement

	Shape of compression zone and confinement reinforcement	$k_{conf,b}$	$k_{conf,s}$
a)	Square members in compression with single confinement reinforcement (Figure 8.3 a))	$k_{conf,b} = \frac{1}{3}\left(\frac{b_{cs}}{b}\right)^2$	$k_{conf,s} = \left(1-\frac{s}{2b_{cs}}\right)^2$ where $1 - \frac{s}{2b_{cs}} \geq 0$
b)	Circular member in compression with circular confinement reinforcement (Figure 8.3 b))	$k_{conf,b} = \left(\frac{b_{cs}}{b}\right)^2$	

	Shape of compression zone and confinement reinforcement	$k_{conf,b}$	$k_{conf,s}$
c)	— Square and rectangular members in compression with multiple confinement reinforcement (Figures 8.3 c) and d)); and — rectangular sections with single confinement reinforcement(Figures 8.3 c)).	$k_{\text{conf,b}} = \dfrac{b_{\text{csx}} \cdot b_{\text{csy}} - \frac{1}{6}\Sigma b_{\text{i}}^2}{b_{\text{x}} \cdot b_{\text{y}}}$ b_{i} are the distances between bends of straight segments (defined by the intersections of centrelines) or anchorages of the confinement reinforcement ($\Sigma b_i^2 = 4b_1^2 + 2b_2^2$ in the example of Figure 8.3c); $\Sigma b_i^2 = 0$ in the examples of Figures 8.3 b) and d))	$k_{\text{conf,s}} = \left(1 - \frac{s}{2b_{\text{csx}}}\right)\left(1 - \frac{s}{2b_{\text{csy}}}\right)$ where $1 - \dfrac{s}{2b_{\text{csx}}} \geq 0$ and $1 - \dfrac{s}{2b_{\text{csy}}} \geq 0$
d)	Compression zones due to bending and axial force (Figure 8.3 e))	$k_{\text{conf,b}} = \dfrac{A_{\text{c,conf}} - \frac{1}{6}\Sigma b_{\text{i}}^2}{A_{\text{cc}}}$ $A_{\text{c,conf}}$ is the confined area within the centrelines of the confinement reinforcement and the neutral axis and A_{cc} is the compressive area	$k_{\text{conf,s}} = \left(1 - \frac{s}{4x_{\text{cs}}}\right)\left(1 - \frac{s}{2b_{\text{csy}}}\right)$ where x_{cs} should not be taken $> b_{\text{csx}}/2$, $1 - \dfrac{s}{4x_{\text{cs}}} \geq 0$ and $1 - \dfrac{s}{2b_{\text{csy}}} \geq 0$

(5) The effect of confinement on the strain limits in concrete may be determined in accordance with Formula (8.16) and Formula (8.17). However, if these enhancements to the strains are included in design, then the concrete area between the free surface and the axis of the confinement reinforcement should not be included in any strength verifications.

$$\varepsilon_{\text{c2,c}} = \varepsilon_{\text{c2}}\left(1 + 5\frac{\Delta f_{\text{cd}}}{f_{\text{cd}}}\right) \tag{8.16}$$

$$\varepsilon_{\text{cu,c}} = \varepsilon_{\text{cu}} + 0{,}2\frac{\sigma_{\text{c2d}}}{f_{\text{cd}}} \tag{8.17}$$

8.2 Shear

8.2.1 General verification procedure

(1) The shear resistance of linear members and the out-of-plane shear resistance of planar members shall be verified according to the following procedure at all critical control sections:

(i) detailed verification of the shear resistance may be omitted, provided that:

$\tau_{\text{Ed}} \leq \tau_{\text{Rdc,min}}$ according to 8.2.1(4)

(ii) no calculated shear reinforcement is required in regions of the members where:

$\tau_{\text{Ed}} \leq \tau_{\text{Rd,c}}$ according to 8.2.2 and 8.4.3;

(iii) otherwise, shear reinforcement shall be designed according to 8.2.3 and 8.4.4:

$\tau_{\text{Ed}} \leq \tau_{\text{Rd}}$

(2) When, on the basis of the design shear calculation, no shear reinforcement is required, minimum shear reinforcement may nevertheless be necessary for linear members, according to Clause 12.

For linear members in statically determinate structures with $d > 500$ mm, minimum shear reinforcement shall be provided. In exceptions where this were not possible, the verification methods according to I.8.3.1(3) shall be used.

(3) In regions of members without geometric discontinuities, the average shear stress over the cross-section τ_{Ed} is defined as:

$$\tau_{Ed} = \frac{V_{Ed}}{b_w \cdot z} \tag{8.18}$$

or

$$\tau_{Ed} = \frac{v_{Ed}}{z} \tag{8.19}$$

where

V_{Ed} is the design shear force at the control section in linear members;

v_{Ed} is the design shear force per unit width in planar members;

b_w is the width of the cross-section of linear members. The width b_w for cross-sections with variable width and for circular cross-sections is defined in 8.2.3(9);

z is the lever arm for the shear stress calculation defined as $z = 0{,}9d$; where d refers to the centroid of tensile reinforcement.

NOTE 1 For d the nominal effective depth d_{nom} or the design value d_d can be used (see A.3). The National Annex can give advice for using d_d.

(4) The minimum shear stress resistance may be calculated as:

$$\tau_{Rdc,min} = \frac{11}{\gamma_V} \cdot \sqrt{\frac{f_{ck}}{f_{yd}} \cdot \frac{d_{dg}}{d}} \tag{8.20}$$

where

γ_V is the partial factor for shear design according to Table 4.3 (NDP) or Tables A.1 (NDP) and A.2 (NDP);

f_{yd} is the design value of the yield strength which has been used to design the flexural reinforcement;

d is the effective depth of the flexural reinforcement. For prestressed members see 8.2.2(6);

NOTE 2 For d refer to NOTE 1.

d_{dg} is a size parameter describing the failure zone roughness, which depends on the concrete type and its aggregate properties. d_{dg} (mm) may be taken as:

— 16 mm + $D_{lower} \leq 40$ mm for concrete with $f_{ck} \leq 60$ MPa;

— 16 mm + $D_{lower}\,(60/f_{ck})^2 \leq 40$ mm for concrete with $f_{ck} > 60$ MPa.

NOTE 3 D_{lower} is the smallest value of the upper sieve size D in an aggregate for the coarsest fraction of aggregates in the concrete permitted by the specification of concrete according to EN 206. In case D_{max} is known, D_{lower} can be replaced by D_{max}. The appropriate choice of aggregate size (D) depends on spacing and density of reinforcement. The definition of D_{lower} via d_{dg} in EN 12620 can lead to a range of aggregate gradings. Similarly EN 206 does not specify a minimum coarse aggregate content. The model is calibrated against tests carried out with typical gradings.

The use of non-typical aggregate gradings where the percentage of larger aggregate sizes in relation to D_{lower} is small can result in different behaviour. This can be avoided by specifying grading parameters in addition to D_{lower}.

In case of prestressed members without ordinary reinforcement, f_{yd} in Formula (8.20) may be replaced by $f_{pd} - \sigma_{pm,\infty}$ where $\sigma_{pm,\infty}$ is the prestress in the tendons after losses.

(5) In planar members (such as solid slabs and shells) with out-of-plane shear forces $v_{Ed,x}$ and $v_{Ed,y}$ acting on the cross-sections perpendicular to the *x* and *y* directions, the design shear force per unit width (v_{Ed}) should be calculated as:

$$v_{Ed} = \sqrt{v_{Ed,x}^2 + v_{Ed,y}^2} \tag{8.21}$$

The effective depth *d* may be taken as a function of the ratio of the shear forces $v_{Ed,y}/v_{Ed,x}$:

— $d = d_x$ for $v_{Ed,y}/v_{Ed,x} \leq 0{,}5$; (8.22)

— $d = 0{,}5\ (d_x + d_y)$ for $0{,}5 < v_{Ed,y}/v_{Ed,x} < 2$; (8.23)

— $d = d_y$ for $v_{Ed,y}/v_{Ed,x} \geq 2$. (8.24)

Alternatively, the effective depth *d* may be taken as:

$$d = d_x \cdot \cos^2\alpha_v + d_y \cdot \sin^2\alpha_v \tag{8.25}$$

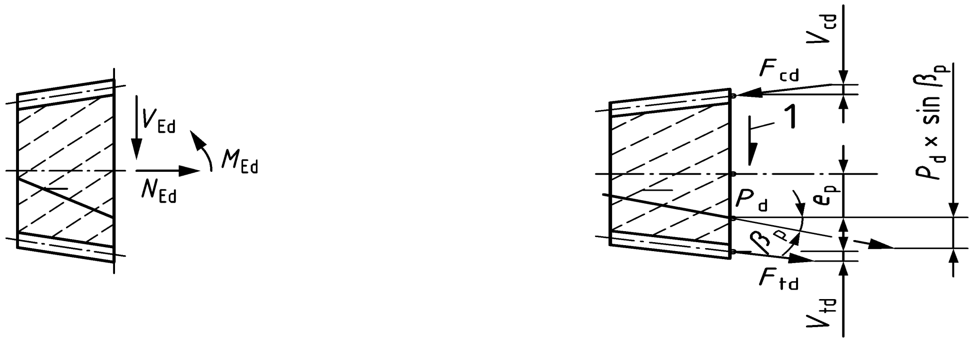
where the angle α_v between the principal shear force and x-axis may be taken as

$$\alpha_v = \arctan(v_{Ed,y}/v_{Ed,x}) \tag{8.26}$$

(6) When the shear force in planar members is not constant along the control section, it may be averaged over a width not larger than $2d$ on both sides from the peak of the shear force, provided that the moment equilibrium after redistribution is fulfilled. If other internal forces are required for calculation of the shear resistance, they may be also averaged over the same width.

(7) In members with inclined chords, the design shear force in the web should account for the influence of inclined forces according to Figure 8.4, replacing V_{Ed} by $V_{Ed} - V_{tcd} - V_{bcd}$.

The favourable effect of V_{tcd} and V_{bcd} should only be considered for members with shear reinforcement according to 8.2.3.



a) internal forces at cross-section

b) compression field, chord and prestressing forces

Key

1 design shear force in web according to (7) or (8) replacing V_{Ed}, N_{Ed} or M_{Ed}

Figure 8.4 — Shear components for members with inclined chords and/or prestressing

(8) In members where the statically determinate part of prestressing is considered as an internal action (i.e. considered as resistance and that prestressing effect is not included in the load combination used to determine V_{Ed}):

— the design shear force in the web should account for the transversal component of the prestressing force according to Figure 8.4, replacing V_{Ed} by $V_{\mathrm{Ed}} - P_{\mathrm{d}} \cdot \sin\beta_{\mathrm{p}}$;

— and where the shear resistance depends on the acting axial force and the bending moment, N_{Ed} and M_{Ed} should be replaced by $N_{\mathrm{Ed}} - P_{\mathrm{d}} \cdot \cos\beta_{\mathrm{p}}$ and $M_{\mathrm{Ed}} - P_{\mathrm{d}} \cdot e_{\mathrm{p}} \cdot \cos\beta_{\mathrm{p}}$, respectively.

— For bonded tendons, the increase of tendon stress due to shear cracking may be considered if compatibility conditions are fulfilled.

(9) When a load is applied through the depth of the member (i.e. at the intersection of primary and secondary beams) or applied in tension to the face of the member (i.e. hanging loads), sufficient reinforcement, in addition to that required for shear, shall be provided to carry the load to the face opposite to the direction of load (i.e. the top face in the case of gravity loads).

(10) In linear members, regions with geometric discontinuities should be designed according to 8.5. In planar members:

— regions with variation of cross-sections should be reinforced according to 12.4.1(4) or designed according to 8.5;

— regions with inserts should be verified according to 8.2.2(11) or designed according to 8.5.

(11) Regions where significant concentrated loads are applied at a distance a_{q} less than d from a support (Figure 8.5) should be designed as discontinuity regions, by using strut-and-tie models or stress fields as in 8.5 or by accounting for a reduced value of τ_{Ed} using the method in 8.2.2(9). In planar members on line supports without shear reinforcement, the shear verification between the load and the support ($a_{\mathrm{q}} < d$) may be omitted, provided that:

— $\tau_{\mathrm{Ed}} \leq 2\tau_{\mathrm{Rdc,min}}$;

— and

— the flexural reinforcement is fully anchored at the support and at the load introduction.

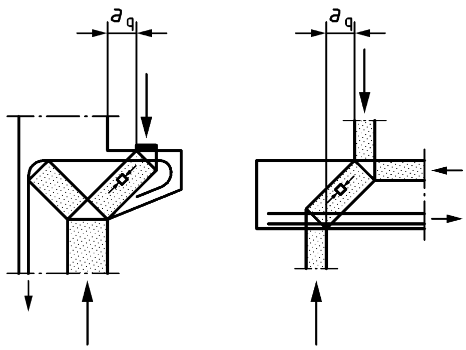


Figure 8.5 — Examples of loads near supports

8.2.2 Detailed verification for members without shear reinforcement

(1) Except where concentrated loads are applied at a distance less than d from the support the detailed verification of the shear resistance may be omitted for control sections that are closer than d from the face of the support or from a significant concentrated load (see Figure 8.6). When significant concentrated loads are applied between d and $2d$ from the face of the support, a control section located at a distance d from the face of the support should be verified according to 8.2.2(9).

a) cases of predominantly distributed loads

b) cases of predominantly concentrated loads near supports according to 8.2.2(9)

Key

1 regions where shear strength verification may be omitted

2 concentrated load

3 support

Figure 8.6 — Regions where shear strength verification may be omitted

(2) The design value of the shear stress resistance should be taken as:

$$\tau_{\mathrm{Rd,c}} = \frac{0{,}66}{\gamma_{\mathrm{V}}} \cdot \left(100\rho_{\mathrm{l}} \cdot f_{\mathrm{ck}} \cdot \frac{d_{\mathrm{dg}}}{d}\right)^{\frac{1}{3}} \geq \tau_{\mathrm{Rdc,min}} \tag{8.27}$$

where

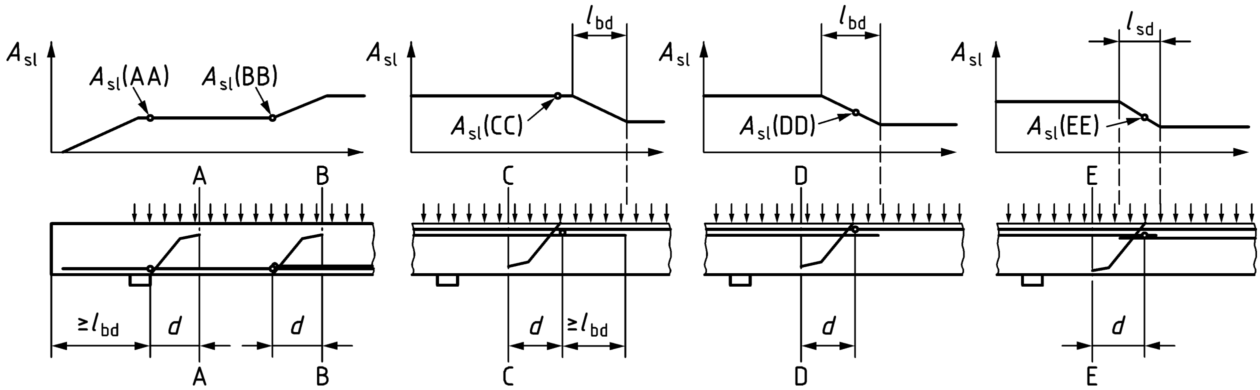
$$\rho_{\mathrm{l}} = \frac{A_{\mathrm{sl}}}{b_{\mathrm{w}} d} \tag{8.28}$$

A_{sl} is the effective area of tensile reinforcement at the distance d beyond the section considered (see Figure 8.7);

b_{w} is the width of the cross-section of linear members. The width b_{w} for cross-sections with variable width and for circular cross-sections is defined in 8.2.3(9);

d_{dg} is defined in 8.2.1(4);

d is the effective depth d_{nom}. The value d may be refined according to (3) and (4) for non-slender members and members with axial force.



Key

Control sections A–A and C–C:	Cases where anchored and curtailed reinforcement may be fully accounted for
Control section B–B:	Case where curtailed reinforcement may not be accounted for
Control sections D-D and E–E:	Cases where curtailed or spliced reinforcement may be partially accounted for

Figure 8.7 — Definition of A_{sl} in Formula (8.28)

(3) The value of d in Formula (8.27) may be replaced by the mechanical shear span a_v, for members with an effective shear span a_{cs} shorter than 4 d:

$$a_v = \sqrt{\frac{a_{cs}}{4} \cdot d} \tag{8.29}$$

Where a_{cs} is the effective shear span with respect to the control section. For reinforced concrete members it may be calculated as a function of the internal forces at control section:

$$a_{cs} = \left|\frac{M_{Ed}}{V_{Ed}}\right| \geq d \tag{8.30}$$

where

M_{Ed} and V_{Ed} include the effects of prestressing according to 8.2.1(8).

(4) In presence of axial forces N_{Ed}, acting at the control section, the value of d in Formula (8.27) or a_v in Formula (8.29) should be multiplied by coefficient k_{vp} according to Formula (8.31):

$$k_{vp} = 1 + \frac{N_{Ed}}{|V_{Ed}|} \frac{d}{3 \cdot a_{cs}} \geq 0{,}1 \tag{8.31}$$

(5) Alternatively to Formula (8.27) in combination with Formula (8.31), the approach considering the effect of compressive normal forces according Formula (8.32) may be used:

$$\tau_{\mathrm{Rdc,min}} \leq \tau_{\mathrm{Rd,c}} = \tau_{\mathrm{Rdc,0}} - k_1 \cdot \sigma_{\mathrm{cp}} \leq \tau_{\mathrm{Rdc,max}} \tag{8.32}$$

where $\tau_{\mathrm{Rdc,0}}$ is the design value of the shear stress resistance defined as follows:

$$\tau_{\mathrm{Rdc,0}} = \frac{0{,}66}{\gamma_{\mathrm{V}}} \cdot \left(100\rho_{\mathrm{l}} \cdot f_{\mathrm{ck}} \cdot \frac{d_{\mathrm{dg}}}{d}\right)^{\frac{1}{3}} \tag{8.33}$$

where

σ_{cp} $= N_{\mathrm{Ed}} / A_{\mathrm{c}}$;

A_{c} is the area of concrete cross-section.

NOTE 1 The factor k_1 can be calculated according to Formula (8.34) unless the National Annex gives another value.

$$k_1 = \frac{0{,}5}{a_{\mathrm{cs,0}}}\left(e_{\mathrm{p}} + \frac{d}{3}\right) \cdot \frac{A_{\mathrm{c}}}{b_{\mathrm{w}} \cdot z} \leq 0{,}18 \cdot \frac{A_{\mathrm{c}}}{b_{\mathrm{w}} \cdot z} \tag{8.34}$$

where

e_{p} is the eccentricity of the prestressing force or of the external load that produces the compressive axial force with respect to the centre of gravity of the cross-section considered as positive towards the tensile side. For statically indeterminate members, the effect of hyperstatic moments due to prestressing should be considered by modifying the tendons eccentricity accordingly;

$a_{\mathrm{cs,0}}$ is determined according to Formula (8.30) without considering in M_{Ed} und V_{Ed} the effect of prestressing or external load that produces the compressive axial force.

For the given factor k_1 according to Formula (8.34), the effective depth d in Formula (8.33) may be replaced by $a_{\mathrm{v,0}}$ where $a_{\mathrm{v,0}}$ is determined according to Formulae (8.29) and (8.30), without considering in M_{Ed} und V_{Ed} the effect of prestressing or external load that produces the compressive axial force.

The value of $\tau_{\mathrm{Rdc,max}}$ may be determined as follows:

$$\tau_{\mathrm{Rdc,max}} = 2{,}15 \cdot \tau_{\mathrm{Rdc,0}} \left(\frac{a_{\mathrm{cs,0}}}{d}\right)^{1/6} \leq 2{,}7 \cdot \tau_{\mathrm{Rdc,0}} \tag{8.35}$$

(6) Prestressing effects according to 8.2.1(8) should be considered in the values of M_{Ed}, V_{Ed} and N_{Ed} to be used in Formula (8.31). For prestressed members with bonded tendons, the effective depth d and the reinforcement ratio ρ_{l} may be calculated as follows:

$$d = \frac{d_{\mathrm{s}}^2 \cdot A_{\mathrm{s}} + d_{\mathrm{p}}^2 \cdot A_{\mathrm{p}}}{d_{\mathrm{s}} \cdot A_{\mathrm{s}} + d_{\mathrm{p}} \cdot A_{\mathrm{p}}} \tag{8.36}$$

$$\rho_{\mathrm{l}} = \frac{d_{\mathrm{s}} \cdot A_{\mathrm{s}} + d_{\mathrm{p}} \cdot A_{\mathrm{p}}}{b_{\mathrm{w}} \cdot d^2} \tag{8.37}$$

The area of prestressed reinforcement A_{p} may be omitted in the calculation of d and ρ_{l} if including A_{p} reduces the shear resistance due to the reduced effective depth, provided that the longitudinal tension reinforcement A_{s} is sufficient to carry M_{Ed} and N_{Ed} taking into account the effect of prestressing.

NOTE 2 The effect of unbonded and external tendons is considered on action side.

(7) In planar members with different reinforcement ratios in both directions, ρ_l should be calculated as a function of the ratio of the shear forces $v_{Ed,y}/v_{Ed,x}$:

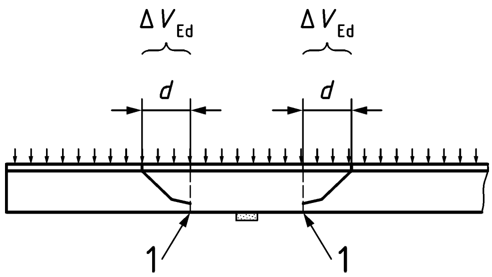
— $\rho_l = \rho_{l,x}$ for $v_{Ed,y}/v_{Ed,x} \leq 0{,}5$; (8.38)

— $\rho_l = \rho_{l,x} \cdot \cos^4\alpha_v + \rho_{l,y} \cdot \sin^4\alpha_v$ for $0{,}5 < v_{Ed,y}/v_{Ed,x} < 2$; (8.39)

— $\rho_l = \rho_{l,y}$ for $v_{Ed,y}/v_{Ed,x} \geq 2$ (8.40)

where α_v is defined in 8.2.1(5).

(8) In case of distributed loads (except high water- or gas pressure as defined in 4.2.1.6) pushing against the member on the tension side (e.g. distributed gravity loads on the top face of continuous members near intermediate supports, see Figure 8.8, or on cantilevers), the design shear force at any control section may be reduced by ΔV_{Ed} calculated as the sum of the distributed loads acting closer than d from the control section, but not larger than ¼ of the total contribution of the distributed load to the design shear force at the control section.



Key

1 control section

Figure 8.8 — Distributed loads pushing on the tension side of the member that may be subtracted from the design shear force V_{Ed}

(9) In case of concentrated forces pushing against each other within a clear distance $d \leq a_q \leq 2\,d$ (e.g. loads and support forces, see Figure 8.5 for the definition on a_q), the contribution of these forces to the design shear force between them may be multiplied by $0{,}5a_q/\,d$. Alternatively to 8.2.1(11), this may be used for $0{,}5d \leq a_q < d$, provided that the longitudinal reinforcement is designed and fully anchored for the tensile force due to the concentrated load. For $a_q < 0{,}5d$ a value of $a_q = 0{,}5d$ may be assumed. The shear stress calculated using the full contribution of these forces should be less than $0{,}6\nu f_{cd}$.

(10) For the design of longitudinal reinforcement, the M_{Ed}-line should be shifted by a distance d in the unfavourable direction as shown in Figure 12.1. Alternatively, the bending moment may be increased by $d \cdot |V_{Ed}|$.

(11) Concreted-in pipes, pipe bundles or slab inserts in the plane of the member and not aligned with the principal shear force direction:

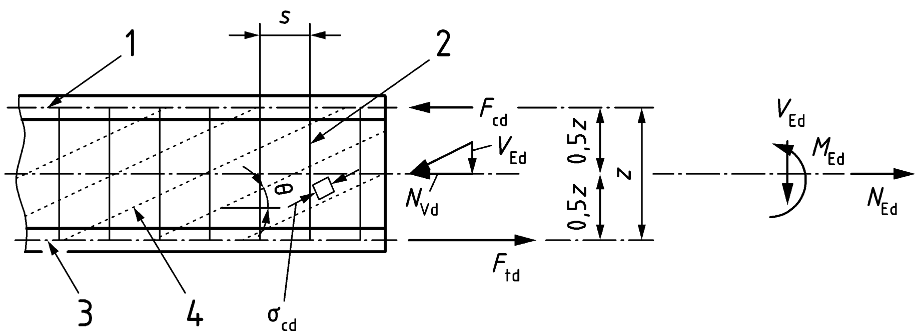
— may be neglected if the width and height are less than $d/6$;

— should be taken into account if their width or height is larger than $d/6$. In that case, the effective shear-resisting depth d should be reduced by the highest value of width and depth.

Pipe bundles with clear distance smaller than $0{,}25d$ should be considered as a single opening.

8.2.3 Members with shear reinforcement

(1) The design of members with shear reinforcement should be based on a compression field (Figure 8.9). Limiting values for the angle θ of the inclined compression field in the web are given in (4). Provisions for inclined shear reinforcement are given in (13).



Key

1 axis of compression chord

2 shear reinforcement

3 axis of tension chord

4 struts (compression field)

Figure 8.9 — Model and notation for shear reinforced members

(2) Control sections should be considered at all critical locations including the face of the support, at a geometrical discontinuity or at a change in shear reinforcement ratio.

In regions where there is no discontinuity of V_{Ed} (e.g. for uniformly distributed loading pushing against the member), the shear reinforcement in any length increment $l = z \cdot \cot\theta$ may be calculated using the smallest value of V_{Ed} in the increment (except in members under high water- or gas pressure).

(3) The lever arm z for the shear calculation (Figure 8.9) may be assumed as in 8.2.1(3).

Prestressed tendons away from the tension chord may be neglected in calculating the centroid of tensile reinforcement, provided that the reinforcement in the tension chord is sufficient to carry the tensile force F_{td} according to (8).

(4) The inclination of the compression field in the web carrying shear may be selected within the following range:

$$1 \leq \cot\theta \leq \cot\theta_{min} \tag{8.41}$$

where the cotangent of the minimal inclination of the compression field θ_{min} should be for shear reinforcement of ductility class B or C:

— $\cot\theta_{min} = 2{,}5$ for ordinary reinforced members without axial force;

— $\cot\theta_{min} = 3{,}0$ for members subjected to significant axial compressive force (average axial compressive stress $\geq |3\text{ MPa}|$) and provided that the depth of the compression chord x determined from a sectional analysis according to 8.1.1 and 8.1.2 is less than $0{,}25d$. Interpolated values between 2,5 and 3,0 may be adopted for intermediate cases. For very high compressive forces ($x > 0{,}25d$), (11) can apply;

— $\cot\theta_{min} = 2{,}5 - 0{,}1 \cdot N_{Ed}/|V_{Ed}| \geq 1{,}0$ for members subjected to axial tension.

For shear reinforcement of ductility class A, $\cot\theta_{min}$ shall be reduced by 20 %.

(5) The shear stress resistance perpendicular to the longitudinal member axis in case of yielding of the shear reinforcement shall be calculated according to:

$$\tau_{Rd,sy} = \rho_w \cdot f_{ywd} \cdot \cot\theta \tag{8.42}$$

where the shear reinforcement ratio ρ_w is defined as:

$$\rho_w = \frac{A_{sw}}{b_w \cdot s} \tag{8.43}$$

The stress in the compression field in all cross-sections shall be verified according to:

$$\sigma_{cd} = \tau_{Ed}(\cot\theta + \tan\theta) \leq \nu \cdot f_{cd} \tag{8.44}$$

NOTE 1 For the case with simultaneous yielding of the shear reinforcement and failure of the compression field, the shear stress resistance results from the solution of Formulae (8.42) and (8.44) as:

$$\tau_{Rd} = \rho_w \cdot f_{ywd} \cdot \cot\theta \leq \frac{\nu \cdot f_{cd}}{2}$$

where $\cot\theta$ is taken from:

$$\cot\theta_{min} \geq \cot\theta = \sqrt{\frac{\nu \cdot f_{cd}}{\rho_w \cdot f_{ywd}} - 1} \geq 1$$

For the case with simultaneous yielding of the tension chord and failure of the compression field, the shear stress resistance results from the solution of Formulae (8.44) and (8.51) with $F_{td} = A_{st} \cdot f_{yd}$.

(6) A value $\nu = 0{,}5$ may be adopted when using the angles of the compression field given in (4).

(7) Angles of the compression field inclination to the member axis lower than θ_{min} given in (4) or values of factor ν higher than according to (6) may be adopted provided that the ductility class of the reinforcement is B or C and that the value of factor ν is calculated on the basis of the state of strains of the member according to:

$$\nu = \frac{1}{1{,}0 + 110 \cdot (\varepsilon_x + (\varepsilon_x + 0{,}001) \cdot \cot^2\theta)} \leq 1{,}0 \tag{8.45}$$

where ε_x is the average strain of the bottom and top chords calculated at a cross-section not closer than $0{,}5 \cdot z \cdot \cot\theta$ from the face of the support or a concentrated load:

$$\varepsilon_x = \frac{\varepsilon_{xt} + \varepsilon_{xc}}{2} \geq 0 \tag{8.46}$$

where the following may be assumed unless more refined methods are used:

$$\varepsilon_{xt} = \frac{F_{td}}{A_{st}E_s} \tag{8.47}$$

$$\varepsilon_{xc} = \frac{-F_{cd}}{A_{cc}E_c} \quad \text{if the flexural compression chord is in compression; and} \tag{8.48}$$

$$\varepsilon_{xc} = \frac{|F_{cd}|}{A_{sc}E_s} \quad \text{if the flexural compression chord is in tension.} \tag{8.49}$$

where

F_{td} and F_{cd}	are the chord forces according to Figure 8.9 and Formulae (8.51) to (8.52) (positive values of F_{cd} refer to compression in the compression chord);
A_{st} and A_{sc}	are the areas of the longitudinal reinforcement in the flexural tension chord and flexural compression chord, respectively;
A_{cc}	is the area of the flexural compression chord.

In prestressed members with bonded tendons, the areas of the longitudinal reinforcement A_{st} and A_{sc} may be increased by $A_p(1/2+e_p/z)$ and $A_p(1/2-e_p/z)$, respectively, where the eccentricity of the tendon e_p is positive on the side of the tension chord.

NOTE 2 For high values of cotθ, the cracking state of the web under serviceability conditions can be governing (see 9.2.3(7)).

(8) The additional tensile axial force, N_{Vd}, due to shear V_{Ed} may be calculated from:

$$N_{Vd} = |V_{Ed}| \cdot \cot\theta \tag{8.50}$$

This force may be added to both chords so that the chord forces F_{td} and F_{cd} (Figure 8.9) are:

$$F_{td} = \frac{M_{Ed}}{z} + \frac{N_{Vd} + N_{Ed}}{2} \tag{8.51}$$

$$F_{cd} = \frac{M_{Ed}}{z} - \frac{N_{Vd} + N_{Ed}}{2} \tag{8.52}$$

In case of direct intermediate support or in the region of concentrated loads, F_{td} may be limited to:

$$\frac{M_{Ed,max}}{z} + \frac{N_{Ed}}{2} \tag{8.53}$$

where $M_{Ed,max}$ is the maximum moment along the member and the internal forces (N_{Ed}, V_{Ed} and M_{Ed}) are applied in the centre of the web with a depth z as shown in Figure 8.9. In these cases, the tension chord may alternatively be designed by shifting the M_{Ed}-line according to 12.3.2. The simplification suggested by 8.2.3(3) for the inner lever arm z in shear design should not be applied for the verification of the chord forces in case of high compression zones (i.e. when the inner lever arm z is significantly less than 0,9d).

For $N_{Vd}+N_{Ed} > 0$, the forces N_{Vd} and N_{Ed} may also be carried totally or partially by a longitudinal web reinforcement. In this case, the sum $N_{Vd}+N_{Ed}$ in Formulae (8.51) and (8.52) may be reduced accordingly.

(9) For sections with variable width:

— b_w may conservatively be taken as the smallest width of the cross-section between the tension chord and the neutral axis (Figures 8.10 a) and b));

— the area A_{sw} to be used in Formula (8.43) shall be multiplied by cosδ (refer to Figure 8.10 a)).

For circular cross-sections:

— the area A_{sw} should be multiplied by the ratio b_w/D_h (refer to Figure 8.10 c)), where D_h is the hoop diameter;

— z is based on a section fitted into the circular section as given in Figure 8.10 c) where the circular segment with depth x_{sb} is the area of the compression chord and the tension bars within b_w define the tension chord. The width b_w can be chosen freely fulfilling equilibrium and resistance conditions, however not larger than D_h.

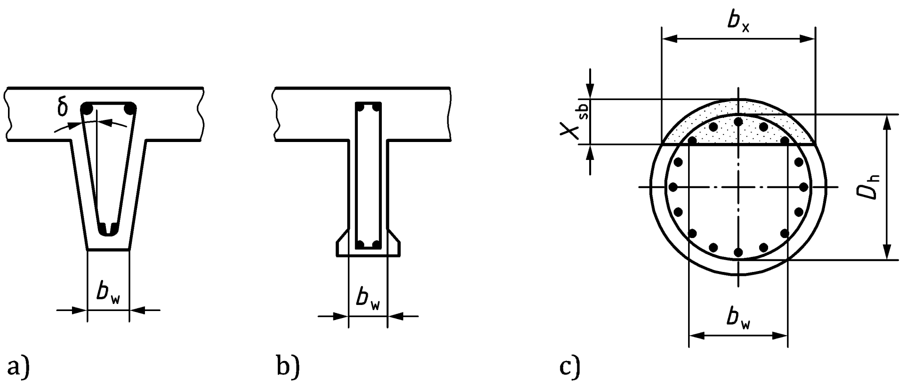


Figure 8.10 — Definition of b_w for sections with variable width

(10) Where the web contains ducts of diameters such that $\Sigma\varnothing_{duct} > b_w/8$, the stress in the compression field according to Formula (8.44) and the shear stress resistance τ_{Rd} according to Formulae in 8.2.3(5), NOTE 1 shall be calculated on the basis of a nominal web width given by:

$$b_{w,nom} = b_w - k_{duct}\Sigma\phi_{duct} \tag{8.54}$$

where ϕ_{duct} is the outer diameter of the duct and $\Sigma\phi_{duct}$ is determined for the most unfavourable level. The value of coefficient k_{duct} should be evaluated depending on the material and filling of the duct as:

— $k_{duct} = 0,5$ for grouted steel ducts;

— $k_{duct} = 0,8$ for grouted plastic ducts with a wall thickness $\leq \max\{0,035\phi_{duct}; 2\text{ mm}\}$;

— $k_{duct} = 1,2$ for non-grouted ducts, for grouted plastic ducts with a wall thickness $> \max\{0,035\phi_{duct}; 2\text{ mm}\}$ or for ducts injected with soft filling material.

In the case of variable cross-section widths, calculations at different heights can be necessary to determine the decisive nominal value of the web width.

The effect of ducts does not need to be considered when checking shear resistance without shear reinforcement unless ducts are not grouted or are injected with soft filling materials.

(11) For members subjected to design axial compression forces N_{Ed}, a portion of the axial force denoted as N_{Edw} can be resisted by the shear zone (web). If $-N_{Edw} \leq |V_{Ed} \cdot \cot\theta|$, and $\cot\theta$ fulfils the recommended values in (4), the shear zone should be calculated as specified in (5) to (8). Otherwise, the design method specified in Annex G should be used for the shear zone.

For members subjected to high design axial forces, N_{Edw} should be chosen so that the depth of the compression chords (i.e. x) carrying $N_{Ed} + N_{Vd}$ and M_{Ed} is not higher than $0,25d$.

(12) In cases where concentrated loads are applied at a distance $a_v = z \cdot \cot\beta_{incl}$ less than $z \cdot \cot\theta$ from a support (Figure 8.11), the shear stress resistance may be enhanced according to:

$$\tau_{Rd} = \nu \cdot f_{cd}\frac{\cot\theta - \cot\beta_{incl}}{1 + \cot^2\theta} + \rho_w \cdot f_{ywd} \cdot \cot\beta_{incl} \leq \nu \cdot f_{cd}\frac{\cot\theta}{1 + \cot^2\theta} \tag{8.55}$$

NOTE 3 The maximum shear resistance can be calculated by optimization varying $\cot\theta$. For the case of a constant value ν according to (6), the optimum is obtained with

$$\cot\theta = \cot\beta_{incl} + \sqrt{1 + \cot^2\beta_{incl}} \leq \cot\theta_{min};$$

for the case of a variable ν according to (7), a reasonable approximation is obtained assuming

$$\cot\theta = 1,3 \cdot a/z,$$

but should not be larger than $\cot\theta$ according to 8.2.3(5), NOTE 1.

Compression field inclinations with $\cot\theta < 1$ are allowed if the yield strength f_{ywd} in Formulae (8.55) and (8.57) is replaced by the stress σ_{swd} in the shear reinforcement according to:

$$\sigma_{swd} = E_s[\cot^2\theta\cdot(\varepsilon_x + 0{,}001) - 0{,}001] \leq f_{ywd} \tag{8.56}$$

where the longitudinal strain ε_x may be calculated according to (7) for a cross-section located midway between the support and the load.

In addition to the axial tensile force, N_{Vd}, due to shear V_{Ed} according to Formula (8.50), also the following moment ΔM_{Ed} should be added to M_{Ed} to be used in Formulae (8.51) to (8.52):

$$\Delta M_{Ed} = (\tau_{Ed} - \rho_w \cdot f_{ywd} \cdot \cot\theta) \cdot z \cdot b_w \cdot \left(\frac{a}{2} - x\right) \tag{8.57}$$

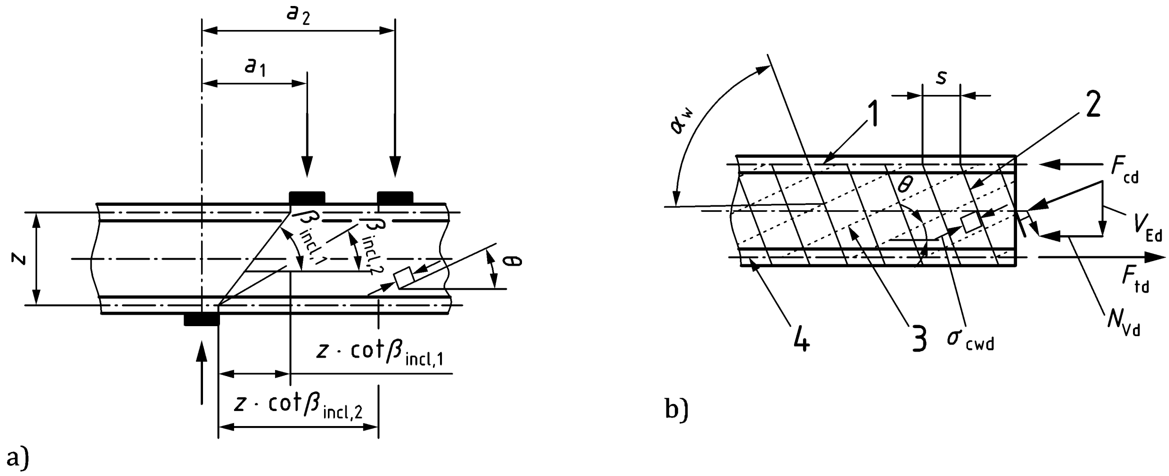
where

a is the distance between the axis of the support and the concentrated force (see Figure 8.11a);

x is the distance between the support and the investigated cross-section.

NOTE 4 The increase of chord forces F_{td} and F_{cd} due to shear according to Formulae (8.50), (8.52) and (8.57) is based on the assumption of a constant compression field inclination θ in the support region. Other solutions respecting equilibrium conditions can be designed according to 8.5 (see Figure 8.26b)).

In case of two or more concentrated forces in the distance $z \cdot \cot\theta$, all potential critical inclinations β_{incl} according to Figure 8.11a) shall be verified.



a) in presence of concentrated loads near the supports

b) truss model and notation for members with inclined shear reinforcement

Key

1 axis of compression chord

2 inclined shear reinforcement

3 struts (compression field)

4 axis of tension chord

Figure 8.11 — Definition of inclinations $\beta_{incl} \geq \theta$ for concentrated loads near the supports and case with inclined shear reinforcement

In case of a distributed load q_d, the term $\rho_w \cdot f_{ywd}$ in Formulae (8.55) and (8.57) may be replaced by $(\rho_w \cdot f_{ywd} + q_d / b_w)$.

(13) For members with inclined shear reinforcement ($45° \leq \alpha_w < 90°$) where α_w is measured positive as shown in Figure 8.11 b), Formulae (8.41), (8.42), (8.44) and (8.50) should be replaced respectively by:

$$\tan\frac{\alpha_w}{2} \leq \cot\theta \leq \cot\theta_{min} \tag{8.58}$$

$$\tau_{Rd,sy} = \rho_w \cdot f_{ywd} \cdot (\cot\theta + \cot\alpha_w) \cdot \sin\alpha_w \tag{8.59}$$

$$\sigma_{cd} = \tau_{Ed}\frac{1+\cot^2\theta}{\cot\theta + \cot\alpha_w} \leq \nu \cdot f_{cd} \tag{8.60}$$

$$N_{Vd} = |V_{Ed}| \cdot (\cot\theta - \cot\alpha_w) \tag{8.61}$$

Angles $\alpha_w > 90°$ should be avoided. For spiral reinforcement, α_w may be assumed as the average angle of both legs, provided that the difference of each leg inclination and the 90 degrees is not greater than 12 degrees.

Formula (8.55) should be replaced by:

$$\begin{aligned}\tau_{Rd} = \nu \cdot f_{cd}\frac{\cot\theta - \cot\beta_{incl}}{1+\cot^2\theta} + \rho_w \cdot f_{ywd} \cdot (\cot\beta_{incl} + \cot\alpha_w)\sin\alpha_w \\ \leq \nu \cdot f_{cd}\frac{\cot\theta + \cot\alpha_w}{1+\cot^2\theta}\end{aligned} \tag{8.62}$$

Compression field inclinations with $\cot\theta < \tan(\alpha_w/2)$ are allowed if the yield strength f_{ywd} in Formula (8.62) is replaced by the stress σ_{swd} in the shear reinforcement according to:

$$\sigma_{swd} = E_s\left[(\varepsilon_x + 0{,}001) \cdot \frac{(\cot\theta + \cot\alpha_w)^2}{1+\cot^2\alpha_w} - 0{,}001\right] \leq f_{ywd} \tag{8.63}$$

8.2.4 In-plane shear and transverse bending

(1) The interaction between shear stress τ_{Ed} and transverse bending m_{Ed} (see Figure 8.12) may be disregarded if $\tau_{Ed}/\tau_{Rd} < 0{,}2$ or $m_{Ed}/m_{Rd} < 0{,}1$

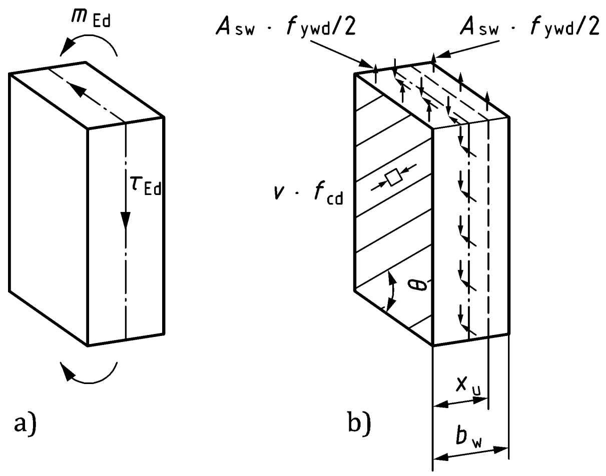
where

- τ_{Rd} is the shear resistance according to Formula in 8.2.3(5), NOTE 1;
- m_{Rd} is the bending resistance without interaction with shear.

(2) The shear stress resistance τ_{Rdm} reduced by the influence of transverse bending, in case of shear reinforcement perpendicular to the longitudinal axis of the member and symmetric to the web middle plane, may be assumed as:

$$\tau_{Rdm} = \tau_{Rd}\sqrt{1 - \frac{m_{Ed}}{m_{Rd}}} \tag{8.64}$$

Alternatively, Annex G may be used.



a) web with shear stress τ_{Ed} and transverse bending moment m_{Ed}

b) eccentric compression field carrying shear and transverse bending

Figure 8.12 — Interaction between shear and transverse bending

8.2.5 Shear between web and flanges

(1) The longitudinal shear stress, τ_{Ed} at the junction between one side of a flange and the web may be determined by the change of the axial (longitudinal) force in the part of the flange considered, according to:

$$\tau_{Ed} = \frac{\Delta F_d}{h_f \cdot \Delta x} \tag{8.65}$$

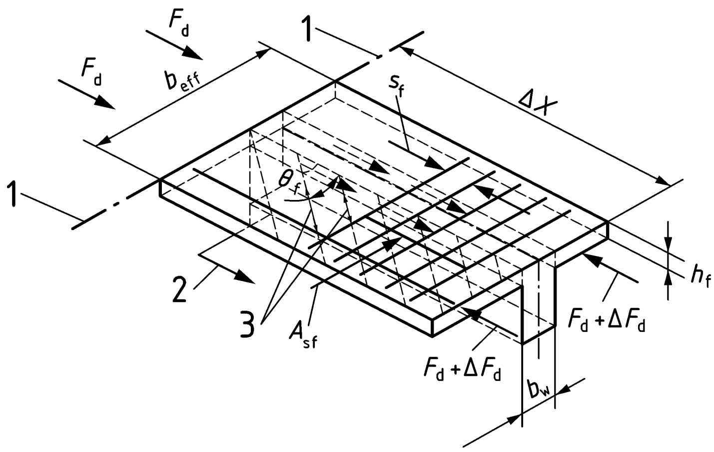
where

h_f is the thickness of the flange at the junctions;

Δx is the length under consideration, see Figure 8.13;

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

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 it is maximum. Where point loads are applied, the length Δx should not exceed the distance between point loads.



Key

1 cross section

2 longitudinal bar anchored beyond this projected point (see 8.2.5(7))

3 struts

Figure 8.13 — Notations for the connection between flange and web

(2) In case the following condition is satisfied, further verification of the shear between web and flanges may be omitted and no extra reinforcement above that for transverse bending is required:

$$\tau_{\mathrm{Ed}} \leq \frac{A_{\mathrm{st,min}}}{s_{\mathrm{f}} \cdot h_{\mathrm{f}}} \cdot f_{\mathrm{yd}} \tag{8.66}$$

where $A_{\mathrm{st,min}}$ is the minimum transverse reinforcement according to Table 12.1 (NDP).

(3) In cases not complying with Formula (8.66), the shear strength of the flange may be calculated by considering the flange as a system of compression fields combined with ties in the form of tensile reinforcement (see Figure 8.13). The inclination of the compression field in the flanges with respect to the longitudinal axis may be selected within the following range:

— $1 \leq \cot\theta_{\mathrm{f}} \leq 3{,}0$ in compression flanges; (8.67)

— $1 \leq \cot\theta_{\mathrm{f}} \leq 1{,}25$ in tension flanges. (8.68)

(4) The transverse reinforcement in the flange A_{sf} may be determined as follows:

$$\tau_{\mathrm{Ed}} \leq \frac{A_{\mathrm{sf}}}{s_{\mathrm{f}} \cdot h_{\mathrm{f}}} \cdot f_{\mathrm{yd}} \cdot \cot\theta_{\mathrm{f}} \tag{8.69}$$

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

$$\sigma_{\mathrm{cd}} = \tau_{\mathrm{Ed}}(\cot\theta_{\mathrm{f}} + \tan\theta_{\mathrm{f}}) \leq \nu \cdot f_{\mathrm{cd}} \tag{8.70}$$

where the following strength reduction factor may be used:

$$\nu = 0{,}5 \tag{8.71}$$

(5) Lower angles of the inclined compression field in the tensile flange than those given in (3) may be adopted provided the value of factor ν is calculated on the basis of the state of strains of the member

according to Formula (8.45) where ε_x is the longitudinal strain in the tensile flange and may be estimated as:

$$\varepsilon_{\mathrm{x}} = \frac{F_{\mathrm{td}}}{A_{\mathrm{st}} E_{\mathrm{s}}} \geq 0 \tag{8.72}$$

where A_{st} and F_{td} are the area of the longitudinal reinforcement and the force in the tension chord, respectively (refer to 8.2.3 (7) and (8)).

(6) The influence of transverse bending may be addressed according to 8.2.4 or Annex G. Alternatively, the reinforcement in the flange may be designed as follows:

— if reinforcement is placed only in the tension zone due to transverse bending, the area of the steel should be the greater of that required for bending and that required for shear;

— if transverse reinforcement is placed at top face as well as at bottom face of the flange (i.e. stirrups with two legs) the area of each leg should be the greater of that required for bending and that required for shear.

(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 cut 1–1 of Figure 8.13).

8.2.6 Shear at interfaces

(1) This clause shall be applied where the static equilibrium depends on shear transfer across a given interface, such as an interface between two concretes cast at different times (see Figure 8.14) or an interface between concrete and a similar material (e.g. concrete cast on rock surfaces).

(2) For very rough interfaces with sufficiently anchored minimum reinforcement according to 12.2 crossing the interface at an angle according to Figure 8.15b), the verification may be omitted. The roughness of the interfaces is defined in (6).

(3) The shear stress at the interface should satisfy the following condition:

$$\tau_{\mathrm{Edi}} \leq \tau_{\mathrm{Rdi}} \tag{8.73}$$

where

τ_{Rdi} is the design shear resistance at the interface. If no reinforcement across the interface is required or if the required reinforcement across the interface is sufficiently anchored, τ_{Rdi} should be calculated by Formula (8.76). In other cases according to (7), Formula (8.77) should be used.

(4) The design value of the shear stress in an interface should be taken as:

$$\tau_{\mathrm{Edi}} = \frac{V_{\mathrm{Edi}}}{A_{\mathrm{i}}} \tag{8.74}$$

where

V_{Edi} is the shear force acting parallel to the interface;

A_i is the area of the interface according to Figure 8.14. For keyed interfaces, A_i should be based on either the key area A1, A2 or A3 according to Figure 8.14 whichever is governing taking into account the corresponding concrete strength.

The longitudinal shear stress between concrete interfaces due to composite action may be taken as:

$$\tau_{\mathrm{Edi}} = \frac{\beta_{\mathrm{new}} \cdot V_{\mathrm{Ed}}}{z \cdot b_{\mathrm{i}}} \tag{8.75}$$

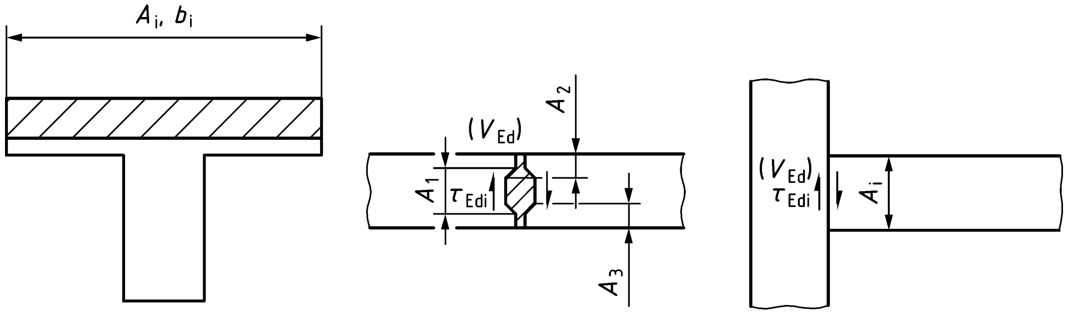
where

β_{new} 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 shear force acting perpendicular to the interface;

z is the lever arm of composite section;

b_i is the width of the interface.



$A_i = min\{A_1; (A_2 + A_3)\}$

Figure 8.14 — Examples of interfaces

(5) The design shear stress resistance at the interface for situations without reinforcement across the interface or if the required reinforcement across the interface is anchored for $\sigma_{sd} = f_{yd}$ may be taken as:

$$\tau_{\mathrm{Rdi}} = c_{\mathrm{v1}} \frac{\sqrt{f_{\mathrm{ck}}}}{\gamma_{\mathrm{C}}} + \mu_{\mathrm{v}}\, \sigma_{\mathrm{n}} + \rho_{\mathrm{i}} f_{\mathrm{yd}}\, (\mu_{\mathrm{v}} \sin\alpha + \cos\alpha) \leq 0{,}30\, f_{\mathrm{cd}} + \rho_{\mathrm{i}} f_{\mathrm{yd}} \cos\alpha \tag{8.76}$$

where

f_{ck} is the lowest compressive strength of the concretes at the interface;

σ_n is the compressive stress over the interface area A_i caused by the minimum external axial force across the interface that acts simultaneously with the shear force. Permanent stresses caused by confinement of surrounding structural parts may be taken into account. When σ_n is a compressive stress σ_n shall not be taken larger than $0{,}60 f_{cd}$;

When σ_n is a tensile stress $\mu_v \cdot \sigma_n$ shall be taken as 0;

ρ_i $= A_{si}/A_i$;

A_{si} is the cross-sectional area of bonded reinforcement crossing the interface and anchored for $\sigma_{sd} = f_{yd}$, including ordinary shear reinforcement (if any), with adequate anchorage according to 11.4 at both sides of the interface. Tensile forces across interfaces shall be carried by reinforcement placed additional to the interface reinforcement A_{si};

α is defined in Figure 8.15b), and is limited to $35° \leq \alpha \leq 135°$; except for very smooth surfaces, where $35° \leq \alpha \leq 90°$;

c_{v1} and μ_v are factors which depend on the roughness of the interface (see Table 8.2 and (6)). The formula assumes the contact surface to be clean and free of laitance, dust or other adhesion-reducing particles.

Table 8.2 — Coefficients depending on the roughness of the surface

	Formula (8.76)		Formula (8.77)		
Surface roughness	c_{v1}	μ_v	c_{v2}	k_v	k_{dowel}
very smooth	0,01[a]	0,5	0	0	1,5
smooth	0,08[a]	0,6	0	0,5	1,1
rough	0,15[a]	0,7	0,08[a]	0,5	0,9
very rough	0,19[a]	0,9	0,15[a]	0,5	0,9
keyed[b]	0,37	0,9	—	—	—

[a] When the interface is subjected to tensile stresses caused by external axial force in perpendicular direction: $c_{v1} = 0$ and $c_{v2} = 0$.

[b] The factors for keyed interfaces shall be applied for the area of each key considering its concrete strength.

(6) The roughness of the concrete interfaces may be classified as follows:

very smooth: a surface cast against steel, plastic or specially prepared wooden moulds;

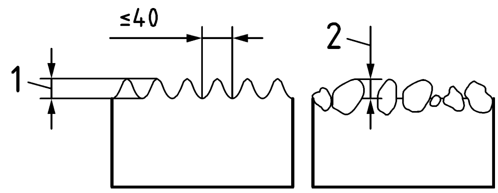
smooth: a surface with less than 3 mm roughness (from peak to valley), e.g. a free surface left without further treatment after compacting;

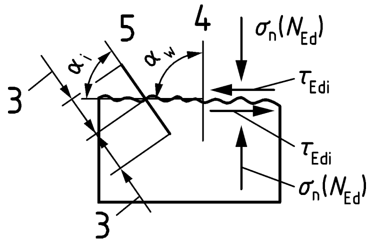
rough: a surface with at least 3 mm roughness (from peak-to-valley, maximum 40 mm spacing), achieved by raking, exposing of aggregate or other methods according to Figure 8.15a);

very rough: a surface with at least 6 mm roughness (from peak-to-valley, maximum 40 mm spacing), achieved by raking, exposing of aggregate or other methods according to Figure 8.15a);

keyed: a surface with shear keys complying with Figure 8.15c).

Dimensions in millimetres



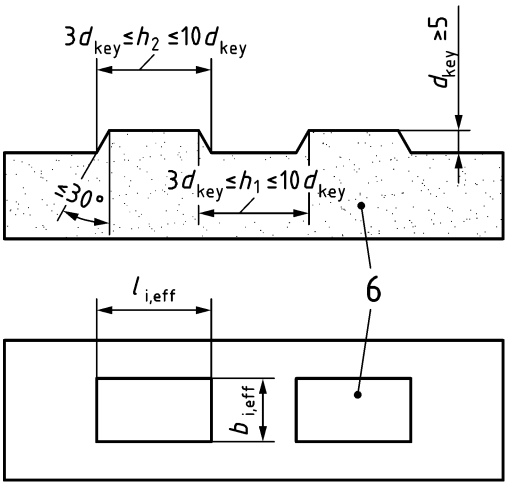


Shear reinforcement: $45° \leq \alpha_w \leq 90°$

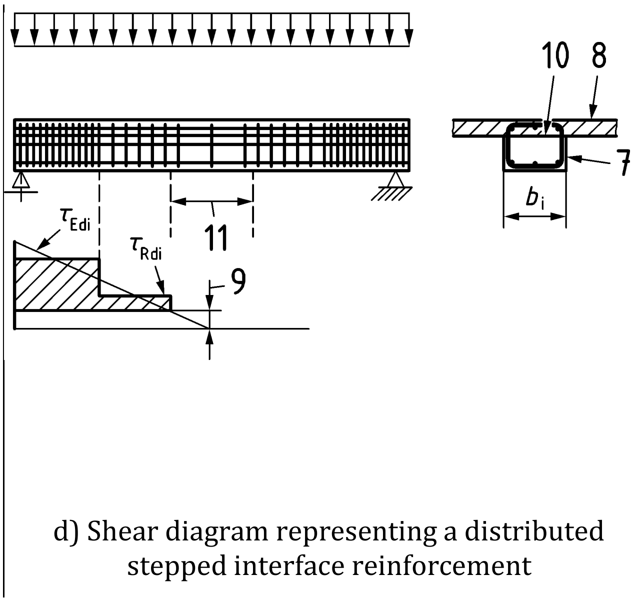
Interface reinforcement: $35° \leq \alpha \leq 135°$

a) Rough/very rough interfaces

b) Shear/interface reinforcement



c) Keyed interfaces



d) Shear diagram representing a distributed stepped interface reinforcement

Key

1 roughness by raking

2 roughness by exposed aggregates

3 anchorage

4 shear reinforcement $45° \leq \alpha_w \leq 90°$

5 interface reinforcement $35° \leq \alpha_i \leq 135°$ (except for very smooth surfaces, where $35° \leq \alpha_i \leq 90°$)

6 keys with $0,5 \leq h_1/h_2 \leq 2,0$; Area of key: $A_i = b_{i,eff} \cdot l_{i,eff}$

7 first concrete pouring section

8 second concrete pouring section

9 interface shear resistance without interface reinforcement

10 interface

11 minimum shear reinforcement $A_{sw,min}$ (if necessary)

Figure 8.15 — Classification of interfaces and definition of interface reinforcement

(7) If yielding of the required reinforcement crossing the interface is not ensured, due to insufficient anchorage (e.g. structural toppings) the shear stress resistance may be taken as:

$$\tau_{Rdi} = c_{v2} \frac{\sqrt{f_{ck}}}{\gamma_C} + \mu_v \sigma_n + k_v \rho_i f_{yd} \mu_v + k_{dowel} \rho_i \sqrt{f_{yd} \cdot f_{cd}} \leq 0,25 f_{cd} \tag{8.77}$$

where

c_{v2}, k_v, k_{dowel} are factors which depend on the roughness of the interface (see Table 8.1 and (6));

μ_v, σ_n as defined in (5).

If the distance of an intersecting reinforcing bar to an edge in the direction of the acting shear force is less than 10ϕ, the coefficient for dowel action of reinforcement (last term in Formula (8.77)) should be taken as $k_{dowel} = 0$. The interface reinforcement should be anchored for a stress of at least $0{,}5f_{yd}$ with a minimum length of embedment of 8ϕ if no other methods of anchorage than by straight bars are applied. For interface reinforcement with α=90° and an embedment length of at least 8ϕ, but anchored for a stress lower than $0{,}5f_{yd}$, Formula (8.77) may be used with $c_{v2}=\mu_v=k_v=0$.

In the case of horizontal shear transfer in slab members with cast-in-place structural toppings and rough or very rough interfaces, the coefficient c_{v2} may be increased by a factor of 1,2 for determining the design value τ_{Rdi} of the interface shear resistance.

(8) The longitudinal shear resistance of grouted joints between (precast) slab or wall elements may be calculated from Formula (8.76). However, in cases where the joint can be significantly cracked, for very smooth, smooth and rough interfaces, c_{v1} should be taken as zero, for very rough interfaces $c_{v1} = 0{,}19$ and for keyed joints $c_{v1} = 0{,}37$ according to Table 8.1 (see also 10.7 in case of fatigue).

(9) If interface reinforcement in composite slabs is required, the spacing between the reinforcing bars crossing the interface should not exceed the following:

— shear transfer direction: $2{,}5h \leq 300$ mm, where h is the depth of the slab;

— perpendicular to shear transfer direction: $5h \leq 750$ mm (≤ 375 mm to the edge).

Along edges of composite slabs where delamination of the topping cannot be prevented by permanent loads (e.g. from walls), the minimum interface reinforcement per unit length along the edge should be calculated by:

$$a_{s,min} = t_{min} f_{ctm}/f_{yk} \tag{8.78}$$

where

t_{min} smaller value of the thickness of new and old concrete layer;

f_{ctm} mean tensile strength of respective concrete layer.

(10) When reinforcement is required across the interface to satisfy Formulae (8.76) or (8.77), a simplified “step approach” may be used, by which each step has a maximum length of $2d$ for linear and $3d$ for planar members, and for which the integral of the design effect is covered by the reinforcement within the step, see Figure (8.15 d)). The spacing of the bars should be designed to ensure that $\tau_{Rdi} > \tau_{Edi}$ calculated in the central point of each step, complying with the spacing requirements defined above.

8.3 Torsion and combined actions

8.3.1 General considerations for torsion

(1) Where a specific stiffness has been considered in the analysis according to 7.1(6), the corresponding internal forces shall be considered in design. If a specific stiffness was neglected in the analysis, e.g. torsional stiffness, then normally the corresponding internal forces, e.g. torque, may be neglected at the ultimate limit state. In such cases, a minimum reinforcement, given in 12.2, 12.3 and 12.6, in the form of stirrups and longitudinal bars should be provided in order to prevent excessive cracking.

(2) Solid sections may be modelled as equivalent thin-walled sections. Where warping torsion may be ignored, cross-sections with complex shapes, such as T-sections, may be divided into a series of sub-

sections, each one of which is modelled as an equivalent thin-walled section. Each thin-walled section may be designed separately according to 8.3.2.

(3) For the purpose of reinforcement design, the distribution of the acting torsional moments over the sub-sections may be in proportion to their uncracked torsional stiffnesses or, alternatively, in proportion to their maximum possible torsional capacity according to Formula (8.84) assuming the same strut inclination for all sub-sections. The reinforcement may then be designed assuming different compression field inclinations.

8.3.2 Internal forces due to torsion in compact or closed sections

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

(2) The shear stress in a wall element of a section subject to a torsional moment T_{Ed} may be calculated from:

$$\tau_{t,i} = \frac{T_{Ed}}{2A_k \cdot t_{eff,i}} \tag{8.79}$$

where

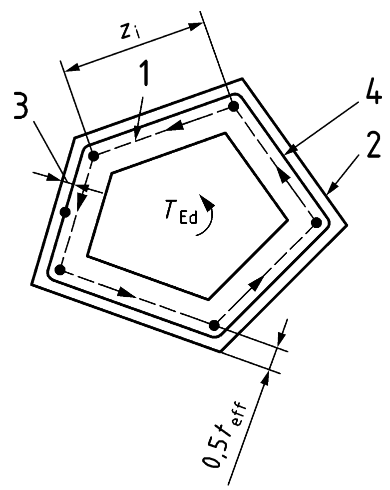
A_k is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas;

$\tau_{t,i}$ is the torsional shear stress in wall i;

$t_{eff,i}$ is the effective wall thickness. It may be taken as A/u, but should not be taken as less than twice the distance between the outer concrete surface and the centre of the longitudinal reinforcement. For hollow sections the actual thickness is an upper limit;

A is the total area of the cross-section, including inner hollow areas;

u is the outer perimeter of the cross-section.



Key

1 centre line

2 outer edge of effective cross-section, circumference u

3 cover

4 torsional web reinforcement A_{sw}

Figure 8.16 — Notations and definitions used in 8.3

(3) The shear force $V_{Ed,i}$ in a wall element i due to torsion may be taken as:

$$V_{Ed,i} = \tau_{t,i}\, t_{eff,i}\, z_i \tag{8.80}$$

8.3.3 Internal forces due to torsion in open sections

(1) In open thin walled members it may be necessary to consider warping torsion. In this case the different parts of the section should be designed according to the rules for bending and longitudinal axial force in 8.1 and shear in 8.2.

8.3.4 Torsional resistance of compact or closed sections

(1) For a single cell, thin-walled section or a sub-section with constant effective wall thickness t_{eff}, the design torsional capacity may be calculated as:

$$\tau_{t,Rd} = \min\{\tau_{t,Rd,sw};\tau_{t,Rd,sl};\ \tau_{t,Rd,max}\} \tag{8.81}$$

Where

$\tau_{t,Rd,sw}$, $\tau_{t,Rd,sl}$ and $\tau_{t,Rd,max}$ are torsional stress resistances given by Formulae (8.82) to (8.84) and determined on the basis of Annex G.

The torsional capacity according to Formula (8.81) should be used when combination of internal forces are verified according to 8.3.6.

(2) The torsional capacity when governed by yielding of the shear reinforcement, $\tau_{t,Rd,sw}$, or the longitudinal reinforcement, $\tau_{t,Rd,sl}$, may be calculated as

$$\tau_{t,Rd,sw} = \cot\theta \frac{A_{sw}}{t_{eff} \cdot s} f_{ywd} \tag{8.82}$$

and

$$\tau_{t,Rd,sl} = \frac{\sum A_{sl}\, f_{yd}}{t_{eff} u_k \cot\theta} \tag{8.83}$$

where

u_k is the perimeter of the area A_k;

f_{yd} is the design yield stress of the longitudinal reinforcement A_{sl};

$\Sigma A_{sl} f_{yd}$ is the yield force of the longitudinal reinforcement, that may be included in the calculation of the torsional capacity. The amount of longitudinal reinforcement considered in Formula (8.83) should have a resultant tensile force that acts at the centroid of the equivalent thin-walled closed cross-section. The reinforcement should generally be distributed according to 12.3.3(9);

f_{ywd} is the design yield stress of the shear reinforcement;

A_{sw} is the cross-sectional area of the shear reinforcement within the effective wall thickness in Figure 8.16;

s is the spacing (in the longitudinal direction) between the shear reinforcement A_{sw};

θ is the angle of compression field with respect to the longitudinal axis.

(3) The torsional strength, when governed by crushing of the compression field in concrete, may be calculated from:

$$\tau_{t,Rd,max} = \frac{\nu \cdot f_{cd}}{\cot\theta + \tan\theta} \tag{8.84}$$

where ν may be determined by the formulae in Annex G. A value of $\nu = 0,60$ may be used when $\cot\theta = 1,0$.

(4) The inclination of the compression field to be used in Formulae (8.82) to (8.84) should for usual cases be chosen within the following range:

$$\frac{1}{\cot\theta_{min}} \leq \cot\theta \leq \cot\theta_{min} \tag{8.85}$$

where $\cot\theta_{min}$ is defined in 8.2.3(4).

(5) When at the same time $b_{max}/b_{min} < 1,5$ and $c > 0,07b_{min}$ calculations of the torsional capacity should be based on a reduced cross-section with reduced cover equal to $0,07b_{min}$.

where

c	is the concrete cover to stirrups;
b_{min}; b_{max}	are the minimum and maximum side lengths of the section (sub-section) , i.e. the width and the height in case of a rectangular cross-section.

NOTE This reduction accounts for a risk for partial spalling of the cover.

8.3.5 Design procedure for combination of actions

(1) Design for combined action of torsion, bending, shear and axial forces may follow:

— the procedure of designing individual wall elements according to (2) or, alternatively;

— a simplified procedure based on interaction formula as presented in 8.3.6.

(2) The design procedure for combination of actions may be based on a thin-walled closed section complying with 8.3.2, in which the sectional forces and moments are replaced by a statically equivalent set of normal- and shear stress distributions. The distribution of normal and shear stresses may be determined by conventional elastic or plastic methods. Calculation of the reinforcement and check of compression in the concrete struts in each individual wall may use the formulae of Annex G, or 8.2.3.

(3) For solid sections, if beneficial, a portion of the axial force and the shear force may also be distributed to the solid core inside the thin-walled section.

8.3.6 Interaction formula

(1) A simplified and conservative verification of the resistance of cross-sections subjected to combination of internal forces may be performed based on the following linear criterion:

$$\sum \left(\frac{S_{Ed}}{S_{Rd}}\right)_i \leq 1,0 \tag{8.86}$$

where

i	is the internal force considered;

S_{Ed}; S_{Rd} are the individual design action and, respectively, the corresponding individual design resistance of the cross-section (e.g. design torsional moment and pure design torsional resistance (refer to 8.3.4), design bending moment and pure design bending moment resistance and design shear stress and shear stress resistance).

(2) The S_{Ed}/S_{Rd}-ratios for shear actions and for the corresponding bending moments need not be inserted simultaneously in Formula (8.86) provided that the bending moment capacity is based on the area of reinforcement that does not include the part reserved for resisting N_{vd} according to 8.2.3(8). In this case, two separate verifications may be carried out considering the following combinations:

a) bending, torsion and axial force; and

b) shear, torsion and axial force.

(3) Alternatively, other safe approximations of the interaction diagram established on the basis of 8.3.5(2) may be used.

8.4 Punching

8.4.1 General

(1) The rules in 8.4 complement those given in 8.2 and cover punching shear in solid slabs and waffle slabs with solid areas over supporting areas (columns, capitals, shearheads, wall ends and wall corners). The rules presented hereafter for supporting areas apply by analogy to loaded areas of planar members and foundations (column bases).

(2) The punching shear resistance shall be verified according to the following procedure:

(i) detailed verification of the punching shear resistance may be omitted, provided that the following condition at the control perimeter $b_{0,5}$ is satisfied:

$$\tau_{Ed} \leq \tau_{Rdc,min} \quad \text{according to 8.2.1(4);} \tag{8.87}$$

(ii) punching shear reinforcement may be omitted when the following condition at the control perimeter $b_{0,5}$ is satisfied :

$$\tau_{Ed} \leq \tau_{Rd,c} \quad \text{according to 8.4.3;} \tag{8.88}$$

(iii) where $\tau_{Ed} > \tau_{Rd,c}$, punching shear reinforcement should be provided in an area in accordance with (v) and designed in accordance with (iv). In addition, the maximum punching shear resistance at the control perimeter $b_{0,5}$ should not be exceeded:

$$\tau_{Ed} \leq \tau_{Rd,max} \quad \text{according to 8.4.4(5) and (6);} \tag{8.89}$$

(iv) where required, the punching shear reinforcement should be provided to satisfy the following condition at the control perimeter $b_{0,5}$:

$$\tau_{Ed} \leq \tau_{Rd,cs} \quad \text{according to 8.4.4(1) to (4) and complying with the detailing rules of 12.5.1;} \tag{8.90}$$

(v) if shear reinforcement is required, a further control perimeter $b_{0,5,out}$ where shear reinforcement is no longer required shall be checked according to 8.4.4(7) or (8).

8.4.2 Shear-resisting effective depth, control perimeter and shear stress

(1) The shear-resisting effective depth of the slab d_v should be taken as the distance from the supporting area to the average level of the reinforcement layers, see Figure 8.17.

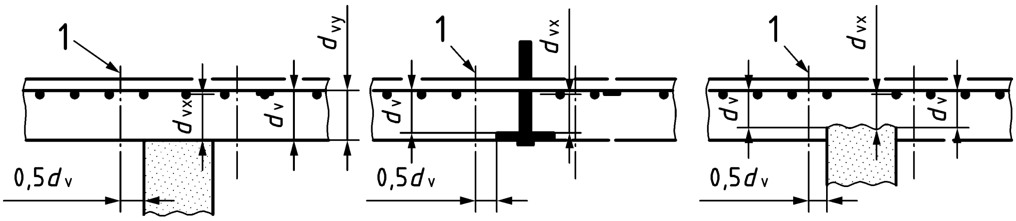
$$d_v = \frac{d_{vx} + d_{vy}}{2} \tag{8.91}$$

where d_{vx} and d_{vy} are the nominal values.

NOTE 1 d_{vx} and d_{vy} apply unless the National Annex permits alternative use of d_{dx} and d_{dy}.

When the column penetrates into the slab by more than $d/20$, d_v should be determined according to Figure 8.17c).

NOTE 2 The tolerance of the column penetration into the slab during construction can be given in the project specification (for instance on the drawings).



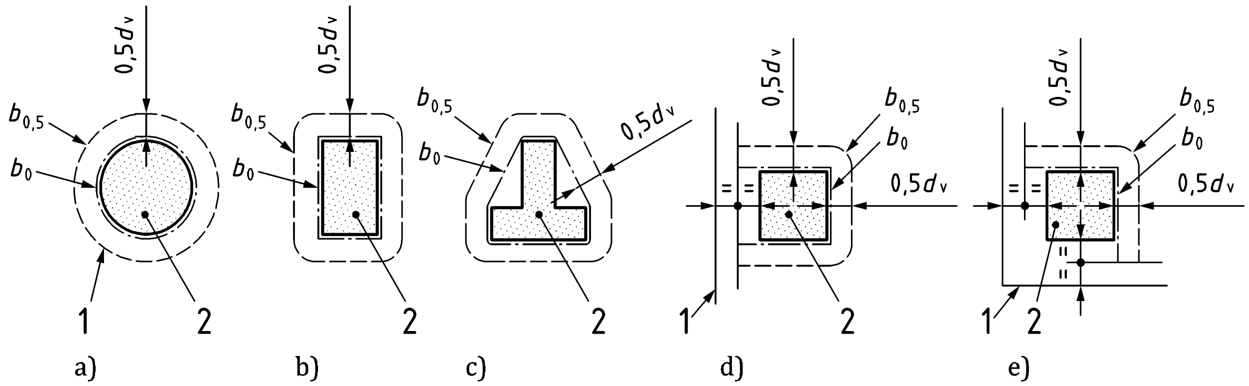
a) direct support b) hanging support with penetration c) penetration by direct support

Key

1 control perimeter

Figure 8.17 — Shear-resisting effective depth of the slab d_v considering effective level of supporting area

(2) The control perimeter $b_{0,5}$ should be taken at a distance 0,5d_v from the face of the supporting area except near to re-entrant corners of the supporting area (see Figure 8.18 c) where the distance between the control perimeter and the face of the column should be increased so that the length of the control perimeter is minimised. For edge and corner columns with overhangs, the extension is limited to half of the overhang (see Figures 8.18 d) and e)). For edge and corner columns with large overhangs, the control perimeter around the column as in internal columns can be governing.



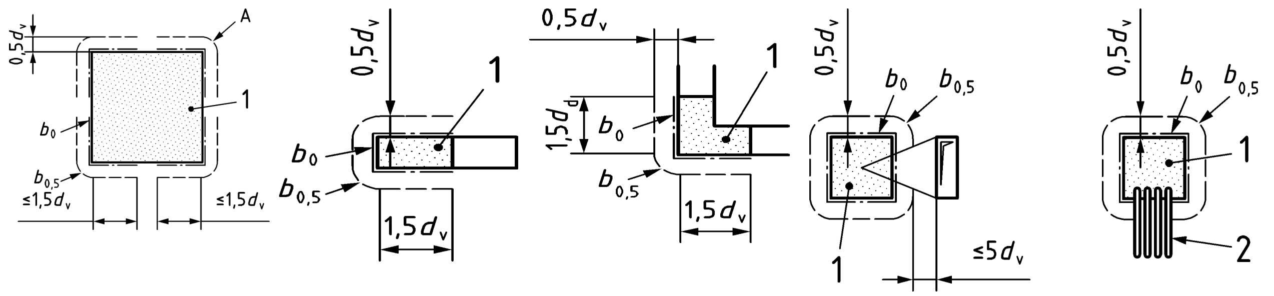
Key

1 slab edge

2 supporting area

Figure 8.18 — Typical control perimeters $b_{0,5}$ and perimeters b_0 around supporting areas (same perimeter shapes)

(3) The effect of concentration of the shear forces at the corners of large supporting areas may be taken into account by reducing the control perimeter assuming that the length of its straight segments does not exceed $3d_v$ for each edge (refer to the reduced control perimeter length shown in Figure 8.19 a). In large columns, without a detailed analysis, only the reduced control perimeter according to Figure 8.19 a) should be accounted for. In wall ends and wall corners, the load carried by the supporting areas defined in Figures 8.19 b) and c) should be verified for punching resistance whereas the load carried outside the supporting areas should be verified for shear according to 8.2.1 and 8.2.2. Columns where the width is larger than $6d_v$ may be considered as wall ends or wall corners. For edge and corner columns with overhangs, the length of the straight segments shown in Figures 8.18 d) and e) is also limited to $3d_v$.



a) around large supporting areas b) around wall end c) around wall corner d) near opening e) with inserts

Key

1 supporting area

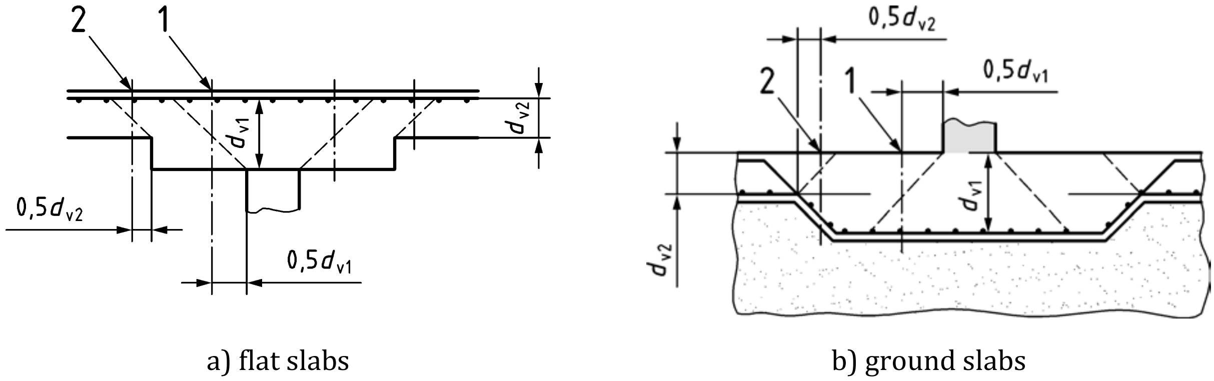
2 inserts

Figure 8.19 — Length of control perimeter $b_{0,5}$ and support perimeter b_0 around supporting areas

(4) The effect of openings or inserts may be neglected if the distance between the nearest face of the opening and the control perimeter exceeds $5d_v$. Otherwise, the part of the control perimeter contained

between the two tangents drawn to the outline of the opening from the centre of the supporting area should be considered to be ineffective (refer to Figures 8.19 d) and e)).

(5) In the case of slabs with variable depth, in addition to the control perimeter near to the supporting area, also other control perimeters with their corresponding shear-resisting effective depth d_v according to Figure 8.20 should be verified.



Key

1 control perimeter 1

2 control perimeter 2

Figure 8.20 — Control perimeter and shear-resisting effective depth of members with variable depth

(6) The design shear stress τ_{Ed} may be calculated as:

$$\tau_{\mathrm{Ed}} = \beta_{\mathrm{e}} \frac{V_{\mathrm{Ed}}}{b_{0,5} \cdot d_{\mathrm{v}}} \tag{8.92}$$

where

V_{Ed} is the design shear force at the control perimeter $b_{0,5}$ (all favourable loads acting on the tensile side of the planar member, soil reactions on foundations and ground slabs and the deviation forces in post-tensioned slabs inside the control perimeter may be deducted from the shear force at centre of supporting area to calculate the design shear force at control perimeter, V_{Ed}). In the case of foundations or ground slabs without shear reinforcement, the soil reaction may be deducted up to a distance of $0,67d_v$ from the face of the column;

β_e is a coefficient accounting for concentrations of the shear forces, which may be adopted from Table 8.3. The approximated values for internal, edge and corner columns may be used only if all following conditions are fulfilled:

— the lateral stability does not depend on frame action of slabs and columns;

— the adjacent spans do not differ in length more than 25 %;

— the slab is only under uniformly distributed loads;

— the moment transferred to the edge and corner columns is not larger than $M_{td,max} = 0,25b_e \cdot d^2 \cdot f_{cd}$ where width b_e is defined in Figure 8.21 c).

Otherwise, the refined values should be adopted. The refined values may also be applied in cases complying with the conditions above for a more refined calculation;

$b_{0,5}$ is the length of the control perimeter.

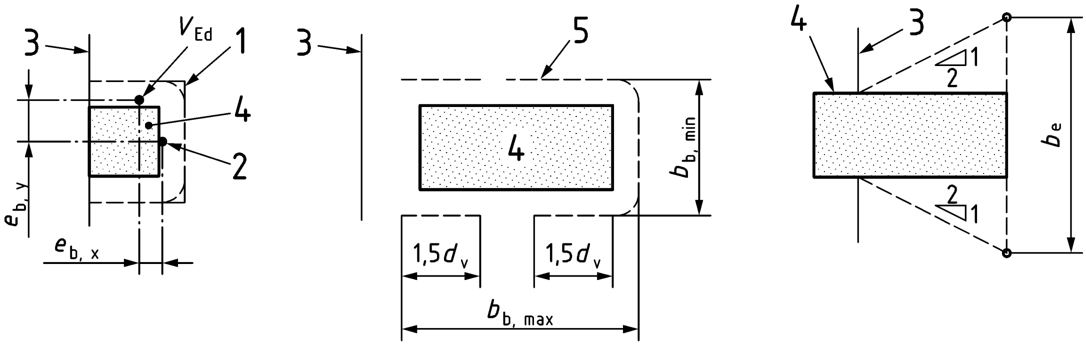
Table 8.3 — Coefficients β_e accounting for concentrations of the shear forces

<table>
<tr><th>Support</th><th>Approximated</th><th colspan="2">Refined[a]</th></tr>
<tr><td>internal columns</td><td>$\beta_e = 1{,}15$</td><td rowspan="3">$\beta_e = 1 + 1{,}1\dfrac{e_b}{b_b} \geq 1{,}05$</td><td>where $e_b = \sqrt{e_{b,x}^2 + e_{b,y}^2}$</td></tr>
<tr><td>edge columns</td><td>$\beta_e = 1{,}4$</td><td>where $e_b = 0{,}5\lvert e_{b,x}\rvert + \lvert e_{b,y}\rvert$</td></tr>
<tr><td>corner columns</td><td>$\beta_e = 1{,}5$</td><td>where $e_b = 0{,}27\left(\lvert e_{b,x}\rvert + \lvert e_{b,y}\rvert\right)$</td></tr>
<tr><td>ends of walls</td><td colspan="3">$\beta_e = 1{,}4$</td></tr>
<tr><td>corners of walls</td><td colspan="3">$\beta_e = 1{,}2$</td></tr>
</table>

[a] $e_{b,x}$, $e_{b,y}$ are the eccentricities of the line of action of the support forces with respect to the centroid of the control perimeter. The line of action of the support forces should be determined accounting for the axial force and the moments in the two directions transferred by the slab to the support (including the internal forces in a column over the slab if present). The control perimeter for calculating its centroid may be simplified replacing parts of circles by corners (see Figure 8.21 a)) and where the straight segments are not limited to $3d_v$ according to (3);

b_b is the geometric mean of the minimum and maximum overall widths of the control perimeter (see Figure 8.21 b). Where the length of straight segments of the control perimeter are limited to $3d_v$ according to (3), the overall width of the control perimeter should not be reduced):

$$b_b = \sqrt{b_{b,min} \cdot b_{b,max}}$$



a) b) c)

a) definition of eccentricities e_{bx} and e_{by}

b) definition of overall widths of $b_{b,min}$ and b_{bmax} for control perimeter $b_{0,5}$

c) definition of width b_e for an edge column

Key

V_{Ed} resultant of support force

1 simplification of the control perimeter with corners instead of parts of circles for calculating its centroid

2 centroid of full control perimeter

3 slab edge

4 column

5 control perimeter

Figure 8.21 — Eccentricity and control perimeter at edge columns for Table 8.3

(7) The design shear stress τ_{Ed} may also be calculated directly from a detailed analysis of the shear stress distribution along the control perimeter using a method accounting for equilibrium and compatibility conditions of the slab (for instance, linear elastic analysis) as:

$$\tau_{\mathrm{Ed}} = \frac{v_{\mathrm{Ed}}}{d_{\mathrm{v}}} \tag{8.93}$$

where the shear force per unit width v_{Ed} may be averaged according to 8.2.1(6) and 2d should be replaced by 2d_v.

(8) In cases where significant concentrated loads ($\geq 0{,}2V_{Ed}$) are applied at a distance from the control perimeter less than 3d_v, the design shear stress τ_{Ed} should be calculated according to (7). In case significant concentrated loads are applied between the supporting area and the control perimeter, the verification should be conducted by using strut-and-tie models or stress fields according to 8.5.

8.4.3 Punching shear resistance of slabs without shear reinforcement

(1) The design punching shear stress resistance should be calculated as follows:

$$\tau_{\mathrm{Rd,c}} = \frac{0{,}6}{\gamma_{\mathrm{V}}} \cdot k_{\mathrm{pb}} \left(100\,\rho_{\mathrm{l}} \cdot f_{ck} \cdot \frac{d_{\mathrm{dg}}}{d_{\mathrm{v}}}\right)^{\frac{1}{3}} \leq \frac{0{,}5}{\gamma_{\mathrm{V}}} \cdot \sqrt{f_{\mathrm{ck}}} \tag{8.94}$$

where

$$\rho_l = \sqrt{\rho_{\mathrm{l,x}} \cdot \rho_{\mathrm{l,y}}} \tag{8.95}$$

$\rho_{l,x}$, $\rho_{l,y}$ are reinforcement ratios of bonded flexural reinforcement in the x- and y-directions respectively. The values of $\rho_{l,x}$ and $\rho_{l,y}$ should be calculated as mean values over the width b_s defined in Figure 8.22. Reinforcement that does not extend at least $2.5d_v+20\phi$ beyond the control perimeter $b_{0,5}$ or 20ϕ beyond the line of contraflexure should not be considered;

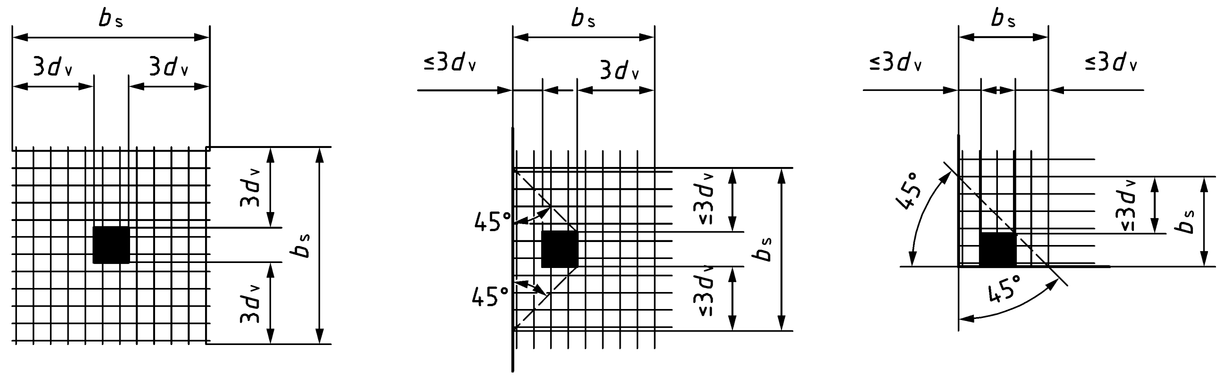
d_{dg} is defined in 8.2.1(4);

k_{pb} is the punching shear gradient enhancement coefficient that may be calculated as:

$$1 \leq k_{pb} = 3{,}6\sqrt{1-\frac{b_0}{b_{0,5}}} \leq 2{,}5 \tag{8.96}$$

b_0 is the length of the perimeter at the face of the supporting area (see Figure 8.18). Near to re-entrant corners of the supporting area and close to slab edges, the perimeter b_0 should be placed parallel to $b_{0,5}$ (see Figures 8.18c)-(e)). For large supporting areas, wall ends, wall corners, slabs with openings and with inserts, the rules of 8.4.2(3) apply (see Figure 8.19);

d_v is calculated according to 8.4.2(1).



a) internal columns b) edge columns c) corner columns

Figure 8.22 — Definition of width b_s

(2) For distances between the centre of the support area and the point of contraflexure in the considered load combination a_p smaller than $8d_v$, the value of d_v in Formula (8.94) may be replaced by:

$$a_{pd} = \sqrt{\frac{a_p}{8} \cdot d_v} \tag{8.97}$$

where

$$a_p = \sqrt{a_{p,x} \cdot a_{p,y}} \geq d_v \tag{8.98}$$

$a_{p,x}$, $a_{p,y}$ are the maximum distances from the centroid of the control perimeter (which may be simplified according to Figure 8.21a) to the two points (on the x- and on the y-axis, respectively) where the bending moments $m_{Ed,x}$, respectively $m_{Ed,y}$, are zero. The distances a_{px} and a_{py} may be calculated according to (3) or using a linear elastic (uncracked) model. The local coordinate system (x,y) has its origin at the centre of the supporting area and coincides with the reinforcement directions (principal directions in case of layers which are not orthogonal). In large columns, wall ends and wall corners, the origin of the local coordinate system may be placed inside the supporting area at a distance $1{,}5d_v$ from the relevant column or wall face if it is more favourable.

(3) For regular flat slabs where the lateral stability does not depend on frame action between the slabs and the columns and that respect the condition $0{,}5 \leq L_x/L_y \leq 2$, the value of a_p may be approximated as $a_{px} \approx 0{,}22L_x$ and $a_{py} \approx 0{,}22L_y$ for the x- and y directions, respectively. For different span lengths in continuous slabs, the largest span length of the bays adjacent to the considered column should be accounted for. For edge columns, in the direction perpendicular to the edge, a_p may be approximated as $a_p = 0{,}22L$ where L is the span perpendicular to the slab edge. For corner columns, a_p may be approximated as $a_p = 0{,}22L$ where L is the largest span length of the adjacent bays in either the x- or y-direction.

(4) For slabs with axial forces and for prestressed slabs, the value of k_{pb} of Formula (8.96) may be multiplied by the coefficient k_{pp}:

$k_{pp} = k_N$ for compressive axial forces (e.g. prestressing); (8.99)

$k_{pp} = 1/k_N$ for tensile axial forces; (8.100)

$$k_N = \sqrt{1 + \frac{0{,}47}{1 - b_0/b_{0,5}} \cdot \frac{|\sigma_d|}{\sqrt{f_{ck}}}} \tag{8.101}$$

where σ_d is the average normal stress over the width b_s defined in Figure 8.22.

In case different axial stresses act in two directions, an average geometric value:

$$k_{pp} = \sqrt{k_{pp,x} \cdot k_{pp,y}} \tag{8.102}$$

may be adopted, where $k_{pp,x}$ and $k_{pp,y}$ are the coefficients accounting for the presence of axial stresses in the x- and y-directions.

For prestressed slabs with eccentric tendons, the beneficial effect of the tendon's eccentricity on the tensile side of the planar member may be considered with:

$$k_N = \sqrt{1 + \frac{0{,}47}{1 - b_0/b_{0,5}} \cdot \frac{|\sigma_d|}{\sqrt{f_{ck}}}\left(1 + 6\frac{e_p}{d}\right)} \tag{8.103}$$

where e_p is the eccentricity of the tendons at axis of the supporting area with the respect to the centre of gravity of the cross-section to be considered as positive for tendons on the tensile side. For statically indeterminate slabs, the effect of hyperstatic moments due to prestressing should be considered by reducing the tendons eccentricity accordingly.

For prestressed slabs with bonded tendons, the effective depth d_v and the flexural reinforcement ratio ρ_l to be used in Formula (8.94) should be calculated according to 8.2.2(6) in both directions and averaged using Formulae (8.91) and (8.95).

The negative influence of tensile forces in the slab, except those related to constraints due to shrinkage and thermal effects, should be accounted for using k_{pp} with a value of less than 1,0.

8.4.4 Punching shear resistance of slabs with shear reinforcement

(1) Where shear reinforcement is required, it should be calculated in accordance to:

$$\tau_{\mathrm{Rd,cs}} = \eta_{\mathrm{c}} \cdot \tau_{\mathrm{Rd,c}} + \eta_{\mathrm{s}} \cdot \rho_{\mathrm{w}} \cdot f_{\mathrm{ywd}} \geq \rho_{\mathrm{w}} \cdot f_{\mathrm{ywd}} \tag{8.104}$$

where

$$\eta_{\mathrm{c}} = \frac{\tau_{\mathrm{Rd,c}}}{\tau_{\mathrm{Ed}}} \tag{8.105}$$

$\tau_{\mathrm{Rd,c}}$ is the punching shear stress resistance of slabs without shear reinforcement according to Formula (8.94).

$$\eta_{\mathrm{s}} = \frac{d_{\mathrm{v}}}{150\phi_{\mathrm{w}}} + \left(15\frac{d_{\mathrm{dg}}}{d_{\mathrm{v}}}\right)^{1/2} \cdot \left(\frac{1}{\eta_{\mathrm{c}} \cdot k_{\mathrm{pb}}}\right)^{3/2} \leq 0{,}8 \tag{8.106}$$

where d_{dg} is defined in 8.2.1(4).

$$\rho_w = \frac{A_{\mathrm{sw}}}{s_{\mathrm{r}} \cdot s_{\mathrm{t}}} \tag{8.107}$$

is the ratio of the vertical shear reinforcement ratio at the investigated control perimeter;

A_{sw} is the area of one leg of shear reinforcement;

s_{r} is the radial spacing of shear reinforcement: $s_{\mathrm{r}} = \max(s_0+s_1/2;\, s_1)$ where s_0 and s_1 are defined in Figure 8.23 a) and should comply with 12.5.1.

s_{t} is the average tangential spacing of perimeters of shear reinforcement measured at the investigated control perimeter (length of the investigated control perimeter divided by the number of shear reinforcement bars arranged on it or near to it).

For shear reinforcement arranged orthogonally over the whole shear reinforced area fulfilling the requirements of 12.5.1, ρ_{w} may be calculated directly on the basis of the spacings in both directions.

(2) Where inclined shear reinforcement is used:

— the shear reinforcement ratio according to Formula (8.107) should be replaced by $\rho_{\mathrm{w}}=A_{\mathrm{sw}}.(\sin\alpha_{\mathrm{w}} + \cos\alpha_{\mathrm{w}})/(s_{\mathrm{r}} \cdot s_{\mathrm{t}})$ for distributed shear reinforcement and by $\rho_{\mathrm{w}}=A_{\mathrm{sw}}.\sin\alpha_{\mathrm{w}}/(d_{\mathrm{v}} \cdot s_{\mathrm{t}})$ for single bent up bars;

— coefficient η_{s} calculated according to Formula (8.106) may be multiplied by the factor $(\sin\alpha_{\mathrm{w}} + \cos\alpha_{\mathrm{w}}) \cdot \sin\alpha_{\mathrm{w}}$ (but respecting that $\eta_{\mathrm{s}} \leq 0{,}8$).

(3) In outer perimeters of shear reinforcement, without an explicit verification, at least the same amount of reinforcement as in the first perimeter should be provided in each perimeter.

(4) Alternatively, the required area of shear reinforcement at the outer perimeters may be calculated according to 8.4.4(1) and (2), where:

— the shear resisting effective depth of the outer shear reinforcement $d_{\mathrm{v,out}}$ should be considered instead of d_{v} for the calculation of τ_{Ed} according to Formula (8.92);

— the control perimeter $b_{0,5,\mathrm{s}}$ placed at the location of the shear reinforcement perimeter (see Figure 8.24) should be considered instead of $b_{0,5}$ for the calculation of τ_{Ed} according to Formula (8.92);

— the flexural reinforcement ratio to be considered according Formula (8.95) for calculating $\tau_{\mathrm{Rd,c}}$ should be anchored with respect to the control perimeter $b_{0,5,\mathrm{s}}$ instead of $b_{0,5}$ and calculated using the full effective depth for bending d in each direction;

— the value of the shear gradient enhancement coefficient k_{pb} for the calculation of $\tau_{Rd,c}$ should be calculated according to Formula (8.96) replacing $b_{0,5}$ by $b_{0,5,s}$ and b_0 by $b_{0,s}$, where the perimeter $b_{0,s}$ is located at a distance $d_{v,out}/2$ from the control perimeter $b_{0,5,s}$ towards the supporting area (see Figure 8.24). In addition, the coefficient k_{pb} may be multiplied by $(d_v/d_{v,out})^{1/2}$ provided that the product respects the limits of Formula (8.96);

— The values d_v in Formula (8.94), a_{pd} in Formula (8.97) and k_{pp} in 8.4.3(4) remain unchanged;

— the shear reinforcement ratio should be calculated assuming s_r=s_1;

— the coefficient β_e accounting for the concentration of shear forces along the control perimeter may be adapted according to the refined method of Table 8.3 by considering the overall dimensions of the control perimeter $b_{0,5,s}$;

— for wall ends, wall corners, large and elongated columns, the perimeters $b_{0,5}$ and $b_{0,5,s}$ may be increased according to Figure 8.24 c), provided that the shear reinforcement is arranged accordingly.

NOTE 1 The value of $d_{v,out}$ is given by:

$$d_{v,out} = \frac{d_x + d_y}{2} - c_v \tag{8.108}$$

where c_v is defined in Figure 8.23 unless the National Annex gives a different value.

(5) The punching shear resistance shall be limited to a maximum of:

$$\tau_{Rd,max} = \eta_{sys} \cdot \tau_{Rd,c} \tag{8.109}$$

NOTE 2 The values of coefficient η_{sys} for shear reinforcement complying with 12.5.1 are:

— $\eta_{sys} = 0{,}70 + 0{,}63 \left(\frac{b_0}{d_v}\right)^{1/4} \geq 1{,}0$ for studs (8.110)

— $\eta_{sys} = 0{,}50 + 0{,}63 \left(\frac{b_0}{d_v}\right)^{1/4} \geq 1{,}0$ for links and stirrups (8.111)

unless the National Annex gives different values.

(6) The maximum punching shear resistance $\tau_{Rd,max}$ at internal columns may be multiplied by η_{pm} considering compressive membrane action provided that:

— no significant openings, inserts or slab edges are present at a distance less than $5d_v$ from the control perimeter $b_{0,5}$; and

— coefficient η_{pm} is not already considered in $\tau_{Rd,c}$ according to I.8.5.1(1).

NOTE 3 The factor is η_{pm} =1,0 for new structures and η_{pm} according to I.8.5.1(1) for existing structures unless the National Annex gives different values.

(7) The outer control perimeter at which shear reinforcement is not required ($b_{0,5,out}$, see Figure 8.24) may be calculated as:

$$b_{0,5,out} = b_{0,5} \cdot \left(\frac{d_v}{d_{v,out}} \cdot \frac{1}{\eta_c}\right)^2 \tag{8.112}$$

where

$b_{0,5}$ is the control perimeter located at a distance $d_v/2$ from the face of the supporting area according to 8.4.2(2);

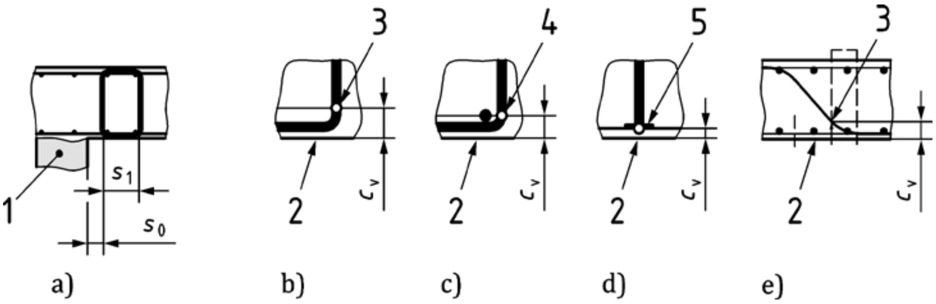
$d_{v,out}$ is the shear-resisting effective depth of the outer shear reinforcement according to Formula (8.108);

η_c as defined in Formula (8.105).

The outermost perimeter of shear reinforcement should be placed at a distance not greater than $0{,}5d_{v,out}$ from the outer control perimeter $b_{0,5,out}$ according to Figure 8.24. Where the spacing of the shear reinforcement exceeds $3d_{v,out}$, the part of $b_{0,5,out}$ greater than $1{,}5d_{v,out}$ from a shear reinforcement assembly, measured parallel to the outer reinforcement perimeter, should be ignored, see Figure 8.24 b).

The flexural reinforcement considered according Formula (8.95) for calculating $\tau_{Rd,c}$ should be anchored with respect to the control perimeter $b_{0,5,out}$ instead of $b_{0,5}$;

(8) Alternatively, the outer control perimeter $b_{0,5,out}$ at which shear reinforcement is not required may be calculated by finding the control perimeter where the punching shear resistance is sufficient without shear reinforcement according to 8.4.3, using the method defined in (4). The value of $b_{0,5,s}$ where shear reinforcement is not required is derived iteratively and becomes $b_{0,5,out}$.



—

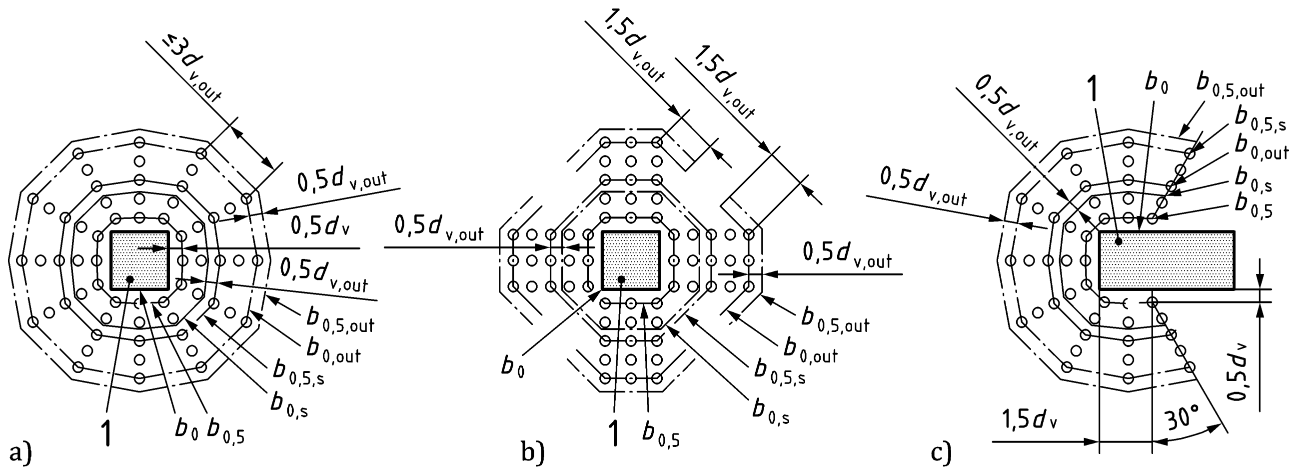
a) definition of radial spacings of punching reinforcement

b) – e) definition of parameter c_v for different anchorage types

Key

1	supporting area (column, wall end or wall corner)	3	level of axis of reinforcement inside the bend
2	limit of bend	4	bottom of stud head

Figure 8.23 — Definition of parameter c_v (for detailing rules, see 12.5.1)



a) control perimeters for radial arrangements ($b_{0,5,s}$ and $b_{0,s}$ refer to the instance of the 3rd perimeter of shear reinforcement)

b) control perimeters for cruciform arrangements

c) control perimeters for wall ends, wall corners, large and elongated columns (instance of wall end with radial arrangement of shear reinforcement)

Key

1 supporting area

Figure 8.24 — Definition of control perimeters for punching reinforcement

8.5 Design with strut-and-tie models and stress fields

8.5.1 General

(1) Strut-and-tie models or stress fields should be used for design and verification of discontinuity regions (i.e. regions where the strain state distribution is not linear such as near concentrated loads or geometric discontinuities) in absence of specific provisions elsewhere in this code or of alternative refined analyses.

NOTE 1 All provisions of 8.2.3 to 8.2.5 and 8.3 and Annex G are consistent with the rules given in 8.5. Linear members with shear reinforcement and without discontinuities can thus also be designed for bending, shear and torsion with the provisions of 8.5. Unless stated otherwise, the provisions in 8.5 apply to cases that can be modelled under plane stress conditions. Alternative and more refined verification methods for struts, ties and nodes can be adopted for special cases (e.g. where triaxial stress conditions apply).

(2) The provisions in 8.5 are applicable without a verification of the deformation capacity provided that the following conditions are fulfilled:

— the member contains a minimum reinforcement according to Clause 12 or the development of uncontrolled cracks is avoided by other means;

— the reinforcement complies with the details according to Clauses 11 and 12;

— the reinforcing steel is either ductility Class B or C.

(3) Strut-and-tie models and stress fields aim at a representation of the stress state in a cracked concrete structure. They shall be in equilibrium with the external actions and shall satisfy strength conditions. They consist of:

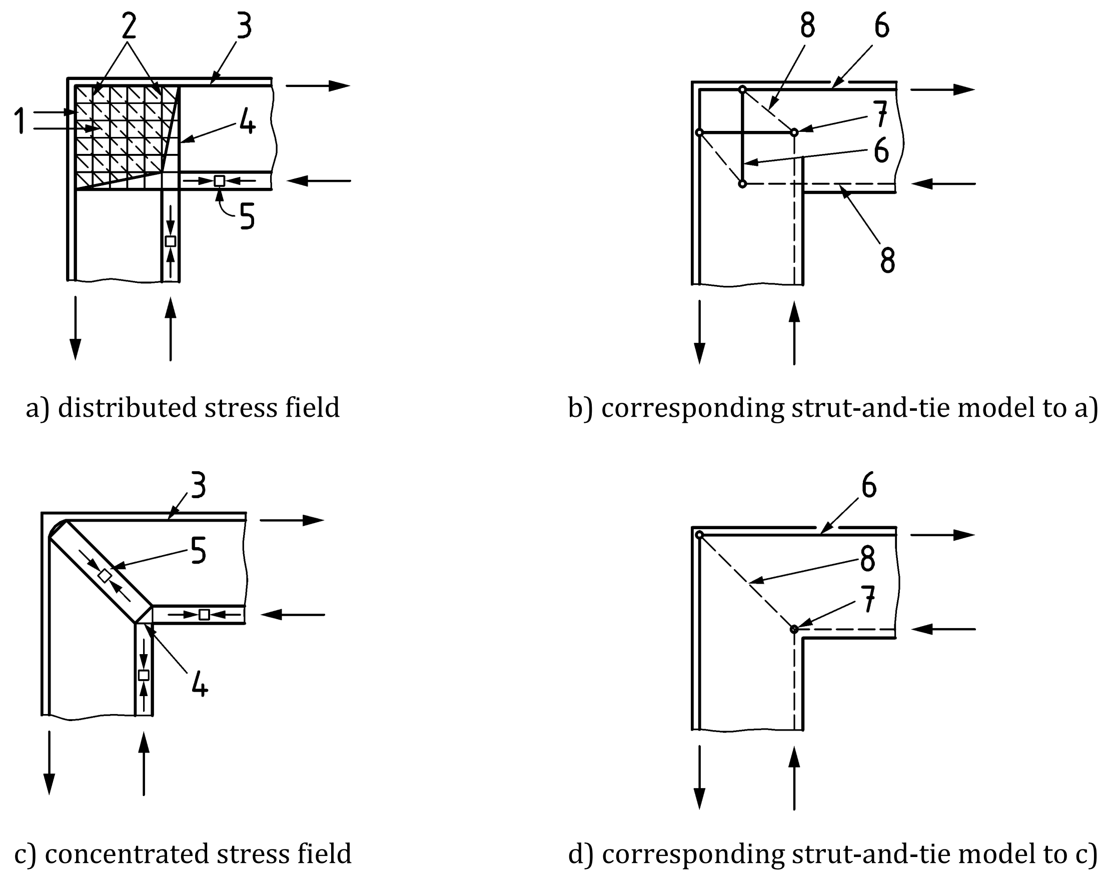
— struts in strut-and-tie models which are idealisations of the compression fields in the stress fields (refer to 8.5.2);

— ties representing the reinforcement (refer to 8.5.3);

— nodal regions where the forces are transferred between the struts or between compression fields and/or the ties (refer to 8.5.4).

NOTE 2 Strut-and-tie models and stress fields are consistent in terms of hypotheses and complementary in terms of use. Strut-and-tie models are idealizations of stress fields aiming at a representation of the force resultants of the compression fields (struts) and ties in the tension fields (see Figure 8.25 b) and d)).

(4) Compression fields and ties in stress fields can be distributed (Figure 8.25 a)) or concentrated (Figure 8.25 c)). Both cases may be designed with the provisions of 8.5. Alternatively, for an element fully occupied by a distributed and uniform compression field and with distributed reinforcement (refer for instance to Figure 8.25 a), the formulae of Annex G may be used for its design. In this latter case, the forces in any chord at the boundaries of the membrane element shall be designed accordingly.



Key

1	distributed compression field	5	concentrated compression field
2	distributed tie field	6	tie
3	concentrated tie	7	node
4	triangular nodal region	8	strut

Figure 8.25 — Examples of stress fields and strut-and-tie models

8.5.2 Struts and compression fields

(1) The width of the struts and of the compression fields shall be verified to respect the strength condition of (3). For the conditions at the ends of struts, 8.5.4 applies.

(2) The stresses in the concrete struts and in the compression fields may generally be assumed as uniformly distributed over its cross-section, so that its value may be calculated as:

$$\sigma_{\mathrm{cd}} = \frac{|F_{\mathrm{cd}}|}{b_{\mathrm{c}} \cdot t} \tag{8.113}$$

where

F_{cd} is the compressive force of the strut;

t is the thickness of the strut which can be limited by the thickness of the member;

b_{c} is the width of the strut at the considered location.

(3) The compressive stress σ_{cd} in a strut or in a compression field developing within the concrete shall fulfil the following condition:

$$\sigma_{\mathrm{cd}} \leq \nu \cdot f_{\mathrm{cd}} \tag{8.114}$$

where ν is the strength reduction factor defined in (4) and (5).

(4) Unless a more refined approach is used, the value of factor ν may be determined as a function of the smallest angle θ_{cs} between the strut representing the resultant of the compression field and any of the ties that cross with the strut as defined in Figure 8.26.

a) for compression fields and struts crossed or deviated by a tie at an angle:

— $20° \leq \theta_{\mathrm{cs}} < 30°$ $\quad \nu = 0{,}4$ (8.115)

— $30° \leq \theta_{\mathrm{cs}} < 40°$ $\quad \nu = 0{,}55$ (8.116)

— $40° \leq \theta_{\mathrm{cs}} < 60°$ $\quad \nu = 0{,}7$ (8.117)

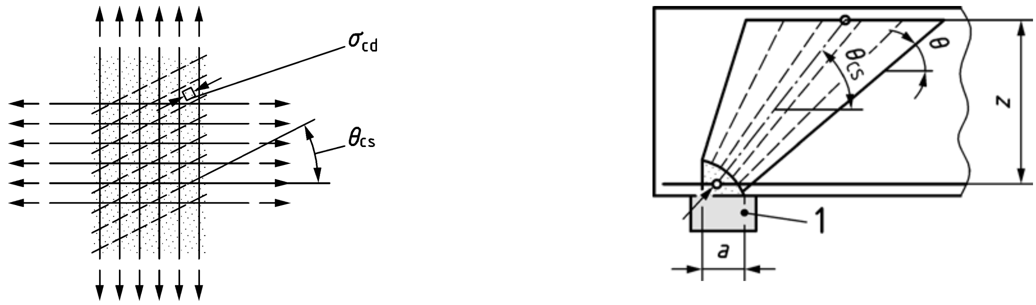
— $60° \leq \theta_{\mathrm{cs}} < 90°$ $\quad \nu = 0{,}85$ (8.118)

Alternatively, the value of factor ν may be determined as:

$$\nu = \frac{1}{1{,}11 + 0{,}22 \cdot \cot^2\theta_{\mathrm{cs}}} \tag{8.119}$$

b) for compression fields and struts in a region without transverse cracking (e.g. when transverse compressive stresses are present)

$$\nu = 1{,}0 \tag{8.120}$$



a) constant width

b) variable width
(example of end support of beams)

Key

1 bearing

Figure 8.26 — Definition of angle θ_{cs} for a compression fields

(5) More refined values for factor ν may also be used for compression fields in cracked zones. They may be determined from the principal tensile strain in the concrete, based on a cracked analysis of the member neglecting the tensile strength of the concrete. In this case, factor ν may be evaluated as:

$$\nu = \frac{1}{1{,}0 + 110\varepsilon_1} \leq 1{,}0 \tag{8.121}$$

where ε_1 is the value of the maximum principal tensile strain.

(6) In zones with confinement reinforcement, the compressive strength may be increased according to 8.1.4. The confinement stress σ_{c2d} to be used in 8.1.4(2) is the minimum of the two principal compressive stresses perpendicular to the strut (in planar members without confinement reinforcement perpendicular to the plane, σ_{c2d} is thus null). The stresses in the confined struts shall be limited to $\sigma_{cd} \leq \nu f_{cd} + \Delta f_{cd}$.

(7) The contribution of compression reinforcement to the strength of a strut may be taken into account up to its yield strength, provided that:

— it is in the same direction as the strut;

— the strength of member according to 8.1 is respected;

— buckling of the reinforcement is prevented according to the rules in 12.6; and

— detailing is performed according to the rules of Clauses 11 and 12.

(8) Where compression fields of width b_c cross ducts of diameters such that $\phi_{duct} > b_c/8$, the resistance should be calculated on the basis of a nominal width based on 8.2.3(10).

8.5.3 Ties

(1) The resistance of a tie F_{Rd} shall fulfil the following condition:

$$F_{td} \leq F_{Rd} = A_s \cdot f_{yd} + A_p \cdot f_{pd} \tag{8.122}$$

where A_s and A_p are the cross-sectional areas of the reinforcements of the tie. If prestressed reinforcement is considered as an external action, then f_{pd} should be replaced by $f_{pd} - \sigma_{pd}$ according to 7.6.5(1).

(2) Formula (8.122) may be used only when ties are suitably anchored at the nodes according to the rules given in 8.5.4. Otherwise, the force in the ties should be limited to the force that can be effectively anchored.

8.5.4 Nodes

8.5.4.1 Definitions

(1) Nodes shall ensure the transfer of forces amongst the different struts and ties (see Figure 8.25).

(2) For nodes with three concurrent struts and ties, four cases can be distinguished:

— CCC nodes, when only struts reach the node (refer to 8.5.4.2);

— CCT nodes, when only one tie is present in the node (refer to 8.5.4.3);

— CTT nodes, when only one strut is equilibrating the reinforcement forces (refer to 8.5.4.4);

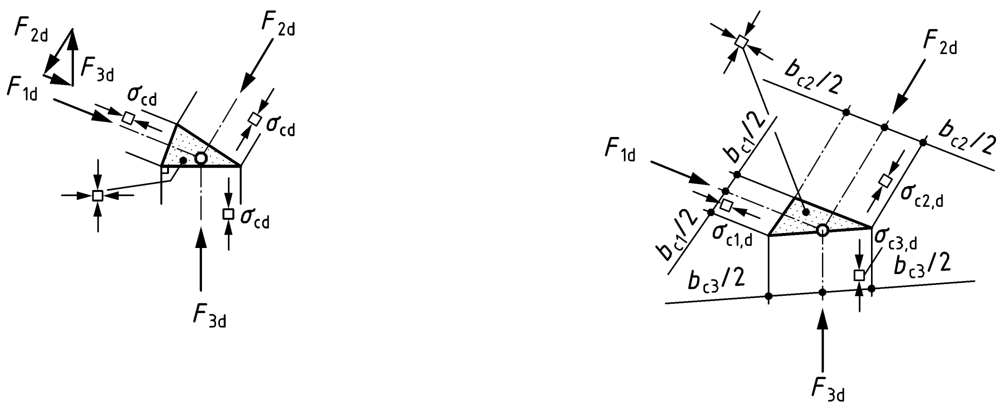
— TTT nodes, where only ties reach the node. TTT nodes may be used only if consistent values of the strength reduction factor ν in the node are adopted accounting for anchorage, detailing and strains according to 8.5.2(5).

(3) Nodes with four or more concurrent struts and ties may be treated as a combination of two or more nodes, each of them with three intersecting strut and ties according to (2).

8.5.4.2 CCC nodes

(1) In a CCC node, verification of the stress state within the nodal region may be omitted as long as the adjoining struts all comply with their stress limits.

(2) The value of coefficient ν at the ends of the converging struts may be taken as 1,0 (see Figure 8.27 a)) or increased according to 8.1.4 or 8.6. The widths of the struts b_c may be adapted accordingly. If the converging struts have different stresses, the geometry of the node may be adapted according to Figure 8.27 b).



a) identical stresses   b) different stresses

Figure 8.27 — CCC node with three struts

8.5.4.3 CCT nodes

(1) The tension force F_{2d} shown in Figure 8.28 a) shall be anchored:

— outside of the nodal region according to Figure 8.28 b); or

— partly or completely inside of the nodal region according to Figures 8.28 c), d) and e).

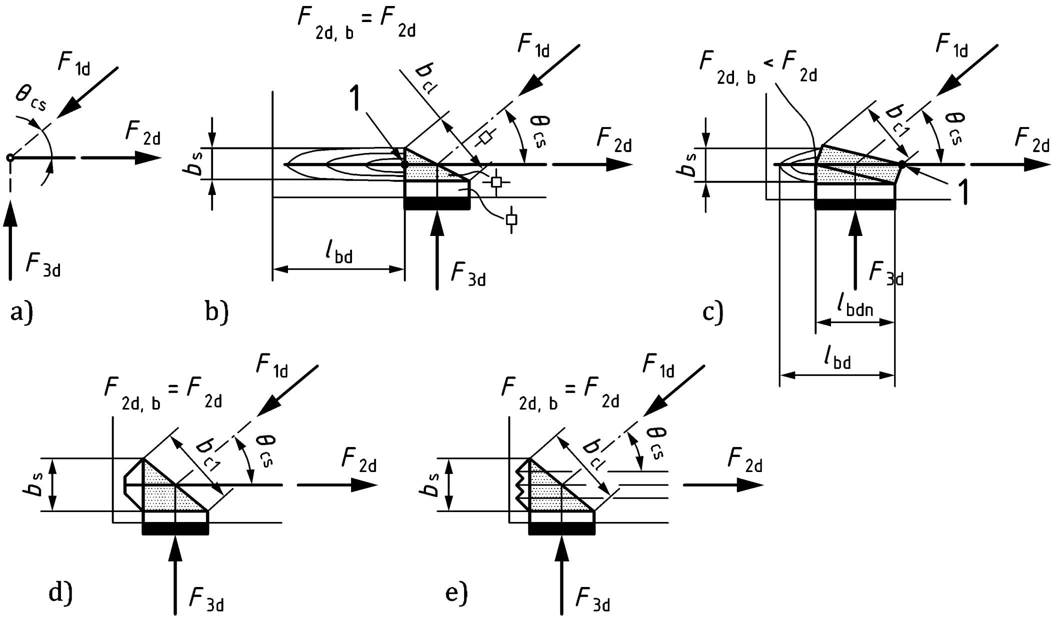
(2) The following methods may be used for anchoring the tie force:

— straight bars according to 11.4.2 where the design anchorage length l_{bd} should not be less than the width of the nodal region (l_{bdn} in Figure 8.28 c);

— bends, hooks (11.4.4) and loops (11.4.6) should not start their bend before the centre of the node nor should the curved segment of bends, hooks and loops be located completely within the node.

— The heads of headed bars (11.4.7) should be placed beyond the nodal region. An additional length according to Figures 8.28 d) to e) should be provided to allow spreading the anchored force over the width b_s and the thickness t (where $b_s \cdot t \geq F_{2d,b}/f_{cd}$, $F_{2d,b}$ being the reinforcement force to be anchored at the backside of the node).

(3) If necessary, the reinforcement carrying the tie force may be distributed or a distributed reinforcement may be added according to Figure 8.28e). In case of distributed stirrups carrying the tie force, 12.3.3 applies.



a) strut-and-tie model

b) with anchorage outside the nodal region

c) anchorage partly inside the nodal region

d) example of anchorage using headed bars outside the nodal region

e) example of anchorage using loops outside the nodal region

Key

1 cross-section where the reinforcement force is fully transferred to concrete in compression

Figure 8.28 — CCT nodes and stress fields

(4) The strength reduction factor ν of concrete in CCT nodes may be assumed as:

— according to 8.5.2(4) in case the tie is fully anchored inside the nodal region as shown in Figure 8.28c) (case with $l_{bdn} = l_{bd}$);

— $\nu = 1{,}0$ in case the tie is fully anchored outside of the nodal region as shown in Figures 8.28b), d) and e);

— a linear interpolation between the previous values depending on the ratio between the required anchorage length outside of the nodal region and the total required anchorage length.

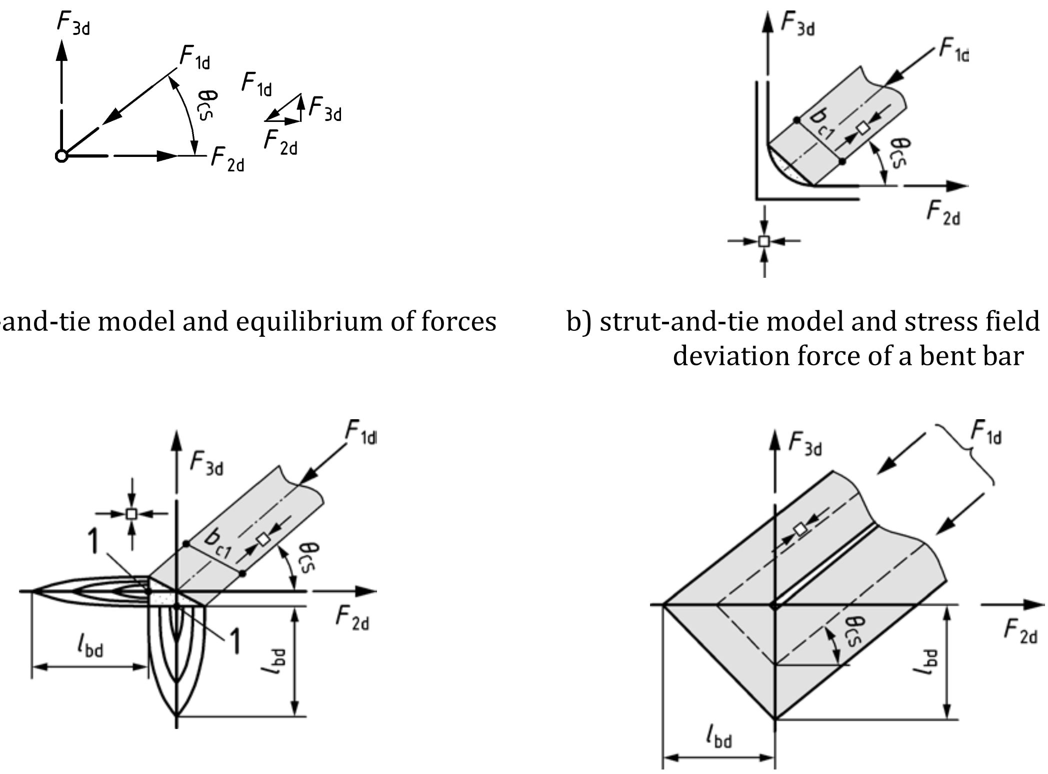
8.5.4.4 CTT nodes

(1) CTT nodes (Figure 8.29) should be designed according to one of following alternatives:

— with bent bars as shown in Figure 8.29b);

— with anchorages outside of the nodal region as shown in Figure 8.29c) or partly inside of the nodal region in a similar manner as in Figure 8.28c). In case the force is anchored with bends, hooks, loops or headed bars, 8.5.4.3(2) applies;

— with straight bars inside the nodal region as shown in Figure 8.29d).



a) strut-and-tie model and equilibrium of forces

b) strut-and-tie model and stress field for the deviation force of a bent bar

c) strut-and-tie model and stress field for anchorages outside the node

d) strut-and-tie model and stress field for anchorages inside the node with deviations occurring due to bond

Key

1 cross-section where the reinforcement force is fully transferred to concrete in compression

Figure 8.29 — CTT node and corresponding stress field

(2) The strength reduction factor ν of concrete at the end of the strut may be assumed as:

— according to 8.5.2(4) in the case of bent bars shown in Figure 8.29b), provided that the requirements of the mandrel diameter of 11.3 are fulfilled, or in the case that the ties are anchored inside of the nodal region as shown in Figure 8.29d);

— $\nu = 1{,}0$ in case the tie is fully anchored outside of the nodal region as shown in Figure 8.29c).

8.5.5 Transfer of concentrated forces into a member

(1) Unless a detailed analysis is performed on the serviceability and ultimate limit state conditions (according for instance to 7.3.4), the transverse reinforcement for concentrated forces applied in a plane stress state and spreading into a member shall satisfy the rules provided in 8.5.5.

(2) The reinforcement for carrying spreading forces may be dimensioned and arranged according to the stress fields or the strut-and-tie model shown in Figure 8.30:

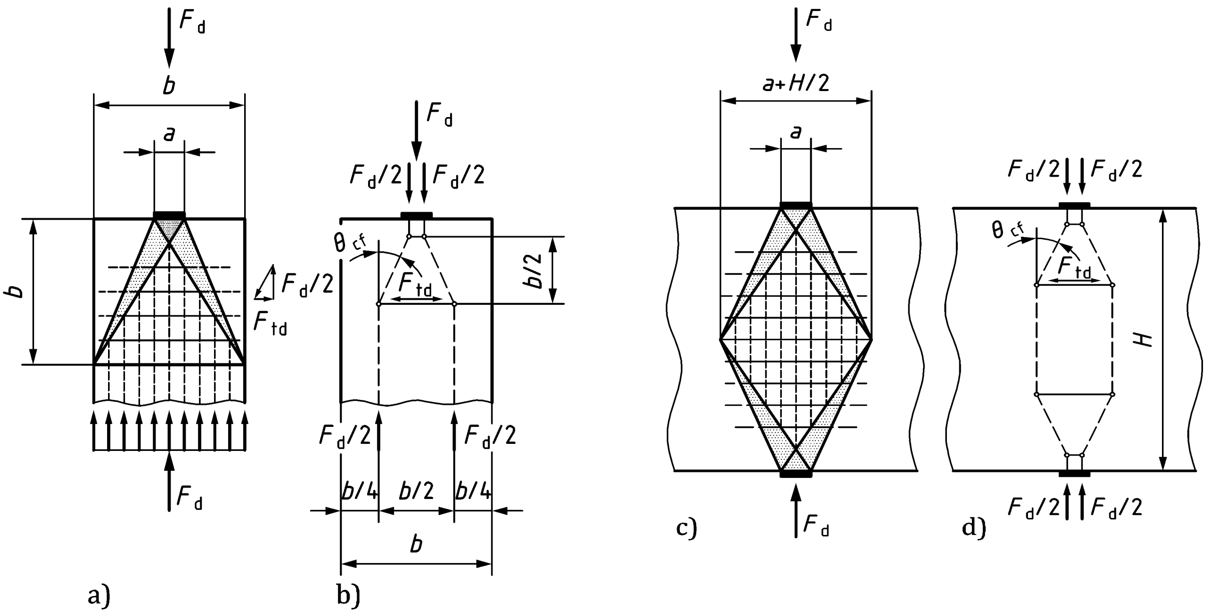
$$F_{\mathrm{td}} = \frac{F_{\mathrm{d}}}{2}\tan\theta_{\mathrm{cf}} \tag{8.123}$$

where the spreading angle θ_{cf} may be assumed as:

$$\tan\theta_{\mathrm{cf}} = \frac{1 - a/b}{2} \tag{8.124}$$

where a and b are defined in Figure 8.30a). For the case shown in Figures 8.30c) and d) ($b > a + H/2$), Formula (8.124) should be replaced by $\tan\theta_{\mathrm{cf}} = 0{,}5$

Alternative approaches for design of the reinforcement may be used, provided that the spreading angles and corresponding reinforcement layout are derived from a stress field fulfilling the requirements of 8.5.2 to 8.5.4.



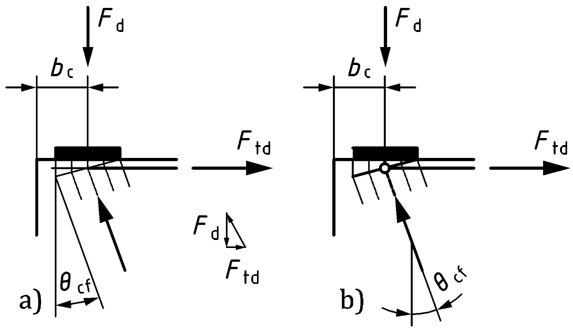
a) narrow element; stress fields
b) strut-and-tie model
c) wide element; stress fields
d) strut-and-tie model

Figure 8.30 — Spreading of concentrated forces, stress fields and strut-and-tie models

(3) In case of concentrated forces acting close to an edge, transverse reinforcement should be placed, to take the force according to the concentrated stress field in Figure 8.31:

$$F_{\mathrm{td}} = F_{\mathrm{d}} \cdot \tan\theta_{\mathrm{cf}} \tag{8.125}$$

where $\tan\theta_{\mathrm{cf}} \geq 1/4$ may be assumed. The nodal region, in this case may be considered as a CCT node.



a) stress field

b) corresponding strut-and-tie model

Figure 8.31 — Concentrated forces near an edge

8.6 Partially loaded areas

(1) For partially loaded areas without horizontal force components, the design resistance according to (2) and transverse tension forces according to (3) and 8.5 shall be considered. The formulae given in (2) below are valid where there is no risk of punching failure. Possible punching failure shall be checked independently according to 8.4.

(2) The design resistance may be verified as follows:

$$\sigma_{R\mathrm{du}} = f_{\mathrm{cd}}\sqrt{\frac{A_{\mathrm{c1}}}{A_{\mathrm{c0}}}} \leq \nu_{\mathrm{part}} f_{\mathrm{cd}} \tag{8.126}$$

where

ν_{part} is a confinement factor:

$\nu_{\mathrm{part}} = 3{,}0$ unless larger resistance can be justified based on refined analysis including tensile stresses due to load or restraint, where relevant;

— For a concentrically loaded area A_{c0} (see Figure 8.32),

$$A_{\mathrm{c0}} = a_0 \cdot b_0 \tag{8.127}$$

is the loaded area with a_0 taken as the length of the loaded area in the direction perpendicular to the closest edge of the load introduction block.

— For eccentric loading A_{c0} should be taken as $A_{\mathrm{c0,red}}$ (see Figure 8.33 a)):

$$A_{\mathrm{c0,red}} = a_{0,\mathrm{red}} \cdot b_{0,\mathrm{red}} \tag{8.128}$$

where

$a_{0,\mathrm{red}} = a_0 - 2e_{\mathrm{a}}$ and $b_{0,\mathrm{red}} = b_0 - 2e_{\mathrm{b}}$,

e_{a} is the eccentricity of the applied load to the center A_{c0} parallel to a_0,

e_{b} is the eccentricity of the applied load to the center A_{c0} parallel to b_0.

$$A_{\mathrm{c1}} = a_1 \cdot b_1 \tag{8.129}$$

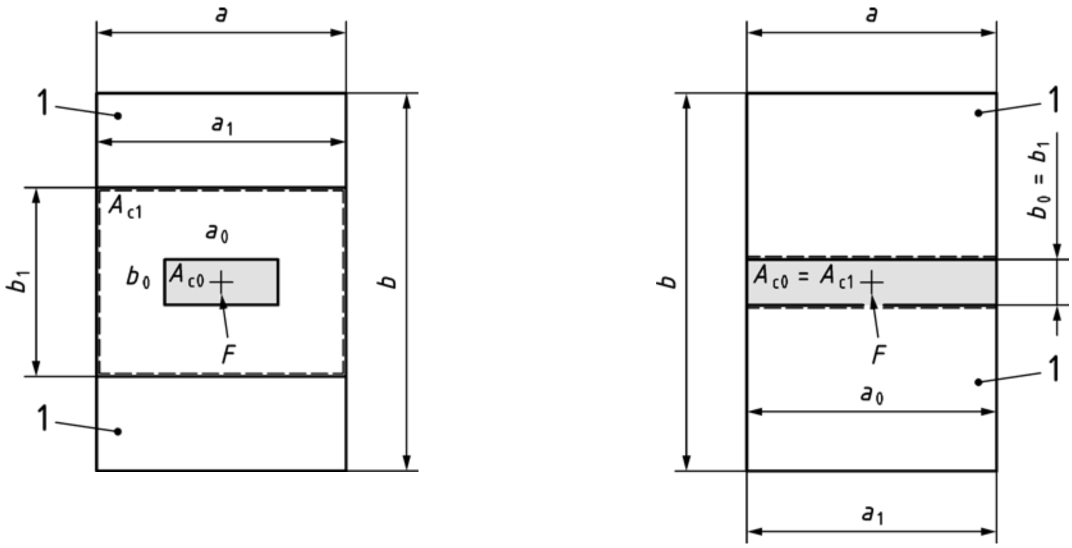
is the contributing concrete area (see Figure 8.32 for a single loaded area and Figure 8.33 b) for two loaded areas of different size),

where

$a_1 = a$ taken as the length of the load introduction block parallel to a_0.

$b_1 = \min\{b_0 + (a_1 - a_0); b\}$

The minimum height of the load introduction block should be $h \geq a_1$.

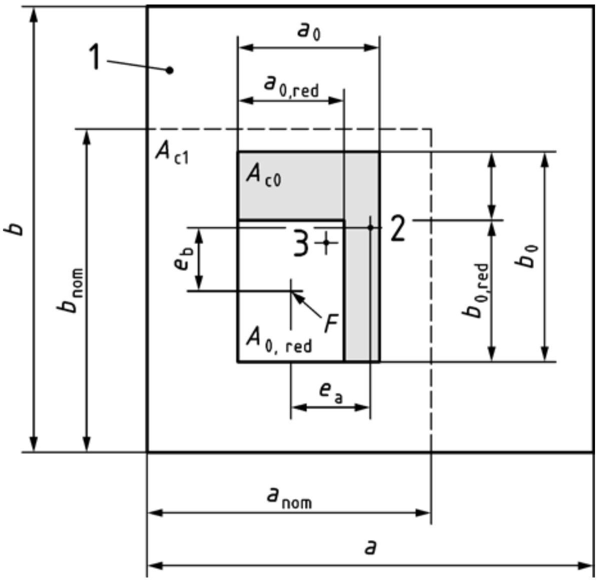


a) 3-dimensional load distribution **b) 2-dimensional load distribution**

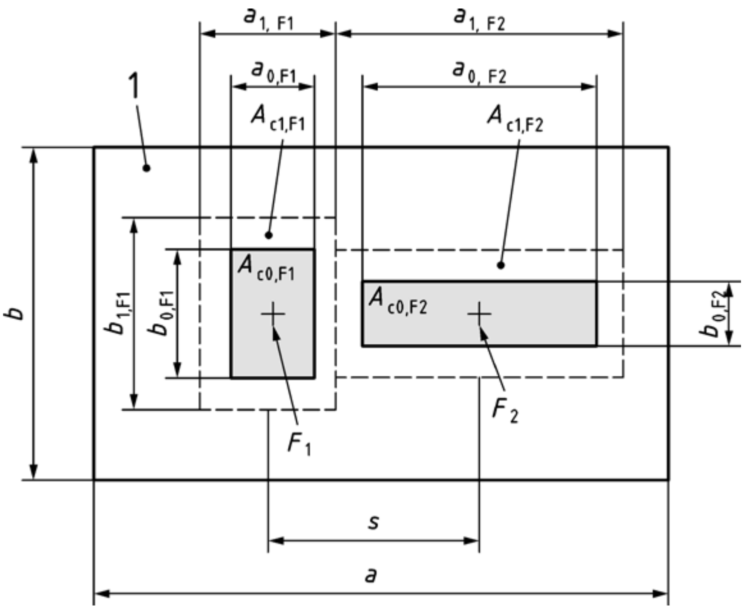
Key

1	load introduction block
a, b	dimensions of load introduction block area
a_0, b_0	dimensions of loaded area A_{c0}
a_1, b_1	dimensions of contributing area A_{c1}

Figure 8.32 — Dimensions of loaded area and contributing concrete area



a)



b)

Key

$$a_{1,F_1} = s + \frac{a_{0,F_1} - a_{0,F_2}}{2}$$

$$a_{1,F_2} = s + \frac{a_{0,F_2} - a_{0,F_1}}{2}$$

1 load introduction block

2 centre of introduction block

3 centre of A_{c0}

Figure 8.33 — Cases of eccentrically applied loads a) **and two loads b)**

(3) Unless justified by more refined analysis, transverse reinforcement should be placed according to 8.5.5 to account for spreading of the applied load.

(4) In lieu of Formula (8.126), the design resistance σ_{Rdu} may be determined by considering the beneficial effect of confinement reinforcement according to 8.1.4. If the area of the confinement core A_{cs}, delimited by the confinement reinforcement, is more than twice as large as the loaded area, the confinement stress obtained from 8.1.4 shall be reduced by the factor $\sqrt{2A_{c0}/A_{cs}} \leq 1$. The confinement reinforcement should be placed in the load introduction block immediately below the loaded area.

(5) When horizontal force components are present, the design resistance σ_{Rdu} may be determined with a reduced contributing area $A_{c1,red}$ located at a depth of $h \geq a_1$ with its centre being on the line of action of the applied load. If the resistance based on the reduced contributing area $A_{c1,red}$ is not sufficient, additional reinforcement should be provided for the entire horizontal force components.

(6) More refined methods, such as three-dimensional stress fields accounting for the beneficial effect of confinement by geometry and by reinforcement may be used for the design of partially loaded areas.

NOTE Additional rules for design in footings and pile caps without shear reinforcement accounting for a concentrated compression zone and the resulting reduced level arm are given in 12.8(3).

9 Serviceability Limit States (SLS)

9.1 General

(1) Clause 9 covers the common serviceability limit states. These are:

stress limitation and crack control (see 9.2);

deflection control (see 9.3);

vibration control (see 9.4).

(2) In the calculation of stresses and deflections, cross sections may be assumed to be uncracked provided that the tensile stress under the characteristic combination of actions does not exceed $f_{ct,eff}$. If the section is assumed to be uncracked a calculation of crack widths is not required.

(3) For the purpose of calculating crack widths and tension stiffening $f_{ct,eff}$ should be taken equal to f_{ctm}. For the calculation of deflections under predominantly flexural stresses, the value of $f_{ct,eff}$ may be taken as f_{ctm} or $f_{ctm,fl}$ (see Formula (9.30)), provided that the calculation of minimum reinforcement areas for crack control is also based on the same value.

NOTE Cracking can be observed for lower values of tensile stresses. This is attributed to non-uniform self-equilibrating stresses due to shrinkage strains, early thermal stresses, sustained loading and to the effective tension area (see Figure 9.3). Further information is given in Annex D.

(4) For the calculation of stresses and deflections, an effective modulus of elasticity, $E_{c,eff}$, may be used for concrete to estimate long term effects due to quasi-permanent actions, according to Formula (9.1):

$$E_{c,eff} = \frac{1{,}05 \cdot E_{cm}}{1 + \varphi(t, t_0)} \tag{9.1}$$

If compressive stresses due to quasi-permanent combination of actions exceed $0{,}40f_{cm}$, the value of $\varphi(t,t_0)$should account for nonlinear creep according to B.5(6).

(5) As a simplified approach for the calculation of stresses, valid for crack width calculations only, a modular ratio $E_s/E_{c,eff}$ equal to 15 may be used for both permanent and variable loads.

9.2 Stress limitations and crack control

9.2.1 General considerations

NOTE 1 Cracking is normal in reinforced concrete structures resulting from either direct loading or restrained imposed deformations.

NOTE 2 Cracks can also arise from other causes such as plastic shrinkage or expansive chemical reactions within the hardened concrete. Such cracks can be unacceptably large but their avoidance and control lie outside the scope of 9.2.

(1) Cracking shall be limited to an extent that will not impair the proper functioning or durability of the structure or cause its appearance to be unacceptable.

(2) Cracks may be permitted to form without any attempt to control their width, provided they do not impair the functioning of the structure. In cases where crack width is not critical and crack width limits for durability are not relevant, verification according to 9.2.2 and 9.2.3 may be omitted and crack control may be assumed to be covered by the provision of minimum reinforcement according to 12.2.

NOTE 3 There are particular risks of large cracks ($w_k > 0{,}4$ mm) occurring in member locations where there are sudden changes of stress, e.g.

at changes of section;

near concentrated loads;

positions where bars are curtailed;

areas of high bond stress, particularly at the end of laps.

(3) If crack width control is required, minimum reinforcement according to 9.2.2 should be provided and the section should be checked for cracking according to 9.2.3.

NOTE 4 Simplified provisions for reinforcement stress limitation and crack control are given in Informative Annex S.

NOTE 5 To control a crack to a specified width a greater area of reinforcement than that required by 9.2.2 may be required.

(4) When lapping large diameter bars ($\phi > 32$ mm), large surface cracks can occur. Surface crack width at the most tensioned fibre may be verified according to 9.2.3.

NOTE 6 Simplified provisions for large bars are given in Informative Annex S.

(5) The measures for limiting the crack widths should be adapted to the causes of cracking, for instance by:

choosing an appropriate structural concept;

prestressing;

limiting the reinforcement stresses and the concrete stresses;

appropriate detailing;

appropriate selection of concrete properties;

curing of concrete.

(6) A limiting value $w_{lim,cal}$ for the calculated crack width $w_{k,cal}$ should be established taking into account the proposed function, the nature of the structure, the costs of limiting cracking and the location where the crack width is considered.

NOTE 7 Unless the National Annex gives different values or a different location, the limits on stresses, crack widths $w_{lim,cal}$ are given in:

Table 9.1 (NDP) for requirements related to appearance;

Table 9.2 (NDP) for requirements related to durability.

NOTE 8 See Annex H for additional conditions related to water-tightness.

Table 9.1 (NDP) — Verifications, stress and crack width limits for appearance

Verification	**Calculation of minimum reinforcement according to 9.2.2**	**Verification of crack width according to 9.2.3**	**Verification of reinforcement stresses to avoid yielding at SLS**
Combination of actions for calculating σ_s	Cracking forces according to 9.2.2	Quasi-permanent combination of actions	Characteristic combination of actions
Limiting value of crack width $w_{lim,cal}$ or stress σ_s	$\sigma_s \leq f_{yk}$	$w_{lim,cal} = 0{,}4$ mm $\sigma_s \leq f_{yk}$	$\sigma_s \leq 0{,}8 f_{yk}$ $\sigma_p \leq 0{,}8 f_{pk}$
NOTE Crack widths are verified at the member surface unless the National Annex gives a different location.			

Table 9.2 (NDP) — Verifications, stress and crack width limits for durability

<table>
<tr><th rowspan="3">Exposure Class</th><th colspan="2">Reinforced members and prestressed members without bonded tendons and with bonded tendons with Protection Levels 2 or 3 according to 5.4.1(4)</th><th colspan="3">Prestressed members with bonded tendons with Protection Level 1 according to 5.4.1(4) and pretensioned members.</th></tr>
<tr><th colspan="2">combination of actions</th><th colspan="3">combination of actions</th></tr>
<tr><th>quasi-permanent</th><th>characteristic</th><th>quasi-permanent</th><th>frequent</th><th>characteristic</th></tr>
<tr><td>X0, XC1</td><td>–</td><td rowspan="2">–</td><td>–</td><td>$w_{\mathrm{lim,cal}} = 0{,}2\ \mathrm{mm} \cdot k_{\mathrm{surf}}$</td><td rowspan="2">–</td></tr>
<tr><td>XC2, XC3, XC4</td><td rowspan="2">$w_{\mathrm{lim,cal}} = 0{,}3\ \mathrm{mm} \cdot k_{\mathrm{surf}}$</td><td>Decom-pression [b]</td><td>$w_{\mathrm{lim,cal}} = 0{,}2\ \mathrm{mm} \cdot k_{\mathrm{surf}}$</td></tr>
<tr><td>XD1, XD2, XD3
XS1, XS2, XS3</td><td rowspan="2">$\sigma_c \leq 0{,}6 f_{ck}$ [a,c]</td><td rowspan="2">–</td><td rowspan="2">Decompression[b]</td><td rowspan="2">$\sigma_c \leq 0{,}6 f_{ck}$ [a,c]</td></tr>
<tr><td>XF1, XF3
XF2, XF4</td><td>–</td></tr>
</table>

NOTE 1 Crack widths are verified at the member surface unless the National Annex gives a different location..

NOTE 2 The factor k_{surf} considers the difference between an increased crack width at the member surface and the required mean crack width according to durability performance of the minimum cover: $1{,}0 \leq k_{\mathrm{surf}} = c_{\mathrm{act}}/(10\ \mathrm{mm} + c_{\mathrm{min,dur}}) \leq 1{,}5$.

c_{act} is a specified actual cover $\geq c_{\mathrm{nom}}$ due to detailing or execution reasons.

[a] This limitation in serviceability conditions is not necessary for stresses under bearings, partially loaded areas and plates of headed bars.

[b] The decompression limit requires that all parts of the bonded tendons or duct lie at least 25 mm within concrete in compression. The decompression check is only relevant in the direction of the prestressed reinforcement.

[c] The compressive stress σ_c may be increased to $0{,}66 f_{ck}$ if the cover is increased by 10 mm or confinement by transverse reinforcement is provided.

(7) Reinforcement should be considered as effective in controlling crack width only within a given area of concrete around the reinforcement bars. This area, $A_{\mathrm{c,eff}}$, is referred to as the effective tension area.

9.2.2 Minimum reinforcement areas to avoid yielding

(1) The required minimum area of reinforcement to avoid yielding shall be calculated by applying the principle that the reinforcement, working at the characteristic yield stress, should balance the moment that cracks the section acting together with the relevant axial force N_{Ed}.

(2) Unless a more refined calculation shows lesser areas to be adequate, the required minimum areas of reinforcement may be calculated as follows. In profiled cross sections like T-beams and box girders, minimum reinforcement should be determined for the individual parts of the section (webs, flanges). The reinforcement necessary in the part of the cross section under consideration may be determined by Formulae (9.2), (9.3) or (9.4), as appropriate.

(i) For pure bending:

$$A_{s,min,w1} \geq \frac{0{,}2k_h f_{ct,eff} A_c}{f_{yk}} \tag{9.2}$$

(ii) For pure tension:

$$A_{s,min,w1} = A_{s,min,w2} \geq \frac{0{,}5k_h f_{ct,eff} A_c}{f_{yk}} \tag{9.3}$$

(iii) For a combination of bending and axial force:

$$A_{s,min,w1} \begin{cases} \geq \dfrac{0{,}3N_{Ed} + 0{,}2k_h f_{ct,eff} A_c}{f_{yk}} \\ \geq 0 \\ \leq \dfrac{0{,}5k_h f_{ct,eff} A_c}{f_{yk}} \end{cases} \tag{9.4}$$

$$A_{s,min,w2} = \frac{N_{Ed}}{f_{yk}} - A_{s,min,w1} \begin{cases} \leq A_{s,min,w1} \\ \geq 0 \end{cases}$$

where

A_c is the full area of the part of the section (web or flange) under consideration;

$A_{s,min,w1}$ is the area of minimum reinforcing steel to be placed at the most tensioned face of the part of the section under consideration to control cracking;

$A_{s,min,w2}$ is the area of minimum reinforcing steel to be placed at the least tensioned face of the part of the section under consideration to control cracking;

N_{Ed} is the design axial force at the serviceability limit state acting on the part of the section under consideration (tensile force positive). N_{Ed} should be determined considering the characteristic values of prestress and axial forces under the relevant combination of actions;

k_h is a coefficient which allows for the effect of non-uniform self -equilibrating stresses, which lead to a reduction of the apparent tensile strength which may be taken as:

$$k_h = 0{,}8 - 0{,}6(\min\{b;h\} - 0{,}3) \begin{cases} \leq 0{,}8 \\ \geq 0{,}5 \end{cases} \tag{9.5}$$

where h and b [m] are the dimensions of the part of the section under consideration.

$f_{ct,eff}$ is the mean value of the tensile strength of the concrete effective at the time when the cracks may first be expected to occur: $f_{ct,eff} = f_{ctm}$ for $t \geq t_{ref}$ or $f_{ct,eff} = f_{ctm}(t)$, if cracking is expected at $t < t_{ref}$

NOTE For more information on $f_{ct,eff}$, see Annex D.

(3) Bonded tendons in the tension zone may be assumed to contribute to crack control within a distance ≤ 150 mm from the centre of the tendon. This may be taken into account by deducting the term $\xi_1 A_p \Delta\sigma_p / f_{yk}$ from the area of minimum reinforcement corresponding to the tensioned face where the prestressing is placed.

where

A_p is the area of pre-tensioning or bonded post-tensioning tendons within the tensile zone;

ξ_1 is the adjusted ratio of bond strength taking into account the different diameters of prestressing and reinforcing steel, according to Formula (9.6):

$$\xi_1 = \sqrt{\xi \frac{\phi}{\phi_p}} \quad (9.6)$$

ξ ratio of bond strength of prestressing and reinforcing steel, according to Table 10.1 in 10.3(2);

ϕ is the diameter of the reinforcing steel bar;

ϕ_p diameter of tendon according to 10.3(2). If only prestressing steel is used to control cracking, $\xi_1 = \xi$;

$\Delta\sigma_p$ stress variation in prestressing tendons from the state of zero strain of the concrete at the same level.

(4) Beams with a downstand depth of more than 600 mm, where the main tension reinforcement is concentrated at the most tensioned fibre, should be provided with additional longitudinal reinforcement given by Formula (9.7), at a spacing not exceeding 300 mm, to control cracking on the side faces of the beam. This reinforcement should be evenly distributed between the level of the reinforcement layer closest to the most tensioned fibre and the neutral axis.

$$A_{s,web} = 0{,}2 \frac{f_{ct,eff}}{f_{yk}} b_w (d - x - a_1) \quad (9.7)$$

where

$A_{s,web}$ is the longitudinal reinforcement to be provided distributed on the two surfaces of the web in a height limited by the neutral axis and (d – a_1) at a spacing not exceeding 300 mm to control cracking;

a_1 equal to 150 mm.

(5) In prestressed members, no minimum reinforcement for crack control is required in sections where, under the characteristic combination of actions and the characteristic value of prestress, the tensile stress in the concrete is below $f_{ct,eff}$.

9.2.3 Refined control of cracking

(1) The calculated crack width $w_{k,cal}$ given in 9.2.3 should only be considered as a nominal value for the crack width at the member surface, to be compared with $w_{lim,cal}$, and not as values actually measured on site.

NOTE 1 Formulae (9.8) to (9.20) provide an estimate of the crack width at the member surface at the most extreme fibre. For other locations “c” can be replaced by the distance from the bar considered and $k_{1/r}$ can be adjusted accordingly.

(2) The calculated surface crack width $w_{k,cal}$ may be determined from Formula (9.8):

$$w_{k,cal} = k_w \cdot k_{1/r} \cdot s_{r,m,cal} (\varepsilon_{sm} - \varepsilon_{cm}) \quad (9.8)$$

where

k_w is a factor converting the mean crack width into a calculated crack width;

NOTE 2 The value of k_w is 1,7 unless the National Annex gives a different value.

$k_{1/r}$ is a coefficient to account for the increase of crack width due to curvature. For the most tensioned fibre this may be defined in accordance with Figure 9.1 as:

$$k_{1/r} = \frac{h - x}{h - a_{y,i} - x} \quad (9.9)$$

When considering the least tensioned fibre of a member with both faces in tension, the effect of the curvature is favourable and therefore may be defined as:

$$k_{1/r} = \frac{|x|}{a_{y,s} + |x|} \quad (9.10)$$

$s_{r,m,cal}$ is the calculated mean crack spacing when all cracks have formed or where not all cracks have formed, the length along which there is slip between the concrete and the reinforcement adjacent to a crack;

ε_{sm} is the mean strain in the reinforcement closest to the most tensioned concrete surface under the relevant combination of actions, including the effect of imposed deformations and taking into account the effects of tension stiffening. Only the additional tensile strain beyond the state of zero strain of the concrete at the same level is considered;

ε_{cm} is the mean strain in the concrete at the same section and level of ε_{sm}.

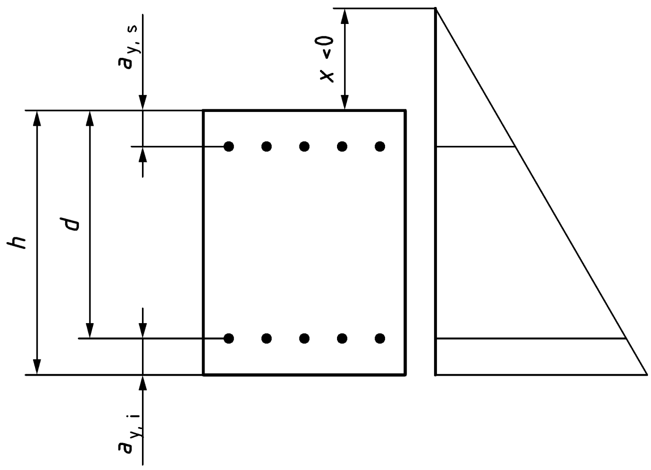
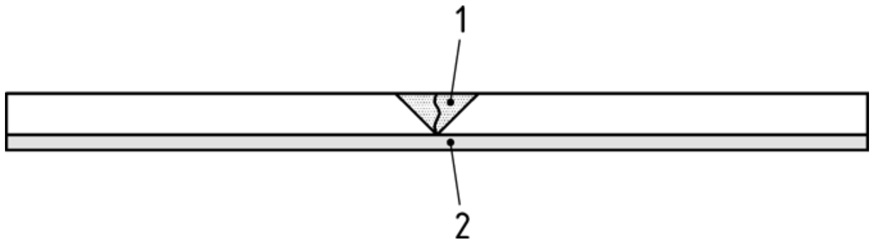
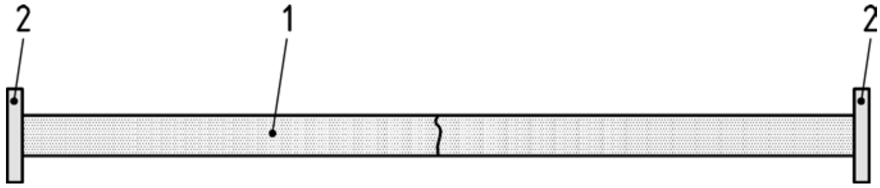


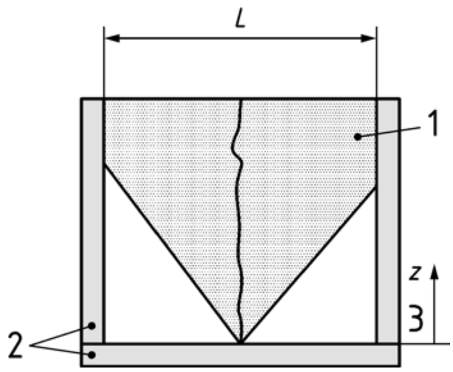
Figure 9.1 — Notation for definition of $k_{1/r}$ in Formulae (9.9) and (9.10): section and strain plane



a) Element restrained at an edge (e.g. jointless wall) — only first crack is represented



b) Element restrained at the ends (typical tie) — only first crack is represented



c) Element restrained at edge and ends (e.g. wall grid) — only first crack is represented

Key

1 area where forces are affected by crack development

2 restraint (at edges or ends)

3 if $z > L/2$, Formula (9.11) for end restraint applies and if $z \leq L/2$, Formula (9.13) for edge restraint applies

Figure 9.2— Types of restraint

NOTE 3 Members subjected to imposed strains can be restrained along the edges (see Figure 9.2 a)) or at the ends (see Figure 9.2 b)). In a member which is restrained along the edges cracking only changes forces locally and crack widths depend on the applied strain. In a member which is restrained at the ends, cracking changes forces globally and crack widths depend on the tensile strength of concrete, but not on the applied strain. This affects the way $\varepsilon_{sm} - \varepsilon_{cm}$ is determined (see (3) and (4)).

(3) For members subjected to direct loads (stabilized cracking) or subjected to imposed strains (crack formation phase) where end restraint dominates (see NOTE in (2) and Figure 9.2 b)), $\varepsilon_{sm} - \varepsilon_{cm}$ may be calculated from Formula (9.11):

$$\varepsilon_{\mathrm{sm}} - \varepsilon_{\mathrm{cm}} = \frac{\sigma_{\mathrm{s}} - k_{\mathrm{t}} \dfrac{f_{\mathrm{ct,eff}}}{\rho_{\mathrm{p,eff}}}\left(1 + \alpha_{\mathrm{e}}\rho_{\mathrm{p,eff}}\right)}{E_{\mathrm{s}}} \geq (1 - k_t)\frac{\sigma_{\mathrm{s}}}{E_{\mathrm{s}}} \tag{9.11}$$

where

σ_s is the stress in the tension reinforcement closest to the tensioned concrete surface assuming a cracked section. For a member subjected to imposed strains, restrained at the ends, the tension in the reinforcement may be derived from the sectional cracking forces. For members with bonded tendons, σ_s may be replaced by $\Delta\sigma_p$, i.e., the stress variation in prestressing tendons from the state of zero strain of the concrete at the same level;

α_e is the ratio E_s/E_{cm};

$$\rho_{p,eff} = \frac{A_s + \xi_1 A_p}{A_{c,eff}} \tag{9.12}$$

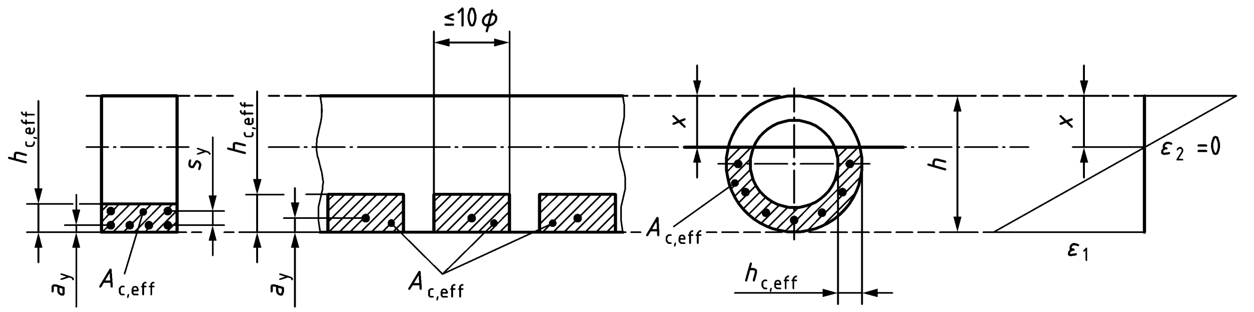
A_s *and* A_p are the non-prestressed and prestressed reinforcement area, respectively, located within the effective area

$A_{c,eff}$ are as defined in Figure 9.3;

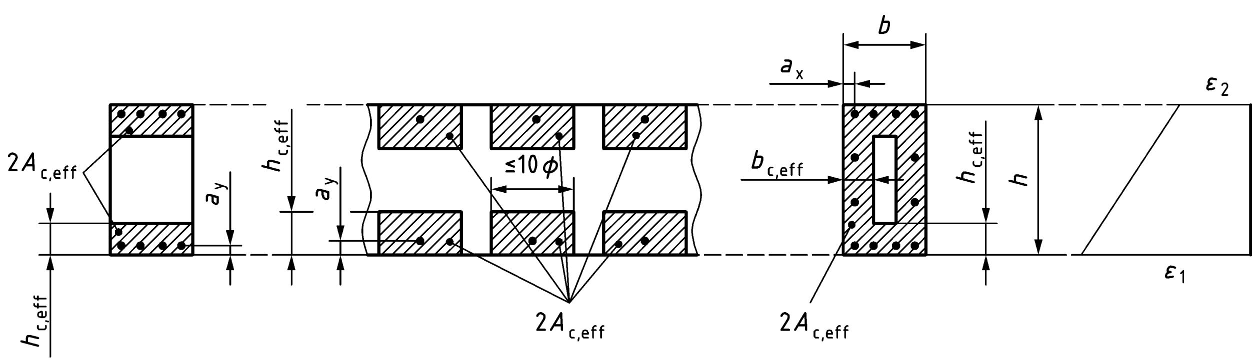
k_t is a coefficient dependent on the duration and nature of the load:

$k_t = 0,6$:
a) for short term loading or;
b) instantaneous loading or;

$k_t = 0,4$:
c) long term and crack formation stage or;
d) repeated loading and crack formation stage;
e) long term and stabilised cracking stage or;
f) repeated loading and stabilised cracking stage.



a) Group of bars (bending) b) Isolated bars (bending) c) Circular cross-section (bending)



d) Group of bars, distributed along perimeter (both faces in tension) e) Group of bars (both faces in tension) f) Isolated bars (both faces in tension)

Key

Fig. a) to b): bending (with or without normal force)

For a single layer of bars: $h_{\mathrm{c,eff}} = \min\left\{a_{\mathrm{y}}+5\phi;\ 10\phi;\ 3{,}5a_{\mathrm{y}};\ h-x;\ \frac{h}{2}\right\}$

For n layers of bars spaced s_{y}: $h_{\mathrm{c,eff}} = \min\{\min\{a_{\mathrm{y}}+5\phi;\ 10\phi;\ 3{,}5a_{\mathrm{y}}\} + (n-1)s_{\mathrm{y}};\ h-x;\ \frac{h}{2}\}$

Fig. c): circular cross section: $h_{\mathrm{c,eff}} = \min\{a + 5\phi;\ 10\phi;\ 3{,}5a\}$

Fig. d) to f): tension (with or without bending)

$h_{\mathrm{c,eff}} = \min\{a_{\mathrm{y}} + 5\phi;\ 10\phi;\ 3{,}5a_{\mathrm{y}};\ 0{,}5h\}$

$b_{\mathrm{c,eff}} = \min\{a_{\mathrm{x}} + 5\phi;\ 10\phi;\ 3{,}5a_{\mathrm{x}};\ 0{,}5b\}$

Figure 9.3 — Effective tension area

(4) For members subjected to restrained imposed strains and restrained at the edges (see NOTE in (2) and Figure 9.2 a)), $(\varepsilon_{\mathrm{sm}} - \varepsilon_{\mathrm{cm}})$ may be calculated from Formula (9.13):

$$\varepsilon_{\mathrm{sm}} - \varepsilon_{\mathrm{cm}} = R_{\mathrm{ax}}\varepsilon_{\mathrm{free}} - k_{\mathrm{t}}\frac{f_{\mathrm{ct,eff}}}{E_{\mathrm{cm}}} \geq 0 \tag{9.13}$$

where

R_{ax} is the restraint factor:

$$R_{\mathrm{ax}} = 1 - \frac{\varepsilon_{\mathrm{restr}}}{\varepsilon_{\mathrm{imp}}} \tag{9.14}$$

For the common case of the base of a wall a simplified value of $R_{\mathrm{ax}} = 0{,}5$ may be assumed;

$\varepsilon_{\mathrm{restr}}$ the strain which develops in the restrained element;

$\varepsilon_{\mathrm{imp}}$ the value of the imposed strain (i.e. free shrinkage, free temperature strain). The ratio $\varepsilon_{\mathrm{restr}}/\varepsilon_{\mathrm{imp}}$ may be estimated according to linear elastic analysis, and may account for staged construction, if relevant;

$\varepsilon_{\mathrm{free}}$ is the imposed strain which develops after the construction stage when restraint is applied (e.g. precast elements).

(5) If a member is restrained both at the ends and at the edges the criterion defined in Figure 9.2c) may be applied.

(6) The mean final crack spacing $s_{\mathrm{r,m,cal}}$ may be calculated as:

$$s_{\mathrm{r,m,cal}} = 1{,}5 \cdot c + \frac{k_{\mathrm{fl}} \cdot k_{\mathrm{b}}}{7{,}2} \cdot \frac{\phi}{\rho_{\mathrm{\rho,eff}}} \leq \frac{1{,}3}{k_w}(h - x) \tag{9.15}$$

where

— For a rectangular cross section subjected to pure bending:

$$k_{\mathrm{fl}} = \frac{h - h_{\mathrm{c,eff}}}{h} \tag{9.16}$$

— In general:

$$k_{\mathrm{fl}} = \frac{1}{2}\left(1 + \frac{\left(h - x_{\mathrm{g}} - h_{\mathrm{c,eff}}\right)}{h - x_{\mathrm{g}}}\right) \qquad (9.17)$$

x_{g} is depth of the neutral axis of the uncracked section;

$$k_{\mathrm{b}} = \begin{cases} 1{,}2 \text{ for poor bond conditions} \\ 0{,}9 \text{ for good bond conditions} \end{cases} \qquad (9.18)$$

ϕ is the bar diameter. For bundled bars, an equivalent diameter according to Formula (11.6) may be used. For a section with n_1 bars of diameter ϕ_1 and n_2 bars of diameter ϕ_2, an equivalent bar diameter ϕ_{eq} according to Formula (9.19) should be used instead of ϕ:

$$\phi_{\mathrm{eq}} = \frac{n_1\phi_1^2 + n_2\phi_2^2}{n_1\phi_1 + n_2\phi_2} \qquad (9.19)$$

c is the clear cover of the bar. For corner bars, the maximum value of the cover applies.

— For pure tension:

$$k_{\mathrm{fl}} = 1{,}00 \qquad (9.20)$$

(7) If, in members reinforced in two orthogonal directions, the angle θ between the axes of principal compressive strain and the direction of the reinforcement in the x-direction is larger than 15° the crack width should be calculated according to G.5.

(8) When using strut-and-tie models or stress field models with the struts or compression fields oriented according to the compressive stress trajectories in the elastic state (cracked or uncracked), Formula (9.8) may be used to evaluate the crack widths by considering the tension reinforcement as a tie with the corresponding effective area around it. The cover to be considered in Formula (9.15) should be taken as the distance from the concrete surface to the edge of the nearest bar, even if the bar is not part of the strut-and-tie or stress field model (e.g. skin reinforcement).

9.3 Deflection control

9.3.1 General consideration

(1) The deformation of a member or structure should be such that it does not adversely affect its proper functioning or appearance. Appropriate limiting values of deflection taking into account the nature of the structure, of the finishes, partitions and fixings and the function of the structure should be established.

(2) For structures within the scope of EN 1990, Annex A1, limits for vertical deflections according to Annex A1.7, Figure A1.1 should be specified for each project and agreed with the client. Suggested maximum values are given in EN 1990, Annex A. For bridges, the provisions of EN 1990, Annex A2 apply.

(3) Deflections may be controlled:

— in buildings by indirectly limiting the span-to-depth ratio, see 9.3.2;

— in buildings by simplified explicit verification of deflections, see 9.3.3;

— in any type of structures by explicit verification, see 9.3.4.

NOTE The actual deformations can differ from the estimated values (either according to 9.3.3 or 9.3.4), particularly if the values of applied moments are close to the cracking moment. The differences will depend on the scatter of the material properties, on the environmental conditions, on the load history, on the restraints at the supports, ground conditions, etc.

9.3.2 Simplified deflection control by span/depth-ratio for buildings

(1) Provided that reinforced concrete beams or slabs in buildings, subjected to predominantly uniformly distributed loads, are dimensioned in compliance with the limits of span to effective depth ratio given in Table 9.3, their deflections may be considered as not exceeding a total deflection of l/250. In such cases explicit verification of the deflections may be omitted.

Table 9.3 — Limiting span/effective depth ratios l/d for buildings

	Structural system	Required mechanical reinforcement ratio[a]								
		$\omega_r = 0,3$			$\omega_r = 0,2$			$\omega_r = 0,1$		
		LL/TL[b]			LL/TL[b]			LL/TL[b]		
		60 %	45 %	30 %	60 %	45 %	30 %	60 %	45 %	30 %
1	Simply supported beam, one-way spanning simply supported slab	15	14	12	17	15	13	22	19	17
2	End span of continuous beam or one-way spanning slab	20	18	16	22	20	17	29	25	22
3	Interior span of continuous beam or one-way spanning slab	23	21	18	26	23	20	33	29	26
4	Cantilever	7	7	6	8	7	6	10	9	8

NOTE This table assumes the quasi-permanent value of the live load with $\psi_2 = 0,3$ and that the deflection limit for long-term deflection is $l/250$, where l is the span of the beam or slab.

[a] $\omega_r = A_{s,req}/(b_w \cdot d) \cdot f_{yd}/f_{cd}$ is the required mechanical tension reinforcement ratio to resist the moment due to the design loads, at mid span for continuous or simply supported elements and at the support for cantilevers. Intermediate values may be interpolated. The limits are conservative for flanged sections.

[b] Characteristic values of: LL = characteristic value of live load (imposed or variable); TL = characteristic value of total load. Intermediate values may be interpolated.

(2) For a different maximum total deflection of l/a, the l/d-values of Table 9.3 should be multiplied by $(250/a)$, where a is the deflection ratio factor different from 250.

(3) The limiting span/effective depth ratios of Table 9.3 may be extrapolated to other support conditions by multiplying them by the cubic root of the ratio of the maximum value of the linear elastic deflection of a simply supported beam of the same span and the maximum value of the linear elastic deflection of the actual structure.

(4) For rectangular 2-way flat slabs supported on columns, with slenderness ratio $l_{\max}/d$, the limiting span/effective depth ratios of Table 9.3 may be multiplied by the following factor:

$$\left(\frac{1}{1+\left(\frac{l_{\min}}{l_{\max}}\right)^4}\right)^{1/4} \tag{9.21}$$

where

$l_{\min}$ is the minimum span of the slab;

$l_{\max}$ is the maximum span of the slab.

(5) For rectangular 2-way flat slabs, simply supported on walls or supported on stiff beams on all four sides, with slenderness ratio $l_{\min}/d$, the limiting span/effective depth ratios of Table 9.3 may be multiplied by the following factor:

$$\left(\frac{1}{1-0{,}65\frac{l_{\min}}{l_{\max}}}\right)^{1/4} \tag{9.22}$$

9.3.3 Simplified calculation of deflections for reinforced concrete building structures

(1) For rectangular sections, long-term deflections may be determined from linear elastic analysis using gross concrete sections and assuming long-term properties (i.e. $E_{\mathrm{c,eff}}$) according to Formula (9.23).

$$\delta = k_{\mathrm{I}}[\delta_{loads} + k_s\delta_{\varepsilon cs}] \tag{9.23}$$

where

δ_{loads} is the linear elastic deflection, which should be calculated using $E_{\mathrm{c,eff}}$, determined for uncracked conditions, due to the quasi-permanent combination of actions;

$\delta_{\varepsilon\mathrm{cs}}$ is the linear elastic deflection, determined for uncracked conditions, due to differential shrinkage determined by applying, on the linear elastic model, the curvature given by Formula (9.24)

$$\left(\frac{1}{r}\right)_{\varepsilon\mathrm{cs}} = \frac{E_{\mathrm{s}}}{E_{\mathrm{c,eff}}}\varepsilon_{\mathrm{cs}}\frac{S_{\mathrm{s}}}{I_{\mathrm{g}}} \tag{9.24}$$

where

S_{s} is the first moment of area of the actual tension and compression reinforcements with respect to the centroid of the gross concrete cross-section;

I_{g} is the second moment of area of the gross concrete cross-section;

k_{I} is a coefficient accounting for cracking and creep defined as follows:

$$k_{\mathrm{I}} = \begin{cases} \zeta\dfrac{I_g}{I_{cr}} + (1-\zeta) \text{ if the section cracks} \\ 1{,}00 \text{ if the section does not crack} \end{cases} \tag{9.25}$$

With

$$\frac{I_g}{I_{cr}} = \frac{1}{2{,}7\left(\alpha_{e,ef}\rho\right)^{0{,}6}\left(\frac{d}{h}\right)^3} \tag{9.26}$$

Where

ρ is the provided tension reinforcement ratio related to $(b \cdot d)$ at mid span for continuous or simply supported elements and at the support for cantilevers;

$\alpha_{e,ef}$ $=E_s/E_{c,eff}$ is the effective modular ratio;

ζ is the zeta factor defined in Formula (9.29) which for beams may be determined on the basis the ratio between the cracking moment (M_{cr}) and the bending moment under the characteristic combination of actions (M_k), assuming a long-term analysis:

$$\zeta = 1 - 0.5\left(\frac{M_{cr}}{M_k}\right)^2$$

k_s is a coefficient to account for the effect of cracking on the shrinkage deflection. k_s may be estimated from Formula (9.27):

$$k_s = \begin{cases} 1{,}00 \text{ if the section does not crack} \\ 455\rho_l^2 - 35\rho_l + 1{,}6 \text{ if the section cracks} \end{cases} \tag{9.27}$$

9.3.4 General method for deflection calculations

(1) For a detailed deflection control the deformations should be calculated under load conditions which are appropriate to the purpose of the check, see EN 1990.

(2) Members which are not expected to be stressed above the level which would cause the tensile strength of the concrete to be exceeded anywhere within the member may be considered to be uncracked. The assessment should allow for tensile stresses induced by restraint of early age thermal and long-term shrinkage strains.

(3) Members which are expected to crack, but may not be fully cracked, should be taken to behave in a manner intermediate between the uncracked and fully cracked conditions according to Formula (9.28):

$$\alpha_\delta = (1 - \zeta)\alpha_{\mathrm{I}} + \zeta\alpha_{\mathrm{II}} \tag{9.28}$$

where

α_δ is the deformation parameter considered which may be, for example, a strain, a curvature, or a rotation (as a simplification α_δ may also be taken as a deflection, see (7));

α_{I}, α_{II} are the values of the deformation parameter calculated for the uncracked and fully cracked conditions respectively;

ζ is a distribution coefficient (allowing for tension stiffening at a section) which may be taken from Formula (9.29):

$$\zeta = 1 - \beta_t\left(\frac{\sigma_{sr}}{\sigma_s}\right)^2 \geq 0 \tag{9.29}$$

$\zeta = 0$ for uncracked sections;

β_t is a coefficient taking into account the influence of the duration of loading or of repeated loading on the average strain; it may be taken equal to:

$\beta_t = 1{,}0$ for a single short-term loading,

$\beta_t = 0{,}5$ for sustained loads or many cycles of repeated loading;

σ_s is the highest stress having occurred up to the moment being analysed in the tension reinforcement calculated on the basis of a cracked section;

σ_{sr} is the stress in the tension reinforcement calculated on the basis of a cracked section under the loading conditions causing first cracking.

NOTE σ_{sr}/σ_s may be replaced by M_{cr}/M for flexure or N_{cr}/N for pure tension, where M_{cr} is the cracking moment and N_{cr} is the cracking force.

(4) For the calculation of deflections under predominantly flexural stresses, the value of $f_{ct,eff}$ may be taken as $f_{ctm,fl}$ (see Formula (9.30) with h [mm]) provided that the calculation of minimum reinforcement for crack control is based on the same value.

$$f_{ctm,fl} = \max\left\{\left(1{,}6 - \frac{h}{1000}\right)f_{ctm}; f_{ctm}\right\} \tag{9.30}$$

(5) For loads with a duration causing creep, the total deformation including creep may be calculated by using an effective modulus of elasticity for concrete according to Formula (9.1).

(6) Shrinkage curvatures may be assessed using Formula (9.24). In this case, S and I should be calculated for the uncracked condition and the fully cracked condition, the final curvature being assessed by use of Formula (9.28) and the deflection due to shrinkage determined by double integration of the curvature.

(7) For a rigorous calculation of deflections using the method given in (3) the curvatures should be computed at frequent sections along the member and then, the deflection calculated by numerical integration. In most cases the deflection may be computed once, assuming the whole member to be uncracked and secondly assuming a fully cracked condition, and then interpolate using Formula (9.28).

9.4 Vibrations

(1) Measures should be taken to prevent vibrations that cause discomfort to people or limit the functional effectiveness of the structure.

(2) Verification of vibrations of concrete structures should be considered following indications in EN 1990:2021 (A1.8.1, A2.8.2, A2.8.3, A2.8.4) and EN 1991-1-4, Annex E, as relevant.

(3) For the calculation of vibrations the dynamic modulus of elasticity may be taken as $1{,}1\ E_{cm}$.

(4) For the verification of vibrations of concrete structures, in the absence of more refined methods, the effective damping ratio ξ_v may be taken as the sum of the effective structural damping, $\xi_{v,st}$, plus the effective damping provided by non-structural components.

(5) Table 9.4 provides values for the effective structural damping, $\xi_{v,st}$.

NOTE Values of effective structural damping and of non-structural components in bridges are provided in Annex K.9(5).

Table 9.4 — Suggested values for effective damping components

Structural Damping ($\xi_{v,st}$) in general		
1	Reinforced concrete	2 %
2	Prestressed concrete	1 %

10 Fatigue

10.1 General

(1) Structures and structural components subjected to significant numbers of repeated load or deformation induced significant stress cycles shall be verified to endure the expected cyclic actions during the required design life.

NOTE A fatigue verification can generally be omitted for the following structures and members:

a) common buildings subjected to a total number of significant load cycles $\leq 2 \cdot 10^4$;

b) prestressing and reinforcing steel, in sections where, under the frequent combination of actions and P_k, only compressive stresses occur at the extreme concrete fibres;

c) external and unbonded tendons, lying within the depth of the concrete section.

(2) The verification shall be performed separately for reinforcement and concrete by:

— simplified methods given in 10.4 to 10.7; or

— refined methods using damage equivalent stresses in E.4 and K.10 where applicable; or

— explicit method using *Palmgren-Miner* Rule in E.5 where applicable;

— EN 1993-1-9 for welds.

10.2 Combination of actions

(1) The fatigue-inducing cyclic action should in general be combined with other actions according to EN 1990:

$$\sum F_{\mathrm{d}} = \sum_i G_{\mathrm{k,i}} + \sum_j \psi_{2,\mathrm{j}} Q_{\mathrm{k,j}} + (P_{\mathrm{k}}) + F_{\mathrm{fat,d}} \tag{10.1}$$

NOTE $Q_{k,j}$ are non-cyclic, non-permanent actions, these should include temperature $Q_{k,temp}$ to determine whether cracked or uncracked sections should be considered. $F_{fat,d}$ is the design value of fatigue action (e.g. traffic load as defined in EN 1991 or other cyclic load) as defined in EN 1991.

(2) For the verification according to 10.4 to 10.7, F_{fat} should be taken as the cyclic component of the frequent load as defined in EN 1991. For road bridges, F_{fat} should be taken as the frequent load of load model 1 according to EN 1991-2. For railway bridges, F_{fat} should be taken as the frequent load of load model 71 according to EN 1991-2.

10.3 Internal forces and stresses for fatigue verification

(1) When determining forces and stresses of the reinforcement, the tensile strength of concrete shall be ignored and a linear stress-strain relationship for concrete under compression shall be used, satisfying compatibility of strains of the two materials.

(2) The effect of different bond behaviour of prestressing and reinforcing steel shall be taken into account. This may be done by calculating the steel stress range with an equivalent area of reinforcement for the prestressing tendons considering the ratio of bond stress ξ:

$$A_{\mathrm{e}} = A_{\mathrm{p}} \sqrt{\xi \frac{\phi}{\phi_{\mathrm{p}}}} \tag{10.2}$$

where

A_e is the equivalent area of reinforcement;

A_p is the area of prestressing tendon or tendons;

ϕ is the largest diameter of reinforcing steel;

ϕ_p is the diameter or equivalent diameter $\phi_{p,eq}$ of prestressing steel:

$\phi_{p,eq} = 1{,}60\sqrt{A_p}$ for bundles of wires or strands,

$\phi_{p,eq} = 1{,}75\phi_{wire}$ for single 7 wire strands,

where ϕ_{wire} is the wire diameter,

$\phi_{p,eq} = 1{,}20\phi_{wire}$ for single 3 wire strands;

ξ is the ratio of bond strength between bonded tendons and ribbed or indented reinforcing steel in concrete. Unless more precise data is available the values given in Table 10.1 may be used.

Table 10.1 — Ratio of bond strength ξ between tendons and reinforcing steel

prestressing steel	Ratio of bond strength ξ		
	pre-tensioned	bonded, post-tensioned	
		$f_{ck} \leq 50$ MPa	$f_{ck} \geq 70$ MPa
Smooth bars and wires	not applicable	0,30	0,15
Strands	0,60	0,50	0,25
Indented wires	0,70	0,60	0,30
Ribbed bars	0,80	0,70	0,35

NOTE 1 Intermediate values may be interpolated between 50 MPa < f_{ck} < 70 MPa.

NOTE 2 Values ξ are valid for tendons directly cast into concrete or contained within corrugated metal ducts.

(3) The redistribution of stresses in concrete in the compression zone may be accounted for by verifying the compression stress at a distance of 100 mm (but not more than 1/3 of the cross-section depth) from the outermost compressed fibre. The stress should not be taken less than 2/3 of the maximum compressive stress for concrete compressive strength up to f_{ck} = 55 MPa and not less than 0,9 for f_{ck} = 100 MPa, respectively, intermediate values may be linearly interpolated.

(4) In the verification for fatigue resistance the inclination of the compressive struts θ_{fat} should be calculated using the compression field inclination θ at ULS from 8.2.3 in Formula (10.3):

$$\cot\theta_{fat} = \sqrt{\cot\theta} \geq 1{,}0 \quad (10.3)$$

For a more refined calculation, θ_{fat} may be taken as θ in G.5, for the maximum shear in the cycle.

(5) In prestressing and reinforcing steel exposed to fatigue loads, the maximum stresses under the relevant fatigue load combination shall not exceed the design yield strength.

10.4 Simplified verification of reinforcing or prestressing steel

(1) Adequate fatigue resistance may be assumed under tension, if the stress range under the fatigue load combination according to 10.2 with the frequent cyclic load with a maximum of 10^8 load cycles complies with:

a) reinforcing steel bars (for bent bars footnote [a)] of Table E.1 (NDP) should be applied):

— $\Delta\sigma_{sd} \leq 90$ MPa unwelded reinforcing bars $\phi \leq 12$ mm;

— $\Delta\sigma_{sd} \leq 73$ MPa unwelded reinforcing bars $\phi > 12$ mm;

— $\Delta\sigma_{sd} \leq 40$ MPa butt and tack welded reinforcing bars $\phi \leq 12$ mm;

— $\Delta\sigma_{sd} \leq 30$ MPa butt and tack welded reinforcing bars $\phi > 12$ mm;

— $\Delta\sigma_{sd} \leq 19$ MPa couplers.

b) prestressing steel for pre-tensioning:

— $\Delta\sigma_{pd} \leq 95$ MPa.

c) prestressing steel for post-tensioning:

— $\Delta\sigma_{pd} \leq 95$ MPa single strands in plastic ducts;

— $\Delta\sigma_{pd} \leq 80$ MPa straight tendons and curved tendons in plastic ducts;

— $\Delta\sigma_{pd} \leq 55$ MPa curved tendons in steel ducts.

NOTE These limits for the design stress ranges (including partial factor γ_{Ff} according to EN 1990) in the reinforcement are based on the S-N curves in Tables E.1 (NDP) and E.2 (NDP) assuming 10^8 load cycles and $\gamma_S = 1{,}15$. Modification of values in Tables E.1 (NDP) and E.2 (NDP) will result in changes of the limits given above.

10.5 Simplified verification of concrete under compression

(1) Adequate fatigue resistance of concrete under compression may be assumed to be met, if the following condition is satisfied:

$$\frac{|\sigma_{cd,max}|}{f_{cd,fat}} \leq 0{,}5 + 0{,}45 \frac{|\sigma_{cd,min}|}{f_{cd,fat}} \leq 0{,}90 \tag{10.4}$$

where

$\sigma_{cd,max}$ is the maximum compressive stress at a fibre under the fatigue load combination according to 10.2;

$\sigma_{cd,min}$ is the minimum compressive stress at the same fibre where $\sigma_{cd,max}$ occurs;

$f_{cd,fat}$ is the design fatigue strength of concrete according to Formula (10.5):

$$f_{cd,fat} = \beta_{cc}(t_0) \cdot \frac{f_{ck}}{\gamma_C} \cdot \eta_{cc,fat} \tag{10.5}$$

where

$\beta_{cc}(t_0)$ is a coefficient for concrete strength at first load application t_0 (see B.4(1));

$\eta_{cc,fat}$ $= \min\{0{,}85\eta_{cc};\ 0{,}8\}$;

NOTE Formula (10.4) is based on 10^7 assumed load cycles.

10.6 Simplified verification of concrete under shear

(1) For members requiring design shear reinforcement at the ultimate limit state, Formula (10.4) may be applied to the struts of members subjected to shear. In this case the design fatigue reference strength of concrete $f_{cd,fat}$ should be reduced by the strength reduction factor ν. A value of $\nu = 0{,}5$ may be used.

(2) For members not requiring design shear reinforcement at the ultimate limit state it may be assumed that the concrete resists fatigue due to shear effects where the following formulae are complied with:

for $\tau_{Ed,min}/\tau_{Ed,max} \geq 0$:

$$\frac{|\tau_{Ed,max}|}{\tau_{Rd,c}} \leq 0{,}5 + 0{,}45\frac{|\tau_{Ed,min}|}{\tau_{Rd,c}} \leq 0{,}90 \tag{10.6}$$

for $\tau_{Ed,min}/\tau_{Ed,max} < 0$:

$$\frac{|\tau_{Ed,max}|}{\tau_{Rd,c}} \leq 0{,}5 - \frac{|\tau_{Ed,min}|}{\tau_{Rd,c}} \tag{10.7}$$

where

$\tau_{Ed,max}$ is the design shear stress due to the maximum applied shear force under the fatigue load combination according to 10.2;

$\tau_{Ed,min}$ is the design shear stress due to the minimum applied shear force under the fatigue load combination according to 10.2 in the cross-section where $\tau_{Ed,max}$ occurs;

$\tau_{Rd,c}$ is the design value for shear resistance stress without shear reinforcement according to Formulae (8.27) or (8.94).

10.7 Simplified verification of shear at interfaces

(1) Interfaces subjected to fatigue actions should be rough, very rough or keyed.

(2) In verification with Formula (10.8) without reinforcement crossing the interface, the values for $c_{v1,fat}$ and $\mu_{v,fat}$ in Table 10.2 should be applied.

$$\tau_{Edi} \leq \tau_{Rdi} = c_{v1,fat}\frac{\sqrt{f_{ck}}}{\gamma_C} + \mu_{v,fat} \cdot |\sigma_n| \leq 0{,}3 \cdot f_{cd} + \rho_i\, f_{yd} \cos\alpha \tag{10.8}$$

(3) The fatigue verification of reinforcement crossing the interface may be omitted in members with shear reinforcement crossing the interface and fulfilling the minimum requirements according to 12.2, if:

— the interface is very rough or keyed as defined in 8.2.6(6); and

— the interface reinforcement is sufficiently anchored and crosses the interface at an angle according to Figure 8.15 b).

(4) If reinforcement crossing the interface is required according to 8.2.6, it should be anchored and the shear strength verified according to Formula (10.9) with $\mu_{v,fat}$ from Table 10.2:

$$\Delta\tau_{Edi} \leq \Delta\tau_{Rdi} = \mu_{v,fat} \cdot |\sigma_n| + \rho\frac{\Delta\sigma_{Rsk}}{0{,}45 \cdot \gamma_S}\left(\mu_{v,fat} \cdot \sin\alpha + \cos\alpha\right) \tag{10.9}$$

where the stress range $\Delta\tau_{Edi}$ is defined by upper and lower load level of Formula (10.1).

The value of $\Delta\sigma_{Rsk}$ should be taken from Table E.1 (NDP) or based on testing appropriate to the design situation under consideration.

Table 10.2 — Coefficients $c_{v1,fat}$ and $\mu_{v,fat}$ for different interface roughness

Surface roughness	without interface reinforcement Formula (10.8)		with interface reinforcement Formula (10.9)
	$c_{v1,fat}$	$\mu_{v,fat}$	$\mu_{v,fat}$
Rough	0,075	0,7	0,7
Very rough	0,095	0,9	0,9
Keyed	0,185	0,9	0,9

11 Detailing of reinforcement and post-tensioning tendons

11.1 General

(1) The rules given in Clause 11 apply to ribbed and indented reinforcement, mesh and post-tensioning tendons.

11.2 Spacing of bars

(1) The clear distance between parallel bars and their arrangement should be sufficient to allow access for vibrators if needed for good compaction of the concrete. Where bars are positioned in separated horizontal layers, there should be sufficient space between the resulting columns of bars to allow access for vibrators and good compaction of concrete.

(2) The clear distance c_s (horizontal and vertical) between individual parallel bars should be not less than max{ϕ; D_{upper} + 5 mm; 20 mm}.

(3) Bars may be arranged in bundles with a maximum of 3 contacting parallel bars. For bundles of vertical bars in compression and for bars in a lapped joint, a maximum of 4 contacting parallel bars may be used. Bars in bundles of 3 and 4 should be arranged so that every bar is in contact with at least two other bars. Clear distance between bundles of bars should be not less than the equivalent diameter ϕ_b defined in 11.4.3(1).

(4) The clear distance between the face of already poured concrete and a bar parallel to it should be at least 5 mm if the surface of the already poured concrete is at least rough according to 8.2.6(6), else the distance should equal the minimum required for bond according to 6.5.2.3(1).

(5) The minimum clear spacing between post-installed reinforcing steel bars or between post-installed reinforcing steel bars and cast-in reinforcing steel bars is given in 11.4.8.

11.3 Permissible mandrel diameters for bent bars

(1) The minimum diameter to which a bar may be bent shall be such as to avoid:

— damaging the reinforcement, see (2); and

— failure of the concrete inside the bend of the bar (crushing, splitting or spalling of reinforcement cover), see (3), (4) and (5).

(2) The mandrel diameter of bars without welds or of bars and welded fabrics with welds at least 3ϕ away from the bend should be at least:

— $\phi_{mand,min} = 4\phi$ for $\phi \leq 16$ mm;

— $\phi_{\mathrm{mand,min}} = 7\phi$ for $\phi > 16$ mm.

(3) Provided that $f_{\mathrm{yd}} \leq 25 f_{\mathrm{cd}}$ and $\gamma_{\mathrm{C}} \leq 1{,}5$, verification of the concrete inside the bend may be omitted for:

— Stirrups and links in compliance with 12.3.3;

— standard hook and bend anchorages complying with Figure 11.6 where:

— the anchorage of the bar does not require a length more than 5ϕ past the end of the bend;

— there is a clear distance $c_{\mathrm{x}} \geq 1{,}5\phi$ from an edge parallel to the bend;

— there is a clear distance between bars $c_{\mathrm{s}} \geq 3\phi$ according to Figure 11.3c; and

— provided that $f_{\mathrm{yk}} \leq 500$ MPa and $f_{\mathrm{ck}} \geq 25$ MPa, all bends with an angle $\alpha_{\mathrm{bend}} \leq 45°$ at a clear distance $c_{\mathrm{x}} \geq 2{,}5\phi$ from an edge parallel to the bend, a clear distance between bars $c_{\mathrm{s}} \geq 5\phi$ and the length of the straight segments between multiple bends is not shorter than 4ϕ.

(4) In cases not complying with (3), the design value of the steel stress σ_{sd} should be verified to avoid concrete failures inside the bend. This may be done according to Formula (11.1):

$$\sigma_{\mathrm{sd}} \leq 0{,}65 f_{\mathrm{cd}} \frac{\phi_{\mathrm{mand}}}{\phi} + \frac{\sqrt{f_{\mathrm{ck}}}}{\gamma_C} \left(\frac{d_{\mathrm{dg}}}{\phi}\right)^{1/3} \left(\frac{c_{\mathrm{d}}}{\phi} + \frac{1}{2}\right) \left(k_{\mathrm{bend}} + 0{,}7 \cdot \frac{\phi_{\mathrm{mand}}}{\phi}\right) \tag{11.1}$$

where

c_{d} is the minimum of the clear distance c_{x} from an edge parallel to the bend and half the clear space between bars c_{s} according to Figure 11.3c);

k_{bend} is a parameter considering the bend angle α_{bend}:

$k_{\mathrm{bend}} = 32 \cdot (45°/\alpha_{\mathrm{bend}})$ (α_{bend} defined in Figure 11.6).

In case of multiple bends, the length of the straight sections between bends shall not be shorter than 4ϕ.

NOTE Practical bending can require longer straight sections.

(5) If transverse bars are placed within the bend, the limit to the steel stress σ_{sd} in Formula (11.1) may be increased by the following factor:

$$k_{\mathrm{trans}} = 1 + 4\, n_{\mathrm{trans}}\, (\phi/\phi_{\mathrm{mand}})(\phi_{\mathrm{trans}}/\phi)^2 (45°/\alpha_{\mathrm{bend}}) \tag{11.2}$$

where:

n_{trans} is the number of transverse bars within the bend; and

ϕ_{trans} is the diameter of transverse bars but not larger than $1{,}35\phi$.

11.4 Anchorage of reinforcing steel in tension and compression

11.4.1 General

(1) Reinforcing steel bars which are either cast-in or post-installed into hardened concrete extending out of a member and/or connecting a member to an adjacent member, and wires or welded fabrics shall be anchored so that their longitudinal forces are safely transmitted to concrete or are transferred to another reinforcement according to 11.5.

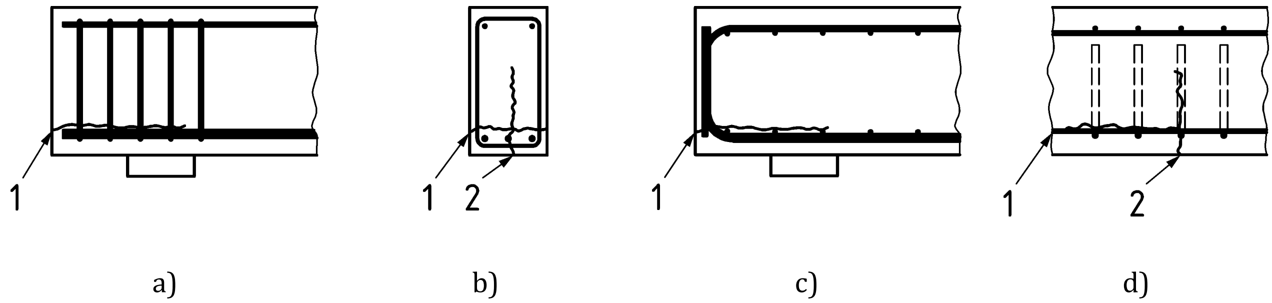
(2) Potential cracking parallel to the anchored reinforcement should be controlled by appropriate transverse or confinement reinforcement. Generally, minimum transverse or shear reinforcement

according to Clause 12 will suffice for bars $\phi \leq 32$ mm. Otherwise, minimum amount of confinement reinforcement according to 11.4.2(7) should be provided.

(3) In linear members, stirrups enclosing the bars as shown in Figures 11.1 a) and b) should be provided to control longitudinal cracking as well as delamination cracking parallel to the anchored reinforcement.

(4) In planar members, transverse bars parallel to the concrete surface placed above or below the anchored reinforcement as shown in Figures 11.1 c) and d) should be provided to control longitudinal cracks. Shear reinforcement, bends in the anchored reinforcement as shown in Figure 11.1 c) or edge reinforcement according to Figure 12.4 should be provided to control delamination cracks.

NOTE Longitudinal cracking in the plane of the anchored reinforcement can considerably reduce the anchorage performance.



Key

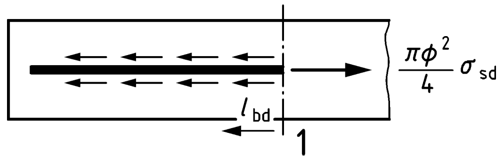
1 potential delamination crack

2 potential longitudinal crack

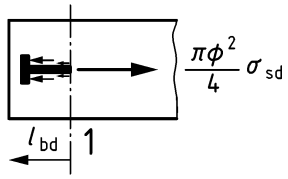
Figure 11.1 — Examples of reinforcement controlling delamination and longitudinal cracks

(5) Methods of anchorage as shown in Figure 11.2 may be used for reinforcing steel in tension and in compression, except method d) which should be used in tensiononly. However, for methods b) and c) used in compression, only the first straight segment may be considered as anchorage, except as given in 11.4.4(3).

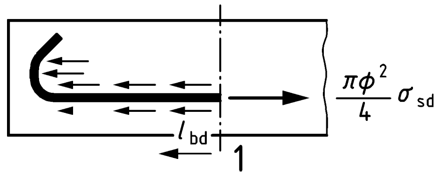
(6) The start of anchorage as shown in Figure 11.2 refers to the cross section where the reinforcement force is fully transferred to the concrete in compression (examples are shown in Figures 8.28 and 8.29). For force transfer to other reinforcements, see 11.5.



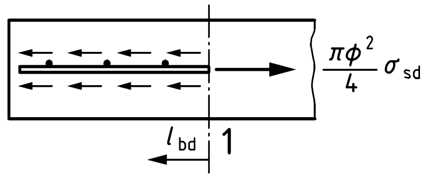
a) Anchorage of straight bars 11.4.2



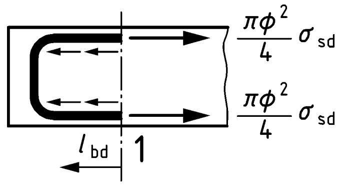
d) Anchorage of headed bars 11.4.7



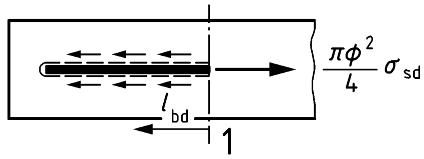
b) Anchorage of bends and hooks 11.4.4



e) Anchorage of welded reinforcement bars 11.4.5



c) U-bar loops 11.4.6



f) Anchorage of bonded post-installled reinforcing steel 11.4.8

Key

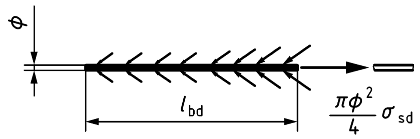
l_{bd} design anchorage length

1 start of anchorage

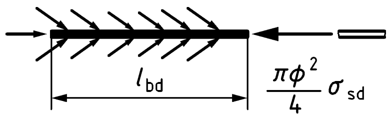
Figure 11.2 — Methods of anchorage (shown for tensile forces)

11.4.2 Anchorage of straight bars

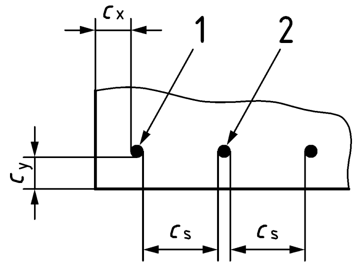
(1) A design anchorage length l_{bd} (Figure 11.3) shall at least be provided to safely transfer the forces from a bar to the surrounding concrete without spalling or bond failures.



a) tension



b) compression



c) conditions for c_d

Key

1 corner bar

2 edge bar

nominal cover $c_d = \min\{0{,}5c_s; c_x; c_y\}$

Figure 11.3 — Definition of design anchorage length for straight bars and c_d

(2) For ribbed bars with $\phi \leq 32$ mm and indented bars with $\phi \leq 14$ mm where $c_d \geq 1{,}5\phi$, $\sigma_{sd} = 435$ MPa and with good bond conditions according to (4) the design anchorage length l_{bd} divided by diameter in tension and compression in persistent and transient design situations may be taken from Table 11.1 (NDP). For bars in poor bond conditions according to (4) the values in Table 11.1 (NDP) should be multiplied by 1,2.

NOTE 1 The values in Table 11.1 (NDP) apply unless the National Annex gives different values.

Table 11.1 (NDP) — Anchorage length of straight bars divided by diameter l_{bd}/ϕ

ϕ [mm]	Anchorage length l_{bd}/ϕ							
	f_{ck}							
	20	**25**	**30**	**35**	**40**	**45**	**50**	**60**
≤ 12	47	42	38	36	33	31	30	27
14	50	44	41	38	35	33	31	29
16	52	46	42	39	37	35	33	30
20	56	50	46	42	40	37	35	32
25	60	54	49	46	43	40	38	35
28	63	56	51	47	44	42	40	36
32	65	58	53	49	46	44	41	38
NOTE The values of Table 11.1 (NDP) are derived from Formula (11.3).								

(3) In cases not complying with the limitations of (2), or for a more detailed calculation, the design anchorage length l_{bd} should be calculated as:

$$l_{bd} = k_{lb} \cdot k_{cp} \cdot \phi \cdot \left(\frac{\sigma_{sd}}{435}\right)^{n_\sigma} \cdot \left(\frac{25}{f_{ck}}\right)^{\frac{1}{2}} \cdot \left(\frac{\phi}{20}\right)^{\frac{1}{3}} \cdot \left(\frac{1{,}5\phi}{c_d}\right)^{\frac{1}{2}} \geq 10\phi \qquad (11.3)$$

Where

Ratios in Formula (11.3) shall be limited to (ϕ/20 mm) ≥ 0,6 and ($25/f_{ck}$) ≥ 0,3:

c_d $c_d = \min\{0{,}5c_s; c_x;\ c_y;\ 3{,}75\phi\}$, see Figure 11.3.

k_{cp} coefficient accounting for casting effects on bond conditions:

- $k_{cp} = 1{,}0$ for bars with good bond conditions according to (4);
- $k_{cp} = 1{,}2$ for poor bond conditions and for all bars used in slipform construction unless it is shown that the vertical bars cannot move during casting;
- $k_{cp} = 1{,}4$ for all bars executed under bentonite or similar slurries unless data is available for the specific slurry to be used;

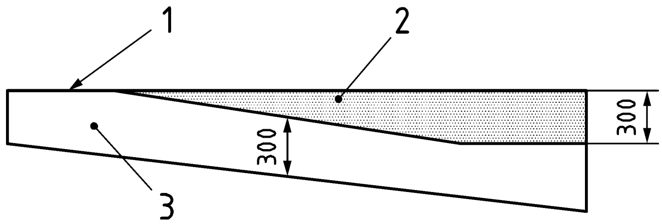
NOTE 2 For anchorages, the following values for k_{lb} and n_σ apply unless the National Annex gives different values:
$k_{lb} = 50$ for persistent and transient design situations with $n_\sigma = 3/2$; and
$k_{lb} = 35$ for accidental design situations with $n_\sigma = 3/2$.

(4) Good bond conditions are defined as:

a) bars with an inclination of 45° to 90° to the horizontal during concreting; and

b) bars with an inclination less than 45° to the horizontal which are up to 300 mm from the bottom of the formwork or at least 300 mm from the free surface during concreting.

Otherwise, poor bond conditions should be assumed (see Figure 11.4).



Key

1 top surface during concreting

2 zone with poor bond conditions for bars with an inclination less than 45° to the horizontal

3 zone with good bond conditions

Figure 11.4 — Description of bond conditions as a function of member depth

(5) In presence of confinement reinforcement crossing the potential splitting surface shown in Figure 11.5a) and placed at a net distance ≤ 5ϕ from the bar to be anchored, or of transverse reinforcement arranged between the bar to be anchored and the free surface (Figure 11.5b)) or/and of external pressure (Figure 11.5c)), the design anchorage length may be reduced by replacing parameter c_d in Formula (11.3), by:

$$c_{\mathrm{d,conf}} = \min\left\{c_{\mathrm{x}}; c_{\mathrm{y}} + 25\frac{\phi_{\mathrm{t}}^2}{s_{\mathrm{t}}}; \frac{c_{\mathrm{s}}}{2};\ 3{,}75\phi\right\} + \Delta c_{\mathrm{d}} \leq 6\phi \tag{11.4}$$

Where $\Delta c_{\mathrm{d}} = (70\rho_{\mathrm{conf}} + 12\sigma_{\mathrm{ccd}}/\sqrt{f_{\mathrm{ck}}})\phi$;

and:

ρ_{conf} is the ratio of the confinement reinforcement referred to the diameter of the bar to be anchored or spliced:

$$\rho_{\mathrm{conf}} = \frac{n_{\mathrm{c}} \cdot \pi \cdot \phi_{\mathrm{c}}^2}{4 \cdot n_{\mathrm{b}} \cdot \phi \cdot s_{\mathrm{c}}} \tag{11.5}$$

ϕ is the diameter of the bar to be anchored or spliced;

ϕ_{c} is the diameter of the confinement reinforcement;

n_{c} is the number of legs of the confinement reinforcement crossing the potential splitting failure surface (see Figures 11.5 a));

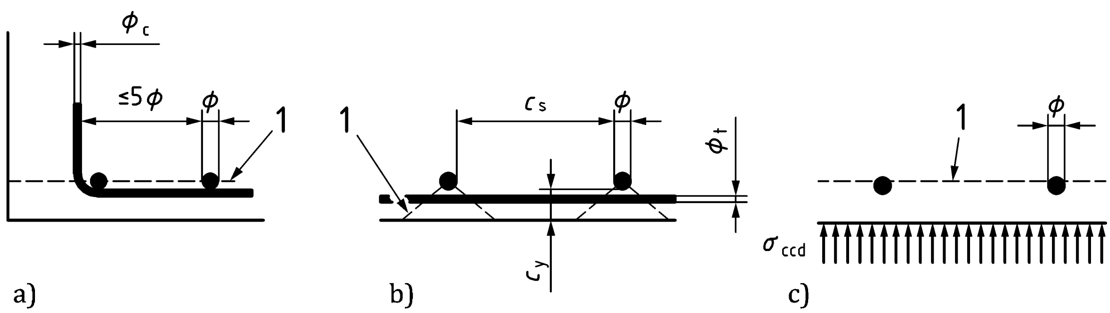
n_{b} is the number of anchored bars or pairs of lapped bars in the potential splitting failure surface;

s_c is the spacing of the confinement reinforcement along the bar to be anchored;

Φ_{t} is the diameter of the transverse reinforcement between the bar to be anchored and the free surface (see Figures 11.5 b));

s_t is the spacing of the transverse reinforcement along the bar to be anchored;

σ_{ccd} is the design value of the mean compression stress perpendicular to the potential splitting failure (see Figure 11.5 c)).



a) confinement reinforcement at spacing s_c; n_c=1, n_b=2

b) transverse reinforcement at spacing s_t

c) external design confinement pressure σ_{ccd}

Key

1 potential splitting surface

Figure 11.5 — Definition of cases where the design anchorage length may be reduced due to confinement or transverse reinforcement

(6) For anchorages of straight bars in compression where the distance measured parallel to the bar axis from the end of the bar to a free surface is not less than 5ϕ, the design anchorage length l_{bd} calculated according to (2) to (5) may be reduced by 15ϕ, but shall not be shorter than 10ϕ.

(7) Transverse and confinement reinforcement, additional to that for shear, should be provided in the anchorage zones of bars ϕ> 32 mm where transverse compression is not present. For straight anchorage lengths the additional reinforcement both transverse and confinement should not be less than:

- transverse reinforcement: $A_{st} \geq 0{,}20\ \phi^2 n_1$; and

- confinement reinforcement: $A_{sc} \geq 0{,}20\ \phi^2 n_2$

where

n_1 is the number of layers with bars anchored at the same point in the member;

n_2 is the number of bars anchored in each layer.

The additional transverse and confinement reinforcement should be uniformly distributed in the anchorage zone and the spacing of bars should not exceed 5ϕ.

11.4.3 Anchorage of bundles

(1) All provisions for anchorage of straight bars may be used also for bundles of bars anchored in one cross section according to 11.4.2(3) with parameter ϕ replaced by an equivalent diameter of the bundle defined as:

$$\phi_b = \sqrt{\frac{4}{\pi} A_s} \tag{11.6}$$

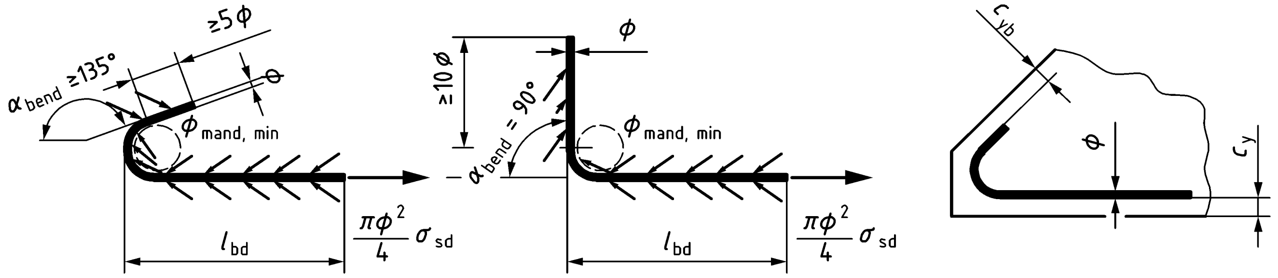
where A_s is the total area of all bars contained in the bundle.

(2) When anchoring one bar in a bundle, the design anchorage length should be based on its own diameter and covers. When anchoring more than one bar of a bundle, the design anchorage length of those bars should be based on their equivalent diameter and covers. Values k_{lb} are according to Formula (11.3).

11.4.4 Anchorage of bars with bends and hooks

(1) The design anchorage length according to 11.4.2(2) or (3) may be reduced by 15ϕ for standard hook and bend anchorages in tension complying with Figure 11.6 (but with $l_{bd} \geq 10\phi$).

Parameter c_d to be used in 11.4.2 is defined in Figure 11.6c).



a) standard hook

b) standard 90° bend anchorages

c) definition of nominal cover

$c_d = \min\{c_s/2; c_x; c_y; c_{yb}\}$

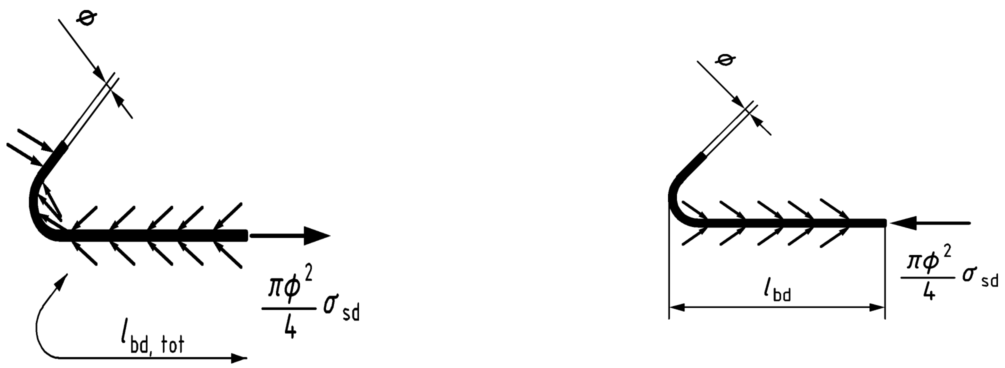
(definition of c_s and c_x, see Figure 11.3 c))

Figure 11.6 — Anchorage with standard hook and bend in tension

(2) For any bend and hook anchorages in tension, the total design anchorage length $l_{bd,tot}$ measured along the centre-line of the bar as defined in Figure 11.7 a) may be calculated using 11.4.2.

Parameter c_d to be used in 11.4.2 is defined in Figure 11.6 c).

(3) For bend and hook anchorages in compression, only the projected first straight segment (l_{bd} in Figure 11.7 b)) may generally be taken to contribute to anchorage, except when all free surfaces perpendicular to the bar are at a distance $\geq 3{,}5\phi$, when the design anchorage length l_{bd} may be reduced by 15ϕ, but shall not be shorter than 10ϕ.



a) general bend or hook in tension

b) standard and general bend or hook in compression

Figure 11.7 — Definition of design anchorage length l_{bd} or $l_{bd,tot}$ for bars with general hooks and bends in tension and for bars with bend or hook in compression

11.4.5 Anchorage of bars with welded transverse reinforcement

(1) The design anchorage length in 11.4.2(2) or (3) of bars with welded transverse reinforcement in tension and compression may be reduced by 15ϕ, (but with $l_{bd} \geq 5\phi$) under the condition that:

— when $\phi_t \geq 0{,}6\phi$ at least one transverse bar is located within the anchorage length (Figure 11.8a)) at least 50 mm back from a face of the support, where applicable;

— when $\phi_t \leq 0{,}6\phi$ a minimum of two transverse bars are located within the anchorage length with 50 mm $\leq s \leq$ 100 mm and for $\phi \leq$ 16 mm (Figure 11.8b)), the second transverse bar from the end at least 50 mm back from a face of the support, where applicable.

NOTE For minimum weld strength, see C.4.1(3).

a) one transverse bar within the anchorage length

b) two transverse bars within the anchorage length

Figure 11.8 — Methods of anchorage with welded transverse reinforcement

11.4.6 Anchorage of U-bar loops

(1) For U-bar loops subject to pure tension, anchorage may be considered to be provided if the loop details comply with 11.3.

(2) Alternatively, the design anchorage length in 11.4.2(2) or (3) with U-bar loops in tension with the minimum mandrel diameter may be reduced by 20ϕ (but with $l_{bd} \geq 10\phi$).

11.4.7 Anchorage of headed bars in tension

(1) A tensile stress in the reinforcing steel bar σ_{sd} = 435 MPa may be considered to be developed without additional anchorage length for heads $\phi_h \geq 3\phi$ with $f_{ck} \geq$ 25 MPa, $\phi \leq$ 25 mm; $d_{dg} \geq$ 32 mm and:

— $a_y \geq 3\phi$ for headed bars in uncracked concrete or $a_y \geq 4\phi$ for headed bars in cracked concrete;

— $a_x \geq 2a_y + 1{,}2\phi_h$;

— $s_x \geq 4a_y$

where

ϕ_h is the diameter of a circular head $\leq 4t_h$ (where t_h is the head thickness) or the diameter of a circle with the same area as that of the actual head

$$\phi_h = 2\sqrt{\frac{A_h}{\pi}} \tag{11.7}$$

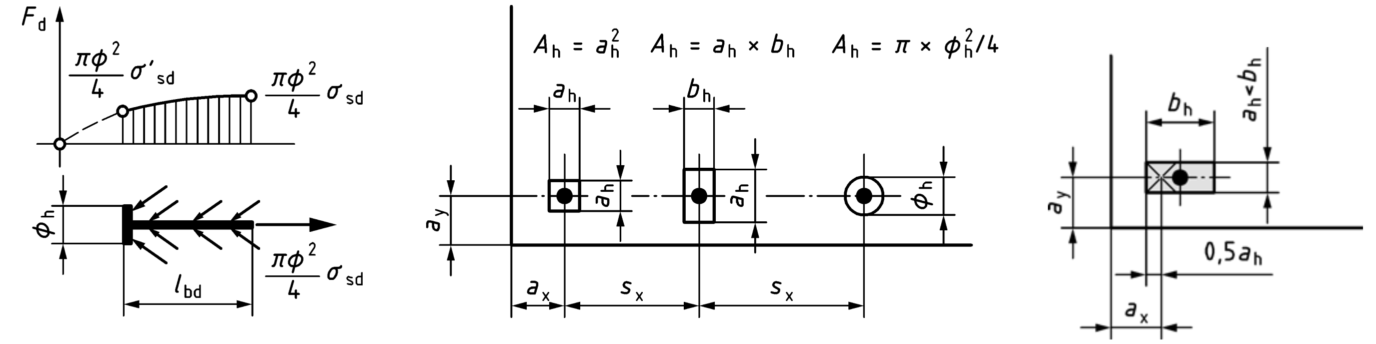
where A_h is the total area of the head defined in Figure 11.9;

a_y is the distance between the bar axis and the nearest edge as defined in Figure 11.9b);

a_x is the minimum distance between the bar axis and a corner as defined in Figure 11.9 b). For rectangular heads elongated in the direction of the considered edge, a_x should be considered according to Figure 11.9 c). In case a_x becomes smaller than a_y, the dimensions a_x and a_y should be inverted in the verification;

s_x is the bar spacing of a group of bars along the considered edge as defined in Figure 11.9 b);

a_h and b_h are the widths of square and rectangular heads which shall not be taken larger than 4 times the thickness of the head plate.



a) definition of design anchorage length for headed bars in tension

b) definition of head dimensions (a_h, b_h, ϕ_h, A_h), distance to the considered edge (a_y), distance to the corner (a_x) and bar spacing (s_x) for square, rectangular and circular heads

c) definition of distance a_x for rectangular heads near to corners

Figure 11.9 — Definition of design anchorage length, head sizes and bar distances for headed bars

(2) In cases not complying with the requirements of (1) or for a more detailed calculation, the maximum tensile stress in the reinforcing steel developed by the head should be calculated as:

$$\sigma'_{sd} = k_{h,A} \cdot f_{cd} + \nu_{part} \frac{\sqrt{f_{ck}}}{\gamma_C} \cdot \frac{a_d}{\phi} \cdot \left(\frac{\phi_h}{\phi}\right)^{\frac{5}{6}} \cdot \left(\frac{d_{dg}}{\phi}\right)^{\frac{1}{3}} \leq k_{h,A} \cdot \nu_{part} \cdot f_{cd} \tag{11.8}$$

where

$k_{h,A}$ is the ratio between the net area of the head and the cross-sectional area of the reinforcement:

$$k_{h,A} = \frac{A_h - A_s}{A_s} = \left(\frac{\emptyset_h}{\emptyset}\right)^2 - 1 \tag{11.9}$$

ν_{part} = 11,0 for uncracked concrete in the region of the head
= 8,0 for concrete cracked in the region of the head

a_d is the nominal value of the distance between the bar and a free surface which may be assumed as:

- single bar near to an edge, bar near to a corner at a distance $a_x \geq 2a_y + 1{,}2\phi_h$ and group of bars along an edge at spacing $s_x \geq 4a_y$, with $a_x \geq 2a_y$: $a_d = a_y$;
- single bar near to a corner at a distance $a_x < 2a_y + 1{,}2\phi_h$: $a_d = 0{,}5 \cdot a_y + 0{,}25 \cdot a_x - 0{,}3 \cdot \phi_h$;
- group of bars along the edge at spacing $s_x < 4a_y$:

$$a_d = a_y \frac{s_x - \emptyset_h}{4a_y - \emptyset_h} + 2{,}3 \left(a_y - \frac{\emptyset_h}{2}\right) \frac{4a_y - s_x}{4a_y - \emptyset_h} \left(1 - \frac{1}{(\emptyset_h/\emptyset)^2}\right) \tag{11.10}$$

σ'_{sd} is defined in Figure 11.9a.

(3) For tensile forces larger than the anchorage capacity of the head, both forces in the head and in the anchorage length of the bar may be accounted for. The design length in the reinforcing bar l_{bd} to develop the remaining stress $\sigma_{sd} - \sigma'_{sd}$ may be calculated according to Formula (11.11) as:

$$l_{bd} = 1{,}1 \cdot \left(l_{bd}(\sigma_{sd}) - l_{bd}(\sigma'_{sd})\right) \tag{11.11}$$

(4) When used for shear or confinement reinforcement, anchorage should rely on the head only.

(5) Where it can be shown that external compressive forces (e.g. from supports) or forces due to opposed active headed bars (e.g. corner joint) acting in the zone of the head are sufficient to balance the tensile blow-out stresses in their direction, the resistance σ'_{sd} may be enhanced according to a more refined model.

11.4.8 Anchorage of bonded post-installed reinforcing steel

(1) This Eurocode applies to post-installed reinforcing steel bars comprising a (de-coiled) straight reinforcing steel bar with properties according to C.4 and an anchoring mortar with established suitability and properties according to C.8 for the intended application, exposure condition and temperature, see C.8(3).

NOTE 1 The design of post-installed reinforcing bars according to this Eurocode assumes that the installation is performed according to the manufacturer's installation instructions by qualified personnel and inspection of the installation is carried out by appropriately qualified personnel.

(2) Unless differently specified in a European Technical Product Specification, the minimum concrete cover $c_{min,b}$ for bond of post-installed reinforcing bars in all directions should be detailed in accordance with Table 11.2, as a function of the intended drilling method and of the execution procedure.

Table 11.2 — Minimum concrete cover $c_{min,b}$ for post-installed reinforcing steel bars

Drilling method	**Bar diameter**	$c_{min,b}$	
		without drilling aid	**with drilling aid**
Rotary percussion drilling / hammer drilling and diamond coring/drilling	$\phi < 25$ mm	$30\text{ mm} + 0{,}06 l_{bd,pi} \geq 2\phi$	$30\text{ mm} + 0{,}02 l_{bd,pi} \geq 2\phi$
	$\phi \geq 25$ mm	$40\text{ mm} + 0{,}06\, l_{bd,pi} \geq 2\phi$	$40\text{ mm} + 0{,}02 l_{bd,pi} \geq 2\phi$
Compressed air drilling	$\phi < 25$ mm	$50\text{ mm} + 0{,}08 l_{bd,pi}$	$50\text{ mm} + 0{,}02 l_{bd,pi}$
	$\phi \geq 25$ mm	$60\text{ mm} + 0{,}08 l_{bd,pi} \geq 2\phi$	$60\text{ mm} + 0{,}02 l_{bd,pi} \geq 2\phi$

(3) Unless specified differently in a European Technical Product Specification, the minimum clear spacing between individual post-installed parallel bars should be $c_{s,pir} = \max\{4\phi; 40\ \text{mm}\}$ and the minimum clear spacing c_s between post-installed reinforcing and cast-in reinforcing bars should not be less than $c_s \geq \max\{2\phi; 20\ \text{mm}\}$.

NOTE 2 The above given limits for concrete cover and spacings have been confirmed for maximum aggregate size up to 25 mm. The above given limits for spacings have been confirmed for boreholes drilled with rotary percussion drilling / hammer drilling and diamond coring/drilling.

(4) The anchorage of post-installed reinforcing steel bars shall be designed considering the requirements of 11.4.1 and 11.4.2. The design anchorage length $l_{bd,pi}$ of post-installed reinforcing steel bars in tension should be calculated as:

$$l_{\mathrm{bd,pi}} = \frac{l_{\mathrm{bd}}}{k_{\mathrm{b,pi}}} \geq 10\phi \cdot \alpha_{\mathrm{lb}} \tag{11.12}$$

where

l_{bd} calculated according to 11.4.2, where the concrete strength considered in Formulae (11.3) and (11.4) shall be limited to $f_{ck} \leq 50$ MPa or the value stated in the European Technical Product Specification (whichever is larger), and the design stress in the reinforcing bar be limited to $\sigma_{sd} \leq 435$ MPa unless tested to higher values;

$k_{b,pi}$ bond efficiency factor, see C.8;

α_{lb} factor accounting for cracks along the bar which may be taken as $\alpha_{lb} = 1{,}5$ in general or as given in the European Technical Product Specification.

(5) The design anchorage length $l_{bd,pi}$ of post-installed reinforcing steel bars in compression shall be determined according to Formula (11.12) but should not be reduced according to 11.4.2(6) unless confirmed by testing in accordance with C.8.

(6) Post-installed reinforcing steel bars may be lapped using $l_{sd,pi}$ with straight cast-in deformed bars according to 11.5.2 and 11.5.3 where the design anchorage length l_{bd} is replaced by $l_{bd,pi}$, calculated according to Formula (11.12) (see also Table 11.3). The lap length $l_{sd,pi}$ should be designed for the minimum concrete cover of either cast-in or post-installed reinforcing steel bars and the bond conditions of the cast-in reinforcing steel bars.

(7) For shear forces across interfaces with post-installed reinforcing steel bars, see 8.2.6.

11.5 Laps of reinforcing steel in tension and compression and mechanical couplers

11.5.1 General

(1) Tension and compression forces may be transmitted from one bar to another by:

— lapping of bars anchored with a method described in Table 11.3 (see 11.5.2);

— mechanical couplers ensuring load transfer in tension and compression or compression only (see 11.5.6);

— full penetration butt welds or fillet welds (see 11.5.7).

(2) Welds shall be designed for the minimum design yield strength of the reinforcing steel at both sides of the connection. In this case, they may take place at the same section, without staggering.

11.5.2 All types of laps

(1) The detailing of laps between bars shall be such that:

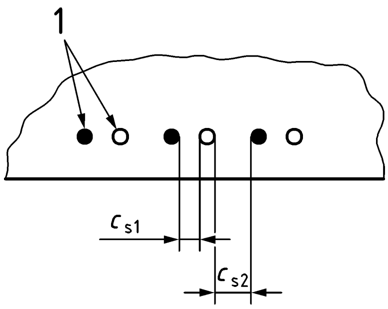
— the transfer of forces from one bar to the next is ensured;

— spalling of the concrete in the vicinity of the lap does not occur;

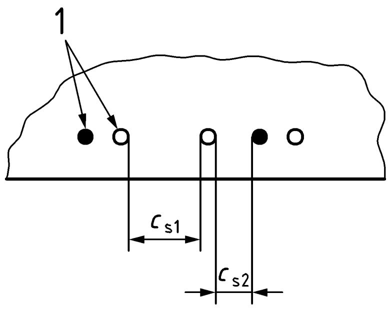
— large cracks parallel to the lapped bars which affect the performance of the structure do not develop. Generally, minimum transverse or shear reinforcement according to Clause 12 will suffice if the required diameter of lapped bars is $\phi < 20$ mm or the percentage of lapped bars in any section is < 25%. Otherwise, minimum amount of transverse or confinement reinforcement should be provided according to 11.5.2(10).

(2) A design lap length l_{sd} between two bars in tension or in compression should be provided, at least equal to the anchorage lengths l_{bd} given in 11.4 multiplied with k_{ls}, where the clear distance $c_s = c_{s1} + c_{s2}$ to calculate cover c_d should be taken according to Figure 11.10.

NOTE 1 The value k_{ls} = 1,2 applies unless the National Annex gives a different value.



a) adjacent laps



b) staggered laps

Key

1 pair of lapped bars

Figure 11.10 — Definition of clear distance c_s in lap splices

(3) All bars in compression may be lapped in one section and the lap length designed for σ_{sd}.

(4) Away from plastic hinge locations, tension laps may be detailed with up to 100 % of bars lapped at any section and the lap length may be designed for σ_{sd}.

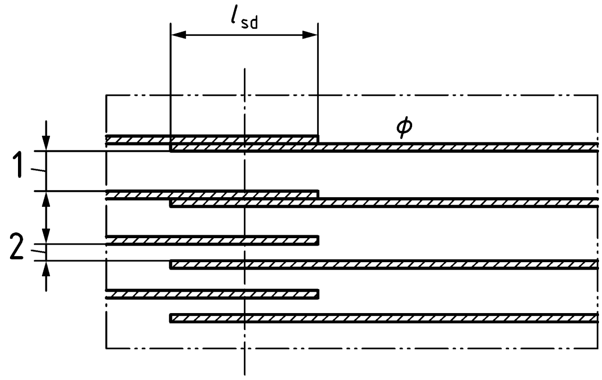
(5) Where tension laps are located across plastic hinge locations, tension lap length may be designed for σ_{sd} if:

— a confinement reinforcement is arranged according to 11.5.2(8); or

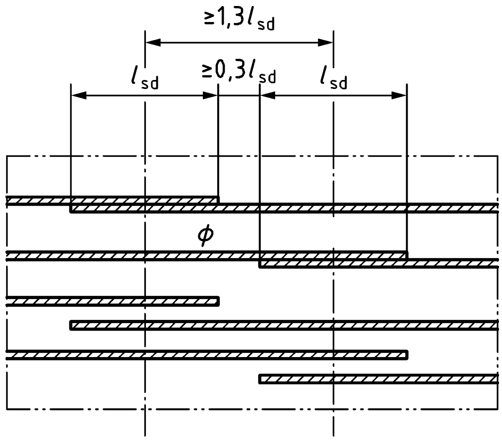
— if the laps are staggered so that the area of lapped bars is ≤ 35 % of the total cross section area of the reinforcement in linear members (beams and columns) or ≤ 50 % in planar members (slabs, walls and shells). If staggering of laps is chosen as an option, the distance between adjacent laps should be at least $0,3l_{sd}$ (see Figure 11.11b)).

Otherwise, tension lap lengths should be designed for $1,2\sigma_{sd}$.

NOTE 2 For laps in links, see 12.3.3(6).



a) 100 %-lap with clear distances between adjacent laps and lapped bars



b) 50 %-lap with clear distances between adjacent laps and lapped bars consistent with planar members

Key

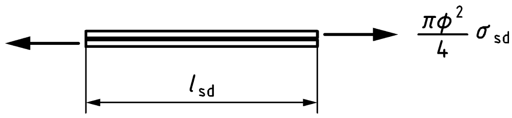
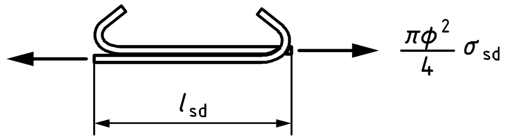
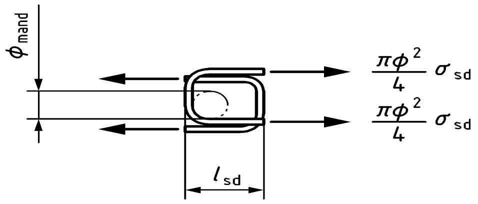
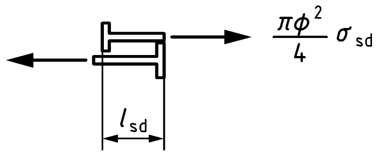
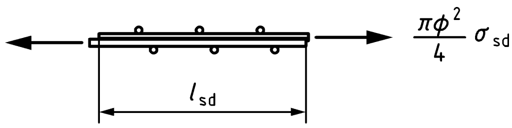
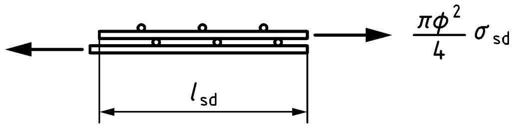
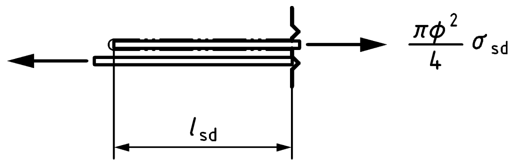
1 clear distance between adjacent laps: $c_{s,laps} \geq \max\{2\phi; 20\text{ mm}\}$

2 clear distance between lapping bars: $c_{s,bars} \leq \min\{4\phi; 50\text{ mm}\}$, see (7)

Figure 11.11 — Examples of lap arrangements

(6) In cases of different methods of anchorage according to Table 11.3, the larger value of the design lap length l_{sd} calculated for both methods shall be used.

Table 11.3 — Types of laps and design lap lengths l_{sd}

Type of lap splice		**Design lap length l_{sd}**	
		Tension laps	**Compression laps**
	straight bars	$l_{sd} = k_{ls} \cdot l_{bd} \geq 15\phi$ where l_{bd} is calculated according to 11.4.2, see also 11.5.3	
	bends and hooks (tension only)	$l_{sd} = k_{ls} \cdot l_{bd} \geq 15\phi$ where l_{bd} is calculated according to 11.4.4	—
	loops (tension only)	l_{sd} is calculated according to 11.5.4, with the limit $l_{sd} \geq \phi_{mand} + 4\phi$	—
	headed bars	l_{sd} is calculated according to 11.5.5	
	intermeshed fabric	$l_{sd} = k_{ls} \cdot l_{bd} \geq \max\{15\phi; 250\text{ mm}\}$ where l_{bd} is calculated according to 11.4.5	
	layered fabric		
	bonded post-installed reinforcement	$l_{sd,pi} = k_{ls} \cdot l_{bd,pi} \geq 15\phi \cdot \alpha_{lb}$ where $l_{bd,pi}$ is calculated according to 11.4.8	

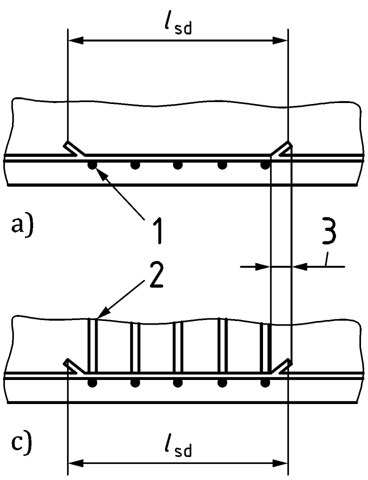
(7) The distance between lapped bars should be as small as possible, generally touch one another. In case the clear distance exceeds the smaller of 50 mm or 4ϕ, the design lap length shall be increased by the centre to centre distance of the lapped bars and adequate transverse reinforcement provided to resist the associated transverse forces developed.

(8) Where confinement or transverse reinforcement in the lap zone is used to reduce the design lap length according to 11.4.2(5) and/or 11.5.2(5), at least 5 bars fulfilling the requirement of Figure 11.12 a) or 11.12 c) should be distributed over the lap length, or alternatively, 3 bars over a length of $0{,}3l_{sd}$ at both ends of the lap according to Figure 11.12b) and 11.12d).

The minimum amount of confinement according to 11.4.2(5) should be dimensioned so that $c_{d,conf}$ according to Formula (11.4) is not less than 3ϕ.

(9) For compression laps, at least one transverse or confinement bar should be placed at each end of the lap within a maximum of 50 mm or 2ϕ of the ends of the lap where ϕ is the diameter of the smaller

lapped bar. These bars or links may be one of the 5 or 3 bars under (8) above or additional, see Figure 11.12b) or 11.12d).



a) transverse reinforcement, tension laps

b) transverse reinforcement, compression laps

c) confinement reinforcement, tension laps

d) confinement reinforcement, compression laps

Key

1 transverse reinforcement

2 confinement reinforcement

3 clear distance to lap end: ≤ max{2ϕ; 50 mm}

Figure 11.12 — Transverse and confinement reinforcement for laps

(10) When required by 11.5.2(1), transverse reinforcement should be provided and have a total area, ΣA_{st} (sum of all legs parallel to the layer of the lapped reinforcement) of not less than the area A_s of one lapped bar ($\Sigma A_{st} \geq 1{,}0\ A_s$). The transverse bars should be placed perpendicular to the direction of the lapped reinforcement.

If more than 50% of the reinforcement is lapped at one point and the distance between adjacent laps at a section is ≤ 10ϕ (see Figure 11.11) ΣA_{st} should be provided as confinement reinforcement formed by links or U bars and anchored into the body of the section.

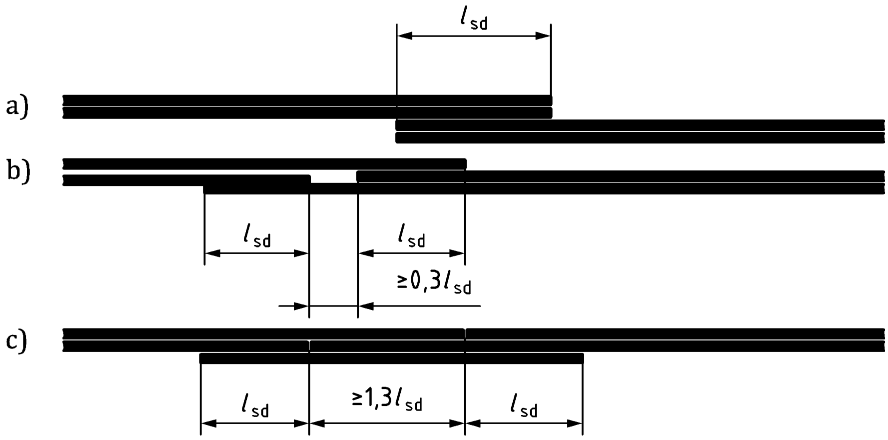
The transverse and confinement reinforcement ΣA_{st} should be positioned at the outer sections of the lap as shown in Figure 11.12.

11.5.3 Laps of bundles

(1) In bundles consisting of two bars, laps may be used without staggering of individual bar interruptions as shown in Figure 11.13a). Design lap lengths may be calculated on the basis of 11.4.2 and 11.5.2(2) and with the equivalent bar diameter ϕ_b according to 11.4.3(1).

For bundles which consist of three bars, laps without staggering of individual bar interruptions shall not be used.

(2) Design lap length l_{sd} of bundles with 2 or 3 bars may be calculated on the basis of the individual bar diameterϕ if laps are staggered with a gap between individual laps ≥ 0,3l_{sd} according to Figure 11.13 b) or with an additional bar as shown in Figure 11.13 c).



a) without staggering

b) with staggering

c) with an additional bar

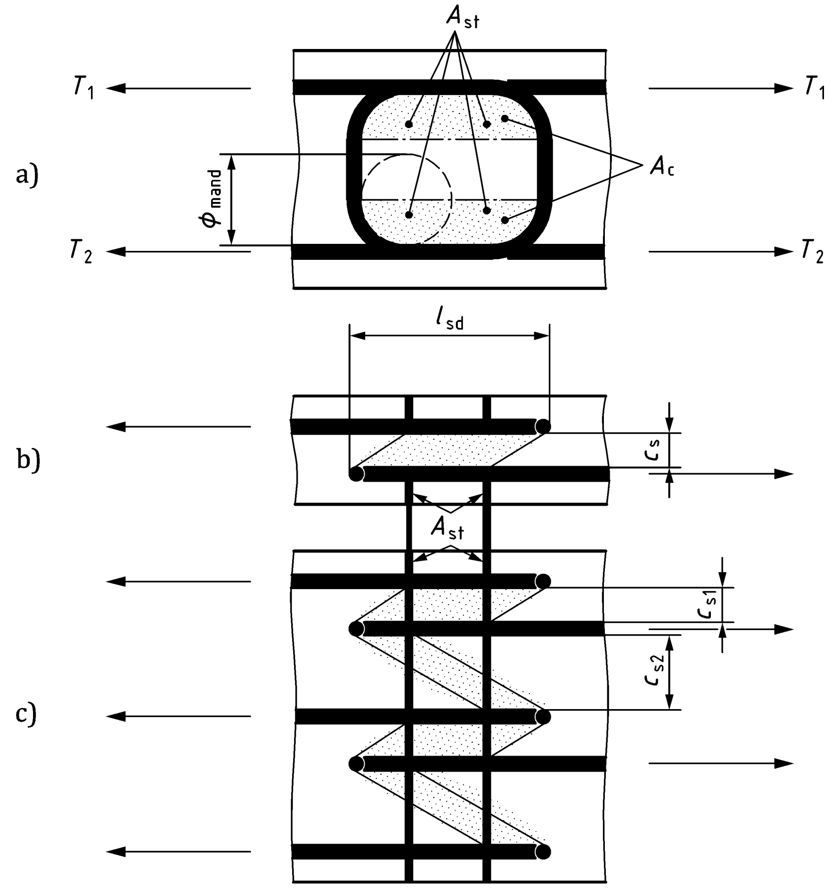
Figure 11.13 — Laps of bundles

(3) Bundles which consist of 4 bars shall not be lapped.

11.5.4 Laps using U-bar loops

(1) Transfer of tension between reinforcing steel bars may be achieved by overlapping U-bar loops (Figure 11.14). Overlapping U-bars may be single (Figure 11.14 b)) or multiple (Figure 11.14 c)) and both legs of each U-bar should be anchored outside the connection for the larger of the two design tension forces T_1 and T_2 (also when T_1 or $T_2 = 0$).

NOTE Design tension forces $T_1 = T_2$ when the lap transfers pure tension. T_1 will according to a sectional analysis differ from T_2 when the lap transfers combinations of normal forces and bending moments.



a) elevation view

b) plan view of single lap

c) plan view of multiple laps

Figure 11.14 — Laps using U-bar loops

(2) The resistance of a single lap splice shown in Figure 11.14 b) should be checked using as criterion the crushing strength of the concrete between the two loops. For $c_s \leq 0{,}5l_{sd}$ crushing of the concrete is prevented when the larger of the two design tension forces T_1 and T_2 (see Figure 11.14a) is smaller than $T_{Rd,c}$, which may be calculated as:

$$T_{Rd,c} = 0{,}2f_{cd} \cdot A_c \cdot \left(\frac{d_{dg}}{l_{sd}}\right)^{\frac{1}{3}} \cdot \left(\sqrt{k_{st} + \left(\frac{c_s}{l_{sd}}\right)^2} - \frac{c_s}{l_{sd}}\right) \tag{11.13}$$

where

A_c is the total effective concrete area within the curved parts of the overlapping U-bars (Figure 11.14a)):

$$A_c = (\phi_{mand} + \phi) \cdot [l_{sd} - 0{,}21(\phi_{mand} + \phi)] \tag{11.14}$$

l_{sd} is the lap length (Figure 11.14);

d_{dg} is a coefficient that takes into account the concrete type and its aggregate properties according to 8.2.1(4);

c_s is the clear spacing of U-bars;

k_{st} is the resistance factor of the confinement reinforcement, which may be taken equal to $k_{st} = 1$ when $\omega \geq 0{,}5$ and to $k_{st} = 4\omega(1-\omega)$ for lower values of

$$\omega = \frac{A_{st} \cdot f_{yd}}{0{,}85 \left(\frac{d_{dg}}{l_{sd}}\right)^{\frac{1}{3}} \cdot A_c \cdot f_{cd}}; \tag{11.15}$$

A_{st} is the total area of the fully anchored confinement reinforcement positioned within A_c.

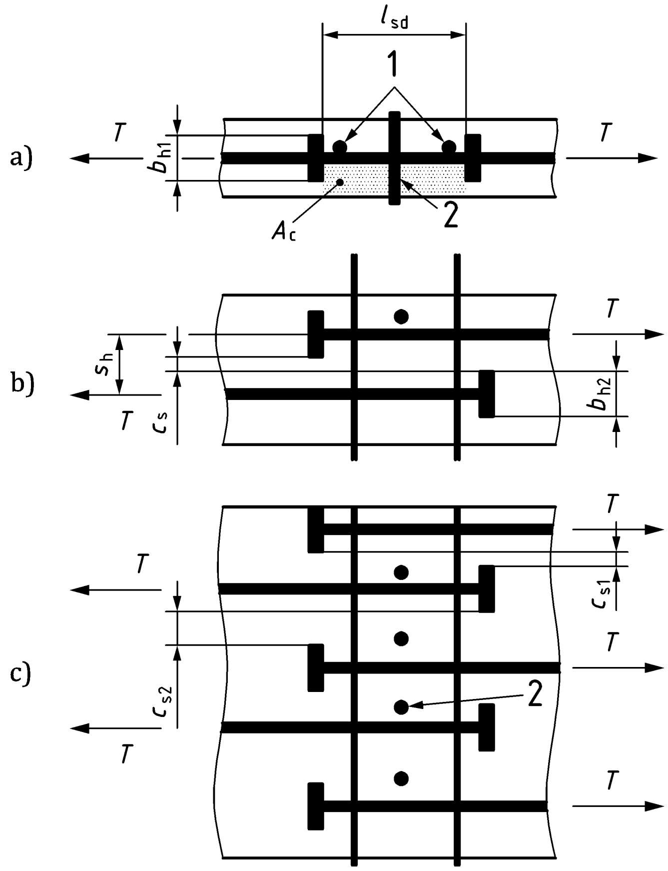
(3) In case of multiple U-bars as shown in Figure 11.14 c), the resistance of Formula (11.13) may be calculated using an average net spacing $c_s = 0{,}5(c_{s1} + c_{s2})$ (see Figure 11.14 c)) and multiplied by $(n_s - 1)$ where n_s is the total number of U-bars ($n_s = 5$ in the example of Figure 11.14 c)). Multiple overlaps may also be treated as an assembly of single overlaps where c_s is taken as the smaller of c_{s1} and c_{s2}.

(4) To avoid brittle behaviour, a minimum amount of confinement reinforcement should be placed in a double symmetric configuration within A_c (see e.g. Figure 11.14 a)):

$$A_{st} \geq 0{,}5\sqrt{f_{ck}} \cdot \frac{A_c}{f_{yk}} \tag{11.16}$$

11.5.5 Laps using headed bars

(1) Transfer of tension between reinforcing steel bars may be achieved by overlapping headed bars (Figure 11.15). Overlapping headed bars may be single (Figure 11.15 b)) or multiple (Figure 11.15 c)) and should be anchored for the design force outside the connection.



a) elevation view

b) plan view of single lap

c) plan view of multiple laps

Key

1 transverse reinforcement A_{st}

2 tie down reinforcement A_{std} symmetrically placed between heads

Figure 11.15 — Laps using headed bars

(2) The geometry of the heads of the headed bars should comply with 11.4.7(1).

(3) The maximum tensile force developed in each bar should be limited by 11.4.7(2).

(4) The resistance of a single headed bar lap shown in Figure 11.15 b) should be checked using as criterion the crushing strength $T_{Rd,c}$ of the concrete between the two heads. For $0 \leq c_s \leq 0{,}5 l_{sd}$, $T_{Rd,c}$ may be calculated as:

$$T_{Rd,c} = 0{,}6 f_{cd} \cdot A_c \cdot \left(\frac{d_{dg}}{l_{sd} - 2\phi}\right)^{\frac{1}{3}} \cdot \left(\sqrt{k_{st} + \left(\frac{c_s}{l_{sd} - 2\phi}\right)^2} - \frac{c_s}{l_{sd} - 2\phi}\right) \tag{11.17}$$

where

A_c is the effective concrete area within the heads of the overlapping bars (Figure 11.15 a)):

$$A_c = (l_{sd} - 2\phi) \cdot b_{h1} \tag{11.18}$$

b_{h1} is effective width of the head perpendicular to the plane of the lap (for circular head with diameter ϕ_h:

$$b_{\mathrm{h1}} = 0{,}5\phi_{\mathrm{h}} \cdot \sqrt{\pi} \tag{11.19}$$

c_s is the clear spacing of headed bars;

k_{st} is the resistance factor of the transverse reinforcement, which may be taken equal to $k_{st} = 1$ when $\omega \geq 0{,}5$ and to $k_{st} = 4\omega(1 - \omega)$ for lower values of

$$\omega = \frac{A_{\mathrm{st}} \cdot f_{\mathrm{yd}}}{1{,}3\left(\frac{d_{\mathrm{dg}}}{l_{\mathrm{sd}} - 2\phi}\right)^{\frac{1}{3}} \cdot A_{\mathrm{c}} \cdot f_{\mathrm{cd}}} \tag{11.20}$$

A_{st} is the total area of the fully anchored transverse reinforcement positioned within A_c.

(5) In case of multiple headed bars as shown in Figure 11.15c), the resistance of Formula (11.17) may be calculated using an average net spacing $c_s = 0{,}5(c_{s1} + c_{s2})$ (see Figure 11.15c)) and multiplied by $(n_s - 1)$ where n_s is the total number of headed bars ($n_s = 5$ in the example of Figure 11.15c)). Multiple overlaps may also be treated as an assembly of single overlaps where c_s is taken as the smaller of c_{s1} and c_{s2}.

(6) To avoid brittle behaviour, a minimum amount of transverse reinforcement should be placed:

$$A_{st} \geq \max\left\{0{,}75 \cdot A_{\mathrm{c}} \cdot \frac{\sqrt{f_{\mathrm{ck}}}}{f_{\mathrm{yk}}};\ \frac{\pi}{8}\phi^2\right\} \tag{11.21}$$

where ϕ is the maximum diameter of the lapped headed bars.

(7) To enhance ductility of laps designed for reinforcement yield, tie down reinforcement with total area A_{std} per single lap should be provided perpendicular to the plane of the headed bars within A_c (Figure 11.15a). The tie down reinforcement should be fully anchored outside A_c and placed symmetrically between the headed bars. The tie down reinforcement may be provided in the form of either double headed shear studs or links. For $l_{sd} \leq 200$ mm, A_{std} may be provided by a single double headed shear stud (Figure 11.15c). The minimum area of tie down reinforcement equals:

$$A_{\mathrm{std}} \geq 0{,}12\phi^2 \tag{11.22}$$

11.5.6 Mechanical couplers

(1) The clear distance (horizontal and vertical) between couplers and between couplers and adjacent bars should be not less than $D_{upper} + 5$ mm and the maximum diameter of the bars. Additional requirements related to installation should be accounted for.

(2) For cover requirements, couplers should be treated as single bars with the cover defined as the minimum clear distance between the outside surface of the coupler and the concrete surface.

11.5.7 Full penetration butt weld and fillet weld splices

(1) Splices with full penetration butt welds or fillet welds shall be detailed according to EN ISO 17660.

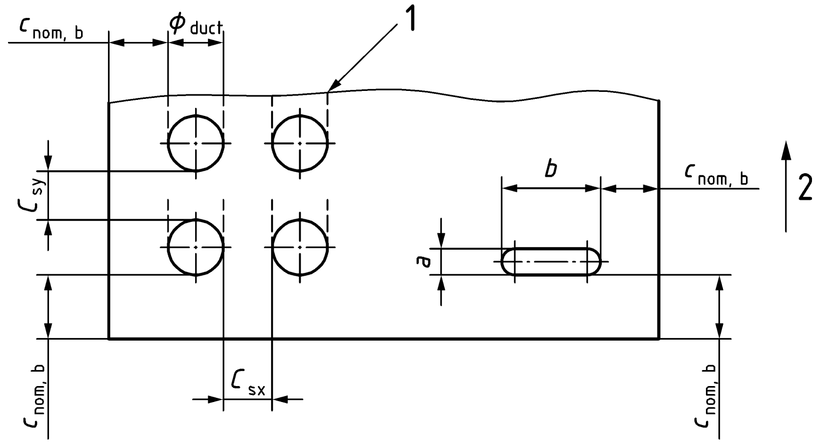
11.6 Post-tensioning tendons

11.6.1 General

(1) Additional provisions for external post-tensioning are given in K.12.3.

11.6.2 Minimum spacing of ducts

(1) The minimum spacing for placing and compacting of concrete and for safe transfer of deviation forces should be in accordance with Figure 11.16. Minimum vertical spacing may be reduced below ϕ_{duct} if adequate transverse reinforcement is provided to cope with the deviation forces from the tendon. Other arrangements may be used provided that satisfactory behaviour in service and at ultimate limit states may be demonstrated. Other requirements for durability are given in Clause 6 and for fire design in EN 1992-1-2.



Key

1 Assumed plane of tendon curvature

2 direction of casting

c_{sx} $\geq \max\{D_{upper} + 5\text{ mm}; \phi_{duct}; 50\text{ mm}\}$

c_{sy} $\geq \max\{D_{upper}; \phi_{duct}; 40\text{ mm}\}$

NOTE Spacing shown for round ducts applies also to rectangular ducts. Cover values of $c_{nom,b}$ are given in 6.5.2 (with $c_{min,b}$ in 6.5.2.3 and Δc_{dev} in 6.5.3).

Figure 11.16 — Minimum clear spacing for internal tendons for post-tensioning

(2) The minimum spacing between bundled tendons should be $s \geq 100$ mm.

(3) Outside the anchorage zone, up to two tendon ducts and up to four greased and sheathed strands may be bundled transversely to the plane of the tendon curvature or in case of straight tendons. Tendons with rectangular ducts shall not be bundled.

(4) The effect of possible deviation forces due to tendon curvature shall be considered according to 11.7.

11.6.3 Minimum radius of curvature and straight length of tendons adjacent to anchorages

(1) The minimum radius of curvature of tendons and the minimum straight length of tendons adjacent to the anchorage devices shall comply with the requirements in the technical documentation of the post-tensioning system depending on the type of duct (metal duct, polymer duct, polymer pipe, steel pipe, polymer sheathing). These values shall not be smaller than those given in (2) unless demonstrated by testing to the relevant standard for prestressing systems.

(2) The minimum radius of curvature of tendons to prevent damage of the unconfined concrete on the inside of the tendon curvature during prestressing and to avoid a reduction of the axial tensile strength of the tendon may be taken as:

$$R_{\min} = \frac{\sigma_{\mathrm{pd}}}{p_{\mathrm{Rd}}}\sqrt{A_{\mathrm{p}}} \qquad (11.23)$$

where

σ_{pd} is the tendon design stress (at the time of tensioning, considering partial factors for prestress, see Table 4.2 (NDP);

p_{Rd} is the maximum transverse bearing stress on the prestressing tendon.

NOTE p_{Rd} is given in Table 11.4 (NDP) unless the National Annex gives other values.

Table 11.4 (NDP) — Maximum transverse bearing stress on the prestressing tendon and additional requirements

Case	maximum transverse bearing stress on the prestressing tendon	additional requirements
Internal tendons with corrugated ducts	$p_{\mathrm{Rd}} = 0{,}75 f_{\mathrm{cd}} \leq 15$ MPa	–
Internal tendons inside a loop in U-shape with duct or pipe without relative movement between tendon and duct (i.e. the tendon fix point during stressing is located in the centre of the U and is not subject to fatigue loading)	$p_{\mathrm{Rd}} = 70$ MPa	see (3)
External tendons with smooth pipe with relative movement between tendon and duct	$p_{\mathrm{Rd}} = 30$ MPa	The bearing stress on the concrete inside the pipe shall be verified.

(3) For tendon loops in U-shape unless more restrictive rules are given in the technical documentation of the post-tensioning system, the following detailing rules should be observed:

— minimum cover transverse to the plane of the tendon loop $\geq 1{,}0\phi_{\mathrm{duct}}$;

— reinforcement transverse to the plane of the tendon loop should be provided for the deviation forces along the inside of the loop;

— half of the deviation forces in the plane of the loop transferred by reinforcement to the concrete on the outside of the loop. This reinforcement should be anchored at the centreline of the loop and may be combined with the bursting reinforcement in the form of U-shaped bars;

— the bearing stress on the confined concrete inside the tendon curvature should be checked according to 8.1.4.

11.6.4 Anchorages, couplers and deviators of post-tensioning tendons

(1) Anchorage devices and couplers used for post-tensioning tendons as well as the zone immediately around and in front of the tendon anchorage where the prestressing force is transmitted to the concrete (local anchorage zone) shall be checked for minimum spacing, minimum edge distance and local anchorage zone reinforcement to be in accordance with the relevant standards for prestressing systems.

(2) The zone where the prestressing force is dispersed over the full cross section of the member (general anchorage zone) should be designed using a strut-and-tie model according to 8.5, or another appropriate model.

(3) If post-tensioning tendons are anchored in a section away from the ends of a member (e.g. at a construction joint, a blister or cast into the member section), stresses should generally be compressive in the entire section in the direction of the anchored prestressing force under frequent load combination. The effect of anchoring or coupling several tendons at or close to the minimum spacing between anchorages or couplers in a single cross section on the strain distribution in the member should be assessed.

NOTE The assumption of linear strain distribution generally does not apply locally where several tendons are anchored or coupled at or close to the minimum spacing in a single cross section inside a member. It is considered good practice that a percentage of the tendon force is transferred with reinforcement (parallel to the tendon) to the member sections behind the anchorage or coupler. Compression forces acting in the member sections behind the anchorage or coupler reduce the required amount of reinforcement.

11.7 Deviation forces due to curved tensile and compressive chords

(1) In the case of curved and kinked tension or compression chords, the effects of the deviation forces shall be accounted for. The same provisions apply for curved post-tensioning tendons near the concrete surface.

(2) Deviation forces in equilibrium with transverse tensile forces as shown in Figure 11.17 should in general be resisted by means of additional transverse reinforcement.

(3) If no specific transverse reinforcement is provided to carry deviation forces, it shall be verified that the deviation forces due to the force in the longitudinal reinforcing steel or due to the force in longitudinal curved post-tensioning tendons can be resisted by the concrete in tension:

$$\frac{F_{\mathrm{td}}}{r \cdot c_{\mathrm{u}}} \leq \frac{0{,}125}{\gamma_{\mathrm{C}}} \sqrt{f_{\mathrm{ck}}} \tag{11.24}$$

where

$c_{\mathrm{u}} = \min\{c_{\mathrm{s}}; 2\sqrt{3}(c_y + 0{,}5\phi)\}$

c_{s} and c_{y} are defined in Figure 11.3c).

For post-tensioning tendons, ϕ shall be replaced by ϕ_{duct}.

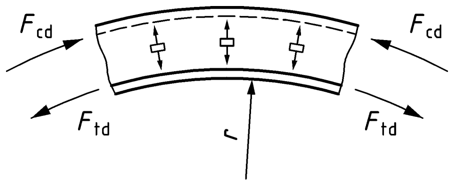


Figure 11.17 — Deviation forces and transverse tensile forces in curved members

(4) In case of laps of curved tensile bars in regions without transverse reinforcement, the interaction between bond and transverse tensile stresses due to deviation forces should be accounted for by means of:

$$\frac{\gamma_C \cdot 8 \cdot F_{\mathrm{td}}}{r \cdot c_u \cdot \sqrt{f_{\mathrm{ck}}}} + \frac{l_{\mathrm{sd}}}{l_{\mathrm{s}}} \leq 1 \tag{11.25}$$

where

l_{s} is the actual lap length.

Calculation of parameter c_{u} according to (3) should account for the presence of additional bars in the splice region potentially reducing net spacing c_{s} as indicated in Figure 11.10.

12 Detailing of members and particular rules

12.1 General

(1) The rules in Clause 12 are intended to detail concrete structures and concrete members such that the assumptions for the behaviour with respect to structural safety, serviceability, durability and robustness inherent in the rules of this Eurocode are met.

(2) Detailing of members shall be consistent with the design models adopted.

(3) Concrete members shall be detailed with due consideration to the constructability and concreting operations.

(4) Sufficient reinforcement shall be provided at all sections to resist the envelope of the acting internal forces taking into account effects such as shear on longitudinal reinforcement. The area of reinforcement in zones of tension shall not be taken less than $A_{s,min}$.

(5) For cable supported members the relevant rules of Annex K may be used.

(6) Members having less longitudinal reinforcement than $A_{s,min}$ given in Clause 12 shall be designed in accordance with Clause 14.

12.2 Minimum reinforcement rules

(1) In members designed as parts of reinforced concrete structures, minimum reinforcement $A_{s,min}$ shall be provided to:

a) ensure distributed cracking and to handle forces from restrained deformations where not considered explicitly in the design ((2) and (4));

b) ensure sufficient deformation capacity to contribute to structural robustness by allowing alternative load paths ((2) and (4));

c) avoid failures due to unpredicted cracking (3);

d) ensure applicability of design models in Clauses 8 and 9 and Annex G;

e) ensure constructability.

NOTE 1 Additional provisions for crack control at SLS are given in 9.2.2.

NOTE 2 The area of minimum reinforcement $A_{s,min}$ can include prestressed and ordinary reinforcement when bonded to the concrete.

(2) The area of minimum reinforcement $A_{s,min}$ shall provide nominal section strength which is at least equal to the effect causing cracking:

a) In members subjected to bending without or with axial force where the compressive axial force, $N_{Ed,min}$, is less than $0.5A_c \cdot f_{cd}$, minimum reinforcement shall be provided so that:

$$M_{R,min}\,(N_{Ed,min}) \geq M_{cr}(N_{Ed,min}) \tag{12.1}$$

where

$M_{R,min}(N_{Ed,min})$	is the moment resistance of the section with $A_{s,min}$ acting at a stress of f_{yk} and in presence of the axial force $N_{Ed,min}$;
$N_{Ed,min}$	is the axial force at ULS (persistent and transient design situation) providing the least compression in the member (or maximum tension in the member if tensile axial forces occur);

$M_{cr}(N_{Ed,min})$ is the cracking moment of the section in presence of $N_{Ed,min}$. The cracking moment may be calculated assuming a linear distribution of normal stresses over the cross section, where the maximum tension stress is taken as the concrete tensile strength f_{ctm}.The influence of the reinforcement may be neglected.

For sections prestressed with internal permanently unbonded tendons or with external tendons the contribution of the tendons to N_{Ed}, $M_{R,min}$ and M_{cr} should be calculated using the $\sigma_{p,mt}(x)$ (see Formula (7.33)).

b) In members with pure tension $A_{s,min}$ may be calculated as:

$$A_{s,min} = A_c \cdot f_{ctm}/f_{yk} \tag{12.2}$$

(3) Where $M_{Ed} < M_{cr}(N_{Ed})$, at ULS and the member is designed as statically determinate without the requirement for distributed cracking, a sudden collapse after cracking may also be avoided with a minimum reinforcement designed for following resistance:

$$M_{Rd,min}(N_{Ed}) = k_{dc} \cdot M_{Ed} \tag{12.3}$$

Where

$M_{Rd,min}(N_{Ed})$ is the design moment resistance of the section in the presence of the axial force N_{Ed};

N_{Ed} is the axial force at ULS acting simultaneous with the bending moment M_{Ed};

$M_{cr}(N_{Ed})$ is the cracking moment of the section in presence of N_{Ed}; and

k_{dc} is a coefficient which depends on the ductility class of the reinforcement:

— $k_{dc} = 1{,}3$ for ductility class A;

— $k_{dc} = 1{,}1$ for ductility class B;

— $k_{dc} = 1{,}0$ for ductility class C.

The calculated reinforcement area does not need to be greater than that given by Formula (12.1).

NOTE 3 Use of Formula (12.3) can result in wide cracks in SLS.

(4) In beams and slabs requiring shear or torsion reinforcement a minimum ratio of such reinforcement $\rho_{w,min}$ shall be placed. $\rho_{w,min}$ should be calculated as:

$$\rho_{w,min} = \frac{A_{sw,min}}{s \cdot b_w \cdot \sin\alpha} \geq 0{,}08 \frac{\sqrt{f_{ck}}}{f_{yk}} \tag{12.4}$$

where

$\rho_{w,min}$ is the minimum reinforcement ratio;

$A_{sw,min}$ is the area of minimum shear reinforcement within the spacing s;

s is the spacing of the shear reinforcement measured along the longitudinal axis of the member;

b_w is the width of the web of the member;

α is the angle between shear reinforcement and the longitudinal axis.

The value of $\rho_{w,min}$ given by Formula (12.4) may be reduced:

— by 10 % when ductility class B reinforcement;

— by 20 % when ductility class C reinforcement;

is used.

Formula (12.4) ensures that the shear model in Clause 8 is valid and that behaviour is reasonably ductile, where alternative shear models are used alternative values may be appropriate.

(5) For members where brittle failure due to tensile stresses is excluded such as members in compression and members with no structural function, (2) to (4) may be disregarded.

(6) Minimum reinforcement should generally be anchored and lapped according to Clause 11 for a design stress of $\sigma_{sd} = f_{yd}\, A_{s,min}/A_s$, where A_s is the actual reinforcement area provided. Reinforcement provided only to satisfy minimum steel requirements may, at simple supports, have its anchorage length reduced providing that it is $\geq 10\phi$ past the inner edge of the support.

12.3 Beams

12.3.1 General

(1) Reinforcement in beams, longitudinal and transverse, should be detailed in accordance with the requirements of Table 12.1 (NDP).

NOTE The values in Table 12.1 (NDP) apply unless the National Annex gives other values.

Table 12.1 (NDP) — Detailing requirements for reinforcement in beams

	Description	Symbol	Requirement
1	Minimum longitudinal reinforcement, in those parts of the section where tension may occur	$A_{s,min}$	12.2(2), see also 12.2(3), 12.2(6)
2	Minimum shear and transverse torsional reinforcement, when required. Minimum torsion reinforcement should be provided to the full perimeter including features not counted part of the thin walled section	$\rho_{w,min}$	12.2(4)
3	Minimum bottom reinforcement at inner supports taking account of unforeseen effects leading to positive moments at the support, e.g. unforeseen settlement, or load reversal due to explosion		$0{,}25\ A_{s,req\ span}$
4	Minimum bottom reinforcement for end supports		$0{,}25A_{s,req\ span}$
5	Maximum longitudinal spacing of shear assemblies/stirrups[a]	$s_{l,max}$	$0{,}75d\ (1 + \cot\alpha)$
6	Maximum longitudinal spacing of bent-up bars[a]	$s_{bu,max}$	$0{,}6d\ (1 + \cot\alpha)$
7	Maximum transverse spacing of shear legs[a]	$s_{tr,max}$	$0{,}75d \leq 600$ mm
8	Minimum ratio of shear reinforcement in the form of stirrups with respect to the required reinforcement ratio (taking account of unforeseen effects e.g. compatibility torsion)	$\rho_{w,stir}$	$\geq 0{,}5\rho_{w,req}$
9	Minimum ratio of torsion reinforcement in the form of closed stirrups with respect to the required reinforcement ratio	$\rho_{w,stir}$	$\geq 0{,}2\rho_{w,req}$
10	Maximum spacing for torsion assemblies/stirrups (u defined in 8.3.2(2))	$s_{stir,max}$	$u/8 \leq \min\{b; h\}$
11	Minimum area and spacing of longitudinal surface reinforcement in beams with downstand ≥ 600 mm to avoid coarse cracks in SLS	$A_{s,web}$ $s_{l,surf,max}$	9.2.2(4) 300 mm
12	Minimum transverse reinforcement in flanges (those part of flanges where tension in the transverse direction may occur)	$A_{st,min}$	12.2(2) see 8.2.5, Figure 8.13

[a] These spacings are consistent with the shear model in 8.2.3. Where alternative models are used alternative spacings may be required.

12.3.2 Longitudinal reinforcement

(1) For members with shear reinforcement, the additional tensile force due to shear, N_{vd}, according to 8.2.3(8) should be considered at the ULS.

Alternatively, for members with constant depth, the moment curve should be shifted at a distance a_l according to:

$$a_l = z\ (\cot\theta - \cot\alpha)/2 \tag{12.5}$$

as indicated in Figure 12.1.

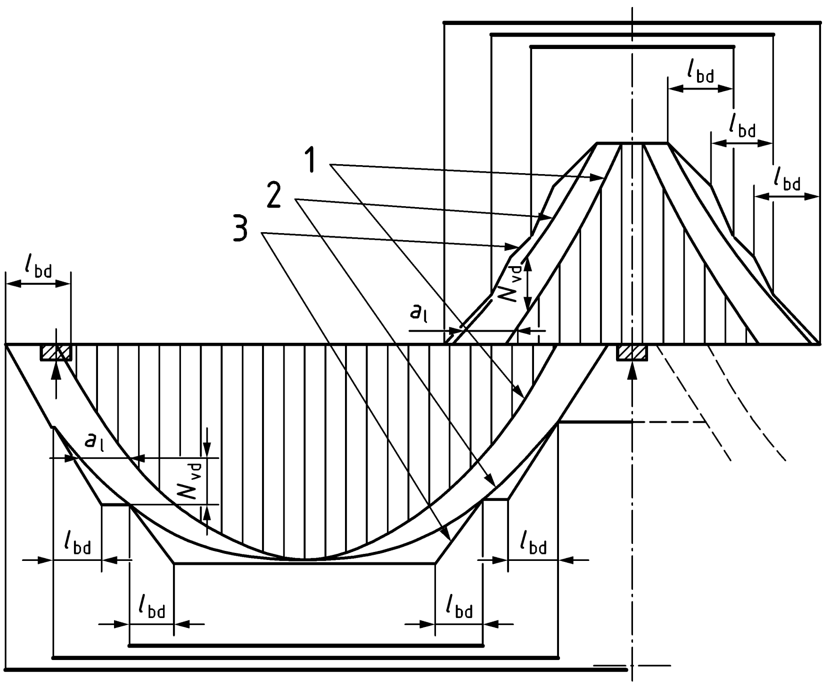
For members without shear reinforcement, it may be assumed:

$$a_l = d \tag{12.6}$$

(2) The resistance of bars within their anchorage lengths may be taken into account, assuming a linear variation of force, see Figure 12.1.

(3) The tensile reinforcement required in flanged cross-sections may be spread over the effective width of the flange or part of it may be concentrated over the web. The tensile reinforcement located outside the web should be extended by a length equal to its distance from the web times $\cot\theta_f$ (see 8.2.5).

(4) The anchorage length of a bent-up bar which contributes to the resistance to shear should be not less than $1{,}3l_{bd}$ in the tension zone and $0{,}7l_{bd}$ in the compression zone. It is measured from the point of intersection of the axes of the bent-up bar and the longitudinal reinforcement.



Key

1 envelope of $M_{Ed}/z + 0{,}5N_{Ed}$

2 acting tensile force F_s

3 resisting tensile force F_{Rs}

Figure 12.1 — Illustration of the curtailment of longitudinal reinforcement in members with constant depth, taking into account the effect of shear and the resistance of reinforcement within anchorage lengths

(5) Bottom reinforcement at intermediate supports should, as a minimum, extend by 10ϕ into the support if there is no tensile force in the reinforcement. In case there could be tension due to settlement of the support, accidental actions or due to other considerations, the reinforcement should be detailed for adequate capacity and continuity.

(6) Any compression longitudinal reinforcement which is included in the resistance calculation should be confined by transverse reinforcement with spacing not greater than the limits given in Table 12.3 (NDP) and 12.6(3).

12.3.3 Shear and torsion reinforcement

(1) The shear reinforcement may consist of a combination of:

— stirrups/links enclosing the longitudinal tension reinforcement and the compression zone according to Figure 12.2a) and b);

— bent-up bars;

— cages, ladders, shear reinforcement assemblies, etc. which are cast in without enclosing the longitudinal reinforcement but are properly anchored in the compression and tension zones according to Figure 12.2c) and d);

— continuous stirrups (spirals);

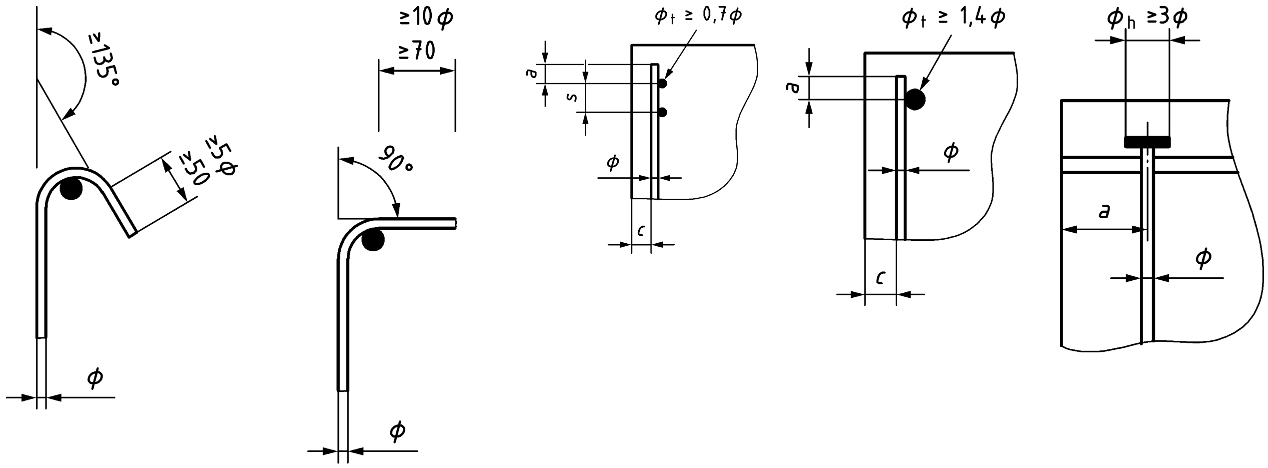
— headed bars as in Figure 12.2e).

(2) When, based on shear design, no shear reinforcement is required, minimum shear reinforcement should nevertheless be provided according to 12.2(4). Minimum shear reinforcement may be omitted in members of minor importance (e.g. lintels with span ≤ 2,0 m) which do not contribute to the overall resistance and stability of the structure.

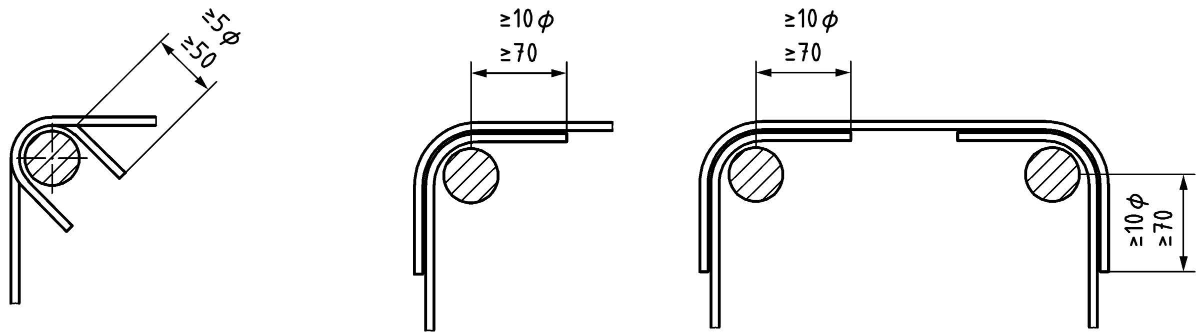
(3) The shear reinforcement shall enclose the tension reinforcement or shall be effectively anchored in the tension zone at one end and in the compression zone at the other. Links and shear reinforcement should normally be anchored by means of bends, hooks, heads or welded transverse reinforcement, see Figure 12.2. A longitudinal bar of minimum diameter equal to not less than the diameter of the stirrup or link should be provided at each corner of stirrup/link and inside end hooks.

(4) Closing of stirrups according to Figure 12.2f) g) and h) may be used in the tension and compression zone of a section. The anchorage of single-leg shear links or of open stirrups according to Figure 12.2b) to d) should be placed in the compression zone of the member. Welding should be carried out in accordance with 5.2.3 and have a weld capacity in accordance with C.4.1(3).

(5) Headed bars according to Figure 12.2e) may be used in the tension and compression zone of a section and should be designed according to 11.4.7 without requiring reinforcement bar length for anchorage.



a) with standard hook | b) with 90°-bend | c) with 2 welded transverse bars | d) with 1 welded transverse bar | e) with head



f) stirrup with 135°-hooks | g) stirrup with 90°-bends | h) closed with 90°-bended U-stirrups

Key

$s \geq \max\{2\phi; 20\text{ mm}\}$ and ≤ 50 mm

$a \geq 10$ mm

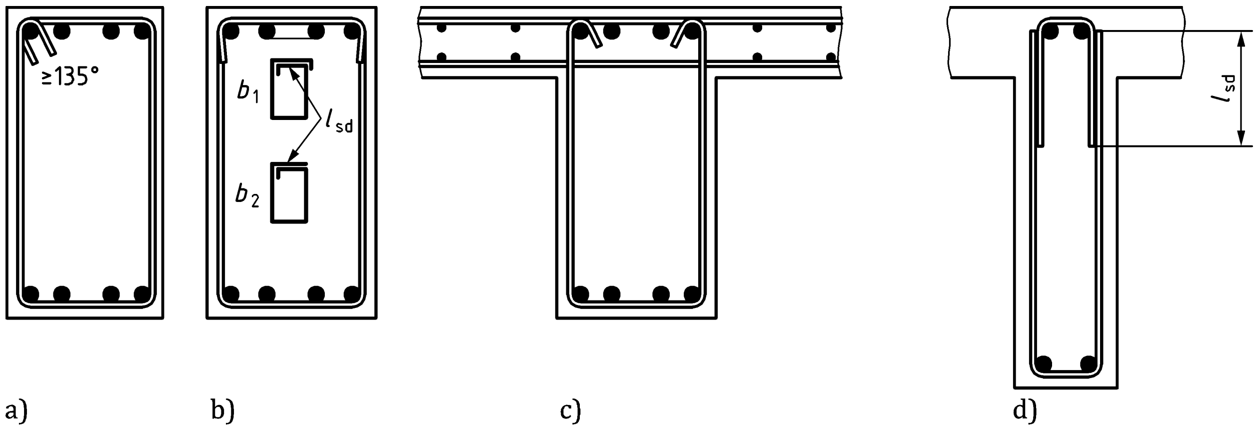
$c \geq \max\{3\phi; 50\text{ mm}\}$

Figure 12.2 — Anchorage of links a)–e) and closing of stirrups f)–h)

(6) Laps on legs of stirrups in shear reinforcement may be used provided they ensure yielding of the stirrup (see Figure 12.3 d)). Laps may be designed according to 11.5 for $\sigma_{sd} = 1{,}2f_{yd}$.

(7) When static equilibrium assumed in the analysis depends on the torsional resistance of elements of a structure, the torsion reinforcement shall comply with the rules given in (8) and Figure 12.3 and shall enclose the whole section. When torsion arises only from compatibility and the structure is not dependent on torsional resistance for its equilibrium, minimum torsional reinforcement shall be provided according to Table 12.1 (NDP).

(8) Torsion links shall be closed and anchored by means of laps or hooked ends (see Figure 12.3), and should be at an angle that does not deviate more than 10° from the ideal angle of 90° to the axis of the structural element. In case of different inclinations, torsion reinforcement should be designed using appropriate stress fields.



a) closed stirrup with hooks bent ≥ 135°

b) closed stirrup with 90°-bends

c) open U-stirrups closed by transversal reinforcement in the flange

d) U-stirrups closed by laps in the height of flanges with l_{sd} to be designed for $1{,}2f_{yd}$

NOTE The alternative b_2) has a full lap length l_{sd} along the shortest side.

Figure 12.3 — Examples of shapes for torsion links

(9) The longitudinal bars should be arranged so that there is at least one bar at each corner, the others being distributed uniformly around the inner periphery of the links, with a spacing not greater than permitted in Table 12.1(NDP).

12.3.4 Suspension reinforcement for indirect support

(1) Where applied loads are introduced in a manner that causes local tensile stresses in the supporting member in accordance with 8.2.1(9), the additional suspension reinforcement should consist of stirrups or links surrounding the principal reinforcement of the supporting/supported member, whichever is at a lower level. The stirrups/links may be located inside the volume of concrete common to the two members or distributed also outside that volume, provided a consistent strut-and-tie model is applied. Stress field models which utilise distributed horizontal reinforcement across the whole depth of the supporting/supported member may be adopted to reduce or substitute the suspension reinforcement.

12.4 Slabs

12.4.1 General

(1) Reinforcement in slabs should be detailed in accordance with the requirements of Table 12.2 (NDP).

NOTE The values in Table 12.2 (NDP) apply unless the National Annex gives other values.

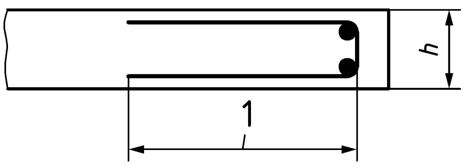
Table 12.2 (NDP) — Detailing requirements for reinforcement in slabs

Description		Symbol	Requirement
1	Minimum longitudinal reinforcement in those parts of the cross-section where tension may occur	$A_{s,min}$	12.2(2) see also 12.2(3), (6)
2	Minimum shear reinforcement, when required	$\rho_{w,min}$	12.2(4) 12.4.2(3)
3	Minimum secondary reinforcement[a]		$0{,}2A_{s,req,span}$[c]
4	Minimum longitudinal bottom reinforcement at inner supports, taking account of unforeseen effects at supports		$0{,}25A_{s,req,span}$[c]
5	Minimum longitudinal bottom reinforcement at end supports		$0{,}25A_{s,req,span}$[c]
6	Minimum top reinforcement at end supports in buildings, without bearings where unintentional restraint may occur. The reinforcement should extend 0,2 length of span from the end support.		$0{,}25A_{s,req,span}$[c] (but $\geq A_{s,min}$ according to 12.2(2))
7	Maximum spacing of bars for concrete in tension	$s_{slab,max}$	$3h \leq 400$ mm
8	Maximum longitudinal spacing of shear assemblies/stirrups[b]	$s_{l,max}$	$0{,}75d \cdot (1 + \cot\alpha)$
9	Maximum longitudinal spacing of bent-up bars[b]	$s_{bu,max}$	d
10	Maximum transverse spacing of shear legs[b]	$s_{tr,max}$	$1{,}5d$
11	Along a free (unsupported) edge, a slab should normally contain longitudinal and transverse reinforcement, generally arranged as shown in Figure 12.4 in order to accommodate torsional moments near the edge.		

[a] To ensure a minimum ability to locally redistribute sectional forces transverse to the span direction, secondary reinforcement is to be placed in areas of slabs which can be considered to behave as one-way slabs.

[b] These spacing are consistent with the shear model in 8.2.3. Where alternative models are used alternative spacings may be required.

[c] $A_{s,req\ span}$ is the required reinforcement for positive bending moments at the span.



Key

1 $\geq \max\{2h; l_{bd}\}$

Figure 12.4 — Edge reinforcement for a slab

(2) Any compression longitudinal reinforcement which is included in the resistance calculation should be confined by transverse reinforcement in accordance with 12.6(3).

(3) Where lifting of a slab-corner is restrained suitable reinforcement shall be provided.

(4) In case of slabs with depth discontinuities on the tension side, the flexural reinforcement should be detailed as shown in Figure 12.5, or alternatively, by using 8.5.

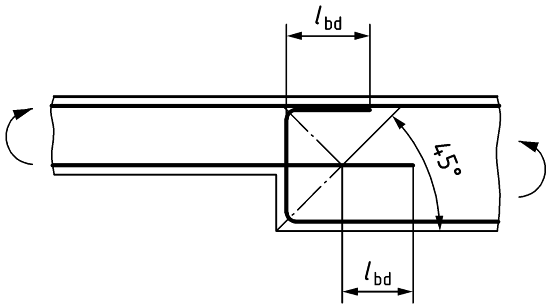


Figure 12.5 — Example of reinforcement detailing in case of slabs with depth discontinuities on the tension side and without shear reinforcement

12.4.2 Shear reinforcement

(1) Shear reinforcement according to 12.3.3(1) may be used.

(2) A slab in which shear reinforcement is provided should have a depth h not less than 200 mm with stirrups, links or headed bars, or 160 mm with bent up bars.

(3) The minimum shear reinforcement according to 12.2(4) may be omitted in solid, ribbed or hollow core slabs or slabs with voids where transverse redistribution of loads is possible.

(4) The shear reinforcement for a one-way slab should be detailed as for a beam, see 12.3.3.

For other slabs, the shear reinforcement may be detailed to enclose all except the outermost layers of reinforcement as shown in Figure 12.6.

(5) In slabs, the shear reinforcement may consist entirely of bent up bars or of shear reinforcement assemblies, where $\tau_{Ed} \leq 0{,}08\, f_{cd}$ for vertical and $\tau_{Ed} \leq 0{,}16\, f_{cd}$ for 45°-alignment (of bent up bars or shear reinforcement assemblies). Intermediate values may be interpolated.

(6) The diameter of shear reinforcement assemblies ϕ_w shall not exceed a maximum value according to 12.5.1(3).

12.5 Slab-column connections and column bases

12.5.1 Punching shear reinforcement

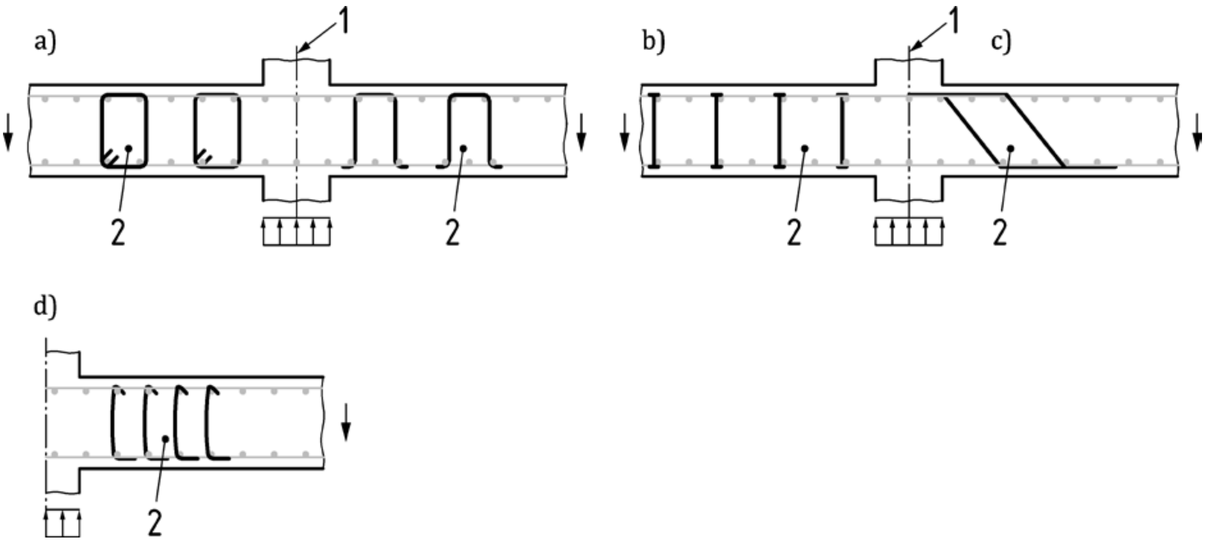
(1) If punching shear reinforcement is required (see 8.4), the following types of shear reinforcement may be placed to increase the punching shear capacity:

— stirrups (Figure 12.6 a)) with anchorage according Figure 12.2 a) or b);

— double-headed studs (Figure 12.6 b)) with anchorage according to Figure 12.2 e);

— bent-up bars (Figure 12.6 c));

— single-leg shear links (Figure 12.6 d) with anchorage according Figure 12.2 a) or b).



a) stirrups

b) double-headed studs and headed bars

c) bent-up bars

d) single-leg shear links

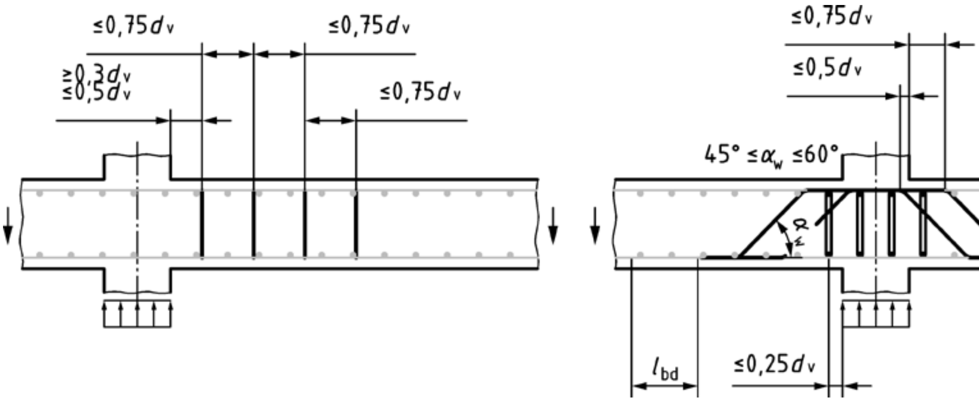
Key

1 center line of support

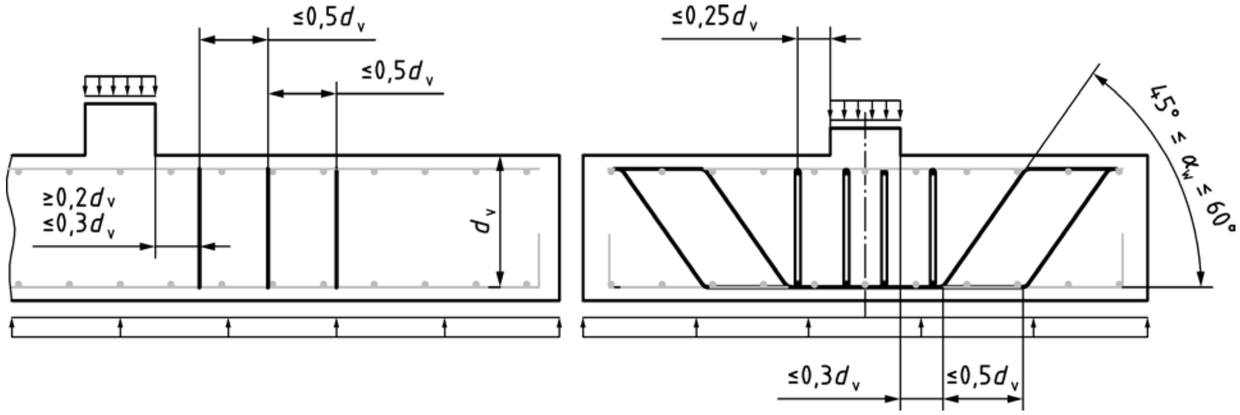
Figure 12.6 — Examples of shear and punching shear reinforcement

(2) Where punching shear reinforcement is required it should be placed between the loaded area/column and $0{,}5d_{v,out}$ inside the control perimeter at which shear reinforcement is no longer required (see Figure 8.24). If stirrups, links or double-headed studs are provided, the shear reinforcement should be installed along at least two perimeters. For bent-up bars anchored as in Figure 12.7 one perimeter may be considered sufficient. If only one perimeter of bent-up bars is provided, their slope may be reduced to 30°. The spacing of the shear reinforcement in radial and tangential directions should satisfy the provisions in Figure 12.7.

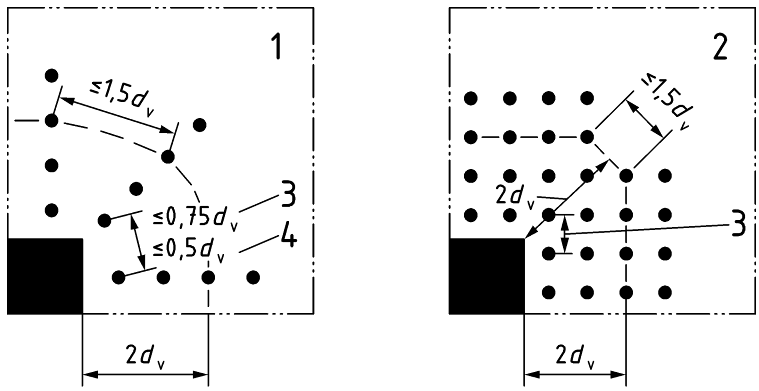
The tangential spacing of shear reinforcement should be limited based on the distance to the column edge (see Figure 12.7 c). For shear reinforcement located at a distance ≤ $2d_v$ from the column edge, the tangential spacing should not exceed $1{,}5d_v$ and it should not exceed $0{,}75d_v$ and $0{,}5d_v$ for flat slabs and column bases, respectively, in the first perimeter. The tangential spacing of shear reinforcement should also meet the requirements of Figure 8.24.



a) Radial spacing rules for flat slabs



b) Radial spacing rules for column bases



c) Tangential spacing rules

Key

1 radial layout

2 orthogonal layout

3 limit spacing on first reinforcement perimeter to ≤0,75d_v for flat slabs, and to ≤0,5d_v for column bases and ground slabs

Figure 12.7 — Spacing of punching shear reinforcement

NOTE Figure 12.7 a) and b) show the position of vertical leg of punching shear reinforcement simplified without anchorage elements.

(3) The maximum effective area of one leg of shear reinforcement should be limited to that of a bar of diameter $\phi_{w,max}$ of:

— for single leg links and open stirrups; $$\phi_{w,max} = 10\sqrt{d/200} \tag{12.7}$$

— for closed stirrups or bars with similar anchorage; $$\phi_{w,max} = 11\sqrt{d/200} \tag{12.8}$$

— for bent up bars and headed bars; $$\phi_{w,max} = 16\sqrt{d/200} \tag{12.9}$$

— where d and $\phi_{w,max}$ are in mm.

(4) Bent-up bars passing through the loaded area or being at a distance not exceeding 0,25d from this area may be used as punching shear reinforcement (see Figure 12.7 a)). For single bent-up bars, the horizontal projection of the bars has to be used for the value of s_r to calculate ρ_w.

12.5.2 Integrity reinforcement against progressive collapse of flat slabs

(1) For buildings in CC2 (refer EN 1990) and higher, an integrity reinforcement of at least two bars in each orthogonal direction should be provided at all columns without punching shear reinforcement or with punching shear reinforcement not fulfilling the requirement of (2). Reinforcing bars provided for other reasons may be accounted for as integrity reinforcement if it fulfils its requirements.

These bars should be:

— of ductility class B or C;

— anchored in the column or pass through it and

— placed on the compression side of the slab within the vertical column reinforcement.

For robustness reasons, they should be designed for a resistance:

$$V_{Rd,int} = \sum A_{s,int} \cdot f_{yd} \cdot k_{int} \geq V_{Ed} \tag{12.10}$$

where

V_{Ed} is the design value of the acting shear force for the accidental design situation;

$A_{s,int}$ is the sum of the cross-sections of all reinforcement bars crossing a column edge (the same bar may be counted twice if it passes through the column and is fully anchored on both sides outside the column edges);

f_{yd} is the yield strength of the integrity reinforcement for the accidental design situation;

k_{int} is a coefficient equal to:

— 0,37 for bars of ductility class B;

— 0,49 for bars of ductility class C.

(2) Integrity reinforcement according to (1) should also be provided in slabs with shear reinforcement if:

$$V_{Rd,w,int} = \rho_w \cdot f_{ywd} \cdot b_{0,5} \cdot d_v < V_{Ed} \tag{12.11}$$

where the ratio ρ_w is defined in 8.4.4(1) for vertical and in 8.4.4(2) for inclined shear reinforcement.

In this case, the integrity reinforcement should be designed according to Formula (12.11) where V_{Ed} may be replaced by $V_{Ed} - V_{Rd,w,int}$.

(3) The hogging reinforcement on the tension side of the slab can contribute in preventing progressive collapse. For slabs without shear reinforcement, the resistance of the integrity reinforcement $V_{Rd,int}$ may be increased by the term:

$$V_{\mathrm{Rd,hog}} = n_{\mathrm{hog}} \frac{\sqrt{f_{\mathrm{ck}}}}{\gamma_{\mathrm{C}}} \phi \cdot b_{\mathrm{ef,hog}} \tag{12.12}$$

Where

n_{hog} is number of bars crossing the control perimeter $b_{0,5}$ and fully anchored at a distance $4d$ from the control perimeter and within the column if considered only once;

γ_C is the partial factor for concrete for accidental design situation;

$b_{ef,hog}$ $=\min\{s\text{-}\phi;\ 6\phi;4c\}$

ϕ is the diameter of the hogging reinforcement considered;

s is the spacing of the hogging reinforcement; and

c is the cover of the hogging reinforcement.

12.6 Columns

(1) Longitudinal and hoop reinforcement of columns should be detailed in accordance with the requirements of Table 12.3 (NDP).

NOTE The values in Table 12.3 (NDP) apply unless the National Annex gives other values.

(2) Every longitudinal bar or bundle of bars placed at a corner shall be restrained by transverse reinforcement.

(3) No bar within a compression zone with a compressive strain exceeding 2 ‰ should be further than 150 mm from a restrained bar. Where additional transverse reinforcement to that required by Table 12.3 (NDP) is required only to restrain a longitudinal bar the spacing may be twice the maximum spacing according to Table 12.3 (NDP).

(4) Where longitudinal bars are bent by more than 5° ($\tan\theta \approx 1/12$), with respect to the axis of the column, the transverse splitting forces produced should be resisted by transverse reinforcement in accordance with a strut-and-tie model or stress field model.

Table 12.3 (NDP) — Detailing requirements for reinforcement in columns

	Description	Symbol	Requirement
1	Minimum amount of longitudinal reinforcement for robustness and to avoid risk of compressive yielding of reinforcement due to creep and shrinkage in SLS When all longitudinal reinforcement is prestressed the $0{,}1N_{Ed}/f_{yd}$ limit may be ignored.		$\max\left\{0{,}1\frac{N_{\mathrm{Ed}}}{f_{\mathrm{yd}}};\ 0{,}002A_{\mathrm{c}}\right\}$
2	Minimum number of longitudinal bars[a]: — polygonal cross-section — circular cross-section		1 at each corner with a spacing ≤ 400 mm 6 evenly distributed with a spacing ≤ 400 mm

	Description	Symbol	Requirement
3	Maximum longitudinal spacing of transverse reinforcement (stirrups/hoops) for columns with dimensions h and b:		
	— intermediate region between the two end regions[b]	$s_{max,col}$	$20\phi_{l,max}$[c] $\leq \min\{h; b; 300\text{ mm}\}$
	— intermediate region between the two end regions, when longitudinal bars are not accounted for column resistance	$s_{max,col}$	$\min\{h; b; 400\text{ mm}\}$
	— end regions, over a length equal to the larger dimension of the column. For concrete with $f_{ck} > 50$ MPa the transverse reinforcement shall provide a minimum confinement of $k \cdot f_{cd}$ in accordance with 8.1.4, Formulae (8.11) to (8.14)[d]		$0{,}6s_{max,col}$
	— at lap area where $\phi_l \geq 14$ mm		$0{,}6s_{max,col}$
4	Minimum bar diameter for transverse reinforcement (bars in stirrups, wires in welded mesh)		$\geq 0{,}25\phi_{l,max}$[a]

[a] For constructability, the diameter of longitudinal bars $\phi_{l,max}$ should be at least 12 mm.

[b] Where all bars are prestressed a spacing of $\min\{h; b; 300\text{ mm}\}$ may be used.

[c] $\phi_{l,max}$ – maximum diameter of longitudinal bars.

[d] This requirement is to provide a minimum level of ductility to higher strength concrete columns. $k = 0{,}02$ unless the National Annex gives a different value.

12.7 Walls and deep beams

(1) For walls subjected predominantly to out-of-plane bending, the rules for slabs should be applied (see 12.4).

(2) Vertical, horizontal and orthogonal-to-the-surface reinforcement in walls, should be detailed in accordance with the requirements of Table 12.4 (NDP).

NOTE The values in Table 12.4 (NDP) apply unless the National Annex gives other values.

Table 12.4 (NDP) — Detailing requirements for reinforcement in walls and deep beams

	Description	Symbol	Requirement
1	Minimum amount of vertical reinforcement (each surface):	$A_{s,min,v}$	
	— where the member carries in-plane normal and shear stresses and designed/verified by use of 8.5 or Annex G		$0{,}25A_c \cdot \frac{f_{ctm}}{f_{yk}}$
	— where the member is only loaded by vertical in-plane compression and out of plane bending		$0{,}001A_c$
2	Minimum amount of horizontal reinforcement (each face):	$A_{s,min,h}$	
	— where the member carries in-plane normal and shear stresses and designed/verified by use of 8.5 or Annex G		$0{,}25A_c \cdot \frac{f_{ctm}}{f_{yk}}$
	— where the member is only loaded by vertical in-plane compression and out of plane bending		$0{,}25A_{s,v}$

Description		Symbol	Requirement
3	Maximum spacing of vertical reinforcement		min{3h[a]; 400 mm}
4	Maximum spacing of horizontal reinforcement		400 mm
5	Maximum spacing of orthogonal-to-the-surface reinforcement where $A_{s,v}$ exceeds 0,02A_c and is utilised in compression (end region is taken as ≥ 4h[a])		see 12.7(3)
[a] h – thickness of wall			

(3) Where the vertical reinforcement is utilized in compression at ULS and is placed outside the horizontal reinforcement a minimum number of stirrups with 4 legs per m^2 of wall area, perpendicular to the surface, should be provided unless welded wire mesh or bars of diameter $\phi \leq 16$ mm are used with concrete cover larger than 2ϕ.

12.8 Foundations

(1) Foundations shall be detailed with due regard to possible deviations in geometry and position of piles, supporting bedrock or soil.

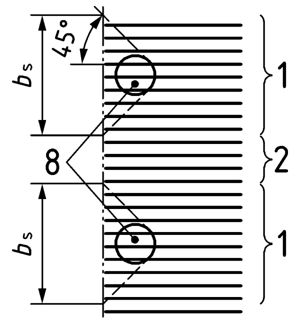
(2) The main tensile reinforcement in pile caps may be designed using strut-and-tie models according to 8.5 or it may also be designed for bending and shear by considering the foundation as a slab if the main reinforcement near to piles is:

— designed according to 8.2.1(11) or 8.2.3(8); and

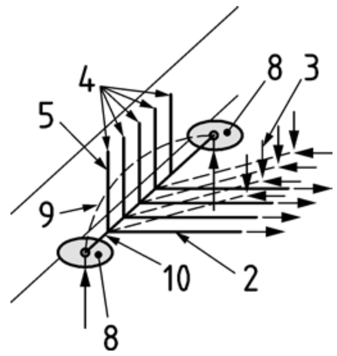
— placed within the width b_s defined in Figure 12.8 a).

In the area confined by the pile reaction according to Figure 12.8 a), the anchorage length of main reinforcement may be reduced according to 11.4.2(5).

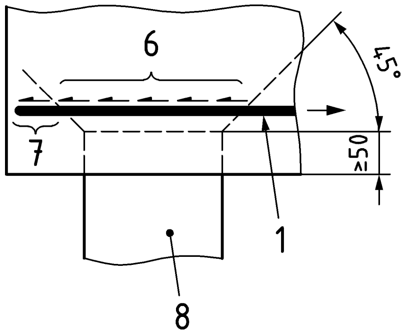
The main reinforcement perpendicular to free edges outside the widths b_s may also be activated if the vertical component of the corresponding struts can be carried to the piles by the system according to Figure 12.8 b).



a) Definition of width b_s



b) load carrying system with bent-up of main reinforcement outside widths b_s as suspension reinforcement



c) definition of anchorage zones with and without confinement due to pile compression

Key

1 main reinforcement in the width b_s and anchored behind the pile

2 main reinforcement outside the width b_s

3 struts carrying shear

4 bent-up part of main reinforcement acting as suspension reinforcement

5 anchorage of the suspension reinforcement

6 anchorage zone of main reinforcement with confinement due to pile compression

7 anchorage zone without confinement due to pile compression

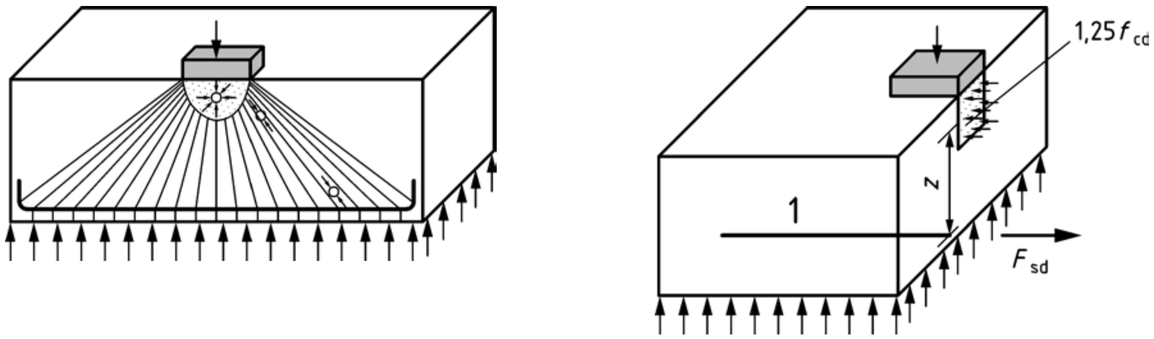
8 pile

9 arched compression strut

10 tie between piles

Figure 12.8 — Anchorage of main reinforcement in pile caps

(3) The main reinforcement in footings and pile caps without shear reinforcement should be designed accounting for the concentrated compression zone and the resulting reduced lever arm of the internal forces under the column according to Figure 12.9a) and b). The increase of concrete strength in vertical direction under the column due to confinement according to 8.5 may be accounted for. Without detailed verification, a concrete strength in horizontal direction under the column up to $1{,}25 f_{cd}$ may be assumed.



a) compression field carrying shear

b) concentration of compression zone under the column and resulting reduced level arm z

Key

1 bottom reinforcement

Figure 12.9 — Concentrated compression zones in pile caps and footings

(4) Top surface and sides of pile caps and footings may be designed without surface reinforcement if there is no tension developing in these areas.

12.9 Tying systems for robustness of buildings

12.9.1 General

(1) Structures require an adequate level of robustness appropriate to the consequences of failure. Structures with normal consequences of failure should have a suitable tying system, to prevent progressive collapse by providing alternative load paths after local damage.

(2) In the absence of detailed analysis, the following ties should be provided as applicable:

a) peripheral ties;

b) internal ties;

c) horizontal column and wall ties;

d) vertical column and wall ties.

(3) Where a building is divided by expansion joints into structurally independent sections, each section should have an independent tying system.

(4) In the design of the ties the reinforcement may be assumed to be acting at its characteristic yield strength and should be capable of carrying tensile forces defined in the following subclauses. Reinforcement of ductility classes B or C should be used. Bonded prestressing steel may also be considered.

(5) Reinforcement provided for other purposes in columns, walls, beams and floors may be regarded as providing part of or the whole of these ties.

12.9.2 Dimensioning of ties

12.9.2.1 Peripheral ties

(1) At each floor and roof level of a structure an effectively continuous peripheral tie within 1,2 m from the edge should be provided according to Table 12.5 (NDP). The tie may include reinforcement used as part of the internal tie.

12.9.2.2 Internal ties

(1) Internal ties should be at each floor and roof level in two directions approximately at right angles. They should be effectively continuous throughout their length and should be anchored to the peripheral ties at each end unless continuing as horizontal ties to columns or walls.

(2) The internal ties may, in whole or in part, be spread evenly in the slabs or may be grouped at or in beams, walls or other appropriate positions. In walls they should be within 0,5 m from the top or bottom of floor slabs.

(3) In each direction, internal ties should be capable of resisting a design value of tensile force according to Table 12.5 (NDP).

12.9.2.3 Horizontal ties to columns and/or walls

(1) Columns and walls should be tied horizontally to the structure at each floor and roof level.

(2) The ties should be capable of resisting a tensile force t_{fac} per metre of the wall. For columns the force need not exceed T_{col}. Connections between columns and/or walls and the structure at each floor and roof level shall – without beneficial effects of actions – be capable of transferring the specified tie forces.

(3) Corner columns should be tied in two directions. Bonded and unbonded reinforcement provided for the peripheral tie may be used as the horizontal tie in this case.

12.9.2.4 Vertical ties

(1) Each column and wall should be tied continuously from the foundations to the roof level. Where a column or wall is supported at its lowest level by a member other than a foundation (e.g. by a beam or flat slab) the consequence of accidental loss of this member should be considered.

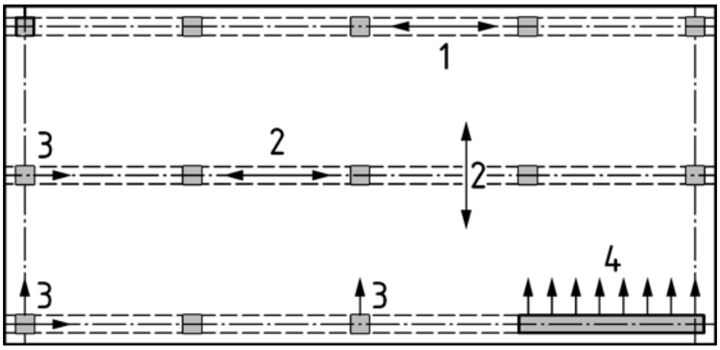
12.9.3 Required resistances for ties

(1) Ties (see Figure 12.10) should be capable of resisting a tensile force given in Table 12.5 (NDP).

NOTE The values in Table 12.5 (NDP) apply unless the National Annex gives other values.

Table 12.5 (NDP) — Resistances for reinforcement in ties

Description		Symbol	Requirement
1	Peripheral ties	T_p	EN 1991-1-7
2	Internal ties	T_i	EN 1991-1-7
3	Horizontal ties to columns	T_{col}	≥ 150 kN
4	Horizontal ties to walls	t_{fac}	≥ 20 kN/m
5	Vertical ties	T_v	EN 1991-1-7



Key

1 peripheral tie

2 internal tie

3 horizontal ties to columns

4 horizontal ties to walls

Figure 12.10 — Ties for robustness

12.10 Supports, bearings and expansion joints

(1) Supports and bearings shall be designed and detailed for the relevant actions (loads, movements, rotations) to ensure correct positioning of the bearing reaction in accordance with the design model, taking into account construction deviations.

(2) For supports and bearings which permit movements, the shift of the bearing reaction should be taken into account in the design of the adjacent members.

(3) For supports which do not permit movements or rotation without overcoming significant restraint, actions due to restrained movements of the adjacent members (elastic, creep, shrinkage, temperature) and misalignment, lack of plumb, etc. should be taken into account in the design of these members.

(4) Positioning and sizing of supports and bearings as well as detailing of reinforcement in supporting and supported members shall ensure effective transfer of forces compatible with the assumed action effects in the members and in the respective nodes of stress field or struts and ties, see Figure 8.28. Ties in the form of bent bars should effectively enclose the node.

(5) The nominal length a_1 of a simple support or bearing as shown in Figure 12.11 should be detailed accounting for:

— the net support or bearing length a_1 with regard to bearing capacity as defined in (6);

— anticipated movements where relevant;

— edge distances of supported and supporting members that may be considered ineffective due to, for example, the possibility of spalling or crumbling or due to incompatible stiffness;

— the allowances for construction deviations Δa_2 and Δa_3 in geometry and position of the supported and supporting members.

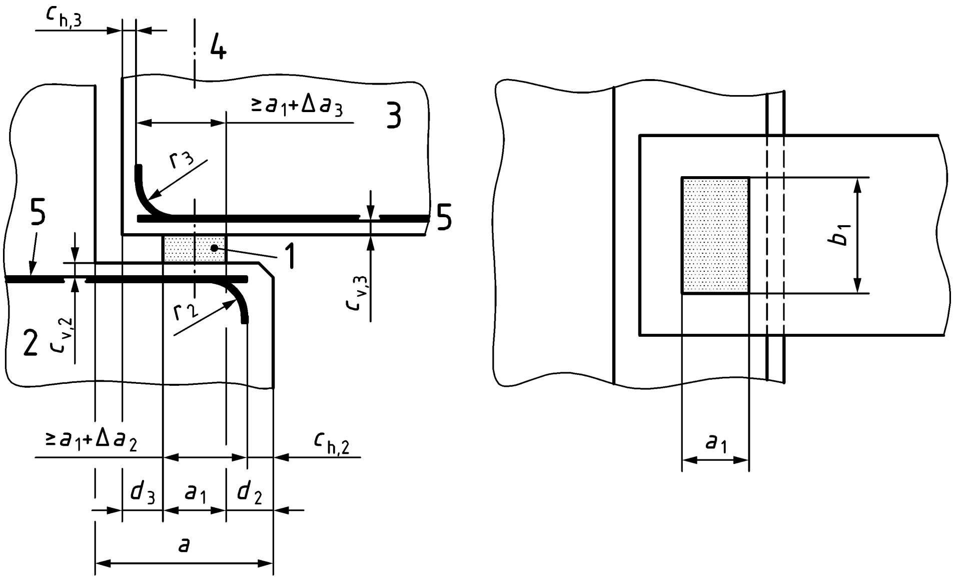
Detailing of members and supports should respect the following conditions, see Figure 12.11:

— $d_i = c_{h,i} + \Delta a_i$ with horizontal U-bar loops or otherwise anchored bars;

— $d_i = c_{h,i} + \Delta a_i + r_i$ with vertically bent bars;

— $c_{v,i} \leq d_i$ level of horizontal U-bar loops.

Where measures are taken to prevent spalling of concrete outside of the vertical bend, dimension d may be reduced, but $a_1/2 + d_i \geq c_{hi} + \Delta a_i + r_i$.



a) cross section

b) elevation

Key

a_1	bearing length	1	bearing
b_1	effective bearing width	2	supporting member
Δa_i	deviations	3	supported member
$c_{v,i}$, $c_{h,i}$	nominal cover (vertical and horizontal)	4	centre-line of support
r_i	Mandrel radius of bend of longitudinal reinforcement	5	anchored longitudinal reinforcement

Figure 12.11 — Example of bearing and lengths definitions

(6) Supported and supporting members shall be verified for the effects of the support or bearing reactions in accordance with 8.6.

(7) In the absence of more detailed specifications, the following values may be used for the bearing strength:

$$f_{Rd} = 0{,}4 f_{cd} \text{ for dry connections} \tag{12.13}$$

$$f_{Rd} = f_{bed} \leq 0{,}85 f_{cd} \text{ for all other cases} \tag{12.14}$$

where

f_{Rd} is the design value of bearing strength; for partially loaded areas see 8.6;

f_{cd} is the lower value of the design strengths of supported and supporting members;

f_{bed} is the appropriate design resistance of the bedding material, consistent with the design model and the limit state being checked.

(8) If a uniform distribution of the bearing pressure can be obtained, e.g. by mortar, neoprene or other pads, the design bearing width may be taken as the actual bearing width. Otherwise b_1 according to Figure 12.11 should not be taken greater than 600 mm.

(9) The choice of the type of bearing and expansion joints should be compatible with the structural system assumed in the design of the structure and suitable for the anticipated movements.

(10) The arrangement of bearings should be such that restraints of the adjacent members are avoided or minimised.

(11) Where the design service life of the bearing or expansion joint is less than the design service life of the member, bearings and expansion joints should be designed to be replaceable.

13 Additional rules for precast concrete elements and structures

13.1 General

(1) The rules in Clause 13 apply to structures made partly or entirely of precast concrete elements and are supplementary to other rules in this Eurocode.

(2) Further development of the provisions for specific precast concrete products – related to design, detailing, tolerances and production –given in EN 13369 and in the respective European product standards should be considered. Provisions for assembly given in EN 13670 should be considered.

13.2 Specific requirements

(1) In design and detailing, the following shall be considered specifically:

— bearings (temporary and permanent), joints and connections;

— transient situations like demoulding, transfer of prestress, transports, storage, erection and assembly.

(2) Where relevant, dynamic effects in transient situations should be taken into account. In the absence of an accurate analysis, static effects may be multiplied by an appropriate factor (for precast products, directions may be provided in specific product standards).

13.3 Concrete

13.3.1 Strength for heat curing

(1) In the case of heat curing, the mean compressive strength of concrete at an age t less than t_{ref}, $f_{\mathrm{cm}}(t)$, may be estimated from Formula (B.1), replacing the concrete age t with the temperature adjusted concrete age obtained by Formula (B.18), where the coefficient $\beta_{\mathrm{cc}}(t)$ should be limited to 1,0.

For the effect of heat curing, Formula (13.1) may be used:

$$f_{\mathrm{cm}}(t) = f_{\mathrm{cmp}} + \frac{f_{\mathrm{cm}}(t_{\mathrm{ref}}) - f_{\mathrm{cmp}}}{\lg(t_{\mathrm{ref}} - t_p + 1)} \lg(t - t_p + 1) \tag{13.1}$$

where

f_{cmp} is the mean compressive strength after the heat curing (i.e. at tendon release), measured by testing samples at the time t_{p} ($t_{\mathrm{p}} < t_{\mathrm{ref}}$) subject to the same heat treatment as the precast concrete product;

t_{ref} is defined in 5.1.3.

13.3.2 Creep and shrinkage

(1) In case of heat cured precast concrete elements, the values of creep deformations may be estimated according to the maturity function of Formula (B.14) in Annex B, in which the age of concrete at loading t_0 (in days) in Formula (B.9) should be replaced by the adjusted concrete age obtained by Formulae (B.17) and (B.18).

(2) In elements subjected to heat curing, drying shrinkage strain and basic shrinkage strain during heat curing may be assumed negligible.

13.4 Structural analysis

13.4.1 General

(1) The analysis shall account for:

— the behaviour of the structural elements at all stages of construction, using the appropriate geometry and properties for each stage, and their interaction with other elements (e.g. composite action with insitu concrete or other precast units);

— the influence of the behaviour of the connections, with particular regard to their actual deformability and strength;

— the uncertainties affecting restraints and force transmission between elements, arising from deviations in geometry and in positioning of elements and bearings;

— the influence of the flexibility of members supporting precast floor elements on the load distribution between the precast floor elements.

(2) In bearings of beams and slabs, beneficial effects of horizontal restraint caused by friction due to gravity loads may be taken into account (using $\gamma_{G,inf}$) and where

— friction is not solely relied upon for overall stability of the structure;

— bearing arrangements preclude the possibility of accumulation of sliding of the elements;

— risk of significant impact loading is excluded.

(3) The effects of relative movements of elements on bearings should be considered in design with respect to the resistance of the structure and the integrity of the connections.

13.4.2 Losses of prestress during heat curing

13.4.2.1 Relaxation losses

(1) For pre-tensioned members, the effect of heat treatment on the relaxation losses shall be considered.

(2) This effect of heat treatment on the relaxation losses may be accounted for by the simplified method given in Formula (13.2). In this case, an equivalent time t_{eq} should be added to the time after tensioning t used in Annex B.

$$t_{eq} = \frac{1{,}14^{T_{max}-20}}{T_{max}-20}\sum_{i=1}^{n}\left(T_{(\Delta t_i)}-20\right)\ \Delta t_i \qquad (13.2)$$

where

t_{eq} is the equivalent time (in hours);

$T_{(\Delta t_i)}$ is the temperature during the time interval Δt_i;

T_{max} is the maximum temperature during the heat treatment.

13.4.2.2 Thermal losses

(1) A specific thermal loss $\Delta\sigma_\theta$ induced by heat treatment should be avoided or taken into account. The thermal loss may be estimated by Formula (13.3):

$$\Delta\sigma_\theta = 0{,}5E_p \cdot \alpha_{c,th} \cdot (T_{max} - T_0) \qquad (13.3)$$

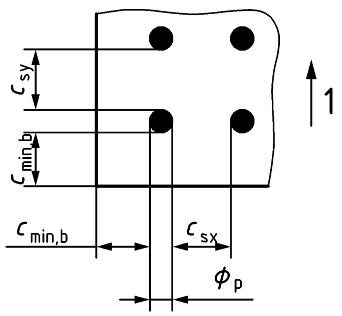
where $(T_{max} - T_0)$ is the difference between maximum and initial concrete temperature near the tendons.

13.5 Design and detailing of pre-tensioning tendons

13.5.1 Arrangement of tendons

(1) The spacing and concrete cover of pre-tensioning tendons shall be such as to ensure that placing and compacting concrete will be carried out satisfactorily and that sufficient bond will be attained between the concrete and the tendons.

(2) The minimum spacing and cover for bond and to avoid splitting should be in accordance with Figure 13.1 and Table 13.1. Depending on, for example, the production method, the level of pre-stressing or the level of release strength, other values of spacing and cover may be used, provided that satisfactory behaviour in service and at ultimate is demonstrated by testing or permitted by the specific European product standard with factory production control.



Clear spaces with $c_{sx} \geq \max\{2\phi_p; D_{upper} + 5\text{ mm}; 20\text{ mm}\}$ and $c_{sy} \geq \max\{2\phi_p; D_{upper}\}$

Key

1 direction of casting

NOTE For $c_{min,dur}$, see 6.5.2.2.

Figure 13.1 — Minimum clear spacing and cover for bond for pre-tensioning tendons

Table 13.1 — Minimum concrete cover $c_{min,b}$ for pre-tensioning tendons

clear spacing s	**Minimum cover** $c_{min,b}$	
	Strand	**Indented wire**
$s = 2\phi_p$	$3{,}0\phi_p$	$4{,}5\phi_p$
$s \geq 2{,}5\phi_p$	$2{,}5\phi_p$	$4{,}0\phi_p$

(3) Tendon ends shall be protected for durability. Methods of protection for durability may be given by the specific European product standard with factory production control or applied otherwise provided that satisfactory behaviour is demonstrated.

13.5.2 Anchorage zones

(1) In anchorage zones of tendons, the following length parameters should be considered, see Figure 13.2 and Figure 13.3:

— Transmission length l_{pt}, over which the prestressing force P_0 is fully transmitted to the concrete, see 13.5.3(1);

— Dispersion length l_{disp}, over which the concrete stresses gradually disperse to a linear distribution across the concrete section, see 13.5.3(3);

— Anchorage length l_{bpd}, over which the tendon force P_{pd}, in the ultimate limit state is fully anchored in the concrete, see 13.5.4(2).

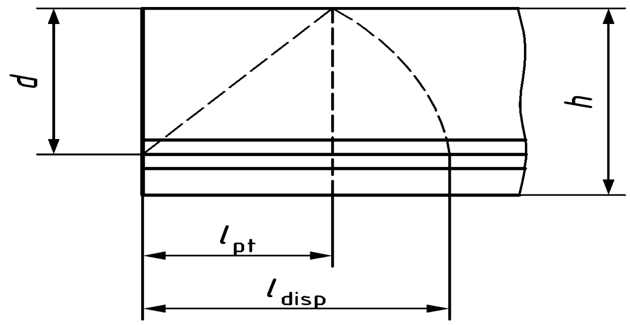
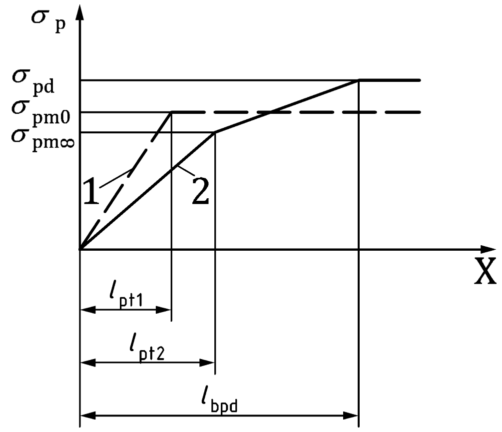


Figure 13.2 — Transfer of prestress in pre-tensioned member; length parameters



Key

X distance from end

1 at release

2 at ULS

Figure 13.3 — Tendon stresses

13.5.3 Transfer of prestress

(1) The basic value of the transmission length l_{pt} may be taken as:

$$l_{\mathrm{pt}} = \frac{\gamma_{\mathrm{C}}}{1{,}5} \cdot \frac{\alpha_1 \cdot \alpha_2 \cdot \sigma_{\mathrm{pm0}}}{\eta_1 \cdot \sqrt{f_{\mathrm{ck}}(t)}} \cdot \phi_{\mathrm{p}} \tag{13.4}$$

where

α_1 = 1,0 for gradual release,

= 1,25 for sudden release;

α_2 = 0,40 for indented wires,

= 0,26 for 3- and 7-wire strands;

ϕ_p is the nominal diameter of the tendon;

σ_{pm0} is the tendon stress just after release;

η_1 = 1,0 for tendons located in favourable positions during concreting (see 11.4.2(4)),

= 0,7 otherwise unless a higher value can be justified with regard to special circumstances in execution;

$f_{ck}(t)$ is the concrete compressive strength at time of release which may be taken as:

$$f_{\mathrm{ck}}(t) = [\beta_{\mathrm{cc}}(t)]^{2/3} \cdot f_{\mathrm{ck}} \tag{13.5}$$

where

$\beta_{cc}(t)$ is the age factor according to B.4, Formula (B.2).

Other values of transmission length may be used, provided that satisfactory behaviour in service is demonstrated by testing or permitted by the specific European product standard with factory production control.

(2) Depending on the design situation, the design value of the transmission length should be taken as:

$l_{pt1} = 0{,}8 l_{pt}$ for the verification of local stresses at release (13.6)

$l_{pt2} = 1{,}2 l_{pt}$ for ultimate limit states (shear, anchorage, etc.) (13.7)

(3) Concrete stresses may be assumed to have a linear distribution in the member cross-section outside the dispersion length, l_{disp}, see Figure 13.2:

$$l_{\mathrm{disp}} = \sqrt{{l_{\mathrm{pt}}}^2 + d^2} \tag{13.8}$$

(4) Alternative models for the transfer of prestressing force may be used, if adequately justified and if the transmission length is modified accordingly.

13.5.4 Anchorage of tensile force at ULS

(1) The anchorage of tendons shall be checked in sections where the concrete tensile stress exceeds $f_{ctk,0,05}$ for the relevant characteristic load combination.

The tendon stress σ_{pd} shall be calculated for a cracked section, including the effect of shear according to 8.2.3(8).

(2) If concrete is uncracked all along the transmission length, the total anchorage length for anchoring a tendon at ultimate limit state with stress σ_{pd} may be taken as:

$$l_{bpd} = l_{pt2} + \frac{\gamma_C}{1,5} \cdot \frac{2 \cdot \alpha_2 \cdot \alpha_3 \cdot (\sigma_{pd} - \sigma_{pm\infty})}{\eta_1 \cdot \sqrt{f_{ck}}} \cdot \phi_p \tag{13.9}$$

where

l_{pt2} is the upper design value of transmission length, see 13.5.3(2);

α_2 is as defined in 13.5.3(1);

α_3 = 1,5 in cases where fatigue verification is required;

= 1,0 in all other cases;

$\sigma_{pm\infty}$ is the tendon stress after all losses;

η_1 is as defined in 13.5.3(1).

Tendon stresses in the anchorage zone are illustrated in Figure 13.3.

13.5.5 Shear resistance of precast members without shear reinforcement

(1) Except for the conditions described in (2), 8.2.2 applies.

(2) Provided that following conditions are fulfilled:

— the member is prestressed;

— the effective depth is not larger than 500 mm unless the size effect is considered by a refined analysis;

— the investigated region is uncracked in bending (where the flexural tensile stress for the persistent and transient design situation, including effects of imposed deformations is smaller than $f_{ctk,0,05}/\gamma_C$).

The shear resistance may be checked by the verification of the principal stress:

$$\sigma_{1Ed}(y) \leq \frac{f_{ctk,0,05}}{\gamma_C} \tag{13.10}$$

where

σ_{1Ed} (y) is the maximum value of the principal tensile stress in the concrete cross-section,

$$\sigma_{1Ed} = \frac{\sigma_{x,Ed}(y)}{2} + \sqrt{\left(\frac{\sigma_{x,Ed}(y)}{2}\right)^2 + \tau_{Ed}^2(y)} \tag{13.11}$$

where

$\sigma_{x,Ed}(y)$ is the normal stress in the longitudinal direction of the structure determined in a fibre at distance y from the centroidal axis, assuming linear stress distribution over the depth. For cross-sections within the transmission length of pre-tensioning tendons l_{pt2} according to Formula (13.7), the contribution of the related prestressing force to N_{Ed} and M_{Ed} should be considered with a linear distribution according to Figure 13.3;

$\tau_{Ed}(y)$ is the shear stress in a fibre at distance y from the centroidal axis:

$$\tau_{Ed}(y) = \frac{V_{Ed} \cdot S(y)}{b(y) \cdot I} \tag{13.12}$$

I is the second moment of area of the concrete cross-section;

$b(y)$ is the width of the concrete cross-section at a distance y from the centroidal axis;

$S(y)$ is the first moment of area of the concrete cross-section above the fibre at distance y to the centroidal axis taken about the centroidal axis;

y is the distance from the considered fibre to centroidal axis of the concrete cross-section, y should be choosen so that the maximum value of $\sigma_{1,Ed}$ is found.

Additional shear stresses, for example by support effects, and/or due to the dispersion of tendon anchorage forces should be considered.

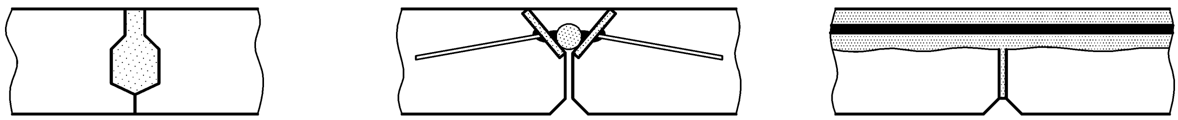
(3) The calculation of the shear resistance, using Formula (13.10), is not required for points above the support and within a line, starting from the face of the support and extending upwards at an angle of 45°.

13.6 Floor systems for buildings

13.6.1 Distribution of loads

(1) Where transverse load distribution between adjacent elements has been taken into account, appropriate shear connections shall be provided across the longitudinal joints.

(2) Transverse shear transfer across longitudinal joints between precast elements may be achieved as shown by examples in Figure 13.4.



a) longitudinal key infilled with mortar or concrete

b) with welded or bolted anchored plates

c) with structural reinforced concrete topping

Figure 13.4 — Connections for vertical shear transfer between parallel precast units

(3) Transverse distribution of vertical loads between adjacent elements should be based on analysis or tests taking into account the likely load variation between precast elements and the strength of the interface.

(4) For building floors with uniformly distributed load and in the absence of a more accurate analysis, the shear force acting in joints to adjacent precast elements per unit length of longitudinal joint may be taken as:

$$v_{Ed} = q_{Ed} \cdot b_e/3 \tag{13.13}$$

where

q_{Ed} is the design value of variable load;

b_e is the width of the element.

(5) Precast concrete elements with a structural concrete topping at least 40 mm thick may be designed as composite elements, if shear at the interface is verified according to 8.2.6.

(6) Transverse reinforcement for bending and other action effects may be placed within the structural concrete topping. In the case of negative bending at a section above a joint (see Figure 13.4 c)) the compression zone should be assumed to be in the structural concrete topping unless the joint between the precast units contains adequately compacted insitu concrete. Concrete may be considered adequately compacted if the width of the joint is greater than the largest of D_{upper} and the height of the precast unit.

(7) Building floor slabs combining parallel beams with unreinforced blocks in between without structural concrete topping may be analysed in shear and bending as solid slabs, if:

(i) cast-insitu transverse ribs are provided with a continuous reinforcement through the longitudinal ribs; and

(ii) the spacing s_{T} of such transverse ribs complies with the limits in Table 13.2.

Table 13.2 — Maximum spacing of transverse ribs s_{T}

Type of actions	$s_{\mathrm{L}} \leq l_{\mathrm{L}}/8$	$s_{\mathrm{L}} > l_{\mathrm{L}}/8$
residential, snow	not required	$s_{\mathrm{T}} \leq 12h$
other	$s_{\mathrm{T}} \leq 10h$	$s_{\mathrm{T}} \leq 8h$

where

s_{L} spacing of longitudinal ribs;

h depth of the floor;

l_{L} length (span) of longitudinal ribs.

(8) Webs or ribs in isolated slab elements should be provided with shear reinforcement as for beams.

(9) The effects of possible restraints of precast units shall be considered.

13.6.2 Diaphragm action

(1) Where precast concrete floors in buildings are assumed to act as horizontal diaphragms transferring horizontal forces to vertical bracing units:

— the diaphragm should form part of a realistic structural model, taking into account the deformation compatibility with bracing units;

— the effects of horizontal deformations should be taken into account for all parts of the structure involved in the transfer of horizontal loads;

— the diaphragm should be reinforced and connected for the tensile forces derived from the analysis;

— stress concentrations at openings and connections should be taken into account in the detailing of the reinforcement.

(2) If tie bars with diameter ϕ are anchored in the concrete infilled in longitudinal joints between parallel elements, the joint width at the height of the tie bars should be $\geq 3\phi$.

(3) Transverse reinforcement provided across longitudinal joints as part of the tying system may be concentrated along the elements' supports and shall be consistent with the structural model.

(4) In diaphragm action provided by untopped building floors made of precast slab elements with concreted or grouted longitudinal joints, their shear resistance should be determined according to shear-friction mechanisms, accounting for the transverse compressive force on the joint surface and for its roughness or presence of keys. For shear at the interface, reference should be made to 8.2.6 or to design assisted by testing.

13.6.3 Tying systems for buildings

(1) Precast structures should be provided with a tying system to secure robustness of the structure according to the rules of 12.9. For building structures the following additional provisions apply.

(2) In planar elements loaded in their own plane, e.g. in multi-storey walls and floor diaphragms composed of precast elements, the necessary interaction among elements for the overall resistance may be obtained by tying them together with peripheral and/or internal ties.

(3) The same ties may also be relied upon to provide overall robustness of the structure, according to 12.9.

(4) Horizontal ties may be provided wholly within the insitu structural concrete topping or at connections of precast members. Where ties are not continuous in one plane, the bending effects resulting from the eccentricities should be considered.

(5) Ties should not be lapped in narrow joints between precast elements. Mechanical anchorage shall be used in these cases.

(6) If internal ties are grouped along lines, surrounding members should be designed to distribute their load back to the internal tie.

13.7 Connections and supports

13.7.1 Connections

(1) Materials for connections shall be:

— stable and durable for the design service life of the structure;

— chemically and physically compatible.

(2) Connections shall be designed to resist all action effects consistent with design assumptions, to accommodate the necessary deformations and ensure robust behaviour of the structure.

(3) The verification of resistance and stiffness of connections may be based on numerical analysis, possibly assisted by testing.

(4) Except in exposure classes X0 and XC1, fasteners for claddings shall be made of corrosion resistant material unless adequately protected against environmental actions.

(5) Shear forces may be ignored in compression connections if they are less than 10 % of the compressive force.

(6) At connections with bedding materials like mortar, concrete or polymeric pads, relative movement between the connected surfaces shall be prevented during hardening of the material.

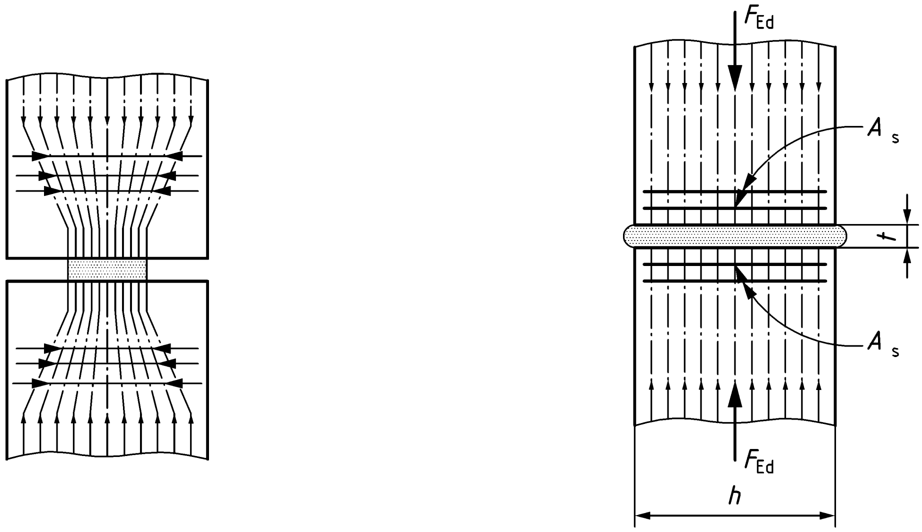
(7) Connections for compressive force without bedding material or mortar bed may only be used where appropriate quality of workmanship can be achieved. The average stress between plane surfaces should not exceed $0{,}4 f_{\mathrm{cd}}$.

(8) Transverse tensile stresses near the ends of adjacent precast concrete elements should be considered. They may be due to concentrated compression according to Figure 13.5 a), or due to the expansion of a soft padding according to Figure 13.5 b). Reinforcement in case a) may be designed and located according to 8.5.5. Reinforcement in case b) should be placed close to the surfaces of the adjacent elements.

(9) In absence of more accurate models, reinforcement in Figure 13.5 b) may be calculated as:

$$A_s = 0{,}25 \frac{t}{h} \cdot \frac{F_{\mathrm{Ed}}}{f_{\mathrm{yd}}} \qquad (13.14)$$

(10) The maximum capacity of compression connections should be determined according to 8.6, or should be based on design assisted by testing.



a) concentrated bearing

b) expansion of soft padding and splitting reinforcement A_s (two layers every side)

Figure 13.5 — Transverse tensile stresses at connections transmitting compression

(11) In building wall elements installed over floor slabs, reinforcement should normally be provided for possible eccentricities and concentrations of the vertical load at the end of the wall.

(12) Specific reinforcement in building walls over a connection may be omitted, if the wall is supported in a mortar bed and the vertical load satisfies

$$F_{Ed} \leq 0{,}5h \cdot f_{cd} \tag{13.15}$$

where h is the wall thickness (see Figure 13.6).

(13) F_{Ed} in Formula (13.15) may be increased to

$$F_{Ed} = 0{,}6h \cdot f_{cd} \tag{13.16}$$

in presence of reinforcement as in Figure 13.6, having diameter $\phi \geq 6$ mm and spacing s not greater than the lesser of h and 200 mm. For higher loads, reinforcement should be designed according to Formula (13.14). A separate check should be made for the lower wall.

a) transverse view b) longitudinal view

Figure 13.6 — Example of reinforcement in a building wall over a connection between floor slabs without mortar joint

(14) Reinforcement for transferring bending moments or tensile forces shall be continuous across the connection and anchored in the adjacent precast concrete elements.

13.7.2 Supports

(1) The provisions of 12.10 should be complied with.

(2) If the bearing of isolated elements allows longitudinal movements, the net bearing length shall be increased to cover possible movements.

(3) If an isolated element is tied other than at the level of its bearing, the net bearing length a_1 shall be increased to cover the effect of possible rotation around the tie.

13.8 Pocket foundations for buildings

13.8.1 General

(1) Pocket foundations, embedding a precast column base within side walls infilled with insitu concrete (see Figs. 13.7 and 13.8, where only the main reinforcement is sketched) and transferring member action effects by shear and bending, may be considered to provide full restraint of the column in the pocket.

NOTE Depending on soil properties, soil-structure interaction can however result in lower restraints than available between column and pocket.

(2) Pockets should be large enough to enable a good concrete filling below and around the column.

(3) Pocket foundations may host members other than a single column, e.g., twin columns, short walls, etc.; in these cases, specific adequate provisions should be devised for ensuring the assumed restraint for these elements.

13.8.2 Pocket foundations with keyed surface

(1) Pocket and hosted column, both provided with adequate keys (see 8.2.6) able to mobilise strut mechanisms (Figure 13.7), may be considered monolithically connected with overlapped reinforcements.

(2) The lap length between the reinforcement of the column and of the pocket according to 11.5.2 should be increased by at least the distance s between the lapped bars. Adequate confinement reinforcement at the lap splice should be provided (see Figure 13.7).

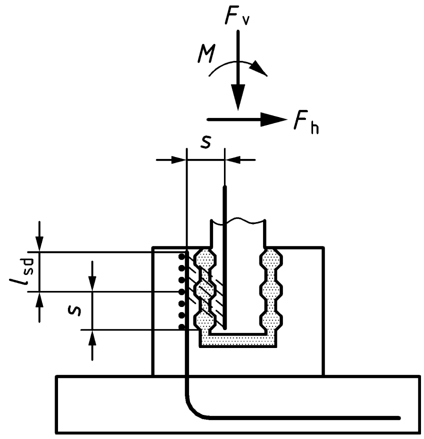


Figure 13.7 — Pocket foundations with keyed surfaces

13.8.3 Pocket foundations with smooth or rough surfaces

(1) In case of pocket and column with smooth or rough surfaces (non-keyed), the action effects from the column may be assumed to be transferred through the concrete filling by the compressive forces F_1, F_2, F$_3$ and the corresponding friction forces $\mu_v F_1$, $\mu_v F_2$, $\mu_v F_3$, as shown in Figure 13.8.

(2) With the assumption of (1) above, the minimum embedded length l should be

$$l \geq 1{,}2h_{col} \text{ for } M_{Ed}/N_{Ed} \leq 0{,}15h_{col} \tag{13.17}$$

$$l \geq 2{,}0h_{col} \text{ for } M_{Ed}/N_{Ed} \geq 2{,}0h_{col} \tag{13.18}$$

where h_{col} is the largest side of the column's section.

l may be interpolated for $0{,}15h_{col} < M_{Ed}/N_{Ed} < 2{,}0h_{col}$.

(3) The coefficient of friction μ_v should be taken from Table 8.2, according to the relevant type of surface.

(4) The transfer of forces to the base slab may be checked at ULS by assuming a strut-and-tie model in accordance with 8.5.

(5) The position of F_1 should be chosen with $a \geq 0{,}1l$.

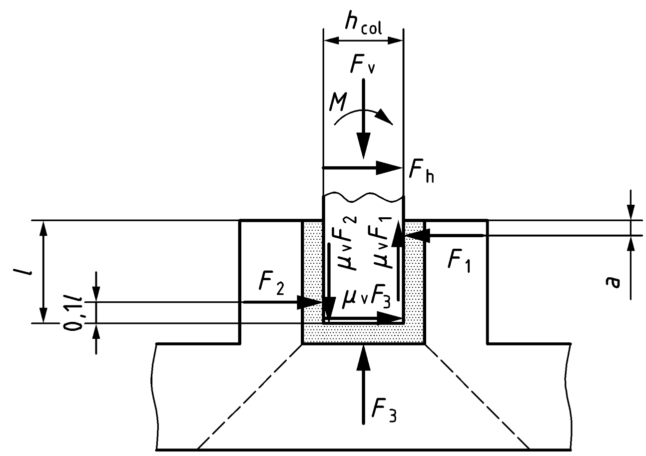


Figure 13.8 — Pocket foundations with smooth or rough joint surface

14 Plain and lightly reinforced concrete structures

14.1 General

(1) Clause 14 provides additional rules for plain concrete structures or where the reinforcement provided is less than the minimum required for reinforced concrete according to Clause 12.

(2) It should be ensured that brittle failure of these members does not lead to collapse of the structure.

(3) Clause 14 applies to members mainly subjected to compression other than that due to prestressing such as:

— walls, columns, arches, vaults, and tunnels;

— strip and pad footings for foundations;

— gravity retaining walls;

— piles with diameter ≥ 600 mm and where $|N_{\mathrm{Ed}}/A_{\mathrm{c}}| \leq 0{,}30f_{\mathrm{ck}}$.

Clause 14 does not apply to members affected by dynamic actions (other than seismic) such as those from rotating machines and traffic loads.

(4) For members made of lightweight aggregate concrete with closed structure according to Annex M or for precast concrete elements and structures covered by this Eurocode, the design rules should be modified accordingly.

(5) Plain concrete members may include steel reinforcement to satisfy serviceability, or partial reinforcement. Such reinforcement may be taken into account for local verification of ultimate limit states and for verification of serviceability limit states.

(6) Members subject to imposed deformations except as noted in 14.4.2 should be designed as reinforced members or constructed with joints to avoid uncontrolled cracking.

14.2 Concrete

(1) For the verification of plain concrete members at ULS, the values for $f_{\mathrm{cd,pl}}$ and $f_{\mathrm{ctd,pl}}$ shall be taken as:

$$f_{\mathrm{cd,pl}} = k_{\mathrm{c,pl}} \cdot f_{\mathrm{cd}} \tag{14.1}$$

$$f_{\mathrm{ctd,pl}} = k_{\mathrm{t,pl}} \cdot f_{\mathrm{ctd}} \tag{14.2}$$

NOTE The values of $k_{\mathrm{c,pl}} = 0{,}8$ and $k_{\mathrm{t,pl}} = 0{,}8$ apply unless the National Annex gives other values.

(2) When tensile stresses are considered for the design resistance of plain concrete members, the stress strain diagram (see 8.1.1) may be extended linearly up to the tensile design strength $f_{\mathrm{ctd,pl}}$.

14.3 Structural analysis

(1) Since plain concrete members have limited deformation capacity, analysis methods according to Clause 7 such as stress fields, strut-and-tie or FE should be used which permit neglecting the tensile strength of concrete, in general.

14.4 Ultimate limit states

14.4.1 General

(1) Beams, slabs and other structural members without axial compression or axial restraint and with predominantely bending moments should not be designed as plain concrete members taking into account

concrete tensile strength. In cases of axially loaded members and shear forces 14.4.3 may be used. Strip and pad footings may be designed according to 14.6.3.

14.4.2 Design resistance to bending with axial force

(1) In the case of walls, subject to the provision of adequate construction details and curing, the imposed deformations due to temperature or shrinkage may be ignored.

(2) The general provisions of 8.1 apply for determining the ultimate resistance of sections to bending with axial force.

(3) The axial resistance N_{Rd} of a rectangular cross-section with a uniaxial eccentricity e in the direction of h (see Figure 14.1), may be taken as:

$$N_{Rd} = f_{cd,pl} \cdot b \cdot h\,(1 - 2\,e/h) \tag{14.3}$$

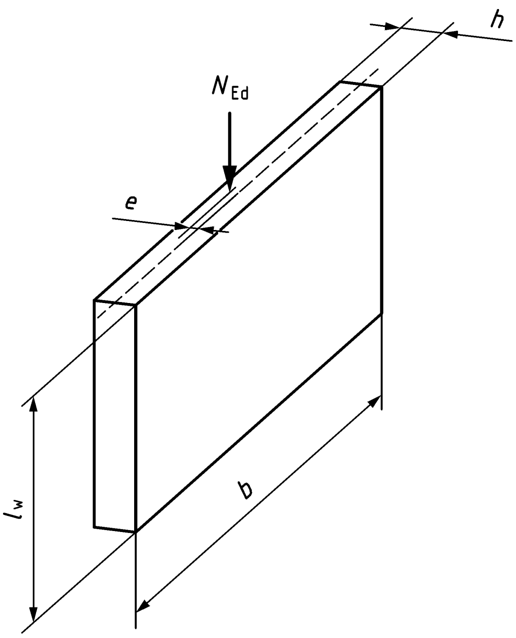


Figure 14.1 — Notation for plain walls

14.4.3 Shear

(1) In members with axial compression, approaches based on compression fields without ties according to 8.5 should be used to calculate shear resistance.

(2) In plain concrete members account may be taken of the concrete tensile strength in the ultimate limit state for shear, provided that either by calculations or by experience, brittle failure is excluded and adequate resistance is ensured.

(3) For a section subject to a shear force V_{Ed} and a normal compressive force $|N_{Ed}|$ acting over a compressive area A_{cc} the absolute value of the components of design stress may be taken as:

$$\sigma_{cp} = |N_{Ed}|/A_{cc} \tag{14.4}$$

$$\tau_{cp} = 1{,}5 \cdot V_{Ed}/A_{cc} \text{ (for rectangular sections)} \tag{14.5}$$

and the following should be satisfied:

$$\tau_{cp} \leq \tau_{Rd,pl}$$

where

$$\text{if } \sigma_{cp} \le \sigma_{c,lim}: \quad \tau_{Rd,pl} = \sqrt{f_{ctd,pl}^2 + \sigma_{cp} f_{ctd,pl}} \tag{14.6}$$

or

$$\text{if } \sigma_{cp} > \sigma_{c,lim}: \quad \tau_{Rd,pl} = \sqrt{f_{ctd,pl}^2 + \sigma_{cp} f_{ctd,pl} - \left(\frac{\sigma_{cp} - \sigma_{c,lim}}{2}\right)^2} \tag{14.7}$$

$$\sigma_{c,lim} = f_{cd,pl} - 2\sqrt{f_{ctd,pl} \cdot \left(f_{ctd,pl} + f_{cd,pl}\right)} \tag{14.8}$$

where $\tau_{Rd,pl}$ is the plain concrete design strength in shear.

(4) A concrete member may be considered to be uncracked at the ultimate limit state if either it remains completely under compression or if the absolute value of the principal concrete tensile stress does not exceed $f_{ctd,pl}$.

(5) Shear forces in construction joints should be verified according to 8.2.6.

14.4.4 Torsion

(1) Cracked members should not be designed to resist torsional moments unless it can be justified otherwise.

14.4.5 Ultimate limit states induced by structural deformation (buckling)

14.4.5.1 Slenderness of columns and walls

(1) The effective length of a column or wall should be calculated as:

$$l_0 = \beta_{Eul} \cdot l_w \tag{14.9}$$

where

β_{Eul} *Euler*-coefficient which depends on the support conditions (*Euler*-cases). For walls supported on 3 or more sides β_{Eul}-values may be taken from Table 14.1.

(2) The β_{Eul} -values should be increased appropriately if the transverse bearing capacity is affected by chases or recesses.

(3) A transverse wall may be considered as a bracing wall if:

— its total thickness is not less than 0,5h, where h is the overall thickness of the braced wall;

— it has the same height l_w as the braced wall under consideration;

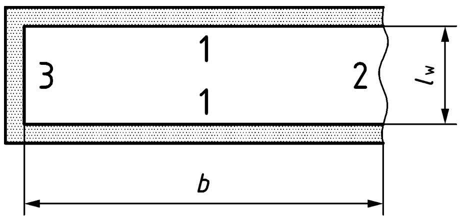
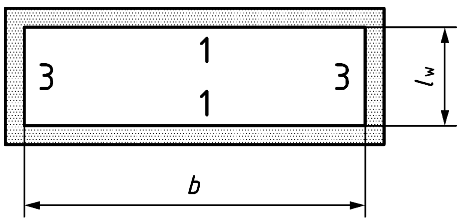
— its length is at least equal to l_w/5, where l_w denotes the clear height of the braced wall;

— within the length l_w/5 the transverse wall has no openings.

(4) In the case of a wall connected along the top and bottom in a flexurally rigid manner by insitu concrete and reinforcement, so that the end moments can be fully resisted, the values for β_{Eul} given in Table 14.1 may be factored by 0,85.

(5) The slenderness of walls in plain concrete cast insitu should generally not exceed $\lambda = 86$ (i.e. $l_0/h = 25$).

Table 14.1 — Values of β_{Eul} for different support conditions

Lateral bearing	Elevation of wall	Formula	Factor β_{Eul}	
			b/l_{w}	β_{Eul}
along three sides		$\beta_{\text{Eul}} = \dfrac{1}{1 + \left(\dfrac{l_{\text{w}}}{3b}\right)^2}$	0,2	0,26
			0,4	0,59
			0,6	0,76
			0,8	0,85
			1,0	0,90
			1,5	0,95
			2,0	0,97
			5,0	1,00
			b/l_{w}	β_{Eul}
along four sides		if $b \geq l_{\text{w}}$: $\beta_{\text{Eul}} = \dfrac{1}{1 + \left(\dfrac{l_{\text{w}}}{b}\right)^2}$ if $b < l_{\text{w}}$: $\beta_{\text{Eul}} = \dfrac{b}{2l_{\text{w}}}$	0,2	0,10
			0,4	0,20
			0,6	0,30
			0,8	0,40
			1,0	0,50
			1,5	0,69
			2,0	0,80
			5,0	0,96

1 – floor slab;
2 – free edge;
3 – transverse wall with thickness $\geq 0{,}5h$ and with a length $\geq l_{\text{w}}/5$

NOTE The information in Table 14.1 assumes that the wall has no openings with a height exceeding 1/3 of the wall height l_{w} or with an area exceeding 1/10 of the wall area. In walls laterally restrained along 3 or 4 sides with openings exceeding these limits, the parts between the openings should be considered as laterally restrained along 2 sides only and be designed accordingly.

14.4.5.2 Simplified design method for walls and columns

(1) In the absence of a more rigorous approach according to 7.4.3.3, the design resistance in terms of axial force for a braced wall or column in plain concrete with $f_{\text{ck}} < 55$ MPa, may be calculated as follows:

$$N_{\text{Rd}} = b \cdot h \cdot f_{\text{cd,pl}} \cdot \Phi \tag{14.10}$$

where

Φ is the factor taking into account eccentricity, including second order effects and considering $\gamma_{ce} = 1{,}20$

$$\Phi = \frac{1 - \left(2{,}1 + 0{,}02\,\frac{l_0}{h}\right)\frac{e_{tot}}{h}}{1 + \left(\frac{l_0}{h}\right)^2 \left(0{,}9 + 6\,\frac{e_{tot}}{h}\right)\left(\frac{0{,}8 + \varphi_{eff}}{1000}\right)\left(\frac{f_{cd,pl}}{20}\right)^{0{,}6}} \tag{14.11}$$

where

$$e_{tot} = e_0 + e_i \tag{14.12}$$

or derived from an equivalent first order end moment as described in O.7.2;

e_0 is the first order eccentricity including, where relevant, the effects of floors (e.g. possible clamping moments transmitted to the wall from a slab) and horizontal actions;

e_i is the additional eccentricity covering the effects of geometrical imperfections, see 7.2.1.

NOTE φ_{eff} is taken according to 7.4.2 unless the National Annex gives a different value.

In some cases, depending on slenderness, the end moment(s) (M_{02}) can be more critical for the structure than the equivalent first order end moment M_{0Ed}. In such cases Formula (14.3) should be used for the verification of the ultimate limit state.

14.5 Serviceability limit states

(1) Stresses should be checked where structural restraint is expected to occur.

(2) The following measures to ensure adequate serviceability should be considered:

a) with regard to crack formation:

- limitation of concrete tensile stresses according to Clause 9;
- provision of supplementary structural reinforcement (surface reinforcement, tying system where necessary);
- provision of joints;
- choice of concrete technology (e.g. appropriate concrete composition, curing);
- choice of appropriate method of construction.

b) with regard to limitation of deformations:

- a minimum section size (see 14.6);
- limitation of slenderness in the case of compression members.

(3) Any reinforcement provided in plain concrete members, although not taken into account for load bearing purposes, should comply with durability requirements according to Clause 6.

14.6 Detailing of members and particular rules

14.6.1 Structural members

(1) The overall thickness of a cast insitu plain concrete wall should not be less than 120 mm.

(2) Local reinforcement may need to be added at chases and recesses to control cracking.

14.6.2 Construction joints

(1) Construction joints should be designed according to 8.2.6.

(2) Where tensile stresses are expected to occur in concrete at constructions joints, reinforcement shall be detailed to provide required strength and to control cracking.

14.6.3 Strip and pad footings

(1) In the absence of more detailed data, axially loaded strip and pad footings (see Figure 14.2) may be designed and constructed as plain concrete member provided that:

$$\frac{0{,}85 \cdot h_{\mathrm{F}}}{a_{\mathrm{F}}} \geq \sqrt{\frac{3\sigma_{\mathrm{gd}}}{f_{\mathrm{ctd,pl}}}} \tag{14.13}$$

As a simplification, the relation in Formula (14.14) may be used:

$$\frac{h_{\mathrm{F}}}{a_{\mathrm{F}}} \geq 2 \tag{14.14}$$

where

h_{F} is the footing depth;

a_{F} is the distance from the footing edge to the column or wall face (see Figure 14.2);

σ_{gd} is the design value of ground pressure.

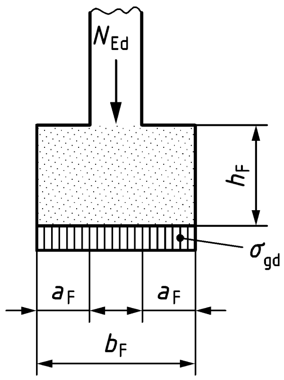


Figure 14.2 — Unreinforced pad footings; notations

Annex A
(informative)

Adjustment of partial factors for materials

A.1 Use of this annex

(1) This informative Annex contains information on the basis of the recommended partial factors for materials and provisions for adjustment of partial factors for materials for different reliability levels based on the national rules.

NOTE 1 National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this Informative Annex, it can be used.

A.2 Scope and fields of application

(1) This informative Annex applies to all clauses and annexes of this standard. It may be used for the cases listed in Tables A.1 and A.2.

A.3 General

(1) The partial factors for materials given in Table 4.3 (NDP) may be adjusted according to this Annex A if at least one of the conditions defined in Tables A.1 (NDP) and A.2 (NDP) apply.

NOTE 2 The values given in Tables A.1 (NDP) and A.2 (NDP) apply unless the National Annex gives different values.

(2) If there is an evidence that the actual statistical data of at least one of following variables:

— material strength;

— dominant geometrical value; or

— model uncertainty.

is more unfavourable than the values defined in Table A.3, the partial factors for materials given in Table 4.3 (NDP) shall be adapted according to (3).

Table A.1 (NDP) — Values of adjusted material factors - General

Condition for adjusted material factors	persistent and transient design situations			accidental design situations		
	γ_S	γ_C	γ_V	γ_S	γ_C	γ_V
a) if the execution ensures that geometrical deviations of Tolerance Class 2 according to EN 13670 are fulfilled	1,08	1,48	1,33	0,97	1,15	1,11
	in case also at least one of the conditions d), e), f) or h) is fulfilled, the partial factors may be calculated according to (3) with the statistical values given in (4) and in (7) for d) or in (8) for e); with the updated values of the resistance model for (f) and with the values given in Table A.4 for h)					
b) if the calculation of design resistance is based on the value of the dominant geometrical data measured in the finished structure and the CoV of the measurement is not larger than the values given in (5)	1,04	1,48	1,29	0,95	1,15	1,08
	in case also at least one of the conditions d), e), f) or h) is fulfilled, the partial factors may be calculated according to (3) with the statistical values given in (5) and in (7) for d) or in (8) for e); with the updated values of the resistance model for (f) and with the values given in Table A.4 for h)					
c) if the calculation of design resistance is based on the design value of the effective depth according to (6)	1,03	1,50	1,29	0,94	1,15	1,07
	in case also at least one of the conditions d), e), f) or h) is fulfilled, the partial factors may be calculated according to (3) with the statistical values given in (6) and in (7) for d) or in (8) for e); with the updated values of the resistance model for (f) and with the values given in Table A.4 for h)					
d) if the insitu concrete strength in the finished structure is assessed according to EN 13791:2019, Clause 8	γ_C and γ_V according to (7)					
e) if the yield strength of the reinforcement is assessed from tests on samples taken from the existing structure	γ_S according to (8)					
f) if the verification of the structure or of the member is conducted according to more refined methods ensuring reduced uncertainties of the resistance model.	γ_S and γ_C according to (3) where the statistical values describing the model uncertainties in Table A.3 are replaced by the actual ones					
g) if the verification of the structure or of the member is conducted using non-linear analysis and the model uncertainty is considered separately according to F.4(1).	1,20	1,46	1,31[a]	1,09	1,16	1,16[a]
h) if the target value for the reliability index β_{tgt} given in Table A.4 is modified in accordance with the relevant authority	γ_S and γ_C according to (3) with the statistical values in Table A.5					

Condition for adjusted material factors	persistent and transient design situations			accidental design situations		
	γ_S	γ_C	γ_V	γ_S	γ_C	γ_V
[a] These values apply for failures modes similar to punching and shear failures in members without shear reinforcement.						

Table A.2 (NDP) – Values of adjusted material factors – Additional provisions for precast members

Condition for adjusting material factors for precast members	persistent and transient design situations			accidental design situations		
	γ_S	γ_C	γ_V	γ_S	γ_C	γ_V
(i) In case (a) of Table A.1 (NDP) is fulfilled and System of AVCP 2+ is applied	1,10	1,40	1,30	0,95	1,05	1,05
(j) In case System AVCP 2+ is applied and the calculation of design resistance is based on critical dimensions, including effective depth, which are either: — the nominal values reduced by tolerances, or — the values measured in the finished member	1,05	1,35	1,28	0,90	1,05	1,05

Table A.3 — Statistical data assumed for the calculation of partial factor defined in Table 4.3 (NDP)

	Coefficient of variation	**Bias factor**[a]
Partial factor for reinforcement γ_S		
Yield strength f_y	$V_{fy} = 0{,}045$	$f_{ym}/f_{yk} = \exp(1{,}645 V_{fy})$
Effective depth d	$V_d = 0{,}050$[b]	$\mu_d = 0{,}95$[b]
Model uncertainty	$V_{\theta s} = 0{,}045$[c]	$\mu_{\theta s} = 1{,}09$[c]
Coefficient of variation and bias factor of resistance for reinforcement	$V_{RS} = 0{,}081$[i]	$\mu_{RS} = 1{,}115$[i]
Partial factor for concrete γ_C		
Compressive strength f_c (control specimen)	$V_{fc} = 0{,}100$	$f_{cm}/f_{ck} = \exp(1{,}645 V_{fc})$[d]
Insitu factor $\eta_{is} = f_{c,ais}/f_c$ [e]	$V_{\eta is} = 0{,}120$	$\mu_{\eta is} = 0{,}95$
Concrete area A_c	$V_{Ac} = 0{,}040$	$\mu_{Ac} = 1{,}00$
Model uncertainty	$V_{\theta c} = 0{,}070$[f]	$\mu_{\theta c} = 1{,}02$[f]
Coefficient of variation and bias factor of resistance for concrete	$V_{RC} = 0{,}176$[i]	$\mu_{RC} = 1{,}142$[i]
Partial factor for shear and punching γ_V (see 8.2.1, 8.2.2, 8.4, I.8.3.1, I.8.5)		
Compressive strength f_c (control specimen)	$V_{fc} = 0{,}100$	$f_{cm}/f_{ck} = \exp(1{,}645 V_{fc})$[d]
Insitu factor $\eta_{is} = f_{c,ais}/f_c$ [e]	$V_{\eta is} = 0{,}120$	$\mu_{\eta is} = 0{,}95$
Effective depth d	$V_d = 0{,}050$[b]	$\mu_d = 0{,}95$[b]
Model uncertainty	$V_{\theta v} = 0{,}107$[g]	$\mu_{\theta v} = 1{,}10$[g]
Residual uncertainties	$V_{res,v} = 0{,}046$[h]	–
Coefficient of variation and bias factor of resistance for shear and punching (members without shear reinforcement)	$V_{RV} = 0{,}137$[i]	$\mu_{RV} = 1{,}085$[i]

[a] The values in this column refer to ratio between mean value and values used in the design formulae (characteristic or nominal).

	Coefficient of variation	Bias factor[a]

[b] These values are valid for $d = 200$ mm. For other effective depths: $V_d = 0{,}05(200/d)^{2/3}$ and $\mu_d = 1 - 0{,}05(200/d)^{2/3}$.

[c] The partial factor γ_S is calibrated for the case of pure bending according to 5.2.4 and 8.1.

[d] This formula replaces relationship given in Table 5.1 for the purpose of Annex A.

[e] Insitu factor η_{is} accounts for the difference between the actual insitu concrete strength in the structure $f_{c,ais}$ and the strength of the control specimen f_c. For strength $f_{c,is}$ assessed on extracted 2:1 cores according to EN 13791, see (7).

[f] The partial factor γ_C is calibrated for the case of axial compression according to 5.1.6 and 8.1.

[g] The partial factor γ_V is calibrated for the case of punching according to 8.4 and applies also for the case of shear without shear reinforcement according to 8.2.2 (similar statistical values).

[h] The residual uncertainties refer to aggregate size, reinforcement area and spacing and column size.

[i] Based on the statistical values above and calculated using Formulae (A.2) to (A.7).

(3) The adjusted partial factors may be calculated as:

$$\gamma_{\mathrm{M}} = \frac{\exp(\alpha_{\mathrm{R}} \cdot \beta_{\mathrm{tgt}} \cdot V_{\mathrm{RM}})}{\mu_{\mathrm{RM}}} \tag{A.1}$$

where

index M is S for reinforcement, C for concrete in compression and V for shear;

α_{R} is the sensitivity factor for resistance according to Table A.4 (NDP);

β_{tgt} is the target value for the 50-year reliability index according to Table A.4 (NDP);

V_{RM} is the coefficient of variation of the resistance which may be calculated from

$$V_{\mathrm{RS}} = \sqrt{V_{\mathrm{fy}}^2 + V_{\mathrm{d}}^2 + V_{\mathrm{\theta s}}^2} \tag{A.2}$$

$$V_{\mathrm{RC}} = \sqrt{V_{\mathrm{fc}}^2 + V_{\mathrm{\eta is}}^2 + V_{\mathrm{Ac}}^2 + V_{\mathrm{\theta c}}^2} \tag{A.3}$$

$$V_{\mathrm{RV}} = \sqrt{\left(\frac{V_{\mathrm{fc}}}{3}\right)^2 + \left(\frac{V_{\mathrm{\eta is}}}{3}\right)^2 + V_{\mathrm{d}}^2 + V_{\mathrm{\theta v}}^2 + V_{\mathrm{res,v}}^2} \tag{A.4}$$

where the coefficients of variation of each uncertainty are defined in Table A.3 or updated (for conditions, see Table A.1 (NDP) and Table A.2 (NDP));

μ_{RM} is the bias factor of the resistance and may be calculated from:

$$\mu_{\mathrm{RS}} = \frac{f_{\mathrm{ym}}}{f_{\mathrm{yk}}} \mu_{\mathrm{d}} \cdot \mu_{\mathrm{\theta S}} \tag{A.5}$$

$$\mu_{\mathrm{RC}} = \frac{f_{\mathrm{cm}}}{f_{\mathrm{ck}}} \mu_{\mathrm{\eta is}} \cdot \mu_{\mathrm{Ac}} \cdot \mu_{\mathrm{\theta C}} \tag{A.6}$$

$$\mu_{\mathrm{RV}} = \left(\frac{f_{\mathrm{cm}}}{f_{\mathrm{ck}}} \cdot \mu_{\mathrm{\eta is}}\right)^{1/3} \cdot \mu_{\mathrm{d}} \cdot \mu_{\mathrm{\theta V}} \tag{A.7}$$

where the bias factors of each uncertainty are defined in Table (A.3) or updated (for conditions, see Tables A.1 (NDP) and A.2 (NDP)).

NOTE 3 The values of β_{tgt} in Table A.4 (NDP) apply unless the National Annex gives different values

NOTE 4 According to EN 1990, the approximated Formula (A.1) can be used for $V_{\mathrm{RM}} < 0{,}20$. The exact Formula to be used for higher coefficients of variation V_{RM} can be found in EN 1990 (which provides 3 % higher values of γ_{M} for $V_{\mathrm{RM}} = 0{,}30$).

Table A.4 (NDP) — Sensitivity factors for resistance α_{R} and target values for the 50-year reliability index β_{tgt}

Design situations/Limit states	**Sensitivity factors for resistance α_{R}**	**target value for the 50-year reliability index β_{tgt}**
Persistent or transient design situation	0,8	3,8
Fatigue design situation	0,8	3,8
Accidental design situation	0,8	2,0
NOTE 1 These values refer to CC2. For others Consequence Classes, refer to EN 1990.		

(4) If the execution is subjected to a quality control system, which ensures that geometrical deviations of Tolerance Class 2 according to EN 13670 are fulfilled, the statistical data of the geometrical values in Table A.3 may be replaced by:

— effective depth d of the reinforcement:

$V_d = 0{,}025$ for $d \geq 200$ mm or $V_d = 0{,}025(200/d)^{2/3}$ otherwise; $\mu_d = 0{,}975$;

— concrete area: $V_{Ac} = 0{,}02$ and $\mu_{Ac} = 1{,}00$.

(5) If the calculation of design resistance is based on the value of the dominant geometrical data (e.g. the effective depth for bending or the concrete area for axial compressive force) measured in the finished structure at the governing cross-section, the statistical data of the geometrical values in Table A.3 may be replaced by:

— effective depth d of the reinforcement: $V_d = 0{,}015$ and $\mu_d = 1{,}00$;

— concrete area: $V_{Ac} = 0{,}015$ and $\mu_{Ac} = 1{,}00$.

The measurements of the effective depth and of the concrete area should be conducted to allow for the coefficients of variation defined above. If this is not possible, the statistical values should be adapted accounting for the actual uncertainties of the measurements.

(6) The statistical data of the effective depth in Table A.3 may be replaced by $V_d = 0{,}00$ and $\mu_d = 1{,}00$ if the calculation of the design resistance is based on the design value of the effective depth d_d:

$$d_d = d_{nom} - \Delta d \tag{A.8}$$

where

Δd is the deviation value of the effective depth:

$\Delta d = 15$ mm for reinforcing and post-tensioning steel;

$\Delta d = 5$ mm for pre-tensioning steel.

NOTE 5 The design value of the effective depth d_d can be used unless the National Annex gives limitations.

(7) If the compressive concrete strength is assessed according to EN 13791:2019, Clause 8, the partial safety factors γ_C and γ_V may be adjusted using Formula (A.1), where Formulae (A.3), (A.4), (A.6) and (A.7) should be replaced by Formulae (A.9) to (A.12), respectively:

$$V_{RC} = \sqrt{V_{fc,is,corr}^2 + V_{Ac}^2 + V_{\theta c}^2} \tag{A.9}$$

$$V_{RV} = \sqrt{\left(\frac{V_{fc,is,corr}}{3}\right)^2 + V_d^2 + V_{\theta v}^2 + V_{res,v}^2} \tag{A.10}$$

and

$$\mu_{RC} = \mu_{fc,is} \cdot \mu_{Ac} \cdot \mu_{\theta c} \tag{A.11}$$

$$\mu_{RV} = \mu_{fc,is}^{1/3} \cdot \mu_d \cdot \mu_{\theta v} \tag{A.12}$$

where

V_{Ac}, $V_{\theta c}$, μ_{Ac} and $\mu_{\theta c}$ are taken from Table A.3 or updated considering the cases (b), (c), (e), (g) or (h) defined in Tables A.1 (NDP) and A.2 (NDP);

$$V_{\mathrm{fc,is,corr}} = \frac{k_{\mathrm{d,n}}}{\alpha_{\mathrm{R}} \cdot \beta_{\mathrm{tgt}}} V_{\mathrm{fc,is}} \quad \text{(A.13)}$$

$$\mu_{\mathrm{fc,is}} = \exp\left(k_{\mathrm{n}} \cdot V_{\mathrm{fc,is}}\right) \quad \text{(A.14)}$$

$k_{\mathrm{d,n}}$ is a parameter which depends on the number of samples according to Table A.5;

$V_{\mathrm{fc,is}}$ is the coefficient of variation of the core strength according to EN 13791, but not smaller than 0,08;

k_{n} is the parameter which depends on the number of samples and has been used to calculate $f_{\mathrm{ck,is}}$ according to EN 13791:2019, Table 6 (see also Table A.5).

(8) If the reinforcement yield strength is assessed from tests on samples taken from the existing structure, the partial safety factor γ_{S} may be adjusted using Formula (A.1), where Formulae (A.2) and (A.5) should be replaced by Formulae (A.15) and (A.16), respectively:

$$V_{\mathrm{RS}} = \sqrt{V_{\mathrm{fy,corr}}^2 + V_{\mathrm{d}}^2 + V_{\theta\mathrm{s}}^2} \quad \text{(A.15)}$$

and

$$\mu_{\mathrm{RS}} = \mu_{\mathrm{fy,corr}} \cdot \mu_{\mathrm{d}} \cdot \mu_{\theta\mathrm{s}} \quad \text{(A.16)}$$

where

V_{d}, $V_{\theta\mathrm{s}}$, μ_{d} and $\mu_{\theta\mathrm{s}}$ are taken from Table A.3 or updated considering the cases (b), (c), (e), (g) or (h) defined in Tables A.1 (NDP) and A.2 (NDP);

$$V_{\mathrm{fy,corr}} = \frac{k_{\mathrm{d,n}}}{\alpha_{\mathrm{R}} \cdot \beta_{\mathrm{tgt}}} V_{\mathrm{fy}} \quad \text{(A.17)}$$

$$\mu_{\mathrm{fy,corr}} = exp\left(k_{\mathrm{n}} \cdot V_{\mathrm{fy}}\right) \quad \text{(A.18)}$$

$k_{\mathrm{d,n}}$ is a parameter which depends on the number of samples according to Table A.5;

V_{fy} is the coefficient of variation of reinforcement obtained from tests;

k_{n} is the parameter which depends on the number of samples and has been used to calculate V_{fy} (see Table A.5).

Table A.5 — Values of k_{n} and $k_{\mathrm{d,n}}$ as function of the number of test results *n* used to evaluate the insitu concrete compressive strength in the test region or to evaluate the yield strength of the reinforcement

n	**4**	**5**	**8**	**10**	**12**	**16**	**20**	**30**	**∞**
k_{n}	2,63	2,33	2,00	1,92	1,87	1,81	1,76	1,73	1,645
$k_{\mathrm{d,n}}$ (for $\alpha_{\mathrm{R}} \cdot \beta_{\mathrm{tgt}}$ = 3,04)	11,40	7,85	5,07	4,51	4,19	3,85	3,64	3,44	3,04

Annex B
(normative)

Time dependent behaviour of materials: strength, creep, shrinkage and elastic strain of concrete and relaxation of prestressing steel

B.1 Use of this annex

(1) This Normative Annex contains more detailed provisions for the time dependent behaviour of materials, i.e. strength, creep, shrinkage and elastic strains and relaxation of prestressing steel.

B.2 Scope and field of application

(1) This Normative Annex applies to all types of members and structures. For members and structures very sensitive to deformations, testing of the elastic modulus, creep and shrinkage should be carried out. For more accurate prediction of relaxation losses, results of relaxation tests on the prestressing steel should be considered.

B.3 General

NOTE 1 Both creep and shrinkage are subdivided into two components, basic creep and drying creep or basic shrinkage and drying shrinkage, respectively, due to the pronounced effect of the ambient climate conditions on the magnitude and the kinetics of the time-dependent deformations.

NOTE 2 The drying shrinkage strain develops slowly, since it is a function of the diffusion controlled migration of the water through the hardened concrete, which is affected by the size and shape, and the temperature of the member. The basic shrinkage strain develops during hardening of the concrete: the major part therefore develops in the early days after casting. Basic shrinkage is a function of the water/cement ratio and thus of the concrete strength. It is particularly relevant when new high strength concrete is cast against hardened concrete. For normal strength concrete ε_{cbs} is rather low. For bond between old and new/young concrete total differential shrinkage should is relevant (risk of cracking).

NOTE 3 The influence of the seasonal change of the ambient conditions during the year (relative humidity and temperature) on the development of the drying shrinkage is not considered in the model. This influence of the season on the development of the drying shrinkage after casting can be considered.

NOTE 4 The data base behind the creep and shrinkage models represents most of the creep and shrinkage investigations conducted around the world within the years 1960 and 2000. New binder type concretes like for instance concretes with a high amount (beyond the maximum values defined in EN 206 or national annexes) of fly ash or ground granulated blast-furnace slag (GGBS) are not sufficiently represented by the models.

NOTE 5 The indices “cm” or “ck” of values $f_{cm} = f_{cm,ref}$ or $f_{ck} = f_{ck,ref}$ mean any specified value between 28 days and 91 days.

(1) The early strength development of concrete with time (slow, normal, rapid) is subdivided into the Classes CS, CN and CR according to Table B.1 (NDP), which are subsequently defined from the

cement/binder characteristics and compositions (applied in Formulae (B.2) and (B.17) and Tables B.2, B.3 and D.1):

Table B.1 (NDP) — Strength development classes of concrete

Class	Composition and properties of the binder		
	Type of cement	Cement strength class [a], e.g.	Composition of the binder (cement and SCM [b])
CS	CEM III CEM II/B	32,5 N 42,5 N	Portland cement clinker and more than 65 M.-% of ground granulated blast furnace slag (ggbs) or more than 35 M.-% of fly ash (fa)
CN	CEM II CEM I	42,5 N 32,5 R	Portland cement clinker and more than 35 M.-% but less than 65 M.-% of ground granulated blast furnace slag (ggbs) or more than 20 M.-% but less than 35 M.-% of fly ash (fa)
CR	CEM I	42,5 R 52,5 N; 52,5 R	—

[a] According to EN 197-1.
[b] Secondary cementitious materials (SCM) considered as cement replacement for the water-binder-ratio according to EN 206.

NOTE 6 Strength development classes are defined in Table B.1 (NDP) unless the National Annex gives different specifications.

(2) The relations for creep and shrinkage given in Annex B predict the time-dependent mean cross-section behaviour of an unreinforced member moist cured at normal temperature for not longer than 14 days. These values should be used in conjunction with 7.3.1(6), 7.3.1(7), 7.4.2, 7.6.4, 9.1(4) and 9.3.3(1).

(3) Unless special provisions are given, the relations are valid for structural concrete (12 MPa $\leq f_{ck} \leq$ 100 MPa) subjected to a constant compressive stress $\sigma_c \leq 0{,}40 f_{cm}(t_0)$ under a quasi-permanent combination at an age at loading t_0 (unless adjusted by Formula (B.17)) and exposed to a mean relative humidity in the range of 20 % to 100 % at a mean temperature in the range of 5 °C to 30 °C. The age at loading should be at least 1 day. The relations consider shrinkage deformation beginning from a concrete age of 1 day. For variable stresses see B.8.

(4) The relations may be applied as well to concrete in tension.

(5) Both basic and drying shrinkage are isotropic strains. Creep is an anisotropic strain. For uncracked concrete, Poisson's ratio may be taken as 0,2 for basic and for drying creep.

B.4 Development of concrete strength and stiffness with time

NOTE 1 The mechanical properties of concrete at an age t depend on the composition of the concrete, the composition and properties of the binder, the temperature history, the curing conditions and the drying conditions.

(1) For a mean temperature of 20 °C and curing in accordance with EN 13670 the compressive strength of concrete at various ages $f_{cm}(t)$ may be estimated from Formulae (B.1) and (B.2). For other temperatures, temperature adjusted concrete age may be considered according to Formula (B.18).

$$f_{\mathrm{cm}}(t) = \beta_{\mathrm{cc}}(t) \cdot f_{\mathrm{cm}} \tag{B.1}$$

with

$$\beta_{\mathrm{cc}}(t) = \exp\left[s_{\mathrm{C}}\left(1-\sqrt{\frac{t_{\mathrm{ref}}}{t}}\right)\sqrt{\frac{28}{t_{\mathrm{ref}}}}\right];\ t \leq t_{\mathrm{ref}} \tag{B.2}$$

where

s_C is a coefficient which depends on early strength development of the concrete and the concrete strength as defined in Table B.2.

The values of s_C in Table B.2 are valid for concrete covered by the data considered, see B.3, Note 4. In other cases, e.g. where there is a high content of pozzolanic binder, experimental verification should be carried out for structures which are sensitive to the development of the concrete strength, deformations and stresses.

Table B.2 — Values of the coefficient s_C for different early strength development of concrete and concrete strength

Concrete strength	Values of the coefficient s_C		
	Class CS[a]	Class CN[a]	Class CR[a]
$f_{ck} \leq 35$ MPa	0,6	0,5	0,3
35 MPa $< f_{ck} <$ 60 MPa	0,5	0,4	0,2
$f_{ck} \geq 60$ MPa	0,4	0,3	0,1
[a] See B.3(1).			

NOTE 2 The development of tensile strength with time is strongly influenced by the early strength cement class, curing and drying conditions as well as by the dimensions of the structural members.

(2) It may be assumed that the tensile strength $f_{ctm}(t)$ is equal to:

$$f_{ctm}(t) = \beta_{cc}{}^{0,6}(t) \cdot f_{ctm} \tag{B.3}$$

where $\beta_{cc}(t)$ follows from Formula (B.2) and the values for f_{ctm} are given in Table 5.1.

(3) Where the development of the tensile strength with time is important, tests should be carried out considering the exposure conditions and the dimensions of the structural member.

(4) The development of the modulus of elasticity with time may be estimated by:

$$E_{cm}(t) = \beta_{cc}{}^{1/3}(t)\ E_{cm} \tag{B.4}$$

where $\beta_{cc}(t)$ follows from Formula (B.2) and E_{cm} from Formula (5.1).

B.5 Basic formulae for determining the creep coefficient

(1) The total mean creep coefficient $\varphi(t,t_0)$ may be calculated from Formula (B.5). For characteristic values of the creep coefficient, see (8).

$$\varphi(t,t_0) = \varphi_{bc}(t,t_0) + \varphi_{dc}(t,t_0) \tag{B.5}$$

where

$\varphi_{bc}(t,t_0)$ is the basic creep coefficient, Formula (B.6);

$\varphi_{dc}(t,t_0)$ is the drying creep coefficient, Formula (B.9);

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

t_0 is the age of concrete at loading in days adjusted according to Formulae (B.17).

(2) The basic creep coefficient $\varphi_{bc}(t,t_0)$ may be estimated from:

$$\varphi_{bc}(t,t_0) = \beta_{bc,f_{cm}} \cdot \beta_{bc,t-t_0} \tag{B.6}$$

where

$\beta_{bc,f_{cm}}$ is a function to describe the effect of concrete strength on basic creep, see Formula (B.7);

$\beta_{bc,t-t_0}$ is a function to describe the time development of basic creep, see Formula (B.8)

$$\beta_{bc,f_{cm}} = \frac{1,8}{(f_{cm,28})^{0,7}} \tag{B.7}$$

$$\beta_{bc,t-t_0} = \ln\left(\left(\frac{30}{t_{0,adj}} + 0,035\right)^2 (t-t_0)+1\right) \tag{B.8}$$

where $t_{0,adj}$ is the adjusted age at loading in days according to Formula (B.17).

(3) The drying creep coefficient φ_{dc} may be estimated from:

$$\varphi_{dc}(t,t_0) = \beta_{dc,f_{cm}} \cdot \beta_{dc,RH} \cdot \beta_{dc,t_0} \cdot \beta_{dc,t-t_0} \tag{B.9}$$

where

$\beta_{dc,f_{cm}}$ is a function to describe the effect of concrete strength on drying creep, see Formula (B.10);

$\beta_{dc,RH}$ is a function to describe the effect of relative humidity and notional size on drying creep, see Formula (B.11);

β_{dc,t_0} is a function to describe the effect of the adjusted concrete age at loading on drying creep, see Formula (B.12);

$\beta_{dc,t-t_0}$ is a function to describe the time development of drying creep, see Formula (B.13);

$$\beta_{dc,f_{cm}} = \frac{412}{{f_{cm,28}}^{1,4}} \tag{B.10}$$

$$\beta_{dc,RH} = \frac{1 - \dfrac{RH}{100}}{\sqrt[3]{0,1\dfrac{h_n}{100}}} \tag{B.11}$$

$$\beta_{dc,t_0} = \frac{1}{0,1 + t_{0,adj}^{0,2}} \tag{B.12}$$

$$\beta_{dc,t-t_0} = \left[\frac{(t-t_0)}{\beta_h + (t-t_0)}\right]^{\gamma(t_{0,adj})} \tag{B.13}$$

with

$$\gamma(t_{0,adj}) = \frac{1}{2,3 + \dfrac{3,5}{\sqrt{t_{0,adj}}}} \tag{B.14}$$

$$\beta_h = 1,5h_n + 250\alpha_{f_{cm}} \leq 1\,500\alpha_{f_{cm}} \tag{B.15}$$

and

$$\alpha_{f_{cm}} = \left(\frac{35}{f_{cm,28}}\right)^{0,5} \tag{B.16}$$

where

RH is the relative humidity [%] of the ambient environment;

h_n $= 2A_c/u$, is the notional size [mm] of the member, where A_c is the cross-section area and u is the perimeter of the structural member in contact with the atmosphere;

$t_{0,adj}$ is the adjusted age at loading in days adjusted according to Formula (B.17);

$t - t_0$ Is the non-adjusted duration of loading [d].

(4) The effect of early strength development of concrete on the creep coefficient of concrete may be considered by modifying the age at loading t_0 to $t_{0,adj}$ according to Formula (B.17):

$$t_{0,\mathrm{adj}} = t_{0,\mathrm{T}} \left[\frac{9}{2 + t_{0,\mathrm{T}}^{1,2}} + 1 \right]^{\alpha_{\mathrm{SC}}} \geq 0,5 \tag{B.17}$$

where

$t_{0,T}$ is the age of concrete at loading in days adjusted according to the concrete temperature as given in Formula (B.18). For $T = 20$ °C, $t_{0,T}$ corresponds to t_0;

α_{SC} is a coefficient which depends on the strength development of concrete (see B.3(1)):

$\alpha_{SC} = -1$ for class CS;

$\alpha_{SC} = 0$ for class CN;

$\alpha_{SC} = 1$ for class CR.

(5) The effect of elevated or reduced concrete temperatures within the range of 0 °C ≤ T ≤ +80 °C on the maturity of concrete may be considered by adjusting the concrete age according to Formula (B.18):

$$t_{\mathrm{T}} = \sum_{i=1}^{n} \Delta t_{\mathrm{i}} \cdot \exp\left(13,65 - \frac{4000}{273 + T(\Delta t_{\mathrm{i}})}\right) \tag{B.18}$$

where

t_T is the temperature-adjusted concrete age in days which replaces t in the corresponding formulae;

Δt_i is the number of days where a temperature T prevails;

$T(\Delta t_i)$ is the mean concrete temperature in °C during the time period Δt_i.

(6) For high stress levels in the range of $0,4f_{cm}(t_0) < \sigma_c \leq 0,6f_{cm}(t_0)$ creep is no longer proportional to the applied stress. This non-linearity of creep may be considered by adjusting the creep coefficient according to Formula (B.19):

$$\varphi_\sigma(t,t_0) = \varphi(t,t_0) \cdot \exp[1,5(k_\sigma - 0,4)] \quad \text{for } 0,4 < k_\sigma \leq 0,6 \tag{B.19}$$

where

$\varphi_\sigma(t,t_0)$ is the non-linear notional creep coefficient;

$k_\sigma = \dfrac{\sigma_c}{f_{cm}(t_0)}$ is the stress-strength ratio.

(7) Creep strains may be calculated by means of the Formula (B.20):

$$\varepsilon_{cc}(t,t_0) = \varphi(t,t_0)\frac{\sigma_c}{E_{c,28}} \tag{B.20}$$

where

$$E_c = \alpha_c \cdot E_{cm,28} \tag{B.21}$$

$$\alpha_c = \frac{1}{0{,}8 + 0{,}2\dfrac{f_{cm,28}}{88}} \geq 1{,}0 \tag{B.22}$$

where E_c is the tangent and E_{cm} is the secant modulus of elasticity.

(8) Lower and upper characteristic values of the creep coefficient φ_k may be taken as:

— $\varphi_{k;0,10} = 0{,}6\varphi(t,t_0)$ and $\varphi_{k;0,05} = 0{,}5\varphi(t,t_0)$;

— $\varphi_{k;0,90} = 1{,}4\varphi(t,t_0)$ and $\varphi_{k;0,95} = 1{,}5\varphi(t,t_0)$.

Characteristics values of creep should be used in cases where displacements are specially designed and accounted for, and they have significant influence, e.g. when displacement range of bearings, joints and expansion devices are designed.

B.6 Basic formulae for determining the shrinkage strain

(1) The total mean shrinkage or swelling strain $\varepsilon_{cs}(t,t_s)$ may be calculated from Formula (B.23). For characteristic values of shrinkage or swelling strain, see (4).

$$\varepsilon_{cs}(t,t_s) = \varepsilon_{cbs}(t) + \varepsilon_{cds}(t - t_s) \tag{B.23}$$

where shrinkage is subdivided into the basic shrinkage $\varepsilon_{cbs}(t)$ which occurs even if no moisture loss is possible:

$$\varepsilon_{cbs}(t) = \varepsilon_{cbs,f_{cm}} \cdot \beta_{bs,t} \cdot \alpha_{NDP,b} \tag{B.24}$$

and the drying shrinkage $\varepsilon_{cds}(t,t_s)$ giving the additional shrinkage if moisture loss occurs:

$$\varepsilon_{cds}(t - t_s) = \varepsilon_{cds,f_{cm}} \cdot \beta_{RH} \cdot \beta_{ds,t-t_s} \cdot \alpha_{NDP,d} \tag{B.25}$$

where

$\varepsilon_{cbs,f_{cm}}$	is the notional basic shrinkage coefficient, see Formula (B.26);
$\varepsilon_{cds,f_{cm}}$	is the notional drying shrinkage coefficient, see Formula (B.28);
$\beta_{bs,t}$	is a function to describe the time development of basic shrinkage, see Formula (B.27);
β_{RH}	is a coefficient to consider the effect of relative humidity on drying shrinkage, see Formulae (B.29) to (B.32);
$\beta_{ds,t-t_s}$	is a function to describe the time development of drying shrinkage, see Formula (B.33);
t_s	is the concrete age at the beginning of drying [d];
$t - t_s$	is the real (non-adjusted) duration of drying [d].

NOTE 1 The values of $\alpha_{NDP,b}$ and $\alpha_{NDP,d}$ are 1,0 unless the National Annex gives different values.

(2) The basic shrinkage $\varepsilon_{cbs}(t)$ according to Formula (B.24) may be estimated by applying Formulae (B.26) and (B.27):

$$\varepsilon_{\mathrm{cbs},f_{\mathrm{cm}}} = \alpha_{\mathrm{bs}} \left(\frac{f_{\mathrm{cm,28}}}{60 + f_{\mathrm{cm,28}}}\right)^{2,5} \cdot 10^{-6} \tag{B.26}$$

$$\beta_{\mathrm{bs,t}} = 1 - \exp\left(-0,2\sqrt{t}\right) \tag{B.27}$$

where

α_{bs} is a coefficient which depends on the early strength development of concrete, see Table B.3.

Table B.3 — Coefficients α_{bs} and α_{ds} used in Formulae (B.26) and (B.28), respectively

Early strength development of concrete[a]	α_{bs}	α_{ds}
Low early strength: Class CS	800	3
Ordinary early strength: Class CN	700	4
High early strength: Class CR	600	6
[a] See B.3(1).		

(3) For drying shrinkage $\varepsilon_{cds}(t\text{-}t_s)$ according to Formula (B.25), Formulae (B.28) to (B.33) may be applied:

$$\varepsilon_{\mathrm{cds},f_{\mathrm{cm}}} = (220+110\alpha_{\mathrm{ds}}) \cdot \exp\left(-0,012 f_{\mathrm{cm,28}}\right) \cdot 10^{-6} \tag{B.28}$$

$$\beta_{\mathrm{RH}} = 1,55 \cdot \left[1-\left(\frac{RH}{RH_{\mathrm{eq}}}\right)^{3}\right] \quad \text{for } 20\ \% \leq RH \leq RH_{\mathrm{eq}} \tag{B.29}$$

$$\beta_{\mathrm{RH}} = 1,55 \cdot \left[1-\left(\frac{RH}{RH_{\mathrm{eq}}}\right)^{2}\right] \quad \text{for } RH_{\mathrm{eq}} < RH < 100\ \% \tag{B.30}$$

$$\beta_{\mathrm{RH}} = 1,55 \cdot \left[1-\left(\frac{RH}{RH_{\mathrm{eq}}}\right)^{2}\right] - 0,25 \quad \text{for } RH = 100\ \% \tag{B.31}$$

$$RH_{\mathrm{eq}} = 99 \cdot \left(\frac{35}{f_{\mathrm{cm,28}}}\right)^{0,1} \leq 99 \tag{B.32}$$

$$\beta_{\mathrm{ds},t-t_s} = \left[\frac{(t-t_s)}{0,035 h_{\mathrm{n}}^2 + (t-t_s)}\right]^{0,5} \tag{B.33}$$

where

α_{ds} is a coefficient depending on the early strength development of concrete, see Table B.3;

RH_{eq} is the internal relative humidity of concrete [%] at equilibrium. It considers self-desiccation (relevant for high performance concrete).

NOTE 2 RH =100% means that capillary suction becomes active, i.e. the member is submerged or sufficient free water is available at the member's surface.

(4) Lower and upper characteristic values of total shrinkage or swelling strain ε_{cs} may be taken as:

— $\varepsilon_{cs;0,10} = 0{,}6\varepsilon_{cs}\ (t,t_s)$ and $\varepsilon_{cs;0,05} = 0{,}5\varepsilon_{cs}\ (t,t_s)$;

— $\varepsilon_{cs;0,90} = 1{,}4\varepsilon_{cs}\ (t,t_s)$ and $\varepsilon_{cs;0,95} = 1{,}5\varepsilon_{cs}\ (t,t_s)$.

Characteristic values of shrinkage should be used when the displacement range of bearings and expansion devices are designed.

B.7 Tests on elastic deformations, creep and shrinkage

(1) If a member or a structure is very sensitive to deformations (e.g. long span prestressed concrete girder bridges) design assumptions for elastic modulus, creep and shrinkage behaviour should be verified by testing before execution.

(2) Tests should also be carried out, for sensitive structures only, if:

— the design with upper and lower bound values of the creep coefficient (see B.5(8)) and of the total shrinkage or swelling strain (see B.6(4)) gives no satisfactory results with respect to the limit states;

— if concrete is used for which the prediction models considered does not apply, see B.3, NOTE 4;

— the environmental conditions at the structure are not covered by the prediction models given in B.5 and B.6.

(3) For the tests, the guidelines given below should be followed:

— elastic deformation, modulus of elasticity: EN 12390-13;

— creep: EN 12390-17;

— shrinkage: EN 12390-16.

(4) For creep and shrinkage tests a minimum duration of loading and drying, respectively, of 3 months shall be kept.

(5) From the test results an adapted creep or shrinkage model may be obtained by means of the subsequent procedure.

The creep coefficient should be derived as:

$$\varphi(t,t_0) = \xi_{\mathrm{bc1}} \cdot \varphi_{\mathrm{bc}}(t,t_0) + \xi_{\mathrm{dc1}} \cdot \varphi_{\mathrm{dc}}(t,t_0) \tag{B.34}$$

where the adapted time development functions (see B.5) may be obtained from:

$$\beta_{\mathrm{bc},t-t_0} = \ln\left(\left(\frac{30}{t_{0,\mathrm{adj}}}+0{,}035\right)^2 \frac{(t-t_0)}{\xi_{\mathrm{bc2}}} + 1\right) \tag{B.35}$$

$$\beta_{\mathrm{dc},t-t_0} = \left[\frac{(t-t_0)}{\beta_{\mathrm{h}}\xi_{\mathrm{dc2}} + (t-t_0)}\right]^{\gamma(t_{0,\mathrm{adj}})} \tag{B.36}$$

The shrinkage strain should be derived from:

$$\varepsilon_{\mathrm{cs}}(t,t_{\mathrm{s}}) = \xi_{\mathrm{bs1}} \cdot \varepsilon_{\mathrm{cbs}}(t) + \xi_{\mathrm{ds1}} \cdot \varepsilon_{\mathrm{cds}}(t-t_s) \tag{B.37}$$

where the adapted time development functions (see B.6) result from:

$$\beta_{\mathrm{bs,t}} = 1 - \exp\left(-0{,}2\xi_{\mathrm{bs2}}\sqrt{t}\right) \tag{B.38}$$

$$\beta_{\mathrm{ds},t-t_s} = \sqrt{\frac{(t-t_s)}{0{,}035\xi_{\mathrm{ds2}} \cdot h_{\mathrm{n}}^2 + (t-t_{\mathrm{s}})}} \tag{B.39}$$

Parameters ξ_{bc1}, ξ_{bc2}, ξ_{dc1} and ξ_{dc2} for creep as well as parameters ξ_{bs1}, ξ_{bs2}, ξ_{ds1} and ξ_{ds2} for shrinkage should be determined such as to minimise the sum of the squares of the differences between the model estimation and the experimental results. The strain readings from the experimental results to be used for the calibration should be taken at constant intervals in the logarithmic time scale, i.e. in a geometric progression of reading times, in accordance with the test guidelines.

Once the parameters are obtained, the inputs of the model may be varied to meet the insitu conditions as long as these conditions remain within the validity of the prediction models according to B.3.

NOTE Calibrating the models with experimental results reduces the coefficient of variation from 30 % to 10 % but does not make the variation disappear.

B.8 Detailed analysis for creep at variable loading

(1) A constant stress $\sigma_{\mathrm{c}}(t_0)$ applied at time t_0 leads to the subsequent stress-dependent strain $\varepsilon_{\mathrm{c}\sigma}(t,t_0)$ at time t, which corresponds to the sum of the elastic strain $\varepsilon_{\mathrm{ci}}(t_0)$ and the creep strain $\varepsilon_{\mathrm{cc}}(t,t_0)$ and may be expressed as:

$$\varepsilon_{\mathrm{c}\sigma}(t,t_0) = \varepsilon_{\mathrm{ci}}(t_0) + \varepsilon_{\mathrm{cc}}(t,t_0) = \sigma_{\mathrm{c}}(t_0)\left[\frac{1}{E_{\mathrm{c}}(t_0)} + \frac{\varphi(t,t_0)}{E_{\mathrm{c}}(28)}\right] = \sigma_{\mathrm{c}}(t_0)J(t,t_0) \tag{B.40}$$

where

- $J(t,t_0)$ is the creep function or creep compliance, representing the total stress-dependent strain per unit stress;
- $E_{\mathrm{c}}(t_0)$ is the tangent modulus of elasticity at the time of loading t_0;
- $E_{\mathrm{c}}(28)$ is the tangent modulus of elasticity at the concrete age of 28 days.

(2) Within the range of service stresses $\sigma_{\mathrm{c}} \leq 0{,}40 f_{\mathrm{cm}}$, creep may be assumed to be linearly related to stress. Superposition may be applied. The strain caused by the stress history $\sigma_{\mathrm{c}}(t)$ may be obtained by decomposing the stress history to small increments $\Delta\sigma_{\mathrm{c}}$ applied at times τ_{i}, and summing up the corresponding strains (see Formula (B.41)):

$$\varepsilon_{\mathrm{c}\sigma}(t,\sigma_{\mathrm{c}}) = \sigma_{\mathrm{c}}(t_0) \cdot J(t,t_0) + \sum_{i=1}^{n} \Delta\sigma_{\mathrm{c}}(\tau_{\mathrm{i}}) \cdot J(t,\tau_{\mathrm{i}}) \tag{B.41}$$

If $\sigma_{\mathrm{c}}(t)$ is a continuous function the expression $\Delta\sigma_{\mathrm{c}}(\tau_{\mathrm{i}})$ may be replaced by $\frac{\partial\sigma_{\mathrm{c}}(\tau)}{\partial\tau} \cdot d\tau$.

With t as the age of concrete at the moment considered and the time τ as the integration variable ranging from $t_0 \leq \tau \leq t$, Formula (B.41) may then be expressed as follows:

$$\varepsilon_{\mathrm{c}\sigma}(t,\sigma_{\mathrm{c}}) = \sigma_{\mathrm{c}}(t_0) \cdot J(t,t_0) + \int_{t_0}^{t} \frac{\partial\sigma_{\mathrm{c}}(\tau)}{\partial\tau} \cdot d\tau \cdot J(t,\tau_{\mathrm{i}}) \tag{B.42}$$

B.9 Relaxation of prestressing steel

(1) The design calculations for the loss of tendon stress due to relaxation of the prestressing steel should be based on the value of ρ_{1000}, the relaxation loss at 1 000 hours after tensioning to an initial load

of either 70 % or 80 % of the actual strength of the prestressing steel and at a mean temperature of 20 °C determined according to EN ISO 15630-3.

(2) The values for ρ_{1000} should be taken from Table B.4 unless there is more accurate data available either from testing of the actual prestressing steel or from testing of the same type of prestressing steel as part of continuous surveillance of the producer.

Table B.4 — Relaxation of prestressing steel

Maximum relaxation at 1 000 hours [a]	$\boldsymbol{\rho_{1000}}$		
Type of prestressing steel	**Wires and Strands**	**Bars** $\boldsymbol{\phi \leq 15}$ **mm**	**Bars** $\boldsymbol{\phi > 15}$ **mm**
for initial stress σ_{pi} of 50 % of actual tensile strength f_p	0 %	0 %	0 %
for initial stress σ_{pi} of 70 % of actual tensile strength f_p	2,5 %	6,0 %	4,0 %
for initial stress σ_{pi} of 80 % of actual tensile strength f_p	4,5 %	–	–

NOTE 1 Values apply for all prestressing steel strength classes.

NOTE 2 In the absence of more detailed information, the relaxation loss can be interpolated linearly between the initial stress values.

NOTE 3 $f_p = f_{pk}$ can be assumed if the actual strength of prestressing steel is not known.

NOTE 4 Relaxation losses are very sensitive to the temperature of the prestressing steel.

[a] Relaxation losses apply at a mean temperature of 20 °C.

(3) The evolution of relaxation loss over time may be assumed to follow Formula (B.43):

$$\frac{\Delta\sigma_{pr}}{\sigma_{pi}} = \rho_{1000}\left(\frac{24 \cdot t}{1\,000}\right)^{k_\rho} \tag{B.43}$$

where

$\Delta\sigma_{pr}$ is the absolute value of loss of stress in the prestressing steel;

σ_{pi} is the initial stress in the prestressing steel;

k_ρ is a coefficient which describes the evolution of relaxation losses over time, in lieu if more accurate data, a value of $k_\rho = 0{,}16$ may be assumed;

ρ_{1000} is the relaxation loss [%] after 1 000 hours at initial stress σ_{pi}.

(4) If relaxation loss is taken from testing in accordance with EN ISO 15630-3 up to a duration of at least 1 000 hours, the evolution of relaxation loss $\Delta\sigma_{pr}$ over time at initial stress σ_{pi} may be assumed as best fit curve of the actual test results according to Formula (B.44):

$$(\Delta\sigma_{pr}/\sigma_{pi}) = b \cdot t^{a} \tag{B.44}$$

where a and b are coefficients of the best fit curve of relaxation tests performed at initial stress σ_{pi}.

(5) The long term (final) value of the relaxation loss may be estimated for a time t equal to the design service life of the structure or as specified for the project using Formulae (B.43) or (B.44).

(6) In cases where the mean temperature of prestressing steel is 30 °C or higher the relaxation loss should be determined based on testing in accordance with EN ISO 15630-3 at initial stress σ_{pi} and

assumed actual mean temperature up to a duration of at least 1 000 hours and then extrapolated over time in accordance with Formula (B.44).

NOTE Where prestressed concrete members are subjected to heat treatment (e.g. by steam), refer to Clause 13 for relaxation losses during heat treatment.

Annex C
(normative)

Requirements for materials

C.1 Use of this annex

(1) This Normative Annex contains additional provisions for material properties with minimum or maximum values or an interval of values for which the design provisions of this Eurocode apply. This Normative Annex also applies to precast concrete products according to EN 13369 designed according to this standard.

C.2 Scope and field of application

(1) This Normative Annex applies to materials and products in accordance with Clause 5 and normative Annex Q.

C.3 Concrete

C.3.1 Normal weight, heavy weight and Lightweight Aggregate Concrete (LWAC)

(1) In the design according to this Eurocode concrete strength classes according to EN 206 should be specified. Intermediate classes may be specified if permitted in the National Annex, see 5.1.3(3). Unless noted otherwise D_{lower} should be equal or greater than 8mm.

C.4 Reinforcing steel

C.4.1 Carbon reinforcing steel

(1) Unless the relevant standard for reinforcing steel gives different values, carbon reinforcing steel products may be classified in strength classes B and in ductility classes and shall comply with the requirements given in Tables C.1 and C.2.

(2) Reinforcing steel shall be clearly and permanently identifiable with respect to strength and ductility class according to Tables C.1 and C.2.

Table C.1 — Strength properties of carbon reinforcing steel

Property	Reinforcing steel strength class[a]					
	B400	**B450**	**B500**	**B550**	**B600**	**B700**
Bars including de-coiled bars[c] Tensile Yield strength R_e or $R_{p0,2}$ [MPa] (5 % quantile)	≥ 400	≥ 450	≥ 500	≥ 550	≥ 600	≥ 700
Fatigue stress range $2\sigma_a$ [MPa] for $N \geq 2 \times 10^6$ cycles based on an upper stress limit of $0{,}6f_{yk}$ (10 % quantile)[b]	160 for bars and de-coiled bars $\phi \leq 12$ mm 140 for bars and de-coiled bars 12 mm $< \phi \leq 16$ mm 130 for bars and de-coiled bars 16 mm $< \phi \leq 20$ mm 130 for straight bars $\phi > 20$ mm 100 for welded fabric $\phi \leq 12$ mm					

Property	Reinforcing steel strength class[a]					
	B400	B450	B500	B550	B600	B700
	80 for welded fabric $\phi > 12$ mm					
Minimum relative rib area f_R (5 % quantile)	0,035 for $\phi \leq 6$ mm 0,040 for 6 mm $< \phi \leq 12$ mm 0,056 for $\phi > 12$ mm					

[a] All strength classes apply unless the National Annex excludes specific classes, see 5.2.2(1).

[b] Different fatigue properties can be set in the relevant standard by specifying higher values of the fatigue stress range and/or increasing the number of cycles that is to be confirmed by testing. No specific fatigue properties are required if the reinforcement is used only for predominantly static loading.

[c] De-coiled products are not part of EN 10080. To provide sufficient confidence in the reinforcement properties after de-coiling National regulations can specify additional requirements.

Table C.2 — Ductility properties of carbon reinforcing steel

Properties for weldable reinforcement	Ductility Class		
	A	B	C
Stress ratio of R_m / R_e (10 % quantile)	$\geq 1{,}05$	$\geq 1{,}08$	$\geq 1{,}15$ $< 1{,}35$
Tensile yield strength ratio of $R_{e,act}/R_{e,nom}$ (10 % quantile)	–	$\leq 1{,}3$	$\leq 1{,}3$
Elongation at maximum force A_{gt} [%](10 % quantile)	$\geq 2{,}5$	$\geq 5{,}0$	$\geq 7{,}5$
Bendability	Pass bend and/or rebend test according to the relevant standard for reinforcing steel. For rebending of reinforcing steel on site see requirements in EN 13670.		

NOTE EN 10080 specifies tensile yield strength R_e as a characteristic value 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 or $R_{p0,2}$. However, based on the methods of evaluation and verification the yield strength R_e given in EN 10080 can be taken as f_{yk}.

(3) In welded fabric the declared shear force shall be not lower than the specified minimum value of the shear force F_s of welded joints. The specified minimum value is:

$$F_s \geq 0{,}25 \cdot R_{e,nom} \cdot A_n \tag{C.1}$$

where $R_{e,nom}$ is the nominal characteristic yield strength and A_n is the nominal cross sectional area of either:

a) the larger wire at the joint in a single wire welded fabric; or

b) one of the twin wires in a twin wire welded fabric (twin wires in one direction).

C.4.2 Stainless reinforcing steel

(1) Unless the relevant standard for stainless reinforcing steel gives different values, stainless reinforcing steel products may be classified in strength classes B and in ductility classes and shall comply with the requirements given in Tables C.3 and C.4.

NOTE prEN 10370 specifies a 0,2 % proof strength $R_{p0,2k}$, as a characteristic, based on the long-term quality level of production. In contrast $f_{0,2k}$ is the characteristic 0,2 % proof strength based on only that reinforcement used in a particular structure. There is no direct relationship between $f_{0,2k}$ and the characteristic $R_{p0,2k}$. However the methods of evaluation and verification of 0,2 % proof strength given in prEN 10370 provide a sufficient check for obtaining $f_{0,2k}$.

(2) Stainless reinforcing steel shall be clearly and permanently identifiable with respect to strength and ductility class according to Tables C.3 and C.4.

Table C.3 — Strength properties of stainless reinforcing steel

Property	Reinforcing steel strength class [a]					
	B400	B450	B500	B550	B600	B700
Bars and de-coiled[c] products Yield strength $R_{p0,2k}$ [MPa] (5 % quantile)	≥ 400	≥ 450	≥ 500	≥ 550	≥ 600	≥ 700
Fatigue stress range $2\sigma_a$ [MPa] for $N \geq 5 \times 10^6$ cycles based on a stress ratio $\sigma_{min}/\sigma_{max} = 0,2$ for strength class B500 only for stainless steel (10 % quantile)[b]	160 for $\phi \leq 12$ mm 140 for 12 mm $< \phi \leq 16$ mm 130 for 16 mm $< \phi \leq 20$ mm 130 $\phi > 20$ mm 100 for welded fabric $\phi \leq$ 12mm 80 for welded fabric $\phi \geq$ 12mm					
Minimum relative rib area $f_{R,min}$ (5 % quantile)	0,039 for $\phi \leq 6$ mm 0,045 for 6,5 mm $\leq \phi \leq$ 8,5 mm 0,052 for 9 mm $\leq \phi \leq$ 10,5 mm 0,056 for 11 mm $\leq \phi \leq$ 50 mm					

[a] All strength classes apply unless the National Annex excludes specific classes, see Q.3(1).

[b] Different fatigue properties can be set in the relevant standard by specifying higher values of the fatigue stress range and/or increasing the number of cycles that is to be confirmed by testing.

[c] De-coiled products are not part of EN 10370. To provide sufficient confidence in the reinforcement properties after de-coiling National regulations can specify additional requirements.

Table C.4 — Ductility properties of stainless reinforcing steel

Properties for weldable reinforcement	Ductility Class		
	A	B	C
Stress ratio $R_m / R_{p0,2k}$, where R_m should be limited to stress at 7 % elongation (10 % quantile)	$\geq 1,05$	$\geq 1,08$	$\geq 1,15$ $< 1,35$

Properties for weldable reinforcement	Ductility Class		
	A	B	C
Tensile yield strength ratio of $R_{e,act}/R_{p0,2,nom}$ (10 % quantile)	–	≤ 1,3	≤ 1,3
Elongation at maximum load A_{gt} [%] (10 % quantile)	≥ 2,5	≥ 5,0	≥ 7,5
Bendability	Pass bend and/or rebend test according to the relevant standard for stainless reinforcing steel. For rebending of reinforcing stainless steel on site see requirements in EN 13670.		

C.5 Prestressing steel

(1) Unless the relevant standard for prestressing steel gives different values, prestressing steel products may be classified in strength classes Y and shall comply with the requirements in Tables C.5 to C.7.

NOTE EN 10138 (all parts) refers to the characteristicvalues of tensile yield strength $R_{p0,1}$ and tensile strength R_{pm} based on the long-term quality level of production. In contrast $f_{p0,1k}$ and f_{pk} are the characteristic tensile yield strength and the tensile strength based on only that prestressing steel required for the structure. There is no direct relationship between the two sets of values. However, based on the methods for evaluation and verification the characteristic values for the tensile yield strength $R_{p0,1}$ and tensile strength R_m , given in EN 10138 (all parts) may be taken as $f_{p0,1k}$ and f_{pk}.

(2) Each consignment of prestressing steel shall be accompanied by a certificate containing all the information necessary for its identification in accordance with 5.3.2(1).

Table C.5 — Strength, fatigue, ductility and relaxation properties of prestressing wires

Property		Wires — strength class			
		Y1570	**Y1670**	**Y1770**	**Y1860**
Strength	Tensile yield strength $R_{p0,1}$ [MPa], 5 % quantile	≥ 1380	≥ 1470	≥ 1550	≥ 1650
	Tensile strength R_m [MPa], 5 % quantile	≥ 1570	≥ 1670	≥ 1770	≥ 1860
	Tensile strength R_m [MPa] 95 % quantile	≤ 1800	≤ 1920	≤ 2030	≤ 2140
Fatigue[a]	Fatigue stress range [MPa] for $N \geq 2 \times 10^6$ cycles with an upper limit of 0,7 f_{pk} (10 % quantile)	200 for plain wire 180 for indented wire			
Ductility	Characteristic value of stress ratio $(R_m/R_{p0,1})_k$(10 % quantile)	≥ 1,10			
	Elongation at maximum load, A_{gt} [%] (10 % quantile)	≥ 3,5			
	Bendability	Pass minimum number of reverse bends according to the relevant standard for prestressing steel			
Relaxation[b]	Relaxation at 1 000 hours for initial stress of 70 % of actual tensile strength, ρ_{1000}	≤ 2,5 %			
	Relaxation at 1 000 hours for initial stress of 80 % of actual tensile strength, ρ_{1000}	≤ 4,5 %			
Stress-corrosion resistance	Stress-corrosion resistance at 80% of actual tensile strength	Stress-corrosion resistance should be tested in accordance with the relevant product standard and comply with corresponding specified values			

NOTE All strength classes apply unless the National Annex excludes specific classes, see 5.3.2(1).

[a] Different fatigue properties can be set in the relevant standard by specifying higher values of the fatigue stress range and/or increasing the number of cycles, that is to be confirmed by testing.

[b] Relaxation losses apply at a mean temperature of 20 °C. Relaxation losses are very sensitive to the temperature of the prestressing steel, see B.9.

Table C.6 — Strength, fatigue, ductility and relaxation properties of prestressing strands

Property		Strands — strength class			
		Y1770	**Y1860**	**Y1960**	**Y2060**
Strength	Tensile yield strength $R_{p0,1}$ [MPa], 5 % quantile	≥ 1560	≥ 1640	≥ 1740	≥ 1830
	Tensile strength R_m[MPa], 5 % quantile	≥ 1770	≥ 1860	≥ 1960	≥ 2060
	Tensile strength R_m [MPa], 95 % quantile	≤ 2030	≤ 2140	≤ 2250	≤ 2370
Fatigue[a]	Fatigue stress range [MPa] for $N \geq 2 \times 10^6$ cycles with an upper limit of 0,7 f_{pk} (10 % quantile)	190 for plain strand 170 for indented strand			
Ductility	Characteristic value of stress ratio $(R_m/R_{p0,1})_k$(10 % quantile)	≥ 1,10			
	Elongation at maximum load, A_{gt} [%] (10 % quantile)	≥ 3,5			
	Maximum D-value of deflected tensile test for 7-wire strand with d ≥ 12.5mm	≤ 28%			
Relaxation[b]	Relaxation at 1 000 hours for initial stress of 70 % of actual tensile strength, ρ_{1000}	≤ 2,5 %			
	Relaxation at 1 000 hours for initial stress of 80 % of actual tensile strength, ρ_{1000}	≤ 4,5 %			
Stress-corrosion resistance	Stress-corrosion resistance at 80% of actual tensile strength	Stress-corrosion resistance should be tested in accordance with the relevant product standard and comply with corresponding specified values			

NOTE All strength classes apply unless the National Annex excludes specific classes, see 5.3.2(1).

[a] Different fatigue properties can be set in the relevant standard by specifying higher values of the fatigue stress range and/or increasing the number of cycles that is to be confirmed by testing.

[b] Relaxation losses are very sensitive to the temperature of the prestressing steel, see B.9. Relaxation losses apply at a mean temperature of 20 °C.

Table C.7 — Strength, fatigue, ductility and relaxation properties of prestressing bars

Property		Bars — Strength class			
		Y1030	Y1050	Y1100	Y1230
Strength	Tensile yield strength $R_{p0,1}$ [MPa], 5 % quantile	≥ 835	≥ 950	≥ 900	≥ 1080
	Tensile strength R_m [MPa], 5 % quantile	≥ 1030	≥ 1050	≥ 1100	≥ 1230
	Tensile strength R_m [MPa] 95 % quantile	≤ 1180	≤ 1210	≤ 1260	≤ 1370
Fatigue[a]	Fatigue stress range [MPa] for $N \geq 2 \times 10^6$ cycles with an upper limit of 0,7 f_{pk} (10 % quantile)	200 for plain bars $\phi_p \leq 40$ mm 150 for plain bars $\phi_p > 40$ mm 180 for threaded bars $\phi_p \leq 40$ mm 120 for threaded bars $\phi_p > 40$ mm			
Ductility	Characteristic value of stress ratio $(R_m/R_{p0,1})_k$ (10 % quantile)	$\geq 1,10$			
	Elongation at maximum load, A_{gt} [%] (10 % quantile)	$\geq 3,5$			
Relaxation[b]	Relaxation at 1 000 hours for initial stress of 80 % of actual tensile strength, ρ_{1000}	≤ 6 % for $\phi_p \leq 15$ mm ≤ 4 % for $\phi_p > 15$ mm			
	Maximum Relaxation at 1 000 hours for initial stress of 80 % of actual tensile strength, ρ_{1000}	Value to be determined by testing if stressing intended to $> 0,7\, f_{pk}$.			
Stress-corrosion resistance	Stress-corrosion resistance at 80% of actual tensile strength	Stress-corrosion resistance should be tested in accordance with the relevant product standard and comply with corresponding specified values			

NOTE All strength classes apply unless the National Annex excludes specific classes, 5.3.2(1).

[a] Different fatigue properties can be set in the relevant standard by specifying higher values of the fatigue stress range and/or increasing the number of cycles that is to be confirmed by testing.

[b] Relaxation losses are very sensitive to the temperature of the prestressing steel, see B.9. Relaxation losses apply at a mean temperature of 20 °C.

C.6 Couplers

(1) Couplers for members designed in accordance with this Eurocode should comply with the relevant standards for couplers.

NOTE 1 The National Annex can specify relevant standards for couplers.

NOTE 2 EAD 160129 is available for the assessment and determination of properties of couplers required for design to this Eurocode.

(2) Couplers for splicing of reinforcing steel bars shall be capable of developing the ultimate tensile strength R_m, and the maximum compressive force with yield strength R_e. For failures in the coupler, or in

the bar inside coupler, the measured elongation on the reinforcing bar outside the length of the mechanical splice shall not be less than $0.7A_{gt}$, where A_{gt} is the characteristic value for the reinforcement bar.

(3) The permanent slip value measured for a simple coupler, consisting of no more than two load-transmitting components, should not exceed 0,10 mm.

For couplers consisting of more than two load-transmitting components,

a) the permanent slip value should not exceed 0,10 mm; or

b) the strain measured across the splice at a load equivalent to 0,65 R_e should not exceed 0,16% or the actual value measured on the reference bar, whichever is the greater.

(4) If couplers are used in members subjected to fatigue, they should comply with fatigue stress range of at least 35 MPa for $N \geq 10^7$ cycles with an upper stress limit of 0,6 f_{yk} of the connected bars (10% quantile) in order to comply with the S-N curve given in Table E.1 (NDP).

NOTE 3 Other fatigue stress ranges need to be specified if $\Delta\sigma_{Rsk}$ in Table E.1 (NDP) is modified in the National Annex.

C.7 Headed bars

(1) Headed bars for members designed in accordance with this Eurocode should comply with the relevant standards for headed bars.

NOTE 1 The National Annex can specify relevant standards for headed bars.

NOTE 2 EAD 160003 and EAD 160012 are available for the assessment and determination of properties of double headed studs and for headed reinforcement, respectively, for design to this Eurocode.

(2) Heads at the end of reinforcing steel bars shall be connected with a strength $\geq R_m A_s$ of the bar. The head shall be of sufficient strength to transfer the anchorage force to the concrete.

(3) If double-headed studs or bars are designed as shear or punching shear reinforcement, the requirements of Table C.8 should be fulfilled.

Table C.8 — Properties of double-headed studs and bars for shear reinforcement.

Property	Requirement
Diameter of heads ϕ_h (related to bar diameter ϕ)	$\phi_h \geq 3\phi$
Characteristic resistance to fatigue loading $\Delta\sigma_{Rsk}$ in testing for $N \geq 2 \times 10^6$ cycles based on an upper stress limit of $0{,}6f_{yk}$	$\Delta\sigma_{Rsk} \geq 80$ MPa

(4) The head should be at 90° to the bar axis and the bearing surface may be sloped ≥ 80° to the bar axis unless justified by testing appropriate to the design situation under consideration.

C.8 Post-installed reinforcing steel systems

(1) The reinforcing steel should comply with the properties according to C.4.

(2) Anchoring mortars for post-installed reinforcing steel systems are for the purpose of this Eurocode grouped in bond efficiency classes which should develop a minimum mean bond strength $f_{bm,rqd}$ according to Table C.9, when tested and assessed in accordance with relevant standards for anchoring mortars accounting for the conditions in C.8(3).

NOTE 1 The National Annex can specify relevant standards for anchoring mortars.

NOTE 2 EAD 330087 is available for the assessment and determination of properties of anchoring mortars required for design to this Eurocode.

NOTE 3 Permissible use conditions and information required for design of the post-installed reinforcing steel systems to this Eurocode can be found in European Technical Product Specifications.

Table C.9 — Required minimum mean bond strength $f_{bm,rqd}$ [MPa]

Bond efficiency class Bond efficiency factor $k_{b,pi}$		Required minimum mean bond strength as a function of concrete strength f_{ck} (MPa)					
		12	**16**	**20**	**30**	**40**	**50**
CPI-1,0	1,0	7,7	8,9	10,0	12,2	14,1	15,8
CPI-0,9	0,9	na	8,0	9,0	11.0	12,7	14,2
CPI-0,8	0,8	na	na	8,0	9,8	11,3	12,6
CPI-0,7	0,7	na	na	na	8,6	9,9	11,1

na = not allowed

NOTE 1 Values for intermediate concrete strength may be interpolated linearly.

NOTE 2 Post-installed reinforcing bars with a mean bond strength $f_{bm,rqd}$ < 7,7 MPa are not covered by this Eurocode.

(3) The specification and determination of the mean bond strength $f_{bm,rqd}$ should take into account the following influencing factors:

a) Environmental conditions: corrosion resistance (accounting for the maximum allowable chloride content of the concrete and the mortar for the intended application according to EN 206), alkalinity and possible sulphurous atmosphere, in-service temperature conditions, freeze/thaw conditions, creep behaviour under sustained loads. Adequate corrosion protection may be assumed if post-installed reinforcing steel bars are used in concrete members in environmental conditions according to exposure classes X0 and XC1 (see Table 6.1) or if they are produced from corrosion resistant steel;

b) Installation conditions: drilling method, drilling tools including drilling aids, hole preparation tool (e.g. roughening tool), cleaning devices, mortar injection devices (dispenser, nozzle, etc.), concrete condition at installation (dry, wet), installation direction (downward, horizontal, overhead), concrete temperature at installation, working and curing time of the anchoring mortar and robustness of installation;

c) Load bearing behaviour: stiffness of anchoring mortar;

d) Intended uses: range of reinforcing steel bar diameters (for each drilling method), concrete strength classes and types of loading (static, quasi-static, fatigue).

Annex D
(informative)

Evaluation of early-age and long-term cracking due to restraint

D.1 Use of this annex

(1) This Informative Annex provides complementary guidance on the evaluation of early-age and long-term cracking due to restraint in general with special emphasis on through-cracking.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this Informative Annex, it can be used.

D.2 Scope and field of application

(1) This Informative Annex applies to all types of members and structures.

D.3 General

(1) The main objective of the analysis is to evaluate the cracking risk, R_{cr}, or provide guidance on crack calculations if cracking is expected to occur. The cracking risk may be expressed in terms of the ratio between the maximum tensile stress, $\sigma_{ct,}$ and the tensile resistance of concrete at the moment which is being considered $f_{ct,eff}(t)$, reduced by a factor of 0,8 in order to account for the effects of sustained loading (see Formula (D.1)):

$$R_{\mathrm{cr}}(t) = \frac{\sigma_{\mathrm{ct}}(t)}{0{,}8 f_{\mathrm{ct,eff}}(t)} \tag{D.1}$$

where $f_{ct,eff}(t)$ may be taken as $f_{ctm}(t)$ according to B.4(2).

It is the age at loading that should be adjusted in the aging functions.

(2) The amount of through-cracking may be reduced by using concretes with low heat production during hydration, concretes with low coefficient of thermal expansion, cooling pipes in the hardening concrete, heating cables in the restraining structural elements, reduced fresh concrete temperature, or by reducing the degree of restraint for the hardening concrete structural element (i.e. if possible, break the bond between the hardening concrete and restraining structure in order to reduce the degree of restraint).

NOTE 1 Figure D.1 shows the assumed temperature history for a structural concrete member where

— T_{ci} is the fresh concrete temperature;

— $T_{c,max}$ is the maximum concrete temperature due to hydration heat;

— T_0 is the temperature of the restraining structure; and

— ΔT_{min} is long term maximum temperature drop.

Correspondingly, Figure D.2 shows the resulting diagram of stresses in concrete.

(3) Compression in concrete due to favourable effects from the heating phase may be considered.

NOTE 2 Assuming that there are no additional external actions, the most unfavourable moment in time for early-age cracking is t_{crit}, corresponding to the moment when thermal equilibrium with the restraining structure is achieved (within 2 ºC) and the greater part of basic shrinkage has already developed.

(4) For long-term cracking, the effects of temperature variation (ΔT_{min}) and drying shrinkage should be considered. In Figure D.2, t_2 is the time when the stresses in the critical position change from compression to tension (determined from the temperature history). Alternatively t_2 may be assumed to be 2 day.

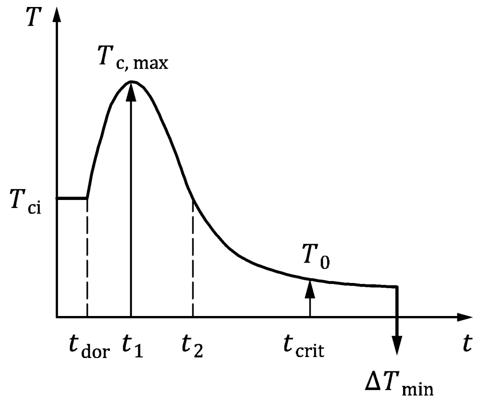
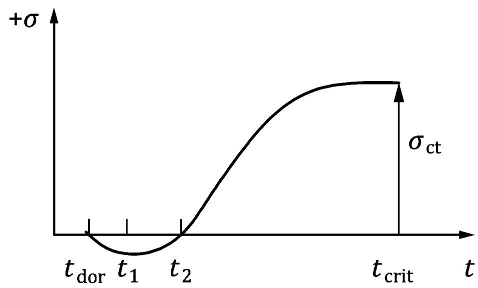


Figure D.1 — Assumed temperature history for a structural concrete member over time



Key

$+\sigma$ tensile stress

Figure D.2 — Corresponding stress diagram due to hydration heat and shrinkage of concrete over time

D.4 Assessment of temperature history

D.4.1 General

(1) A reliable temperature calculation or estimate should be made to achieve accurate stress/strain or crack width calculation. For this purpose, computer programs, hand calculations and diagrams from handbooks and guidelines may be used.

NOTE The most decisive parameters, referred to Figure D.1 are:

— the fresh concrete temperature (T_{ci});

— the start time for stress development (t_{dor});

— ambient temperature history;

— insulation conditions;

— climatic conditions, such as wind velocity and solar radiation;

— temperature of the restraining structure (T_0); and

— the additional, long-term maximum temperature drop (ΔT_{min}).

where t_{dor} marks the end of the dormant phase. Typically, it varies between 8 and 13 hours after placing, the default value being 10 hours. This parameter can be strongly influenced by admixtures, and it is important that its value be taken into consideration when evaluating the mechanical properties of concrete (tensile strength and modulus of elasticity) as well as the value of shrinkage and the temperature development curves.

(2) t_{dor} may be determined from the compressive strength development, from ultrasonic stiffness or heat release measurements, or from restrained shrinkage tests. In absence of more accurate values those provided in Table D.1 may be used.

Table D.1 — Values for the start time of stress development t_{dor}

Concrete strength f_{ck}	Start time of stress development t_{dor} [days]		
	Class CR[a]	Class CN[a]	Class CS[a]
$\leq$ 35 MPa	0,35	0,45	0,50
$>$ 35 MPa $<$ 60 MPa	0,30	0,40	0,45
$\geq$ 60 MPa	0,30	0,35	0,40
[a] See B.3(1).			

(3) In absence of better data, T_0 may generally be taken as the ambient temperature.

D.4.2 Material properties related to temperature development

(1) In order to determine the temperature history of concrete due to cement hydration, the following material properties should be known:

— Total amount of hydration heat ($Q(t)$ (kJ/kg binder)) released during hydration and its development with time. $Q(t)$ may be determined in accordance with EN 12390-14 or EN 12390-15;

— Heat conductivity: The values for the heat conductivity may vary within the range 1,2 to 3,0 J/(s · m · K). In absence of better data, a default value of 2,5 J/(s · m · K) may be adopted;

— Surface convectivity: The surface convectivity may vary in the range from about 3,3J/(s · m^2 · K) for 18 mm plywood formwork and no wind to 15 J/(s m^2 · K) for a free concrete surface with 5 m/s wind;

— Heat capacity: The heat capacity corresponds to the specific heat multiplied by the concrete density. The specific heat varies typically in the range 0,85 to 1,15 kJ/(kg · K), and the default value may be assumed as 1,0 kJ/(kg · K).

NOTE The amount of heat of hydration and development with time are strongly dependent on binder fineness and composition and on the water/binder (w/b)-ratio. Furthermore, the early strength class of cement strongly influences the binder content, and therefore also the amount of hydration heat.

D.5 Stress calculations

(1) The following stress calculations may be used when aiming to limit the risk of cracking.

(2) It may be assumed that the risk that the member will crack is sufficiently low if Formula (D.2) is met:

$$R_{cr}(t) < R_{ea,cr} \tag{D.2}$$

where $R_{ea,cr}$ is a project specific parameter related to the admissible risk of cracking, where $R_{ea,cr}$ may be assumed as 1,0.

(3) When assessing $R_{cr}(t)$, the dormant time, t_{dor}, should be subtracted from the concrete age to determine the tensile resistance and the modulus of elasticity of concrete. Coefficient β_{cc}, describing the evolution of concrete properties with time (see Formula (B.2)) should then be corrected as follows (Formula (D.3)):

$$\beta_{cc}(t) = \exp\left[s_C\left(1-\sqrt{\frac{t_{ref}-t_{dor}}{t_T-t_{dor}}}\right)\cdot\sqrt{\frac{28-t_{dor}}{t_{ref}-t_{dor}}}\right] \tag{D.3}$$

NOTE Formula (B.2) is a simplified version of Formula (D.3) which assumes that $t_{dor} = 0$.

(4) $R_{cr}(t)$ should be determined at least for the following situations:

— At the time when temperature equilibrium between the recently cast element and the restraining structure is achieved. For this verification, it is not necessary to consider drying shrinkage or long-term temperature variations.

The maximum tensile stress to be considered for this verification may be determined using Formula (D.4):

$$\sigma_1 = R_{ax,1}\frac{E_c(t_2)}{1+\chi\varphi_{st}}\left(k_{Temp}\alpha_{cth}\left(T_{c,max}-T_0\right)+\left[\varepsilon_{cbs}(t_{crit})-\varepsilon_{cbs}(t_2)\right]\right) \tag{D.4}$$

where

$R_{ax,1}$ is the restraint factor (see 9.2.3(4)) corresponding to the boundary conditions present after concreting;

$E_c(t_2)$ is the modulus of elasticity of concrete at time t_2 (see Figure D.2). t_2 may be replaced by maturity, i.e. corrected for the temperature history in accordance with Formula (B.18);

$T_{c,max}$ is the maximum temperature in concrete after casting due to hydration heat;

T_0 is the temperature of the restraining structure;

k_{Temp} is a factor accounting for the reduction in temperature from t_1 to t_2 (see Figure D.1) and may be taken as 0,9;

$\chi\varphi_{st}$ accounts for short term creep relaxation which is significant, in spite of the short time, due to the low maturity of concrete and the presence of hydration heat. Its value may be estimated as 0,55.

— Long term analysis: for this verification, the effects of long-term temperature variation and drying shrinkage should be considered. The maximum tensile stress may be determined using Formula (D.5):

$$\begin{aligned}\sigma_1 = {} & R_{ax,1}\frac{E_{c,28}}{1+\chi\varphi(t,t_2)}\left(k_{Temp}\alpha_{cth}\left(T_{c,max}-T_0\right)+\left[\varepsilon_{cbs}(t)-\varepsilon_{cbs}(t_2)\right]\right)\\ & + R_{ax,2}E_{c,28}\alpha_{cth}\Delta T_{min}\\ & + R_{ax,3}\frac{E_{c,28}}{1+\chi\varphi(t,t_s)}\cdot\left[\varepsilon_{cds}(t)-\varepsilon_{cds}(t_2)\right]\end{aligned} \tag{D.5}$$

where

$R_{ax,2}$ is the restraint factor (see 9.2.3(4)) corresponding to the boundary conditions present when the maximum temperature drop is expected to occur;

$R_{ax,3}$ is the restraint factor (see 9.2.3(4)) corresponding to the boundary conditions prevalent during the development of drying shrinkage;

ΔT_{min} is the maximum characteristic temperature drop with respect to the temperature considered at casting T_0;

χ is the aging coefficient which may be taken as 0,8;

$E_{c,28}$ is the tangent modulus of elasticity of concrete at an age of 28 days.

Formula (D.5) may be modified to account for the release of restraint that may occur in time, such as when formwork is removed.

D.6 Crack width calculations

(1) If cracking due to the imposed deformations does occur ($R_{cr} > R_{ea,cr}$), the crack width may be determined according to 9.2.3.

(2) For members restrained at the ends Formula (9.11) applies.

(3) For members restrained at the edges, in Formula (9.13), $R_{ax}\varepsilon_{free}$ may be taken as:

— for early-age cracking:

$$R_{ax}\varepsilon_{free} = R_{ax,1}\left(k_{Temp}\alpha_{cth}\left(T_{c,max}-T_0\right)+[\varepsilon_{cbs}(t_{crit})-\varepsilon_{cbs}(t_2)]\right) \tag{D.6}$$

— for long-term cracking:

$$\begin{aligned} R_{ax}\varepsilon_{free} = {} & R_{ax,1}\left(k_{Temp}\alpha_{cth}\left(T_{c,max}-T_0\right) + [\varepsilon_{cbs}(t) - \varepsilon_{cbs}(t_2)]\right) \\ & + R_{ax,2}\alpha_{cth}\Delta T_{min} + R_{ax,3} \cdot [\varepsilon_{cds}(t) - \varepsilon_{cds}(t_2)] \end{aligned} \tag{D.7}$$

Annex E
(normative)

Additional rules for fatigue verification

E.1 Use of this annex

(1) This Normative Annex contains additional provisions to Clause 10 for a refined fatigue verification of structures.

E.2 Scope and field of application

(1) This annex covers fatigue verification of structures using:

— damage equivalent stresses according to E.4; or

— explicit verifications using Palmgren-Miner Rule according to E.5.

E.3 General

(1) For explicit verifications of bridges according to Annex E, the fatigue load models defined in EN 1991-2 shall be used. For other structures, the fatigue load should be defined according to EN 1990 on a project-specific basis.

E.4 Verification using damage equivalent stress range

E.4.1 General

(1) In the method of damage equivalent stress range the actual operational loading is represented by N^* cycles of a single equivalent stress range under the equivalent fatigue load models.

E.4.2 Verification for reinforcement

(1) For reinforcing or prestressing steel and couplers adequate fatigue resistance may be assumed if Formula (E.1) is satisfied:

$$\gamma_{\mathrm{Ff}} \cdot \Delta\sigma_{\mathrm{S,equ}}(N^*) \leq \frac{\Delta\sigma_{\mathrm{Rsk}}(N^*)}{\gamma_{\mathrm{S}}} \tag{E.1}$$

where

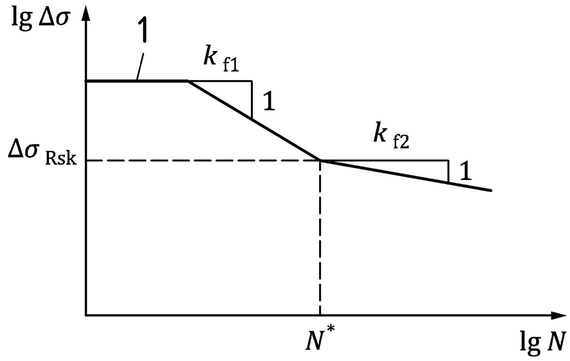
$\Delta\sigma_{\mathrm{Rsk}}(N^*)$ is the stress range at N^* cycles from the appropriate *S-N* curves given in Figure E.1 and Tables E.1 (NDP) and E.2 (NDP);

$\Delta\sigma_{\mathrm{S,equ}}(N^*)$ is the damage equivalent stress range for different types of reinforcement and considering the number of loading cycles N^*. For buildings $\Delta\sigma_{\mathrm{S,equ}}(N^*)$ may be approximated by $\Delta\sigma_{\mathrm{S,max}}$;

$\Delta\sigma_{\mathrm{S,max}}$ is the maximum reinforcing steel stress range under the relevant load combinations.

NOTE 1 Tables E.1 (NDP) and E.2 (NDP) apply unless other values of the fatigue stress range and/or the number of cycles are given in the National Annex and confirmed by testing in accordance with the relevant standards for reinforcing steel and prestressing steel.

NOTE 2 For road and railway bridges $\Delta\sigma_{\mathrm{S,equ}}(N^*)$ is given in K.10.2.1.



Key

1 reinforcing and prestressing steel at yield

Figure E.1 — Shape of the characteristic fatigue strength curve (*S-N* curves for reinforcing and prestressing steel)

Table E.1 (NDP) — Design parameters for *S-N* curves for carbon reinforcing steel

<table>
<tr><th rowspan="3">Type of reinforcing steel</th><th rowspan="3">Diameter</th><th colspan="4">$\Delta\sigma_{Rsk}$ 5 %-quantile (Test σ_{max} 0,6f_{yk})</th></tr>
<tr><th rowspan="2">$\Delta\sigma_{Rsk}$ [MPa]</th><th rowspan="2">N^*</th><th colspan="2">stress exponent</th></tr>
<tr><th>k_{f1}</th><th>k_{f2}</th></tr>
<tr><td rowspan="4">Bars[a]</td><td>$\phi \leq 12$ mm</td><td>160</td><td rowspan="6">$2 \cdot 10^6$</td><td rowspan="4">5</td><td rowspan="4">9</td></tr>
<tr><td>12 mm $< \phi \leq 16$ mm</td><td>140</td></tr>
<tr><td>16 mm $< \phi \leq 20$ mm</td><td>130</td></tr>
<tr><td>$\phi > 20$ mm</td><td>130</td></tr>
<tr><td rowspan="2">Tack welded bars[b,] and welded fabrics</td><td>$\phi \leq 12$ mm</td><td>100</td><td rowspan="2">3</td><td rowspan="2">5</td></tr>
<tr><td>$\phi > 12$ mm</td><td>80</td></tr>
<tr><td>Couplers[c]</td><td>–</td><td>35</td><td>10^7</td><td>3</td><td>5</td></tr>
</table>

[a] Values for bent parts of bars should be obtained using a reduction factor $\zeta_f = 0{,}35 + 0{,}026 \cdot \phi_{mand} / \phi$. The reduction factor ζ_f may be omitted for shear reinforcement with 90° stirrups $\phi \leq 16$ mm and depth $h \geq 600$ mm.

[b] Values for $\Delta\sigma_{Rsk}$ of tack welded bars apply for a distance of 5ϕ at each side of the weld.

[c] Values for couplers apply unless more accurate *S-N* curves are available and confirmed by testing.

NOTE The 10 % quantile values for material according to Table C.1 and Table C.3 are based on a confidence level of 90 % whereas confidence levels probabilities for design $\Delta\sigma_{Rsk}$ (5 % quantile values) are 75 % according to EN 1990, Annex D.

Table E.2 (NDP) — Parameters for *S-N* curves for prestressing steel

S-N curve of prestressing steel	N^*	stress exponent		$\Delta\sigma_{Rsk}$ [MPa] at N^* cycles[a]
		k_{f1}	k_{f2}	
pre-tensioning	10^6	5	9	185
post-tensioning				
— single strands in plastic ducts	10^6	5	9	185
— straight tendons[b] or curved tendons[b] in plastic ducts	10^6	5	9	150
— curved tendons[b] in steel ducts	10^6	3	7	120
— anchoring devices and couplers	10^6	5	5	80

NOTE Values in Table E.2 (NDP) apply for prestressing steel complying with Tables C.3 to C.7 and prestressing systems complying with 5.4.

[a] Values correspond to prestressing steel embedded in concrete.

[b] Applies to tendons with wires and strands; tendons with bars are not covered.

E.4.3 Verification for concrete

(1) A satisfactory fatigue resistance may be assumed for concrete under compression, if the following condition in Formula (E.2) is fulfilled:

$$\frac{|\sigma_{cd,max,equ}|}{f_{cd,fat}} + 0{,}43\sqrt{1 - \left|\frac{\sigma_{cd,min,equ}}{\sigma_{cd,max,equ}}\right|} \leq 1 \tag{E.2}$$

where

$f_{cd,fat}$ is the design fatigue strength of concrete according to Formula (10.5);

$|\sigma_{cd,max,equ}|$ is the upper stress of the damage equivalent stress amplitude for $N = 10^6$ cycles;

$|\sigma_{cd,min,equ}|$ is the lower stress of the damage equivalent stress amplitude for $N = 10^6$ cycles.

NOTE For road and railway bridges, $\sigma_{cd,equ,max}$ and $\sigma_{cd,equ,min}$ are given in K.10.2.

E.5 Explicit verifications using *Palmgren-Miner* Rule

E.5.1 Verification conditions

(1) Calculation of design service life at varying stress amplitudes may be based on linear damage theory (*Palmgren-Miner* Rule) according to Formula (E.3). Action effects due to cyclic loads may be arranged in design stress-levels (action effect-levels) each with constant amplitude and a corresponding number of load cycles, n_i.

$$\sum_{i=1}^{m} \frac{n_i}{N_i} \leq 1 \tag{E.3}$$

where

n_i is the number of acting stress cycles for each design stress-level "i";

N_i is the number of cycles to fatigue failure according to E.5.2 for each design stress-level "i" with constant amplitude;

m is the number of design stress-levels with constant amplitude.

E.5.2 Verification procedure for reinforcing and prestressing steel

(1) The damage of a single stress range $\Delta\sigma$ may be determined by using the corresponding S-N curves (Figure E.1) for reinforcement.

$$N_i(\Delta\sigma_{Ed,s}) = \left(\frac{\Delta\sigma_{Rsk}(N^*)}{\gamma_S \cdot \gamma_{Ff} \cdot \Delta\sigma_s}\right)^{k_f} \cdot N^* \tag{E.4}$$

where

$$\text{if } \Delta\sigma_s \geq \frac{\Delta\sigma_{Rsk}}{\gamma_S \cdot \gamma_{Ff}}: \qquad k_f = k_{f1} \tag{E.5}$$

$$\text{if } \Delta\sigma_s < \frac{\Delta\sigma_{Rsk}}{\gamma_S \cdot \gamma_{Ff}}: \qquad k_f = k_{f2} \tag{E.6}$$

NOTE The parameters for reinforcement are given in Table E.1 (NDP) and Table E.2 (NDP) respectively.

E.5.3 Verification procedure for concrete under compression

(1) The number of cycles to fatigue failure N_i may be calculated for each stress-level using the S-N curves for concrete under compressive fatigue loading in Figure E.2, Formula (E.7).

$$N_i = 10^{k_i} \tag{E.7}$$

where

$$k_i = C \cdot \frac{1 - \frac{|\sigma_{cd,max,i}|}{f_{cd,fat}}}{\sqrt{1 - \left|\frac{\sigma_{cd,min,i}}{\sigma_{cd,max,i}}\right|}} \tag{E.8}$$

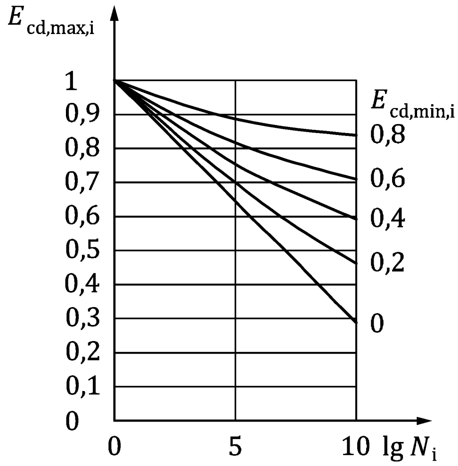
where

C = 14 may be taken for concrete under compression and not permanently submerged in water;

$\sigma_{cd,max}$ is the maximum compressive stress in stress-level "i", $\sigma_{cd,max} = \gamma_{Ff} \cdot \sigma_{c,max}$;

$\sigma_{cd,min}$ is the minimum compressive stress in stress-level "i", $\sigma_{cd,min} = \gamma_{Ff} \cdot \sigma_{c,min}$;

$f_{cd,fat}$ is the design fatigue strength of concrete according to Formula (10.5).



Lower compressive stress level „i“: $E_{cd,min,i} = \frac{\sigma_{cd,min,i}}{f_{cd,fat}}$

Upper compressive stress level „i“: $E_{cd,max,i} = \frac{\sigma_{cd,max,i}}{f_{cd,fat}}$

Figure E.2 — Shape of the characteristic fatigue strength curves (*S*-*N*-curves for concrete under compression)

(2) When verifying struts (e.g. for checking shear), $f_{cd,fat}$ in Formula (E.8) should be replaced by $\nu f_{cd,fat}$.

Annex F
(informative)

Safety formats for non-linear analysis

F.1 Use of this annex

(1) This Informative Annex provides supplementary guidance to non-linear analyses procedures for concrete structures.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this Informative Annex, it can be used.

F.2 Scope and field of application

(1) This Informative Annex covers the use of the partial factor method, the global factor method and full probabilistic method for non-linear analyses of concrete structures at ULS.

F.3 General

(1) Verification of ultimate limit states by non-linear analysis should be performed by numerical simulations in compliance with the requirements of 7.3.4.

(2) Annex F shall be applied consistently to EN 1990.

(3) The software used in non-linear verification of the ultimate limit state should be validated by comparison between numerical and experimental benchmark results. Similarly, the choices made with respect to the specific numerical model should be tested by sensitivity analysis.

This covers general validation attempts:

— basic material tests;

— structural tests with the characterization of relevant failure modes;

and project specific attempts:

— mesh sensitivity tests;

— solution method tests.

(4) The analysis process adopted by a non-linear simulation in order to evaluate the structural resistance shall be defined according to engineering judgement consistently with the considered combination of the actions at ultimate limit state.

NOTE The analysis process may account for the loading sequence due to staged construction methods.

(5) The verification requires that the design value of the actions in the considered combination, determined according to EN 1990, shall not be greater than the design value of the associated structural resistance (see Formula (F.1)):

$$F_{\mathrm{d}} \leq R_{\mathrm{d}} \tag{F.1}$$

where

F_{d} is the design value of the actions;

R_d is the design value of the structural resistance.

(6) The design value of the structural resistance may be determined by using one of the following safety formats:

— Partial factor method (PFM) as described in F.4;

— Global factor method (GFM) as described in F.5;

— Full probabilistic method (FPM) as described in F.6.

F.4 Partial factor method (PFM)

(1) The design value of the structural resistance is obtained as

$$R_d = \frac{R\{X_d;a_d\}}{\gamma_{Rd}} \tag{F.2}$$

where

$R\{...\}$ is the structural resistance based on the numerical simulation;

X_d is the design value of the material property calculated adopting partial safety factors according to Table A.1 line (g) accounting for materials and geometrical uncertainties;

a_d is the design value of the geometric property according to Annex A or EN 1990, if not accounted for in X_d;

γ_{Rd} is the partial safety factor which accounts for model uncertainty according to F.7.

NOTE The structural resistance $R\{...\}$ can be evaluated for different levels of appropriate actions which can be increased from their initial values by incremental steps, such that the design values of the actions in the considered combination are reached in the same step. The incremental process of the actions should be continued until structural failure is reached. The structural resistance $R\{...\}$ corresponds to the values of the actions which lead to structural failure.

(2) Alternatively, in case the concrete tensile strength is neglected, the design value of the resistance may be calculated on the basis of the design values f_{cd}, f_{yd} and f_{pd} according to 5.1.6, 5.2.4 and 5.3.3(1). If the statistic values of the model uncertainties according to F.7 are more favourable than the values given in Table A.3, then the partial safety factors γ_C and γ_S defined in Table 4.3 (NDP) may be used. Otherwise, they shall be adapted according to Annex A.

F.5 Global factor method (GFM)

F.5.1 General

(1) The design value of structural resistance is obtained as

$$R_d = \frac{R_m\{X;a\}}{\gamma_R^* \cdot \gamma_{Rd}} \tag{F.3}$$

where

$R_m\{...\}$ is the mean value of structural resistance based on the numerical simulation and shall be estimated by means of a probabilistic analysis including uncertainties related to material X and geometric a properties.

Alternatively, the approximated value derived by Formula (F.4) may be adopted:

$$R_m\{X;a\} \cong R\{X_m;\ a_{nom}\} \tag{F.4}$$

where

$R\{...\}$ is the structural resistance based on the numerical simulation;

X_m is the mean value of the material property;

a_{nom} is the nominal value of the geometric property according to EN 1990;

γ_R^* is the global resistance factor for the uncertainties of materials properties and geometry according to F.5.2;

γ_{Rd} is the partial safety factor accounting for the model uncertainty according to F.7.

NOTE The structural resistance R{...} can be evaluated for different levels of appropriate actions which can be increased from their initial values by incremental steps, such that the design values of the actions in the considered combination are reached in the same step. The incremental process of the actions should be continued until structural failure is reached. The structural resistance R{...} corresponds to the values of the actions which lead to structural failure.

F.5.2 Determination of the global resistance factor

(1) The global resistance factor may be evaluated with Formula (F.5):

$$\gamma_R^* = \exp\left(\alpha_R \cdot \beta_{tgt} \cdot V_R^*\right) \tag{F.5}$$

where

α_R is the sensitivity factor for dominant resistance variable;

β_{tgt} is the target value of the reliability index according to Table A.4 (NDP);

V_R^* is the coefficient of variation of structural resistance. It accounts for material and geometrical uncertainties and shall be estimated by means of a probabilistic analysis of the structural resistance.

Alternatively, the approximated value derived by Formula (F.6) may be adopted:

$$V_R^* = \sqrt{V_{R,M}^2 + V_{R,G}^2} \tag{F.6}$$

where

$V_{R,M}$ is the coefficient of variation of structural resistance accounting for uncertainties of material properties evaluated according to Formula (F.7):

$$V_{R,M} = \frac{1}{1{,}65} \ln\left(\frac{R\{X_m;\ a_{nom}\}}{R\{X_k;\ a_{nom}\}}\right) \tag{F.7}$$

where X_k is the characteristic value of the material property;

$V_{R,G}$ is the coefficient of variation of structural resistance related to geometrical uncertainties evaluated in line with Annex A.

NOTE 1 As a further simplification, in absence of a numerical estimation of the value of $V_{R,M}$ on the safety side, $V_{R,M} = 0{,}15$ can be used, unless the National Annex gives other specifications.

NOTE 2 The bias factor in Formula (F.4) is equal to 1,00 assuming that the bias factors related to geometric and material uncertainties are considered in the mean and characteristic values of material parameters.

NOTE 3 Formulae (F.5) and (F.7) are determined assuming that bias factors on structural resistance related to geometric and material uncertainties are considered in the mean values of material parameters.

NOTE 4 According to EN 1990, Table C.4.1, the approximated Formula (F.5) may be used for $V_R^* < 0{,}20$. The exact formula to be used for higher coefficients of variation V_R^* can be found in EN 1990, C.4.4 (which provides 3 % higher values of γ_R^* for $V_R^* = 0{,}30$).

F.5.3 Additional material parameters

(1) Information to be used for the description of the mean values of the material properties can be found in Annex A.

(2) Other material properties used in the analysis should be included in the evaluation of the model uncertainty determined according to F.7.

(3) The design values and the characteristic values of other concrete properties not defined in this standard may be determined from the corresponding strength values, i.e. f_{cd}, f_{ck} or f_{td}, f_{tk} according to Annex A.

F.6 Full probabilistic method

(1) The full probabilistic method shall be according to EN 1990, Annex C.

(2) The uncertainties of the random variables which are taken into account shall be consistent with EN 1990 and Annex A.

(3) The model uncertainty random variable shall be characterised according to F.7.

F.7 Model uncertainty

(1) The partial safety factor γ_{Rd} should be derived by probabilistic calibrations.

NOTE 1 The value of γ_{Rd} can be different depending on the adopted software, choices made with respect to the analysis and also depends on the relevant failure modes.

(2) If probabilistic calculation according to (1) is not performed, γ_{Rd} should be set to fixed value.

NOTE 2 Unless the National Annex gives different values, the value of γ_{Rd} can be taken as:

— $\gamma_{Rd} = 1{,}30$ for numerical models in general accounting for also statistical uncertainty;

— $\gamma_{Rd} = 1{,}06$ when 1D beam elements are used and bending failure is the determining failure mode.

The proposed values for γ_{Rd} refer to a reliability index $\beta_{tgt} = 3{,}8$ for a 50-years reference period.

(3) In case of probabilistic calibration, the model uncertainty is described by the ratio θ_i:

$$\theta_i = \frac{R_{exp,i}}{R_{num,i}} \tag{F.8}$$

where

$R_{exp,i}$ is the structural resistance obtained from experiment i;

$R_{num,i}$ is the structural resistance obtained from non-linear numerical simulation of experiment i.

(4) The material parameters for the material models used in the non-linear analyses with the scope to perform probabilistic calibration of model uncertainty shall be derived from the mean parameters available from benchmark experiments. If their derivation from known measured parameters is not available, default values should be used throughout the validation study.

(5) The statistical parameters of model uncertainty may be represented by its mean value (or bias factor) μ_θ and coefficient of variation V_θ based on a lognormal distribution. The statistical uncertainty should be reflected in the methods used for estimation of the statistical parameters μ_θ and V_θ.

(6) The partial factor γ_{Rd} for model uncertainty may be obtained assuming a lognormal probabilistic distribution as:

$$\gamma_{Rd} = \frac{\exp(\alpha_R \cdot \beta_{tgt} \cdot V_\theta)}{\mu_\theta} \quad \text{(F.9)}$$

where

α_R is the sensitivity factor for non-dominant variables: $\alpha_R = 0{,}32$;

β_{tgt} is the reliability index according to EN 1990 or Annex A.

(7) When the design structural resistance is evaluated according to F.4 and F.5, in case the response of the non-linear numerical model of the structure turns out to be sensitive to influence of the probabilistic distributions of materials properties, the partial safety factor γ_{Rd} derived according to (1) or (2) shall be increased.

(8) The sensitivity of the response of non-linear numerical model to influence of the probabilistic distributions of materials properties may be assessed by means of three preliminary non-linear numerical simulations using:

a) mean concrete properties combined with design reinforcement properties;

b) design concrete properties combined with mean reinforcement properties;

c) design concrete properties combined with design reinforcement properties.

If the results in terms of structural resistance of (a) and/or (b) are unfavourable with respect to the one of (c), the response of the non-linear numerical model is sensitive.

NOTE 3 Unless more detailed probabilistic investigation of structural resistance is performed or the National Annex gives other specifications, an increase of the partial safety factor γ_{Rd} of 15 % applies.

Annex G
(normative)

Design of membrane-, shell- and slab elements

G.1 Use of this annex

(1) This Normative Annex contains additional provisions for the design of planar members. The formulations presented in G.3 and G.4 are consistent with the clauses and design provisions in Clause 8 and G.5 gives additional provisions to 9.2.

G.2 Scope and field of application

(1) This Normative Annex covers the design of planar members without discontinuities. Other more refined methods complying with 7.3.3 or 7.3.4 may be used.

(2) Effects of prestressing may be taken into account according to 7.6.1.

G.3 Design of membrane elements in ULS

(1) In locations where σ_{Edx} and σ_{Edy} are both compressive and $\sigma_{Edx} \cdot \sigma_{Edy} > \tau^2_{Edxy}$, design of reinforcement is not required. However, the maximum principal compressive stress should not exceed f_{cd}. Otherwise, the membrane forces are carried by tensile reinforcements in *x*- and *y* directions and a compression field inclined at an angle θ to *x*-axis as shown in Figure G.1 with a strength of $\nu \cdot f_{cd}$.

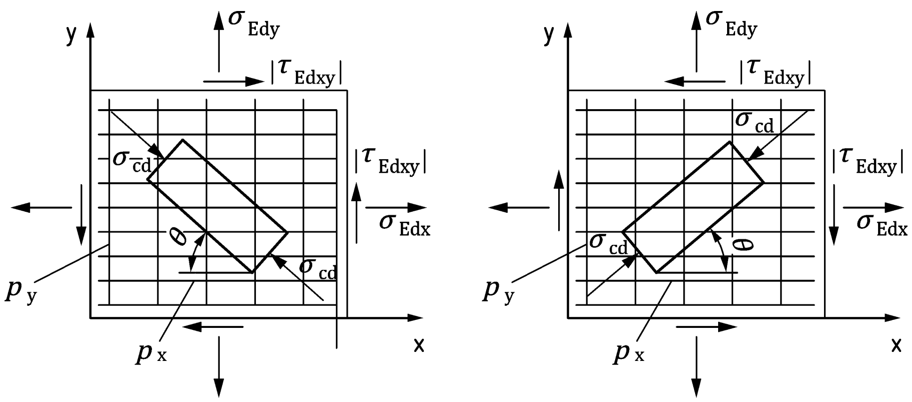


Figure G.1 — In-plane stresses in membrane element and definition of compression stress field inclination θ

(2) The tensile strengths provided by reinforcement are:

$$f_{tdx} = \rho_x f_{yd,x} \tag{G.1}$$

$$f_{tdy} = \rho_y f_{yd,y} \tag{G.2}$$

where ρ_x and ρ_y are the geometric reinforcement ratios along the *x*- and *y*-axes, respectively.

NOTE The yield strength can be different in the x- and y-directions.

(3) The necessary reinforcement and the compressive concrete stress may be determined by:

$$\sigma_{\mathrm{Edx}} + |\tau_{\mathrm{Edxy}}|\cot\theta \leq f_{\mathrm{tdx}} \tag{G.3}$$

$$\sigma_{\mathrm{Edy}} + \frac{|\tau_{\mathrm{Edxy}}|}{\cot\theta} \leq f_{\mathrm{tdy}} \tag{G.4}$$

$$\sigma_{\mathrm{cd}} = |\tau_{\mathrm{Edxy}}|(\cot\theta + \tan\theta) \leq \nu \cdot f_{\mathrm{cd}} \tag{G.5}$$

where $\cot\theta$ should be chosen to avoid negative values of f_{tdx} and f_{tdy}.

The choice of $\cot\theta$ to obtain the least amount of required reinforcement resistances f_{tdx}* and f_{tdy}* (optimum reinforcement) is given in Table G.1,

where

- θ* is the strut inclination corresponding to the optimal reinforcement for membrane elements;
- f_{tdx}*, f_{tdy}* are the tensile strength along the *x*- and *y*-axes, respectively, corresponding to the optimal reinforcement for membrane elements;
- σ_{cd}* is the compression stress in the concrete inclined at the angle θ* corresponding to the optimal reinforcement for membrane elements.

In order to ensure the required deformation capacity, and unless more refined calculations are performed, the reinforcement derived from Formulae (G.3) and (G.4) for each direction should not deviate significantly from the reinforcement resistances f_{tdx}* and f_{tdy}* determined according to Table G.1. These limitations may be expressed by:

$$0{,}4 f_{\mathrm{tdx}}^* \leq f_{\mathrm{tdx}} \leq 2{,}5\, f_{\mathrm{tdx}}^* \quad \text{and} \tag{G.6}$$

$$0{,}4 f_{\mathrm{tdy}}^* \leq f_{\mathrm{tdy}} \leq 2{,}5\, f_{\mathrm{tdy}}^*. \tag{G.7}$$

Table G.1 — Optimum reinforcement, concrete stress and strut inclination for membrane elements

Conditions	**cot *θ****	**f_{tdx}***	**f_{tdy}***	**σ_{cd}***
$\sigma_{\mathrm{Edx}} \geq -\lvert\tau_{\mathrm{Edxy}}\rvert$ $\sigma_{\mathrm{Edy}} \geq -\lvert\tau_{\mathrm{Edxy}}\rvert$	1	$\sigma_{\mathrm{Edx}} + \lvert\tau_{\mathrm{Edxy}}\rvert$	$\sigma_{\mathrm{Edy}} + \lvert\tau_{\mathrm{Edxy}}\rvert$	$2\lvert\tau_{\mathrm{Edxy}}\rvert$
$\sigma_{\mathrm{Edx}} \leq \sigma_{\mathrm{Edy}}$ $\sigma_{\mathrm{Edx}} < -\lvert\tau_{\mathrm{Edxy}}\rvert$ $\sigma_{\mathrm{Edx}}\sigma_{\mathrm{Edy}} \leq \tau_{\mathrm{Edxy}}^2$	$\frac{-\sigma_{\mathrm{Edx}}}{\lvert\tau_{\mathrm{Edxy}}\rvert}$	0	$\sigma_{\mathrm{Edy}} + \frac{\tau_{\mathrm{Edxy}}^2}{\lvert\sigma_{\mathrm{Edx}}\rvert}$	$\lvert\sigma_{\mathrm{Edx}}\rvert\left(1+\left(\frac{\tau_{\mathrm{Edxy}}}{\sigma_{\mathrm{Edx}}}\right)^2\right)$
$\sigma_{\mathrm{Edx}} \geq \sigma_{\mathrm{Edy}}$ $\sigma_{\mathrm{Edy}} < -\lvert\tau_{\mathrm{Edxy}}\rvert$ $\sigma_{\mathrm{Edx}}\sigma_{\mathrm{Edy}} \leq \tau_{\mathrm{Edxy}}^2$	$\frac{\lvert\tau_{\mathrm{Edxy}}\rvert}{-\sigma_{\mathrm{Edy}}}$	$\sigma_{\mathrm{Edx}} + \frac{\tau_{\mathrm{Edxy}}^2}{\lvert\sigma_{\mathrm{Edy}}\rvert}$	0	$\lvert\sigma_{\mathrm{Edy}}\rvert\left(1+\left(\frac{\tau_{\mathrm{Edxy}}}{\sigma_{\mathrm{Edy}}}\right)^2\right)$

(4) The reinforcement should be fully anchored at free edges, e.g. by headed bars, U-bars or similar.

(5) In absence of a refined calculation, the reduction factor of concrete strength may be assumed as:

$$\nu = 0{,}4 \tag{G.8}$$

(6) A more refined calculation of the reduction factor of concrete strength may be performed using:

$$\nu = \frac{1}{1{,}0 + 110 \cdot \varepsilon_1} \leq 1{,}0 \tag{G.9}$$

where ε_1 is the principal strain transverse to the direction of the concrete stress field accounting for strain compatibility and considering concrete as cracked without tensile strength.

(7) The value of the principal strain ε_1 in Formulae (G.9) may be estimated from one of the two following formulae:

$$\varepsilon_1 = \varepsilon_x + (\varepsilon_x + 0{,}001)\cot^2\theta \tag{G.10}$$

$$\varepsilon_1 = \varepsilon_y + \frac{(\varepsilon_y + 0{,}001)}{\cot^2\theta} \tag{G.11}$$

where ε_x and ε_y are the normal strains (positive as tension) in the x- and y-axes. A simple estimate of ε_1 may be obtained by using the following guidelines in cases where $\cot\theta$ is known:

a) Formulae (G.10), with ε_x taken as f_{yd}/E_{Sd}, may be used in cases where $f_{tdy} = 0$. Formulae (G.11), with ε_y taken as f_{yd}/E_{Sd}, may be used in cases where $f_{tdx} = 0$.

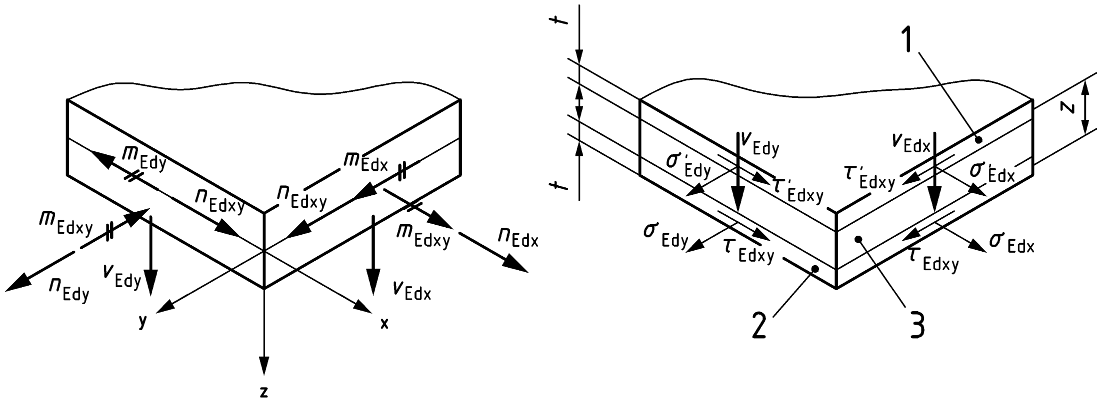
b) ε_1 is the larger of Formulae (G.10) and (G.11), with ε_x and ε_y taken as f_{yd}/E_{Sd}, in cases where reinforcement is required in both directions.

c) ε_1 may be taken as f_{ctd}/E_{cd} if f_{tdx} and f_{tdy} according to calculations are smaller than f_{ctd}, and provided the member is not cracked due to other types of action.

d) In (a) and (b) above, f_{yd}/E_{Sd} may be replaced by $f_{yd}/(\lambda E_{Sd})$, if the reinforcement in both directions is increased by a factor of $\lambda > 1$ beyond the required amount, where λ is the ratio between the provided amount and the required amount.

G.4 Design of shell- and slab elements in ULS

(1) Tension reinforcement to resist combinations of in-plane forces, bending moments and torsional moments in shell elements (Figure G.2 a)) may be determined by adopting a sandwich model (Figure G.2 b)), where the sectional forces are transformed into a set of statically equivalent in-plane stresses acting in the top and bottom layer of the sandwich model. The design tension reinforcement in each of the two layers may then be calculated by the method outlined above for in-plane stress conditions (membrane elements, see G.3).



a) shell element b) sandwich model

Key

1 top layer

2 bottom layer

3 intermediate layer

m_{Edx}; m_{Edy}	design bending moments per unit length
m_{Edxy}	design torsional moment per unit length
n_{Edx}; n_{Edy}	design in-plan normal force per unit length
n_{Edxy}	design in-plan shear force per unit length
v_{Edx}; v_{Edy}	design out of plan shear force per unit length

All design actions are positive as shown.

Figure G.2 — Statically equivalent set of in-plane stresses

(2) The set of statically equivalent in-plane stresses may be calculated as follows:

a) Stresses in bottom layer:

$$\sigma_{\mathrm{Edx}} = \frac{\frac{1}{2} n_{\mathrm{Edx}}}{t} + \frac{m_{\mathrm{Edx}}}{t \cdot z} \tag{G.12}$$

$$\sigma_{\mathrm{Edy}} = \frac{\frac{1}{2} n_{\mathrm{Edy}}}{t} + \frac{m_{\mathrm{Edy}}}{t \cdot z} \tag{G.13}$$

$$\tau_{\mathrm{Edxy}} = \frac{\frac{1}{2} n_{\mathrm{Edxy}}}{t} - \frac{m_{\mathrm{Edxy}}}{t \cdot z} \tag{G.14}$$

b) Stresses in top layer:

$$\sigma'_{\mathrm{Edx}} = \frac{\frac{1}{2} n_{\mathrm{Edx}}}{t} - \frac{m_{\mathrm{Edx}}}{t \cdot z} \tag{G.15}$$

$$\sigma'_{\mathrm{Edy}} = \frac{\frac{1}{2} n_{\mathrm{Edy}}}{t} - \frac{m_{\mathrm{Edy}}}{t \cdot z} \qquad \text{(G.16)}$$

$$\tau'_{\mathrm{Edxy}} = \frac{\frac{1}{2} n_{\mathrm{Edxy}}}{t} + \frac{m_{\mathrm{Edxy}}}{t \cdot z} \qquad \text{(G.17)}$$

where

t is the thickness of the top and bottom layer;

z is the internal lever arm between the top and bottom layer.

(3) If beneficial, the intermediate layer may also be utilized to carry portions of the in-plane forces (n_{Edx}, n_{Edy} and n_{Edxy}). In that case, the set of equivalent stresses in the layers should be modified accordingly.

(4) Out of plane shear forces, v_{Edx} and v_{Edy}, shall be verified according to 8.2 using the design shear force according to Formula (8.21).

(5) The particular case where $n_{\mathrm{Edx}} = n_{\mathrm{Edy}} = n_{\mathrm{Edxy}} = 0$ (slabs without membrane forces) and $|m_{\mathrm{Edxy}}| < 0{,}07 d^2 f_{\mathrm{cd}}$ may be treated according to 8.1.3; Tables G.2 and G.3 giving the least amount of required reinforcement (optimum reinforcement) or the following formulae:

$$m_{\mathrm{Rdx}} \geq m_{\mathrm{Edx}} + \cot\theta_{\mathrm{bot}} \left| m_{\mathrm{Edxy}} \right| \qquad \text{(G.18)}$$

$$m_{\mathrm{Rdy}} \geq m_{\mathrm{Edy}} + \frac{\left| m_{\mathrm{Edxy}} \right|}{\cot\theta_{\mathrm{bot}}} \qquad \text{(G.19)}$$

$$m'_{\mathrm{Rdx}} \geq -m_{\mathrm{Edx}} + \cot\theta_{\mathrm{top}} \left| m_{\mathrm{Edxy}} \right| \qquad \text{(G.20)}$$

$$m'_{\mathrm{Rdy}} \geq -m_{\mathrm{Edy}} + \frac{\left| m_{\mathrm{Edxy}} \right|}{\cot\theta_{\mathrm{top}}} \qquad \text{(G.21)}$$

Where θ_{top} and θ_{bot} denote the inclination of the compression stress fields in the top and the bottom layers carrying torsion, respectively. The values for $\cot\theta_{\mathrm{top}}$ and $\cot\theta_{\mathrm{bot}}$ are positive and may be chosen freely, but should for each of the four formulae lead to a required moment capacity that is between 0,4 to 2,5 times the corresponding optimum moment capacity (see Tables G.2 and G.3).

Table G.2 — Optimum tension reinforcement in bottom layer expressed as required design moment capacities for slab elements

Conditions	Required capacity m_{Rdx}	Required capacity m_{Rdy}
$m_{\mathrm{Edx}} \geq -\lvert m_{\mathrm{Edxy}}\rvert$ $m_{\mathrm{Edy}} \geq -\lvert m_{\mathrm{Edxy}}\rvert$	$m_{\mathrm{Rdx}} \geq m_{\mathrm{Edx}} + \lvert m_{\mathrm{Edxy}}\rvert$	$m_{\mathrm{Rdy}} \geq m_{\mathrm{Edy}} + \lvert m_{\mathrm{Edxy}}\rvert$
$m_{\mathrm{Edx}} \leq m_{\mathrm{Edy}}$ $m_{\mathrm{Edx}} < -\lvert m_{\mathrm{Edxy}}\rvert$ $m_{\mathrm{Edx}} m_{\mathrm{Edy}} \leq m_{\mathrm{Edxy}}^2$	0	$m_{\mathrm{Rdy}} \geq m_{\mathrm{Edy}} + \frac{m_{\mathrm{Edxy}}^2}{\lvert m_{\mathrm{Edx}}\rvert}$
$m_{\mathrm{Edx}} \geq m_{\mathrm{Edy}}$ $m_{\mathrm{Edy}} < -\lvert m_{\mathrm{Edxy}}\rvert$ $m_{\mathrm{Edx}} m_{\mathrm{Edy}} \leq m_{\mathrm{Edxy}}^2$	$m_{\mathrm{Rdx}} \geq m_{\mathrm{Edx}} + \frac{m_{\mathrm{Edxy}}^2}{\lvert m_{\mathrm{Edy}}\rvert}$	0
$m_{\mathrm{Edx}} < 0$ $m_{\mathrm{Edy}} < 0$ $m_{\mathrm{Edx}} m_{\mathrm{Edy}} > m_{\mathrm{Edxy}}^2$	0	0
NOTE Hogging moments with negative sign.		

Table G.3 — Optimum tension reinforcement in top layer expressed as required design moment capacities for slab elements

Conditions	Required capacity m'_{Rdx}	Required capacity m'_{Rdy}
$m_{\mathrm{Edx}} \leq \lvert m_{\mathrm{Edxy}}\rvert$ $m_{\mathrm{Edy}} \leq \lvert m_{\mathrm{Edxy}}\rvert$	$m'_{\mathrm{Rdx}} \geq -m_{\mathrm{Edx}} + \lvert m_{\mathrm{Edxy}}\rvert$	$m'_{\mathrm{Rdy}} \geq -m_{\mathrm{Edy}} + \lvert m_{\mathrm{Edxy}}\rvert$
$m_{\mathrm{Edx}} \geq m_{\mathrm{Edy}}$ $m_{\mathrm{Edx}} > \lvert m_{\mathrm{Edxy}}\rvert$ $m_{\mathrm{Edx}} m_{\mathrm{Edy}} \leq m_{\mathrm{Edxy}}^2$	0	$m'_{\mathrm{Rdy}} \geq -m_{\mathrm{Edy}} + \frac{m_{\mathrm{Edxy}}^2}{\lvert m_{\mathrm{Edx}}\rvert}$
$m_{\mathrm{Edx}} \leq m_{\mathrm{Edy}}$ $m_{\mathrm{Edy}} > \lvert m_{\mathrm{Edxy}}\rvert$ $m_{\mathrm{Edx}} m_{\mathrm{Edy}} \leq m_{\mathrm{Edxy}}^2$	$m'_{\mathrm{Rdx}} \geq -m_{\mathrm{Edx}} + \frac{m_{\mathrm{Edxy}}^2}{\lvert m_{\mathrm{Edy}}\rvert}$	0
$m_{\mathrm{Edx}} > 0$ $m_{\mathrm{Edy}} > 0$ $m_{\mathrm{Edx}} m_{\mathrm{Edy}} > m_{\mathrm{Edxy}}^2$	0	0
NOTE Hogging moments with negative sign.		

G.5 Refined control of cracking in membrane elements in SLS

(1) If in members reinforced in two orthogonal directions the angle θ between the axis of principal compressive strain and the direction of the reinforcement in the *x*-direction is larger than 15° and smaller than 75°, the crack width may be calculated with Formula (9.8), provided that $s_{\mathrm{r,m,cal}}$ and $(\varepsilon_{\mathrm{sm}} - \varepsilon_{\mathrm{cm}})$ are determined with account for the crack inclination θ, as follows.

(2) The mean spacing between cracks inclined at angle θ with the x-direction may be determined as follows:

$$s_{\mathrm{r,m,cal}} = \frac{1}{\dfrac{\sin\theta}{s_{\mathrm{r,m,cal,x}}} + \dfrac{\cos\theta}{s_{\mathrm{r,m,cal,y}}}} \tag{G.22}$$

where $s_{\mathrm{r,m,cal,x}}$ and $s_{\mathrm{r,m,cal,y}}$ correspond to the mean crack spacing related to pure tension in the x- and y-direction, respectively, and may be calculated according to 9.2.3(6).

(3) The strain difference ($\varepsilon_{\mathrm{sm}} - \varepsilon_{\mathrm{cm}}$) in the direction perpendicular to the inclined cracks (direction of principal tensile strain) may be calculated as follows:

$$\varepsilon_{\mathrm{sm}} - \varepsilon_{\mathrm{cm}} = (\varepsilon_{\mathrm{smx}} - \varepsilon_{\mathrm{cmx}}) + (\varepsilon_{\mathrm{smy}} - \varepsilon_{\mathrm{cmy}}) + |\varepsilon_2| \tag{G.23}$$

with

$$\varepsilon_{\mathrm{smx}} - \varepsilon_{\mathrm{cmx}} = \max\left(\frac{\sigma_{\mathrm{sx}} - k_{\mathrm{t}} \cdot \dfrac{s_{\mathrm{r,m,cal}}}{s_{\mathrm{r,m,cal,x}}\sin\theta} \cdot \dfrac{f_{\mathrm{ct,eff}}}{\rho_{\mathrm{px,eff}}} \cdot \left(1 + \alpha_{\mathrm{e}}\rho_{\mathrm{px,eff}}\right)}{E_{\mathrm{s}}}; \frac{(1-k_{\mathrm{t}})\sigma_{\mathrm{sx}}}{E_{\mathrm{s}}}\right) \tag{G.24}$$

$$\varepsilon_{\mathrm{smy}} - \varepsilon_{\mathrm{cmy}} = \max\left(\frac{\sigma_{\mathrm{sy}} - k_{\mathrm{t}} \cdot \dfrac{s_{\mathrm{r,m,cal}}}{s_{\mathrm{r,m,cal,y}}\cos\theta} \cdot \dfrac{f_{\mathrm{ct,eff}}}{\rho_{\mathrm{py,eff}}} \cdot \left(1 + \alpha_{\mathrm{e}}\rho_{\mathrm{py,eff}}\right)}{E_{\mathrm{s}}}; \frac{(1-k_{\mathrm{t}})\sigma_{\mathrm{sy}}}{E_{\mathrm{s}}}\right) \tag{G.25}$$

$$|\varepsilon_2| = \frac{\left|\tau_{\mathrm{Edxy}}\right| \cdot (\cot\theta + \tan\theta)}{E_{\mathrm{cm}}} \tag{G.26}$$

where

$\sigma_{\mathrm{s,x}}$ and $\sigma_{\mathrm{s,y}}$	are the reinforcement stresses in the x- and y-direction, respectively, assuming cracked sections;
$\rho_{\mathrm{px,eff}}$, $\rho_{\mathrm{py,eff}}$ and k_{t}	may be determined according to 9.2.3(3);
$f_{\mathrm{ct,eff}}$	may be taken as $0{,}5f_{\mathrm{ctm}}$;
$\cot\theta$	may be determined from a linear elastic analysis.

(4) A better estimate of $\cot\theta$ may be obtained by solving Formula (G.27):

$$\frac{\left|\tau_{\mathrm{Edxy}}\right|}{\rho_{\mathrm{x}}}\cot^4\theta + \frac{\sigma_{\mathrm{Edx}}}{\rho_{\mathrm{x}}}\cot^3\theta - \frac{\sigma_{\mathrm{Edy}}}{\rho_{\mathrm{y}}}\cot\theta - \frac{\left|\tau_{\mathrm{Edxy}}\right|}{\rho_{\mathrm{y}}} = 0 \tag{G.27}$$

where σ_{Edx}, σ_{Edy} and τ_{Edxy} are the stresses (see Figure G.1) at the point where the crack width is being determined and ρ_{x} and ρ_{y} are defined in G.3(2).

(5) The provisions for crack width control in membrane elements above may also be applied to the layers of the sandwich model (see Figure G.2).

Annex H
(informative)

Guidance on design of concrete structures for water-tightness

H.1 Use of this annex

(1) This informative annex provides supplementary guidance on the design of concrete structures that shall be tight against leakage either from water inside storage volumes, or external water like ground water.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this Informative Annex, it can be used.

H.2 Scope and field of application

(1) This Informative Annex applies mainly to water tightness of concrete structures. It may be used also for other liquids if it can be shown that the liquid behaviour is similar to water.

H.3 General

(1) Concrete in watertight structures should have an adequate composition to obtain low permeability i.e. should have a low water-cement ratio, be rich in fines and meet limitation of maximum aggregate size when the dimensions are close to the minimum values given in (2).

(2) Minimum thickness of members where tightness class TC 0 (see Table H.1) is satisfactory should be 120 mm. A minimum thickness of 150 mm should be provided for other tightness classes.

(3) The concrete sections should be designed with adequate reinforcement to ensure good crack distribution and limitation of crack-widths in accordance with 9.2.

H.4 Tightness classes

H.4.1 Classification

(1) Water retaining structures may be classified in relation to the degree of required protection against leakage according to Table H.1.

NOTE 1 All concretes permit the passage of small quantities of liquids and gases by diffusion.

Table H.1 — Classification of tightness

Tightness Class	Requirements for leakage
TC 0	Some degree of leakage acceptable, or leakage of water irrelevant.
TC 1	Leakage to be limited to a small amount. Some surface staining or damp patches acceptable.
TC 2	Leakage to be minimal. Appearance not to be impaired by staining.
TC 3	No leakage is permitted.

NOTE 2 The tightness classes can be defined quantitatively on a project-specific basis by specifying the leakage rate and relating it to the crack opening by using H.4.2(7).

H.4.2 Tightness requirements

(1) Appropriate limits to cracking depending on the tightness class of the member considered should be selected, paying due regard to the required function of the structure. In the absence of more specific requirements, the following may be adopted:

TC 0 – the provisions of 9.2.1 may be adopted.

TC 1 – the width of any cracks expected to pass through the full thickness of the section should be limited to $w_{k,lim1}$ given in (2) and the provisions of (5) and (6) should be satisfied. 9.2.1 applies where the full thickness of the section is not cracked.

TC 2 – cracks which may be expected to pass through the full thickness of the section should generally be avoided unless appropriate measures (e.g. liners, water proof membranes or water barriers) have been incorporated.

TC 3 – generally, special measures (e.g. liners or prestress) will be required to ensure water-tightness.

(2) The values of $w_{k,lim1}$ may be defined on a project-specific basis. They may be defined as a function of the ratio of the hydrostatic head h_D to the thickness of the member h_D/h, as follows:

— for $h_D/h \leq 5$: $w_{k,lim,1} = 0{,}20$ mm;

— for $h_D/h \geq 35$: $w_{k,lim,1} = 0{,}05$ mm.

For intermediate values of h_D/h, linear interpolation may be used. For the purposes of design $w_{k,cal}$ may be assumed to be ≤ $w_{k,lim}$.

(3) Limitation of the crack widths to these values typically results in the effective sealing of the cracks within a relatively short time. To account for pressurized structures, in the previous expressions h_D may be substituted by the ratio of the pressure to the specific weight of water (p/γ_{water}).

(4) To provide adequate assurance for structures of classes TC 2 or TC 3 that cracks do not pass through the full width of a section, the design value of the depth of the compression zone should be at least x_{min} under the quasi-permanent combination of actions. Where a section is subjected to alternate actions, cracks should be considered to pass through the full thickness of the section unless it can be shown that some part of the section thickness will always remain in compression. This thickness of concrete in compression should normally be at least x_{min} under all appropriate combinations of actions. The action effects may be calculated on the assumption of linear elastic material behaviour assuming that concrete in tension is neglected.

NOTE 1 x_{min} is the lesser of 50 mm or $0{,}2h$ where h is the member thickness unless the National Annex gives different values.

(5) Cracks through which water flows may be expected to heal in members which are not subjected to significant changes of loading or temperature during service. In the absence of more reliable information, healing may be assumed where the expected range of strain in the reinforcement, assuming cracked properties, at a section under service conditions is less than $150 \cdot 10^{-6}$.

NOTE 2 If self-healing is unlikely to occur, any crack which passes through the full thickness of the section can lead to leakage, regardless of the crack width.

(6) Design of members that are subjected to tensile stresses due to the restraint of shrinkage or thermal movements should account for these effects according to Annex D.

NOTE 3 Annex D gives information on the evaluation of early-age and long-term cracking due to restraint.

(7) The leakage rate q may be approximately estimated on the basis of an equivalent crack width under the assumption of a laminar flow given by:

$$q = w_{\mathrm{k,cal,e}}{}^{3} \cdot \Delta p \cdot l_{\mathrm{w,p}} / (96\, h \cdot \eta_{\mathrm{visc}}) \tag{H.1}$$

where the equivalent crack width is

$$w_{\mathrm{k,cal,e}} = \sqrt[3]{\frac{2 w_{\mathrm{k,cal,1}}^{2} \cdot w_{\mathrm{k,cal,2}}^{2}}{w_{\mathrm{k,cal,1}} + w_{\mathrm{k,cal,2}}}} \tag{H.2}$$

Δp is the pressure difference [N/m²];

$l_{\mathrm{w,p}}$ is the length of the passing crack [m];

η_{visc} is dynamic viscosity ($\eta = 1{,}3 \cdot 10^{-3}$ Ns/m² for water);

h is depth of the cross-section [m].

$w_{\mathrm{k,cal,1}}$ and $w_{\mathrm{k,cal,2}}$ are the crack widths on the two surfaces calculated for the nominal concrete cover.

NOTE 4 As the leakage rate depends on the crack width, which exhibits a large scatter both in itself as well as in the actual cracking pattern which will form in a real structure, real values of leakage will deviate from the calculated values.

Annex I
(informative)

Assessment of Existing Structures

I.1 Use of this annex

(1) This informative annex provides supplementary guidance for the assessment of existing structures in plain, reinforced and prestressed concrete.

NOTE 1 National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this Informative Annex, it can be used.

NOTE 2 The Eurocodes provide rules that are primarily intended for the design of new structures, although the principles of EN 1990 can also be applied for existing structures, with additional or amended provisions.

NOTE 3 EN 1990-2 provides additional provisions for the assessment of existing structures (for example, regarding the target reliability index to be used, assessment methods or related to using updated data in assessment) which should be used with this Annex.

I.2 Scope and field of application

(1) This Informative Annex applies to:

— the assessment of existing concrete structures;

— the assessment of the retained parts of existing concrete structures, that are being modified, extended, strengthened or retrofitted, in case of projects where new structural parts are to be combined with retained parts from the existing concrete structures.

(2) This Informative Annex does not apply to design of new structural parts that will be integrated in an existing concrete structure.

(3) This informative annex covers:

— additional rules for materials and system not defined in Clause 5 (e.g. plain bars);

— additional rules for assessing existing structures where detailing does not comply with the provisions in Clauses 11 and 12;

— additional rules for anchorage of plain bars;

— some considerations for deterioration of existing structures.

I.3 General

(1) All clauses of this Eurocode are generally applicable to the assessment of existing concrete structures unless substituted by the provisions given in Annex I.

NOTE 1 The rules for the assessment of existing structures given in EN 1990-2 can be followed.

(2) This annex does not provide predictive methods for estimating deterioration rates associated with the various deterioration mechanisms for concrete structures. These should be undertaken using methods specified by the relevant authority or, where not specified, as agreed for a specific assessment by the relevant parties.

(3) Design values determined in accordance with this Eurocode may be interpreted as assessment values for the purpose of Annex I.

(4) The following assumptions apply for the assessment of existing concrete structures:

— the structure will be used and maintained in accordance with the assessment assumptions;

— adequate supervision and quality control is provided during the assessment process;

— reasonable skill and care appropriate to the circumstances is exercised in the assessment, based on the knowledge and good practice generally available at the time the structure is assessed;

— the assessment of the structure is made by appropriately qualified and experienced personnel.

I.4 Basis of assessment

I.4.1 General rules

I.4.1.1 Basic requirements

(1) For details not fulfilling the requirements of this Eurocode, the consequence for the structural safety should be identified. In these cases, assessment based on adequate models and testing (in accordance with EN 1990) may be used.

I.4.1.2 Effects to be considered in the assessment of deteriorated structures

(1) In case of concrete structures affected by deterioration, where applicable the assessment should take into account the following effects:

— reduced concrete section due to delamination and spalling;

— reduction of cross sectional area and ductility of the reinforcement;

— stress concentration due to localized corrosion (e.g. prestressing steel);

— stress corrosion (e.g. prestressing steel);

— reduced concrete-steel bond;

— loss of mechanical properties of concrete (e.g. sulphate attack, *AAR* and *DEF*, frost attack, leaching and acid attack);

— cracking or expansion of concrete (swelling due to *AAR* and *DEF*).

NOTE 1 The most common deterioration mechanisms which can affect concrete structures are:

— reinforcement corrosion;

— external sulphate attack;

— Alkali-Aggregate Reaction (*AAR*) and other expansive reactions (e.g. Delayed Ettringite Formation, *DEF*);

— frost attack;

— leaching and acid attack.

NOTE 2 Formulae given in this Eurocode can be invalid in case of deterioration (e.g. Young modulus of concrete in case of AAR, steel ductility in case of pitting corrosion).

NOTE 3 Pitting corrosion due to chlorides is often not accompanied by other types of corrosion and hence it can be hardly detected by visual inspection of the structural member surface.

NOTE 4 Some older types of quenched and tempered prestressing steel can have a lower toughness and a higher risk for stress corrosion.

NOTE 5 The effect of deterioration can influence also model uncertainties, geometrical aspects etc. and, hence, the resistance models.

I.4.2 Verification by the partial factor method

I.4.2.1 Partial factors for assessment

(1) In the absence of tests made on the existing structures to assess the mechanical characteristics of materials, the partial factors for materials given in Table 4.3 (NDP) should be used for assessment.

(2) If tests are made on the existing structure to assess the mechanical characteristics of materials or update geometric parameters, the partial factors for materials given in Table 4.3 (NDP) may be adjusted by using the actual mean values and coefficients of variation derived by tests unless the coefficient of variation of a measured parameter is significantly greater than the value normally assumed as reference value for new structures, in that case the corresponding partial factors should be adjusted as follows:

— For concrete strength the partial factor should be adjusted if the coefficient of variation of concrete core strength $V_{\mathrm{fc,is}}$ is greater than $V_{\mathrm{fc,is,lim}}$. The results of tests undertaken earlier (e.g. during the construction) may also be used for this purpose. $V_{\mathrm{fc,is,lim}}$ is a parameter which depends on the number of test results according to Table I.1 (NDP).

— For reinforcement samples the partial factor should be adjusted if the coefficient of variation of yield strength V_{fy} is greater than $V_{\mathrm{fy,lim}}$. The results of tests undertaken earlier (e.g. during the construction) may also be used for this purpose. $V_{\mathrm{fy,lim}}$ is a parameter which depends on the number of test results according to Table I.2 (NDP).

— For other parameters the partial factor should be adjusted in a comparable way

NOTE 1 The values of $V_{\mathrm{fc,is,lim}}$ given in Table I.1 (NDP) and $V_{\mathrm{fy,lim}}$ given in Table I.2 (NDP) apply unless the National Annex gives different values.

NOTE 2 Information from test results can be combined with prior information if available.

NOTE 3 The values given in Table I.1 (NDP) and Table I.2 (NDP) are derived assuming a target reliability index $\beta_{\mathrm{tgt}} = 3{,}8$. If a different target value is used (see EN 1990), the values in Table I.1 (NDP) and Table I.2 (NDP) can be rearranged. For a target reliability index $\beta_{\mathrm{tgt}} < 3{,}8$ the values in the tables are conservative.

NOTE 4 For the adjustment of partial factors for materials, see Annex A.

Table I.1 (NDP) — Values of $V_{\mathrm{fc,is,lim}}$ as function of the number of test results n

n[a]	8	10	12	16	20	30	∞
$V_{\mathrm{fc,is,lim}}$	0,13	0,15	0,17	0,20	0,21	0,23	0,29

[a] n is the number of test results available for the evaluated test region

Table I.2 (NDP) — Values of $V_{\mathrm{fy,lim}}$ as function of the number of test results n

n	5	8	10	12	16	20	30	∞
$V_{\mathrm{fy,lim}}$	0,025	0,050	0,060	0,065	0,080	0,085	0,095	0,120

I.5 Materials

I.5.1 General

(1) For materials not meeting the requirements of the relevant standards referred to from this document, the mechanical characteristics to be used according to this Eurocode, should be derived from tests and/or design and construction records and the use of the provisions in this Eurocode or of alternative approaches present in technical and scientific literature should be justified.

(2) Testing of materials should be undertaken in accordance with the relevant standards. If such tests cannot be performed, alternative test methods may be used when specified by the relevant authority or, where not specified, agreed for a specific assessment by the relevant parties. The effect of the alternative test method should be taken into account when deriving the mechanical characteristics to be used according to this Eurocode.

I.5.2 Concrete

I.5.2.1 General

(1) EN 13791:2019, Clause 8 should be used for the determination of the characteristic value $f_{\mathrm{ck,is}}$ insitu compressive strength of concrete cores.

(2) The investigation of concrete should aim mainly to determine the compressive strength in specific areas of the structure. If deterioration does not significantly influence the other mechanical properties (e.g. modulus of elasticity, tensile strength), these may be determined indirectly based on the compressive strength, by using the formulae included in 5.1, if no specific investigation is conducted.

(3) If f_{ck} is assessed by using insitu testing, this may be estimated from $f_{\mathrm{ck,is}}$ according to Formula (I.1):

$$f_{\mathrm{ck}} = \frac{f_{\mathrm{ck,is}}}{k_{\mathrm{\mu fc}}} \tag{I.1}$$

where the values of are given in Table I.3 (NDP).

NOTE The values of $k_{\mathrm{\mu fc}}$ in Table I.3 (NDP) apply unless the National Annex provides other values.

Table I.3 (NDP) — Parameter $k_{\mathrm{\mu fc}}$ considering the representativeness of the insitu compressive concrete strength assessed according to EN 13791:2019, Clause 8 in Formula (I.1)

Regions and conditions of the structural member where the cores are extracted[a]	$k_{\mathrm{\mu fc}}$
a) Cores extracted only from the bottom parts of the concrete masses during casting (lower 70 % of the depth of concrete during casting) not necessarily representing the governing region for the verification	0,95
b) Cores extracted from different regions representing all conditions in the structural member, but not necessarily representing the governing region for the verification	0,90
c) Cores extracted from the region governing for the verification	0,85
[a] The insitu concrete compressive strength can exhibit significant variations depending on the location (strength typically smaller in the upper part of the element during casting)	

(4) If D_{lower} or D_{upper} are not known, they may be replaced by D_{max}. D_{max} may be assessed by testing on the existing structure or by using original design and construction records.

I.5.2.2 Assessment assumptions

(1) When the concrete strength is assessed in an existing structure, the factor k_{tc} in 5.1.6(1) shall take into account the effects of stress level and duration of loading.

NOTE The following value of k_{tc} applies unless the National Annex gives different values:

— $k_{tc} = 0{,}85$ in case the effect of the permanent action (and/or variable actions of duration > 1 hour) represents 100 % of the total effect at assessment level;

— $k_{tc} = 1{,}00$ in case the effect of the variable action with duration < 1 hour represents at least 20 % of the total effect at assessment level;

for intermediate cases (effect of variable actions with duration < 1 hour is between 0 % and 20 % of the total load effect at assessment level), k_{tc} can be determined by linear interpolation between 0,85 and 1,00.

I.5.3 Reinforcing steel

(1) Where the characteristic values of the properties of reinforcing steel are assessed from testing samples extracted from a structure, the number, location and size of the test specimens should be selected to be representative of the members being assessed. In this case, the properties of reinforcing steel should be tested in accordance with EN ISO 15630 (all parts). Shorter samples may be tested if:

— the test results give no grounds for justified doubts;

— no signs of deterioration are discerned;

— from experience it is known that the ultimate resistance is not sensitive to applied length of specimens.

NOTE 1 The strengths of different size and types of reinforcing steel can have significantly different probability distributions.

NOTE 2 As it is not usually known which reinforcing steel originates from a single batch, there can be a need to investigate the possibility of multiple batches being used within the location being assessed. This can require a larger number of specimens being tested and some engineering judgement to be used regarding the data analysis and the scope of application of the resulting strengths.

NOTE 3 Information from test results can be combined with prior information if available.

(2) Where the characteristic value of the properties of reinforcing steel is calculated based on sample testing, the characteristic value X_k should be determined as given in Formula (I.2):

$$X_k = e^{(m_X - k_n \cdot s_X)} \tag{I.2}$$

where

$$m_X = \frac{1}{n}\sum \ln x_i \tag{I.3}$$

where

n is the number if samples and the value of k_n may be taken from Tables I.4 and I.5 for 5 % and 10 % characteristic value respectively.

For the evaluation of s_X and when using Tables I.4 and I.5, one of two cases should be considered:

— **Case 1:** The row 'V_X known' should be used if the coefficient of variation V_X, or a realistic upper bound of it, is known from prior knowledge; in this case s_X should be taken from Formula (I.4):

$$s_X = \sqrt{\ln(V_X^2 + 1)} \tag{I.4}$$

— **Case 2:** The row 'V_X unknown' should be used if the coefficient of variation V_X is not known from prior knowledge and so needs to be estimated from the sample; in this case s_X and V_X should be taken from Formulae (I.5) and (I.6), respectively:

$$s_X = \sqrt{\frac{1}{n-1}\sum(\ln x_i - m_x)^2} \tag{I.5}$$

$$V_X = \frac{s_X}{m_X} \tag{I.6}$$

Table I.4 — Values of k_n for the 5 % characteristic value

n	1	2	3	4	5	6	8	10	15	20	30	∞
V_X Known	2,31	2,01	1,89	1,83	1,80	1,77	1,74	1,72	1,70	1,68	1,67	1,64
$V_X$unknown	–	–	3,37	2,63	2,33	2,18	2,00	1,92	1,82	1,76	1,73	1,64

Table I.5 — Values of k_n for the 10 % characteristic value

n	1	2	3	4	5	6	8	10	15	20	30	∞
V_X known	1,81	1,57	1,48	1,43	1,40	1,38	1,36	1,34	1,32	1,31	1,30	1,28
V_X unknown	–	–	2,18	1,83	1,68	1,59	1,50	1,45	1,39	1,36	1,33	1,28

(3) If sufficient information is available, the mechanical characteristics of ribbed bars may be determined based on marking on the bar surface.

I.5.4 Prestressing steel

(1) I.5.3 applies also for prestressing steel.

I.6 Durability - Minimum cover for bond

(1) If the actual cover is less than $c_{min,b}$ the effect on the anchorage and lap length may be determined according to I.11.4.1.

NOTE The actual cover to evaluate the anchorage and lap length can be assessed by using original design and construction records and/or testing on the existing structure.

(2) $c_{min,b}$ for square section plain bars should be based on $\phi_{sq,eq}$ defined in I.11.4.1.

I.7 Structural analysis

I.7.1 Methods of analysis

I.7.1.1 General

(1) When linear analysis with redistribution, plastic analysis or non-linear analysis is performed, the effect of reduced ductility of deteriorated concrete and reinforcement should be taken into account, where relevant.

I.7.1.2 Linear elastic analysis with redistribution

(1) In addition to 7.3.2, linear elastic analysis with redistribution under the ultimate limit state may be carried out if shear forces and support reactions used in assessment are taken as the greater of those calculated either prior to, or after redistribution.

I.7.2 Prestressed members and structures

I.7.2.1 General

(1) When tendon corrosion is encountered in the assessment, the normal rules for prestressed concrete should be modified by taking into account the following:

a) Strands, wires or bars which have suffered sectional loss that has resulted in them being unable to sustain their prestress force should be considered ineffective at that section. The strength of a section at the ultimate limit state should be based on the remaining cross sectional area of the effective strands, wires or bars only.

b) Bonded post-tensioning tendons which are ineffective locally can re-anchor and become fully effective elsewhere. Such tendons should be considered in the assessment only if the quality of grouting in the ducts allows anchorage of the design strength of the prestressing steel.

c) Where there is evidence of extensive inadequate grouting, the possible re-anchorage of tendons should not be considered in the assessment without further investigation. If the grouting is too poor to allow re-anchorage of tendons, the member should be treated as unbonded and assessed accordingly.

d) The reduction of ductility due to corrosion should be taken into account.

I.7.2.2 Prestressing force

(1) The maximum prestressing force assumed to be applied should be derived from the original design and construction records or, if not available from documented information for the applied prestressing system valid at the time of construction. If the prestressing level is not known, the effect of variation in the prestressing force should be subject to sensitivity analysis. The actual prestressing force may be measured by insitu testing if no sectional loss is detected.

I.8 Ultimate Limit States (ULS)

I.8.1 General

(1) The section properties used to assess section resistance of deteriorated structures should be consistent with those used in the analysis where relevant.

(2) The following should be considered as possible consequences of reinforcement corrosion, causing cracking and spalling of concrete cover, on cross sectional dimensions of concrete to be considered:

— for corrosion penetration depth $P_x \geq 0{,}2$ mm to 0,4 mm, or crack widths ≥ 1mm, a reduced concrete section may be considered due to spalling ignoring the cover depth around the corroded bars;

— for low/medium P_x (i.e. $P_x < 0{,}2$ mm to 0,4 mm), or crack widths < 1 mm, it may be assumed that the complete concrete section contributes to the resistance with a reduced compressive strength of concrete due to cracking.

NOTE Concrete spalling does not only depend on the level of corrosion but also on the ratio of longitudinal and transverse reinforcement, diameter of the bars, etc.

(3) The following should be considered as possible consequences of corrosion on ordinary reinforcement:

— for compressed ordinary reinforcement, a reduced reinforcement strength should be considered due to possible bar buckling before the maximum load is reached, if stirrups are heavily corroded (relevant P_x or pits);

— in shear the possibility of premature failure of stirrups due to pitting corrosion should be considered;

— for pitting corrosion or $P_x \geq 0{,}2$ mm to 0,4 mm:

  — a reduction of elongation at maximum stress can be expected and should be considered for the verifications at ULS;

  — a concentration of the active stress at pits should be considered;

— values of $P_x < 0{,}2$ mm to 0,4 mm may be assumed not to affect the stress-strain deformation relationships of ordinary reinforcement.

I.8.2 Bending with or without axial force

(1) Where aspects of detailing are present that do not comply with the provisions of this Eurocode the effects on the resistance should be assessed.

NOTE In addition to the effects on bond, low concrete cover can be relevant for other effects such as buckling of bars in compression.

I.8.3 Shear

I.8.3.1 Detailed verification of members not requiring shear reinforcement

(1) Where more refined verification is needed for the purpose of assessment of existing structures, as an alternative to 8.2.2(2) to (5), the design value of the shear stress resistance of members without shear reinforcement may be calculated as follows:

$$\tau_{\mathrm{Rd,c}} = 0{,}33 \cdot \frac{\gamma_{\mathrm{def}}^{2/3}}{\gamma_{\mathrm{V}}^{2}} \cdot \frac{\sqrt{f_{\mathrm{ck}}}}{1 + 24\gamma_{\mathrm{def}} \cdot \varepsilon_{\mathrm{v}} \cdot \dfrac{d}{d_{\mathrm{dg}}}} \tag{I.7}$$

where

ε_{v} is the strain in the longitudinal reinforcement according to (2) which considers implicitly all effects covered by 8.2.2(3) to (5);

γ_{def} is a partial safety factor which covers the uncertainties related to the calculation of the deformation.

NOTE $\gamma_{\mathrm{def}} = 1{,}33$ unless the National Annex gives a different value.

The shear stress resistance τ_{Rd} shall be not smaller than the design value of the shear stress τ_{Ed} calculated according to 8.2.1(3) for a cross section defined with the principles of 8.2.2(1), but located not closer than $d/2$ from a support, a concentrated load or a discontinuity.

(2) The strain in the longitudinal tensile reinforcement ε_v at control section to be used in Formula (I.7) should be calculated according to the assumptions of 8.1.1. For planar members, it refers to the principal direction of the shear force according to 8.2.1(5), a non-linear cross-sectional analysis of the structure may be performed and the obtained internal forces as well as the strain ε_v may be averaged over the same width defined in 8.2.1(6).

(3) For linear members with an effective depth $d > 500$ mm, the approach described in (1) and (2) should be used instead of the method of 8.2.2 or, alternatively, the resistances according to Formula (8.27) should be multiplied with following coefficient:

$$k_{\mathrm{vd}} = 1{,}35\left(100\rho_{\mathrm{l}}\frac{d_{\mathrm{dg}}}{d}\right)^{1/10} \leq 1{,}0 \tag{I.8}$$

I.8.3.2 Detailed verification of members requiring design shear reinforcement

(1) The assessment of members requiring design shear reinforcement may be conducted stepwise as follows:

(i) with the simplified method according to 8.2.3(1) to (6) without explicit calculation of strains;

(ii) with the refined method according to 8.2.3(6) with explicit calculation of strains;

(iii) according to Annex G where the strain compatibility is accounted for;

(iv) with a refined nonlinear analysis according to 7.3.4.

(2) For members not complying with the requirement of maximum longitudinal spacing $s_{c,max}$ of shear reinforcement according to Table 12.1 unless more refined models are used, the shear resistance $\tau_{Rd,sy}$ according to Formula (8.42) should be multiplied with coefficient k_{ns} and the stress in the compression field σ_{cd} according to Formula (8.44) should be divided by coefficient k_{ns} which can be calculated as follows:

$$k_{\mathrm{ns}} = 1 - \frac{s}{z\cot\theta} \tag{I.9}$$

(3) If the transverse spacing of shear legs in an existing member is larger than the maximum value given in 12.3.1 and 12.4.1 for beams and slabs respectively, and if both 8.2.1(1)(i) and 8.2.1(1)(ii) are not fulfilled, the shear resistance should be evaluated by considering the value for the width of the cross section b_w given in Formula (I.10) unless more refined models (e.g. models consistent with 8.3.5(2)) that consider the existing shear legs are used to evaluate shear resistance.

$$b_{\mathrm{w}} = \sum_{i=1}^{n} b_{\mathrm{w,i}} \tag{I.10}$$

where

n is the number of legs in the cross section;

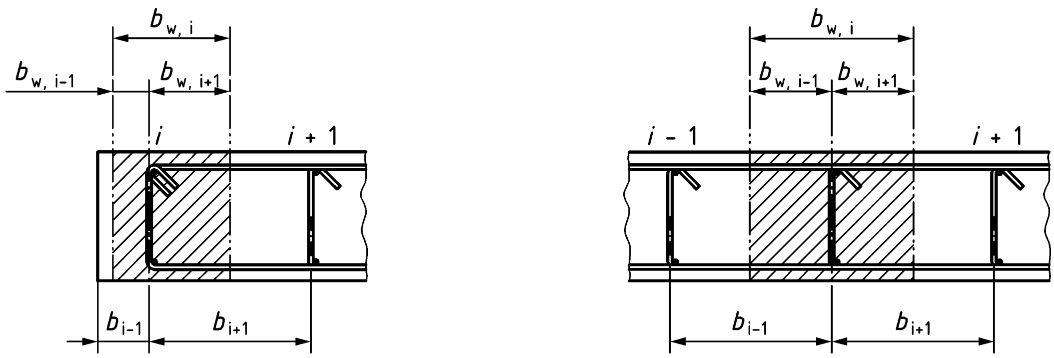
$$b_{\mathrm{w,i}} = b_{\mathrm{w,i-1}} + b_{\mathrm{w,i+1}} \tag{I.11}$$

$$b_{\mathrm{w,i-1}} \leq \begin{cases} \dfrac{b_{\mathrm{i-1}}}{2} \\ \dfrac{s_{\mathrm{tr,max}}}{2} \end{cases} \tag{I.12}$$

$$b_{\mathrm{w,i+1}} \leq \begin{cases} \dfrac{b_{i+1}}{2} \\ \dfrac{s_{\mathrm{tr,max}}}{2} \end{cases} \tag{I.13}$$

$s_{\mathrm{tr,max}}$ is the maximum value of the transverse spacing of shear legs given in in 12.3.1 and 12.4.1 for beams and slabs respectively;

$b_{\mathrm{w,i-1}}$ and $b_{\mathrm{w,i+1}}$ are defined in Figure I.1.



a) edge leg

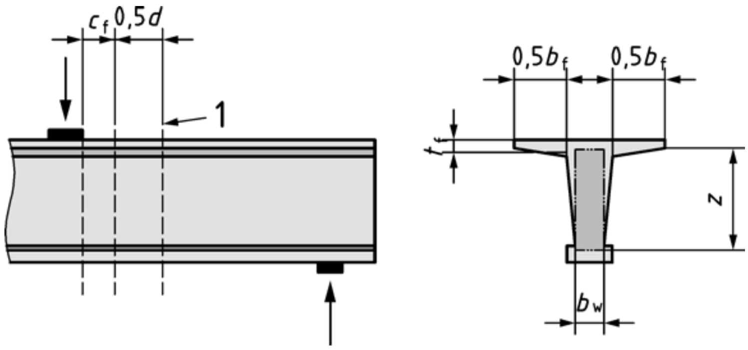
b) interior leg

Figure I.1 — Definition of $b_{\mathrm{w,i}}$ (beam/slab cross section)

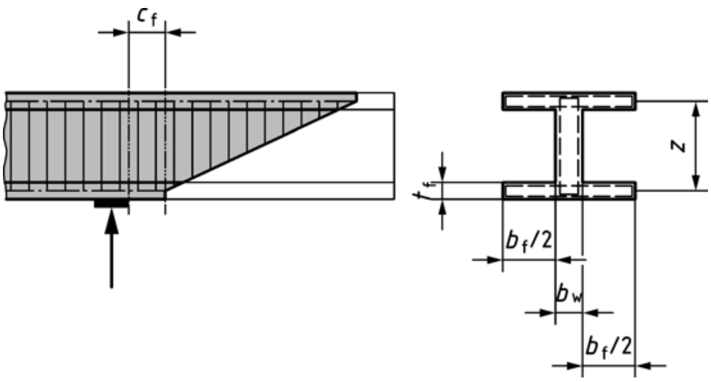
(4) The favourable effect of the presence of compression flanges may be accounted for by determining the location of the governing control section considering an additional distance c_{f} with respect to concentrated loads or reaction forces acting on compression flanges as shown in Figure I.2:

$$c_{\mathrm{f}} = 3t_{\mathrm{f}}\sqrt{\frac{A_{\mathrm{f}}}{A_{\mathrm{w}}}} \tag{I.14}$$

where $A_{\mathrm{f}} = \min\{b_{\mathrm{f}} \cdot t_{\mathrm{f}};\ 6t_{\mathrm{f}}^2\}$ and $A_{\mathrm{w}} = \min\{b_{\mathrm{w}} \cdot z;\ 8b_{\mathrm{w}}^2\}$.



a) near concentrated load



b) near bearing

Key

1 control section

Figure I.2 — Location of control section in presence of compression flanges with or without shear reinforcement

I.8.3.3 Shear at interfaces

(1) In the parts of a composite slab where the spacing between interface reinforcement in the shear transfer direction does not fulfil the requirements of 8.2.6(9), the interface reinforcement should be considered in interface shear verifications only for the parts of the slab (in the shear transfer direction) that have a length, for each reinforcing bar crossing the interface, equal to $1{,}25h \leq 150$ mm per each side (in the shear transfer direction) of the bar unless more refined models that consider the existing interface reinforcement are used to evaluate shear resistance at the interface.

(2) In the parts of a composite slab where the spacing between interface reinforcement perpendicular to the shear transfer direction does not fulfil the requirements of 8.2.6(9), the interface reinforcement should be considered in shear at interface verifications only for that part of the slab that has a width, for each reinforcing bar crossing the interface, equal to $2{,}5h \leq 375$ mm ($\leq$ the distance from the edge for an edge bar) per each side of the bar unless more refined models that consider the existing interface reinforcement are used to evaluate shear resistance at interface.

I.8.4 Torsion and combined actions

(1) Unless more refined methods of analysis are used, stirrups having spacing larger than the minimum of $u/8$, b and h, shall not be considered in the evaluation of torsional resistance.

I.8.5 Detailed verification of punching

I.8.5.1 Punching shear resistance of slabs without shear reinforcement

(1) The favourable effect of compressive membrane action around internal columns without significant openings, inserts or slab edges at a distance less than $5d_{\mathrm{v}}$ from the control perimeter $b_{0,5}$, may be considered by multiplying parameter k_{pb} in Formula (8.94) by the following enhancement factor:

$$\eta_{\mathrm{pm}} = 0{,}95\left(\frac{h}{d}\right)^{\frac{1}{2}} \cdot \left(\frac{\sqrt{f_{\mathrm{ck}}}}{\rho_l \cdot f_{\mathrm{yk}}}\right)^{\frac{1}{4}} \geq 1 \tag{I.15}$$

(2) As an alternative to 8.4.3, the design punching shear stress resistance τ_{Rd} of slabs without shear reinforcement to be compared with τ_{Ed} according to 8.4.2(6) or (7) may be calculated as follows:

$$\tau_{\mathrm{Rd,c}} = 0{,}75 \cdot \frac{\gamma_{\mathrm{def}}^{2/3}}{\gamma_{\mathrm{v}}^{2}} \cdot \frac{\sqrt{f_{\mathrm{ck}}}}{1 + 15\gamma_{\mathrm{def}} \cdot \psi \cdot \dfrac{d_{\mathrm{v}}}{d_{\mathrm{dg}}}} \tag{I.16}$$

where

ψ in radians is the maximum rotation of the slab around the supported area according to (3) which considers explicitly all effects covered by (1) and 8.4.3(2) to (4).

NOTE For γ_{def} see I.8.3.1(1).

(3) The rotation ψ may be calculated on the basis of a non-linear analysis of the structure and accounting for cracking, tension-stiffening effects, yielding of the reinforcement, membrane action and any other non-linear effects relevant for providing an accurate assessment of the structure. The governing value of ψ is the maximum relative rotation between centre of the supporting area and a distance $2d_{\mathrm{v}}$ from the control perimeter.

I.8.5.2 Punching shear resistance of slabs with shear reinforcement

(1) The design punching shear stress resistance of slabs with shear reinforcement, in case the strain-based approach described in I.8.5.1(2) and (3) is used, may be calculated as:

$$\tau_{\mathrm{Rd,cs}} = \tau_{\mathrm{Rd,c}} + \eta_{\mathrm{s}} \cdot \rho_{\mathrm{w}} \cdot f_{\mathrm{ywd}} \geq \rho_{\mathrm{w}} \cdot f_{\mathrm{ywd}} \tag{I.17}$$

where

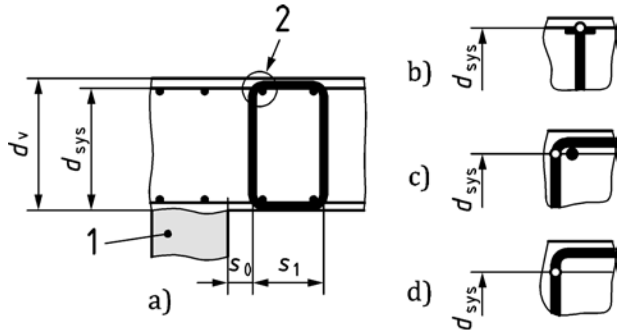
$\tau_{\mathrm{Rd,c}}$ should be calculated according to Formula (I.16);

$$\eta_{\mathrm{s}} = \frac{E_{\mathrm{s}} \cdot \psi}{7{,}5 \cdot f_{\mathrm{ywd}}} \cdot \left(1 + \frac{d_{\mathrm{v}}}{150 \cdot \phi_{\mathrm{w}}}\right) \leq 0{,}8 \tag{I.18}$$

(2) For the verification of the maximum shear resistance τRd,max, the coefficient ηsys in Formula (8.109) may be calculated in a more refined manner according to:

$$\eta_{\mathrm{sys}} = 1{,}15\frac{d_{\mathrm{sys}}}{d_{\mathrm{v}}} + 0{,}63\left(\frac{b_0}{d_{\mathrm{v}}}\right)^{1/4} - 0{,}85\frac{s_0}{d_{\mathrm{sys}}} \geq 1{,}0 \tag{I.19}$$

where d_{sys} and s_0 are defined in Figure I.3. For variable distances s_0, the average over the control perimeter should be used in Formula (I.19). For slabs with shear reinforcement not complying with the detailing rules of Clause 12, I.8.5.3 applies.



a) overview
b) top of head
c) level of axis of reinforcement inside the bend
d) limit of bend

Key

1 support

2 anchorage details in b) to d)

Figure I.3 — Definition of parameter d_{sys}

(3) Alternatively, the maximum shear resistance $\tau_{Rd,max}$ may be calculated with the strain-based approach described in 8.5.1(2) and (3) according to:

$$\tau_{Rd,max} = \eta_{sys,sb} \cdot \tau_{Rd,c} \tag{I.20}$$

where

$\tau_{Rd,c}$ is calculated according to Formula (I.16) using the rotation ψ due to the external actions or with the actions which correspond to $\tau_{Rd,max}$ (in this case, an iteration is needed), and

$\eta_{sys,sb}$ may be assumed as:

$$1.0 \leq \eta_{sys,sb} = 3 \cdot \left(\frac{d_{sys}}{d_v}\right)^{3/2} \cdot \left(\frac{b_0}{d_v}\right)^{1/2} \frac{1}{1 + 5 \cdot \frac{s_{0,t}}{d_v}} \leq 4.0 \tag{I.21}$$

where

$s_{0,t}$ $=0.8 \cdot s_0 + 0.2 \cdot s_t \geq s_0$

(4) The verification for punching outside the shear reinforced zone, in case the strain-based approach described in I.8.5.1 (2) and (3) is used, may be conducted alternatively to 8.4.4(7) and (8) in a similar manner as slabs without shear reinforcement by considering the shear resistant effective depth $d_{v,out}$ according to Figure 8.23 and the outer control perimeter defined in Figure 8.24.

I.8.5.3 Verification of punching in slabs with shear reinforcement not complying with the detailing rules of Clause 12

(1) The maximum punching shear resistance of slabs with shear reinforcement not complying with the requirements of 12.5.1 may be calculated according to I.8.5.2(2) or (3) with following modifications:

— if the distance s_0 between the column edge and the first reinforcement unit is smaller than the lower limit according to Figure 12.7: the distance s_1 instead of s_0, should be used in Formulae (I.19) or (I.21),

— if the distance s_0 between the column edge and the first reinforcement unit is larger than the upper limit according to Figure 12.7: Formulae (I.19) or (I.21) are applicable,

— if the shear reinforcement doesn't enclose at least the 3rd layer of longitudinal reinforcement according to Figure 12.6, coefficient 1,15 in Formula (I.19) should be replaced by 0,85.

I.8.6 Partially loaded areas

(1) If there are no external tensile forces nor restraint which can induce cracking in the direction parallel to the direction of the load, and no expected or known pre-existing cracks in this direction, tensile reinforcement may be omitted, provided that the design stress applied on the loaded area does not exceed $\sigma_{Rd,t}$:

$$\sigma_{\mathrm{Rd,t}} = \frac{\sqrt{f_{\mathrm{ck}}}}{\gamma_{\mathrm{C}}} \cdot k_{\mathrm{part}} \tag{I.22}$$

if $\max\{a_0; b_0\} \leq 3d_0$:

$$k_{\mathrm{part}} = \frac{A_{\mathrm{c1}}}{A_{\mathrm{c0}}} \cdot \frac{1}{1 - \sqrt{\frac{A_{\mathrm{c0}}}{A_{\mathrm{c1}}}}} \tag{I.23}$$

if $\max\{a_0; b_0\} > 3d_0$:

$$k_{\mathrm{part}} = \frac{d_1}{d_0} \cdot \frac{1}{1 - \frac{d_0}{d_1}} \tag{I.24}$$

where

d_0 = $\min\{a_0; b_0\}$ according to 8.6(2);

d_1 block dimension parallel to d_0.

I.9 Serviceability Limit States (SLS)

I.9.1 General

(1) Assessment of serviceability limit states by calculation according to Clause 9 should be performed when:

(i) investigating existing serviceability problems;

(ii) the assessment of structural safety relies on particular serviceability criteria being satisfied;

(iii) required by I.9.1(2).

In the other cases the serviceability limit state verifications may be performed using site-based observations and/or measurements.

(2) If the target value for reliability index β for ultimate limit states is reduced to a value lower than that given in EN 1990, the reinforcement and the concrete stresses at the characteristic combination of actions should be limited.

NOTE The limits on reinforcement and concrete stresses at the characteristic combination of actions are given in Table I.6 (NDP) unless the National Annex gives different values.

Table I.6 (NDP) — Limits on reinforcement and the concrete stresses at the characteristic combination of actions

$\sigma_c \leq 0{,}60 f_{ck}$	$\sigma_s \leq 0{,}80 f_{yk}$	$\sigma_p \leq 0{,}80 f_{pk}$

I.9.2 Crack control

(1) For plain bars, Formula (9.15) shall not be applied and Formula (I.25) may be used.

$$s_{r,m,cal} = 2c + 0{,}56 \frac{\phi}{\rho_{p,eff}} \leq 1{,}3(h - x) \tag{I.25}$$

I.10 Fatigue

(1) Where there is a particular concern regarding the fatigue performance of the structure, the need for a fatigue verification should be specified by the relevant authority or, where not specified, agreed for a specific assessment by the relevant parties.

NOTE 1 Examples of particular concerns regarding the fatigue performance can include:

(i) fatigue-sensitive members where it is known that there is:

— welded reinforcement;

— corroded reinforcement;

— mechanically connected reinforcing bars;

— bent parts of bars with mandrel diameter smaller than the minimum values according to specifications;

— reduced cross sectional area and/or reduced bond area of bars which can cause stress concentrations.

(ii) inadequately restrained external tendons can vibrate excessively and be more susceptible to fatigue failure.

NOTE 2 The influence of the surface quality of reinforcement (e.g. corrosion, notches) on the fatigue strength can be considered by taking into consideration the loading history and stress concentration arising from deterioration.

(2) When the rules of Annex E are used for the assessment of an existing structure, once corrosion has started the fatigue resistance of bars may be determined by reducing the stress exponent in Table E.1 (NDP) for straight and bent carbon reinforcing steel to $k_{f2} = 5$.

I.11 Detailing of reinforcement and post-tensioning tendons

I.11.1 General

(1) Provisions of Clause 11 apply also to plain bars unless Annex I gives different provisions.

I.11.2 Spacing of bars

(1) For the calculation of anchorage length, if the clear distance c_s (horizontal and vertical) between individual parallel bars is lower than $D_{upper} + 5\text{mm}$, the bars should be considered as arranged in bundles.

NOTE The actual c_s to evaluate the anchorage length can be assessed by using original design and construction records and/or testing on the existing structure.

(2) For the calculation of anchorage and lap length, if the clear distance c_s is less than 2ϕ the effect on the anchorage and lap length may be determined according to I.11.4.1.

(3) 11.2(4) shall not be applied in assessment of existing structures.

I.11.3 Permissible mandrel diameter for bent bars

NOTE For bars that do not comply with EN 10080, the minimum mandrel diameter to satisfy 11.3(1) can be found in the national technical standards in force at the time of the original design.

I.11.4 Anchorage of reinforcing steel in tension and compression

I.11.4.1 Anchorage of straight bars

(1) The nominal cover, as defined by Figure 11.3 c), may be assessed by using original design and construction records and/or updated based on testing on the existing structure.

(2) The minimum value of l_{bd} in Formula (11.3) and in 11.4.2(6) (i.e. 10ϕ) may be reduced to 5ϕ if visual inspections demonstrate that the area where anchorage develops is uncracked in the present condition.

(3) For plain straight bars with $\sigma_{sd} \leq 300$ MPa, the design anchorage length l_{bd} may be taken from Formula (I.26):

$$\frac{l_{bd}}{\phi} = (130\eta_1) \cdot \left(\frac{\gamma_c}{1{,}5}\right)^{\frac{3}{2}\eta_2} \cdot \left(\frac{\sigma_{sd}}{435}\right)^{\frac{5}{4}\eta_3} \cdot \left(\frac{25}{f_{ck}}\right)^{\frac{2}{3}\eta_4} \cdot \left(\frac{1{,}5\phi}{c_d}\right) \geq 10 \tag{I.26}$$

where

— ratio $\left(\frac{1{,}5\phi}{c_d}\right) \geq 0{,}5$;

— η_1, η_2, η_3 and η_4 are coefficients that take account of the position of the bars during concreting and in particular:

 — $\eta_1 = \eta_2 = \eta_3 = \eta_4 = 1{,}0$ for bars in good bond conditions;

 — $\eta_1 = 3{,}1$, $\eta_2 = 1{,}6$, $\eta_3 = 0{,}9$ and $\eta_4 = 0{,}6$ in all other cases.

NOTE 1 For details on good bond conditions, see Figure 11.4.

NOTE 2 Some studies on small diameter (i.e. $\phi \leq 16$ mm) cold drawn plain reinforcement demonstrate that such bars have a much smoother surface and exhibit lower bond strengths attained at higher slips compared with hot rolled bars. Thus, the design anchorage length of small diameter cold drawn plain bars is longer than that evaluated by using Formula (I.26).

(4) All provisions for anchorage of plain bars may be used also for square section plain bars with parameter ϕ replaced by an equivalent diameter of the square bar $\phi_{sq,eq}$ given by Formula (I.27):

$$\phi_{\mathrm{sq,eq}} = \sqrt{\frac{4}{\pi} A_{\mathrm{sq}}} \qquad \text{(I.27)}$$

where A_{sq} is the area of the square cross section of the plain bar.

(5) 11.4.2(5) and 11.4.2(6) shall not be applied to plain bars.

(6) 11.4.2(5) should not be applied to ribbed and indented bars if $c_{\mathrm{d}} < \phi$.

(7) If $c_{\mathrm{d}} < \phi$ all provisions for anchorage of ribbed, indented and plain bars may be used with parameter c_{d} replaced by ϕ and by increasing l_{bd} in Formulae (11.3) and (I.26) by a factor $k_{\mathrm{lbs,c}}$. $k_{\mathrm{lbs,c}}$ should be taken from Table I.7.

NOTE 3 Corner and edge bars are defined in Figure 11.3.

Table I.7 — Parameter $k_{\mathrm{lbs,c}}$ for ℓ_{bd} when $c_{\mathrm{d}} < \phi$

Case	Position of bar	$c_{\mathrm{s}}/2$	c_{x}	c_{y}	$k_{\mathrm{lbs,c}}$
1	corner	$\geq \phi$	$\geq \phi$	$< \phi$	$\frac{1}{0{,}5 + 0{,}5\frac{c_{\mathrm{y}}}{\phi}}$
2[b]	corner	$< \phi$	$\geq \phi$	$< \phi$	
3	edge	$\geq \phi$	–	$< \phi$	
4[b]	edge	$< \phi$	–	$< \phi$	
5	corner	$\geq \phi$	$< \phi$	$\geq \phi$	$\frac{1}{0{,}5 + 0{,}5\frac{c_{\mathrm{x}}}{\phi}}$
6[b]	corner	$< \phi$	$< \phi$	$\geq \phi$	
7[a]	corner	$\geq \phi$	$< \phi$	$< \phi$	$\frac{1}{\frac{c_{\mathrm{min,xy}}}{\phi}}$ if $\frac{1}{3} \leq \frac{c_{\mathrm{min,xy}}}{\phi} < 1$ $\frac{1}{0{,}1 + 0{,}7\frac{c_{\mathrm{min,xy}}}{\phi}}$ if $\frac{c_{\mathrm{min,xy}}}{\phi} < \frac{1}{3}$
8	edge	$< \phi$	–	$\geq \phi$	$\frac{1}{0{,}5 + 0{,}5\frac{c_{\mathrm{s}}}{2\phi}}$

[a] In case 7 $c_{\mathrm{min,xy}} = \min(c_{\mathrm{x}}; c_{\mathrm{y}})$

[b] In cases 2, 4 and 6, bars having $c_{\mathrm{s}}/2 < \phi$ shall be considered as bundled.

(8) For ribbed bars having f_{R} lower than values $f_{\mathrm{R,min}}$ of Table C.1, l_{bd}, evaluated according to 11.4.2, should be increased depending on the actual f_{R}.

(9) The reduction in bond strength due to corrosion, where relevant, which highly depends on the confinement to the bar, concrete quality and environment, should be assessed and l_{bd} in Formulae (11.3) and (I.26) increased accordingly.

(10) For corroded reinforcement, in addition to (9), the effect on bond of concrete cover spalling should be considered and it may be taken into account by using the reduced cover thickness in Formulae (11.3) and (I.26).

(11) The effect of surface scaling on bond due to frost attack may be accounted for by use of the reduced cover in Formulae (11.3) and (I.26).

Residual bond capacity of ribbed bars not confined by links and of plain bars in freeze-thaw damaged concrete may be assessed using tensile strength measurements on cores taken from the affected structure. Concrete compressive strength f_{ck} in Formulae (11.3) and (I.26) may be substituted by residual characteristic compressive cylinder strength of concrete after freeze-thaw attack $f_{\mathrm{ck,ft}}$ given by Formula (I.28):

$$f_{\mathrm{ck,ft}} = 3{,}3 \cdot f_{\mathrm{ctk,is;0,05}}^{1,5} \tag{I.28}$$

where

$f_{\mathrm{ctk,is;0,05}}$ is the characteristic measured insitu tensile strength of concrete (5 % fractile).

NOTE 4 Bond strength of ribbed bars is not degraded as severely where bars are properly confined by secondary reinforcement.

I.11.4.2 Anchorage of bars with bends and hooks

(1) For existing structures, the provisions of 11.4.4 for standard hook and bend anchorage in tension complying with Figure 11.6 should be substituted by the following.

The design value of the reinforcement stress at the cross section $\sigma_{\mathrm{sd}}^{'}$ to be anchored by bond over l_{bd} may be taken from Formula (I.29):

$$\sigma_{\mathrm{sd}}^{'} = \sigma_{\mathrm{sd}} - \Delta\sigma_{\mathrm{sd}} \geq 0 \tag{I.29}$$

where

$$\Delta\sigma_{\mathrm{sd}} = 38 \cdot \delta_1 \cdot \left(\frac{\gamma_{\mathrm{c}}}{1{,}5}\right)^{-\delta_2} \cdot \left(\frac{f_{\mathrm{ck}}}{25}\right)^{\frac{1}{2}} \cdot \left(\frac{c_{\mathrm{d}}}{\phi}\right)^{\frac{1}{4}} \tag{I.30}$$

ratio $\left(\frac{c_{\mathrm{d}}}{\phi}\right) \leq 3{,}0$;

δ_1 and δ_2 are coefficients that take account of the position of the bars during concreting and in particular:

— $\delta_1 = \delta_2 = 1{,}0$ for bars in good bond conditions;

— in all other cases:

$\delta_1 = 0{,}8$ and $\delta_2 = 1{,}0$ for ribbed and indented bars;

$\delta_1 = 0{,}3$ and $\delta_2 = 2{,}0$ for plain bars.

Formulae (I.29) and (I.30) apply if:

— for ribbed bars:

$\frac{l_{\mathrm{bd}}}{\phi} \geq 5$ if the provision of I.11.4.1(2) is fulfilled;

$\frac{l_{\mathrm{bd}}}{\phi} \geq 10$ in all other cases;

— $\frac{l_{\mathrm{bd}}}{\phi} \geq 10$ for plain bars;

NOTE For the definition of good bond conditions, see 11.4.2(4).

I.11.5 Laps of reinforcing steel in tension and compression and mechanical couplers

(1) The minimum values of the design lap length given in Table 11.3 may be reduced to:

— 5ϕ for ribbed and indented bars if the laps are made with straight bars or with bends and hooks (tension only) and if the provision of I.11.4.1(2) is fulfilled;

— 10ϕ for plain bars if the laps are made with straight bars or with bends and hooks (tension only).

I.12 Detailing of members and particular rules - Minimum reinforcement rules

(1) 12.1(6) may be neglected for existing structures. However, for members having less longitudinal reinforcement than $A_{s,min}$ given in 12.2 the consequence for the structural safety should be identified.

(2) If the provisions of 12.2(6) are not fulfilled, the reinforcement resistance should be evaluated by using the actual anchorage length.

Annex J
(informative)

Strengthening of Existing Concrete Structures with CFRP

J.1 Use of this annex

(1) This informative Annex contains rules for strengthening of existing structures assessed in accordance with this Eurocode comprising plain, reinforced, and/or prestressed normal weight concrete.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this Informative Annex, it can be used.

J.2 Scope and field of application

(1) This Informative Annex applies to the strengthening of plain, reinforced, and/or prestressed normal weight concrete structures assessed in accordance with this Eurocode, comprising adhesively bonded reinforcement (ABR) which is either externally bonded to the surface (EBR) or near surface mounted in the concrete (NSM). The reinforcement material can be in the form of

— Prefabricated carbon FRP (CFRP) strips (EBR or NSM) or bars (NSM);

— Insitu lay-up carbon fibre (CF) sheets (EBR).

NOTE For general aspects of existing structures, Annex I can be considered unless specifically omitted or supplemented in Annex J.

J.3 General

(1) All provisions of this Eurocode in Clauses 1 to 14 apply unless specifically omitted or supplemented in Annex J.

J.4 Basis of design

(1) Partial factors for ABR reinforcement γ_f and γ_{BA} shall be applied.

NOTE The values γ_f and γ_{BA} are those given in Table J.1 (NDP) unless the National Annex gives different values.

Table J.1 (NDP) — Partial factors for ABR strengthening

Design situation	Tensile strength		Bond strength
	CFRP strips and bars	Insitu lay-up CF sheets	Failure in concrete or adhesive
Designation	γ_f		γ_{BA}
Persistent and transient	1,30	1,40	1,50
Accidental	1,10	1,15	1,15
Serviceability	1,00	1,00	1,00
Fatigue	1,30	1,40	1,50

J.5 Materials

J.5.1 General

(1) ABR strengthening systems suitable for design in accordance with this Eurocode shall satisfy the requirements of J.15.

(2) ABR strengthening systems used for structures in accordance with this Eurocode shall comply with the relevant product standards.

NOTE 1 Where harmonized product standards are not available for a strengthening system, the National Annex can specify the relevant product standard for ABR strengthening systems.

NOTE 2 ISO 10406 can be used for determination of selected properties.

(3) Annex J provides design rules for members strengthened with ABR reinforcement within the following limits of declared properties:

— interlaminar shear strength of CFRP strips according to EN ISO 14130 shall be equal or larger than the adhesive bond strength for any system;

— mean modulus of elasticity of CFRP strips: 150 000 MPa ≤ E_{f} ≤ 250 000 MPa;

— elastic stiffness per unit width of CF sheets: 20 kN/mm ≤ $E_{\mathrm{f}} \cdot A_{\mathrm{f}}/b_{f}$ ≤ 400 kN/mm;

— total CF cross section per unit width of CF sheets in the total of all layers determined in the direction of the tension action effect applied to the system:
100 mm^2/m ≤ $A_{\mathrm{f}}/b_{\mathrm{f}}$ ≤ 1 800 mm^2/m.

— characteristic tensile strength of the adhesive f_{Atk}, determined in accordance with EN 1504-4 shall be $f_{\mathrm{Atk}} \geq 14$ N/mm^2.

(4) ABR shall not be used for concrete substrates where deterioration is imminent or present unless proper concrete repair provisions have been taken.

J.5.2 Properties

(1) The following properties for CFRP strips and CF sheets are required for design of ABR strengthening systems in accordance with this standard:

— f_{fuk} characteristic short-term tensile strength of the ABR, determined in accordance with ISO 10406 (all parts);

— η_{f} reduction factor applied to the tensile strength;

— E_{f} average mean value of modulus of elasticity of the ABR in longitudinal direction, determined in accordance with ISO 10406 (all parts);

— $\varepsilon_{\mathrm{fuk}}$ characteristic ultimate strain;

— A_{f} cross sectional area.

(2) The cross sectional area of CFRP strips should be taken as $A_{\mathrm{f}} = b_{\mathrm{f}} \cdot t_{\mathrm{f}}$ where b_{f} is the width and t_{f} the thickness of the cross section.

(3) The cross sectional area of CF sheets should be obtained from relevant production data.

(4) For CF sheets the thickness t_{f}, which is an effective value, has to be determined considering the number of layers as follows:

$$t_{\mathrm{f}} = {n_{\mathrm{f}}}^{k_{\mathrm{f}}} \cdot A_{\mathrm{f}}/b_{\mathrm{f}} \tag{J.1}$$

Where

n_f is the number of layers;

A_f is the cross sectional area of the fibres per meter of a single layer of CF sheet;

k_f = 0,85 for $n_f > 3$, or 1 otherwise.

(5) The following properties for the adhesive are required for design of ABR strengthening systems in accordance with this standard:

— characteristic compressive strength of the adhesive f_{Ack} determined in accordance with EN 1504-4;

— characteristic tensile strength of the adhesive f_{Atk}, determined in accordance with EN 1504-4.

NOTE 1 Material properties can vary with temperature. Hence, maximum and minimum temperatures for the design life of the strengthening system, calculated in accordance with prEN 1991-4 for the application of the system to be used, need to be known and specified.

NOTE 2 Material properties can vary with environmental conditions and exposure to alkali. Hence, environmental conditions and exposure to alkali that will be encountered through the design life of the strengthening system for the application of the system to be used, need to be known and specified.

J.5.3 Design assumptions

(1) The value of design tensile strength of the ABR (CFRP and CF sheets) shall be taken as:

$$f_{fud} = \frac{\eta_f \cdot f_{fuk}}{\gamma_f} \tag{J.2}$$

where

η_f is a reduction factor applied to the tensile strength of the ABR for the relevant exposure conditions determined in accordance with ISO 10406 (all parts) as appropriate.

(2) η_f may be taken as 0,7 unless more accurate information is available based on test data for the ABR.

(3) The strain corresponding to the short-term design strength, ε_{fud}, shall be calculated according to the following Formula (J.3):

$$\varepsilon_{fud} = \frac{f_{fud}}{E_f} \tag{J.3}$$

(4) The design stress-strain diagram for carbon fibre reinforcement should be taken as in Figure J.1.

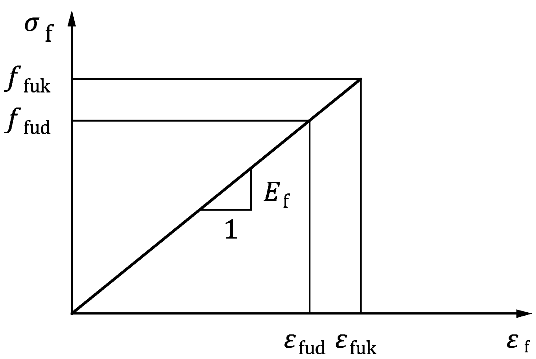


Figure J.1 — Design stress- strain diagram for CFRP reinforcement

(5) A method for determining $f_{\mathrm{ctm,surf}}$ is provided in Formula (J.34). Where this is applied it shall be validated by site testing in accordance with EN 1542 prior to execution.

J.6 Durability

(1) The durability of the strengthening system should be ensured, for the design life of the structure, considering the exposure classes in accordance with Table 6.1, with additional protective measures if necessary.

(2) The durability of the strengthening system should be ensured, for the design life of the structure, taking into account exposure to direct UV radiation (direct or indirect solar radiation by reflection on snow or water) and the risk of repeated or permanent penetration of moisture, with additional protective measures if necessary.

(3) Thermal effects shall be considered from e.g. asphalt on a strengthened bridge deck.

J.7 Structural analysis

(1) Unless more rigorous analysis is undertaken, members strengthened with ABR should not be analysed using linear elastic analysis with redistribution or plastic analysis.

(2) A member should be assessed against accidental loss of ABR, including the following situations:

— The ABR should not be designed to withstand permanent action effects in a manner that the structure would not be able to withstand collapse without ABR unless the following is addressed in design:

  — The strengthening is detailed in a manner that protects it against damage from vandalism or accidental mechanical damage;

  — Member collapse will not result in progressive collapse of the structure;

  — Protection from fire damage is provided.

J.8 Ultimate Limit States (ULS)

J.8.1 Bending with or without axial force

J.8.1.1 General

(1) When determining the ultimate moment resistance of reinforced or prestressed concrete cross sections strengthened in flexure with ABR, the following assumptions in addition to 8.1.1(2) should be made:

— the compressive strength of ABR is ignored;

— the slip between CFRP reinforcement and the concrete substrate is neglected.

(2) The strain state of the existing reinforcement and concrete members being strengthened in bending shall be determined prior to strengthening under the relevant effects of actions. Strains arising from additional bending effects after strengthening should be superimposed to these when verifying the capacity of the strengthened member.

(3) Unless more rigorous analysis is undertaken, the provisions in this Eurocode should not be applied to concrete with $f_{\mathrm{ck}} \leq 12$ MPa or $f_{\mathrm{ck}} > 50$ MPa.

(4) The tensile force in the ABR shall be determined with the strain distribution according to 8.1.1.

(5) The pre-strain of the reinforcement in the existing structure, the yield point of the reinforcement and the strain limits of the ABR according to J.5 shall be considered.

NOTE 1 The tension resistance of the EBR strengthening is generally limited by the bond resistance calculated in accordance with J.11.

NOTE 2 The tension resistance of a NSM system is sometimes limited by the bond resistance calculated in accordance with J.11.

J.8.1.2 Concrete columns confined with closed CFRP wrapping

(1) CFRP reinforcement for confinement should be applied for the full length of the column being strengthened.

(2) The confining effect provided by adhesively bonded CFRP may be considered in design of axially-loaded members under the following conditions, where:

— the characteristic concrete strength f_{ck} is less than 50 MPa;

— the diameter of a circular column, D, or effective diameter of a rectangular or square column, $D_{eq} \geq$ 150 mm;

— the first order eccentricity satisfies the condition $\frac{e_0}{D_{eq}} \leq 0{,}20$;

— the slenderness satisfies the condition $l_0/D_{eq} \leq 40$;

— the corner radius for rectangular cross sections is $r_c \geq 20$ mm;

The increase in compressive strength from CFRP confinement shall be considered in determining slenderness effects.

(3) For the design of CFRP confined concrete cross sections the following simplified bilinear stress-strain relationship may be used:

$$\sigma_c = \frac{f_{cd} \cdot \varepsilon_c}{0{,}00175} \qquad \text{for } 0 \leq \varepsilon_c \leq 0{,}00175 \tag{J.4}$$

$$\sigma_c = f_{cd} + \Delta f_{cd} \frac{(\varepsilon_c - 0{,}00175)}{(\varepsilon_{cu} - 0{,}00175)} \qquad \text{for } 0{,}00175 \leq \varepsilon_c \leq \varepsilon_{cu} \tag{J.5}$$

NOTE 1 The value of ε_{cu} can be taken as 0,006 unless more accurate information is available.

(4) The increase in compressive strength of concrete Δf_{cd} in columns resulting from confinement using FRP may be calculated as follows:

For circular columns:

$$\Delta f_{cd} = 0 \qquad \text{for } \frac{t_f \cdot f_{fud}}{D \cdot f_{cd}} < 0{,}07 \tag{J.6}$$

$$\Delta f_{cd} = k_{cc} \cdot \frac{t_f}{D} \cdot f_{fud} \qquad \text{for } \frac{t_f \cdot f_{fud}}{D \cdot f_{cd}} \geq 0{,}07 \tag{J.7}$$

where D is the diameter of the circular column.

NOTE 2 The value of k_{cc} can be taken as 2,5 for circular columns unless more accurate information is available.

Where helical and/or intermittent wrapping is used on circular columns, the value of f_{fud} in Formulae (J.6) and (J.7) should be factored by k_h according to Formula (J.8), where geometrical parameters are defined in Figure J.2.

$$k_{\mathrm{h}} = \left(1 - \frac{(s_{\mathrm{f}} - b_{\mathrm{f}})}{2 \cdot D}\right)^2 \cdot \left(\frac{1}{1 + (\tan\beta_{\mathrm{f}})^2}\right) \tag{J.8}$$

For rectangular columns:

$$\Delta f_{\mathrm{cd}} = 0 \qquad \text{for } \left(\frac{\mathrm{b}}{\mathrm{h}}\right)^2 k_{\mathrm{e}} \frac{t_{\mathrm{f}} \cdot k_{\mathrm{r}} \cdot f_{\mathrm{fud}}}{D_{\mathrm{eq}} \cdot f_{\mathrm{cd}}} < 0{,}07 \tag{J.9}$$

$$\Delta f_{\mathrm{cd}} = k_{\mathrm{cc}} \cdot \left(\frac{b}{h}\right)^2 \cdot k_{\mathrm{e}} \cdot \frac{t_{\mathrm{f}}}{D_{\mathrm{eq}}} \cdot k_{\mathrm{r}} \cdot f_{\mathrm{fud}} \qquad \text{for } \left(\frac{b}{h}\right)^2 k_{\mathrm{e}} \frac{t_{\mathrm{f}} \cdot k_{\mathrm{r}} \cdot f_{\mathrm{fud}}}{D_{\mathrm{eq}} \cdot f_{\mathrm{cd}}} \geq 0{,}07 \tag{J.10}$$

where

$$D_{\mathrm{eq}} = \frac{2 \cdot b \cdot h}{b + h} \tag{J.11}$$

$$k_{\mathrm{e}} = 1 - \frac{(b - 2 \cdot r_{\mathrm{c}})^2 + (h - 2 \cdot r_{\mathrm{c}})^2}{3 \cdot b \cdot h} \tag{J.12}$$

$$k_{\mathrm{r}} = \begin{cases} 1{,}0 \cdot \dfrac{r_c}{50} \cdot \left(2 - \dfrac{r_c}{50}\right) & \text{for } r_{\mathrm{c}} < 50 \text{ mm} \\ 1{,}0 & \text{for } r_{\mathrm{c}} \geq 50 \text{ mm} \end{cases} \tag{J.13}$$

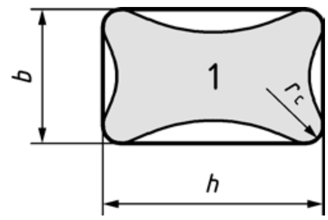
Where helical and/or intermittent wrapping is used on rectangular or square columns (see Figure J.2), the value of f_{fud} in Formulae (J.9) and (J.10) should be factored by k_{h} according to Formula (J.14), where geometrical parameters are defined in Figure J.2.

$$k_{\mathrm{h}} = \left(1 - \frac{s_{\mathrm{f}} - b_{\mathrm{f}}}{2 \cdot b}\right) \cdot \left(1 - \frac{s_{\mathrm{f}} - b_{\mathrm{f}}}{2 \cdot h}\right) \cdot \left(\frac{1}{1 + (\tan\beta_{\mathrm{f}})^2}\right) \tag{J.14}$$

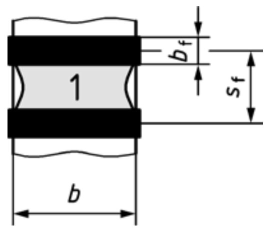
NOTE 3 The value of k_{cc} can be taken as 1,5 for square and rectangular columns unless more accurate information is available.

NOTE 4 Unless more rigorous analysis is undertaken, Formulae (J.9) and (J.10) should not be used for columns with h/b>2.

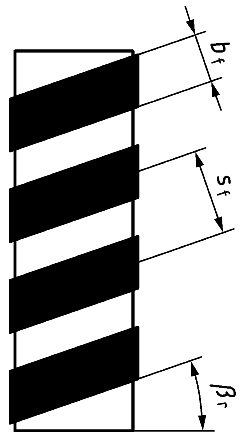
(5) The confining effect of CFRP may be ignored in creep calculations for concrete columns.



a) effectively confined concrete in rectangular cross-section



b) confinement with discrete strips



c) confinement with helically applied strips

Key

1 confined concrete

Figure J.2 — Configuration of CFRP wrapping

J.8.2 Shear

J.8.2.1 General verification procedure

(1) In principle, shear resistance shall be in accordance with 8.2. If a shear resistance analysis according to 8.2 shows unsufficient shear resistance, shear strengthening in accordance with J.8.2.3 may be applied.

J.8.2.2 Detailed verification for members not requiring design shear reinforcement

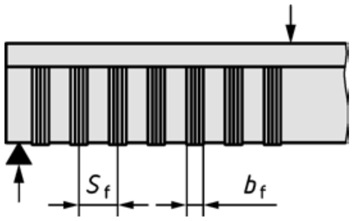
(1) When determining the design value of the shear stress resistance in accordance with Formulae (8.27) or (8.33), the FRP reinforcement applied for strengthening should not be considered in the term A_{sl}.

J.8.2.3 Members requiring design shear reinforcement

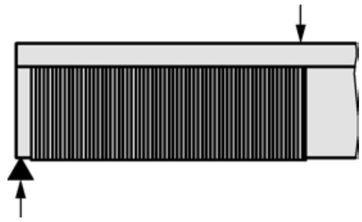
(1) In principle, shear resistance of internal reinforcement shall be calculated in accordance with 8.2.3.

(2) The provisions in this Annex apply for shear strengthening of rectangular and T-shaped cross sections without anchorage devices.

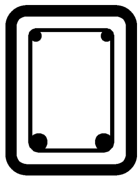
(3) The provisions apply to strengthening systems applied discrete or continuous (see Figure J.3).



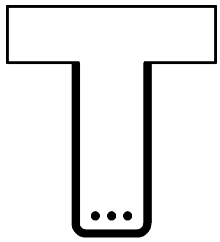
a) Discrete application of the strengthening system



b) Continuous application of the strengthening system



c) Closed wrapped CFRP system



d) **Open CFRP strengthening applications**

Figure J.3 — Definition of notation used to describe CFRP strengthening applications

(4) The shear resistance of a section strengthened with CFRP may be taken as:

$$\tau_{\text{Rd,CFRP}} = \tau_{\text{Rd}} + \tau_{\text{Rd,f}} \leq 0{,}5 \cdot \nu \cdot f_{\text{cd}} \tag{J.15}$$

where

$$\tau_{\text{Rd,f}} = \frac{A_{\text{f}}}{s_{\text{f}}} \cdot \frac{f_{\text{fwd}}}{b_{\text{w}}} \cdot (\cot\theta + \cot\alpha_{\text{f}}) \cdot \sin\alpha_{\text{f}} \tag{J.16}$$

$$\frac{A_{\text{f}}}{s_{\text{f}}} = \begin{cases} \dfrac{2 \cdot t_{\text{f}} \cdot b_{\text{f}}}{s_{\text{f}}} \cdot \sin\alpha_{\text{f}} & \text{for discrete CFRP strips or CF sheets} \\ 2 \cdot t_{\text{f}} \cdot \sin\alpha_{\text{f}} & \text{for continuous CF sheets} \end{cases} \tag{J.17}$$

α_{f} is the angle formed between the strengthening system and the longitudinal member axis;

f_{fwd} is design shear strength of the strengthening system calculated in accordance with Formulae (J.18), (J.19), (J.20) or (J.23)

τ_{Rd}, ν are in accordance with 8.2.

Unless more rigorous analysis is undertaken, θ should be taken as 45 degrees for the calculation of τ_{Rd} and $\tau_{\text{Rd,f}}$.

(5) The following may be used to determine the design shear strength of closed wrapped CFRP systems as defined in Figure J.3 c).

$$f_{\text{fwd}} = 0{,}8 \cdot k_r \cdot f_{\text{fud}} \tag{J.18}$$

where

f_{fud} should be determined using Formula (J.2) and k_{r} should be determined using Formula (J.13).

(6) Formulae (J.19) and (J.20) may be used to determine the design shear strength of open discrete CFRP systems as defined in Figure J.3. In both cases f_{fwd} is limited by the value determined with Formula (J.18).

Where the anchorage length into the compression zone of the member of all CFRP strips, l_{bf}, is less than $l_{\mathrm{bf,max,k}}$, f_{fwd} should be determined using the Formula (J.19), where θ and α_{f} are defined in Figure J.4.

$$f_{\mathrm{fwd}} = \frac{2}{3}\frac{n \cdot s_f}{l_{\mathrm{bf,max,k}} \cdot \left[\left(\cot\theta + \cot\alpha_f\right) \cdot \sin\alpha_f\right]} f_{\mathrm{bfRd}} \tag{J.19}$$

Where the anchorage length into the compression zone of the member of some CFRP strips, l_{bf}, is less than $l_{\mathrm{bf,max,k}}$, f_{fwd} should be determined using Formula (J.20):

$$f_{\mathrm{fwd}} = \left[1 - \left(1 - \frac{2}{3}\frac{m \cdot s_{\mathrm{f}}}{l_{\mathrm{bf,max,k}} \cdot \left[\left(\cot\theta + \cot\alpha_f\right) \cdot \sin\alpha_f\right]}\right)\frac{m}{n}\right] f_{\mathrm{bfRd}} \tag{J.20}$$

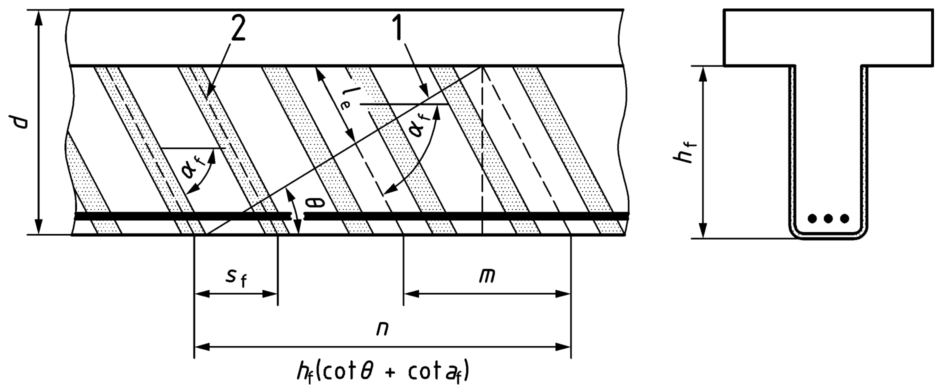
Where the parameters α_{f}, m and n are defined in Formulae (J.21), (J.22) and in Figure J.4 and $l_{\mathrm{bf,max,k}}$ and f_{bfRd} shall be determined using J.11.1.1.

$$n = \text{integer}\left(\frac{h_{\mathrm{f}}(cot\theta + cot\alpha_{\mathrm{f}})}{s_{\mathrm{f}}}\right) \tag{J.21}$$

$$m = \text{integer}\left(\frac{l_{\mathrm{bf,max,k}}(cot\theta + cot\alpha_{\mathrm{f}})sin\alpha_{\mathrm{f}}}{s_{\mathrm{f}}}\right) \tag{J.22}$$

(7) Formula (J.23) may be used to determine the design shear strength of open continuous CFRP sheet systems.

$$f_{\mathrm{fwd}} = \begin{cases} \dfrac{2}{3} \cdot \dfrac{h_f/sin\alpha_{\mathrm{f}}}{l_{bf,max,k}} \cdot f_{bfRd} & \text{for } \dfrac{h_f}{sin\alpha_{\mathrm{f}}} < l_{bf,max,k} \\ \left[1 - \dfrac{1}{3}\dfrac{l_{bf,max,k}}{h_f/sin\alpha_{\mathrm{f}}}\right] \cdot f_{bfRd} & \text{for } \dfrac{h_f}{sin\alpha_{\mathrm{f}}} \geq l_{bf,max,k} \end{cases} \tag{J.23}$$



Key

1 shear crack

2 strips with inclination α_f

m strips with $l_{bf} < l_{bf.max,k}$

n strips intersecting shear crack

Figure J.4 — Illustration of ABR stirrups intersecting shear crack

J.8.3 Torsion and combined actions

(1) When applying 8.3, the CFRP reinforcement shall not be considered.

J.8.4 Punching

(1) This Annex does not apply for strengthening for punching shear.

J.8.5 Design with strut-and-tie models and stress fields

(1) ABR systems may be used as tension reinforcing in ties according to the provisions of 8.5 and J.11 subject to strain compatibility being demonstrated.

(2) Anchorage of ABR and transfer of forces to nodes used for strengthening in ties should be ensured in accordance with 8.5.

J.9 Serviceability Limit States (SLS)

(1) Deflections of beams or slabs strengthened with ABR may be estimated by ignoring the slip between the CFRP and concrete and transforming the area of CFRP to steel by taking account of the modular ratio. Pre-existing deflections shall be considered by using the appropriate provisions of EN 1990.

(2) The stress in the carbon fibre reinforcement shall be limited as follows:

$$\sigma_f \leq 0{,}8 \cdot f_{yk} \cdot E_f / E_s \tag{J.24}$$

(3) Verification of stresses in existing reinforcement and concrete stresses according to 9.2.1 and Annex I.9 shall be done considering the stress states before and after strengthening.

J.10 Fatigue

J.10.1 Basic fatigue analysis for externally bonded CFRP systems

(1) A fatigue check for EBR may be omitted where the following condition is satisfied:

$$\Delta F_{\mathrm{fE,equ}} \leq \Delta F_{\mathrm{fRd,fat1}} = 0{,}35 \cdot {f_{\mathrm{ctm,surf}}}^{1/4} \cdot f_{\mathrm{bfRd}} \cdot b_{\mathrm{f}} \cdot t_{\mathrm{f}} \tag{J.25}$$

where

$\Delta F_{\mathrm{fRd,fat1}}$ is the basic fatigue resistance;

f_{bfRd} is the limiting design strength of the bond in the area being considered calculated in accordance with J.11;

$f_{\mathrm{ctm,surf}}$ may either be determined by testing or estimated with Formula (J.34);

$$\Delta F_{\mathrm{fE,equ}} = \max\{b_{\mathrm{f}} \cdot t_{\mathrm{f}} \cdot \Delta f_{\mathrm{fEd,max}}\,; F_{\mathrm{fEd,cr}}\} \tag{J.26}$$

$\Delta f_{\mathrm{fEd,max}}$ is the maximum difference in CFRP stress under the relevant load combination between cracks (refer to Figure J.5) within the strengthened area. $F_{\mathrm{fEd,cr}}$ is the force in CFRP at first crack in the strengthened area.

J.10.2 Refined fatigue analysis for externally bonded CFRP systems

(1) If the condition in Formula (J.25) cannot be satisfied, the following condition should be assessed under the combination of actions stated in Clause 10 according to:

$$\Delta F_{\mathrm{fEd,fat}} \leq \Delta F_{\mathrm{fRd,fat2}} = \alpha_{\mathrm{fat2}} \cdot \frac{\Delta F_{\mathrm{fk,B}}}{\gamma_{\mathrm{BA}}} \tag{J.27}$$

where

$\Delta F_{\mathrm{fEd,fat}}$ is the design force range due to forces at the crack edge, $\Delta F_{\mathrm{f,max}} - \Delta F_{\mathrm{f,min}}$, and;

$\Delta F_{\mathrm{f,min}}$ is the minimum value of $b_{\mathrm{f}} \cdot t_{\mathrm{f}} \cdot \Delta f_{\mathrm{fEd}}$ under the relevant combination of actions specified in 10.2;

$\Delta F_{\mathrm{f,max}}$ is the maximum value of $b_{\mathrm{f}} \cdot t_{\mathrm{f}} \cdot \Delta f_{\mathrm{fEd}}$ under the relevant combination of actions specified in 10.2.

The difference in CFRP tension stress between cracks Δf_{fEd} calculated according to Formula (J.28) is defined in Figure J.5.

$$\Delta f_{\mathrm{fEd}} = \frac{F_{\mathrm{fEd,b}} - F_{\mathrm{fEd,a}}}{b_{\mathrm{f}} \cdot t_{\mathrm{f}}} \tag{J.28}$$

where

$\Delta F_{\mathrm{fk,B}} = b_{\mathrm{f}} \cdot t_{\mathrm{f}} \cdot \Delta f_{\mathrm{fk,B}}$ and $\Delta f_{\mathrm{fk,B}}$ is calculated according to Formula (J.48);

$$\alpha_{\mathrm{fat2}} = -c_{\mathrm{fat}} \cdot \frac{\Delta F_{\mathrm{f,max}}}{\Delta F_{\mathrm{fRd}}} + c_{\mathrm{fat}} \tag{J.29}$$

$\Delta F_{\mathrm{f,max}}$ is the maximum value of ΔF_{f} under the combination of actions according to 10.2;

$$c_{\mathrm{fat}} = 0{,}35 \cdot \left(\frac{N^*}{2 \cdot 10^6}\right)^{-\frac{1}{k_{\mathrm{f3}}}} \tag{J.30}$$

N^* the number of stress cycles;

$k_{f3} = 23{,}2$ for $N^* < 2 \cdot 10^6$;

$k_{f3} = 45{,}4$ for $N^* \geq 2 \cdot 10^6$.

J.10.3 Near surface mounted CFRP strips

(1) Near surface mounted strips that satisfy the following conditions under the combination of actions stated in Clause 10 may be deemed adequate in fatigue.

a) The number of stress cycles is less than $2 \cdot 10^6$;

b) The maximum force in the NSM CFRP system, taking the shift of the tension envelope into account, does not exceed $F_{f,NSM,max}$, where $F_{f,NSM,max}$ is calculated using Formula (J.31);

c) The strip stress range $\Delta\sigma_f$ complies with the provisions stated in Formula (J.32).

$$F_{f,NSM,max} = 0{,}6 \cdot f_{bfRd} \cdot b_f \cdot t_f \qquad (J.31)$$

$$\Delta\sigma_f \leq \frac{500}{t_f} \qquad (J.32)$$

where

$$\Delta\sigma_f = \frac{\Delta F_{f,max} - \Delta F_{f,min}}{b_f \cdot t_f} \qquad (J.33)$$

J.11 Bond and anchorage of CFRP systems

J.11.1 Anchorage of ABR strengthening systems

J.11.1.1 Basic anchorage resistance — CFRP to concrete for EBR strengthening

J.11.1.1.1 General

(1) The required anchorage length of EBR, calculated in accordance with J.11.1.1, should be curtailed as described in J.11.1.2.

(2) The surface tensile strength $f_{ctm,surf}$ of the prepared concrete surface to be bonded is of decisive importance for the bond resistance. If the surface tensile strength cannot be determined on the member, it may be estimated as a function of the position during concreting according to Formula (J.34).

$$f_{ctm,surf} = k_{c,surf} \cdot f_{ctm} \qquad (J.34)$$

where

$$k_{c,surf} = \begin{cases} 0{,}3 + 0{,}6 \cdot \left(\frac{f_{ck}}{60} - 0{,}2\right) & \text{for conreting position: top} \\ 0{,}4 + 0{,}5 \cdot \left(\frac{f_{ck}}{60} - 0{,}2\right) & \text{for conreting position: side} \\ 0{,}6 + 0{,}3 \cdot \left(\frac{f_{ck}}{60} - 0{,}2\right) & \text{for conreting position: bottom} \end{cases}$$

J.11.1.1.2 Simplified method

(1) The following simplified method may be used to determine the anchorage resistance for EBR.

$$f_{\mathrm{bfRd}} = \frac{0{,}2}{\gamma_{\mathrm{BA}}} \beta_1 \sqrt{\frac{E_{\mathrm{f}}}{t_{\mathrm{f}}} \left(f_{\mathrm{cm}} \cdot f_{\mathrm{ctm,surf}}\right)^{0{,}5}} \tag{J.35}$$

where

$$\beta_1 = \begin{cases} \dfrac{l_{\mathrm{bf}}}{l_{\mathrm{bf,max,k}}}\left(2 - \dfrac{l_{\mathrm{bf}}}{l_{\mathrm{bf,max,k}}}\right) < 1 & \text{if } l_{\mathrm{bf}} < l_{\mathrm{bf,max,k}} \\ 1 & \text{if } l_{\mathrm{bf}} \geq l_{\mathrm{bf,max,k}} \end{cases} \tag{J.36}$$

$$l_{\mathrm{bf,max,k}} = 1{,}5 \sqrt{\frac{E_{\mathrm{f}} \cdot t_{\mathrm{f}}}{\left(f_{\mathrm{cm}} \cdot f_{\mathrm{ctm,surf}}\right)^{0{,}5}}} \tag{J.37}$$

J.11.1.1.3 Refined method

(1) If more accurate data for the EBR system is known based on production data, the following more refined analysis may be used to determine anchorage resistance. The design bond strength of the anchorage, of the EBR system may be taken as the following:

$$f_{\mathrm{bfRd}} = \frac{\sqrt{\eta_{\mathrm{cc}} \cdot k_{\mathrm{tc}} \cdot k_{\mathrm{tt}}}}{\gamma_{\mathrm{BA}}} \cdot \begin{cases} f_{\mathrm{bfk,max}} \cdot \dfrac{l_{\mathrm{bf}}}{l_{\mathrm{bf,max,k}}}\left(2 - \dfrac{l_{\mathrm{bf}}}{l_{\mathrm{bf,max,k}}}\right) & \text{where } l_{\mathrm{bf}} < l_{\mathrm{bf,max,k}} \\ f_{\mathrm{bfk,max}} & \text{where } l_{\mathrm{bf}} \geq l_{\mathrm{bf,max,k}} \end{cases} \tag{J.38}$$

where

$$l_{\mathrm{bf,max,k}} = \frac{2}{k_{\mathrm{sys,b3}}} \cdot \sqrt{\frac{E_{\mathrm{f}} \cdot t_{\mathrm{f}} \cdot s_{\mathrm{f0k}}}{\tau_{\mathrm{f1k}}}} \tag{J.39}$$

$$f_{\mathrm{bfk,max}} = \sqrt{\frac{E_f \cdot \tau_{\mathrm{f1k}} \cdot s_{\mathrm{f0k}}}{t_{\mathrm{f}}}} \tag{J.40}$$

$$s_{\mathrm{f0k}} = 0{,}2 \cdot k_{\mathrm{sys,b2}} \tag{J.41}$$

$$\tau_{\mathrm{f1k}} = 0{,}37 \cdot k_{\mathrm{sys,b1}} \cdot \left(f_{\mathrm{cm}} \cdot f_{\mathrm{ctm,surf}}\right)^{0{,}5} \tag{J.42}$$

NOTE The value of $k_{\mathrm{sys,b1}}$ = 1,0, $k_{\mathrm{sys,b2}} = 1{,}0$ and $k_{\mathrm{sys,b3}} = 1{,}0$ can be used unless more accurate information is available based on production data of the EBR system.

J.11.1.2 EBR anchorage requirements — flexure

J.11.1.2.1 General

(1) Anchorage of the strengthening system to the concrete surface of a member in flexure shall be provided to avoid the following failure mechanisms:

— end anchorage as described in J.11.1.2.2;

— intermediate crack debonding as described in J.11.1.2.3;

— end cover separation as described in J.11.1.2.4;

— shear induced separation as described in J.11.1.2.5.

J.11.1.2.2 End anchorage

(1) The EBR shall be anchored by an anchorage length beyond the section where the design resistance of the unstrengthened existing section is at least as great as the design effects resulting from the relevant limit state.

(2) EBR shall be curtailed according to one of the following conditions:

- where member strengthening is undertaken, the curtailment shall take account of a_l, calculated in accordance with 12.3.2;
- where local strengthening is undertaken, the EBR should extend a distance $l_{bf} + h$ beyond the section where it is needed.

(3) The anchorage resistance f_{bfRd} should be calculated using Formulae (J.35) or (J.38).

J.11.1.2.3 Intermediate crack debonding

(1) Where J.11.1.1.3 is used to determine the anchorage length of EBR, the anchorage capacity between flexural cracks shall be sufficient to resist the difference in tensile forces in the system between cracks.

(2) Formula (J.43) may be used to determine the capacity of the strengthening system between adjacent flexural cracks.

(3) Formula (J.43) may be applied where the strain in the CFRP does not exceeds 10 mm/m.

$$\Delta f_{fEd} \leq \Delta f_{fRd} \tag{J.43}$$

where

Δf_{fEd} is calculated according to Formula (J.28);

Δf_{fRd} is the bond resistance between adjacent cracks.

(4) Unless a more accurate analysis is undertaken, Δf_{fRd} and Δf_{fEd} should be calculated using the minimum crack spacing, $s_{cr,min}$ (see Figure J.5), where $s_{cr,min}$ may be determined as follows:

$$s_{cr,min} = 1{,}5 \cdot \frac{M_{cr}}{0{,}85 \cdot h \cdot F_{bsm}} \tag{J.44}$$

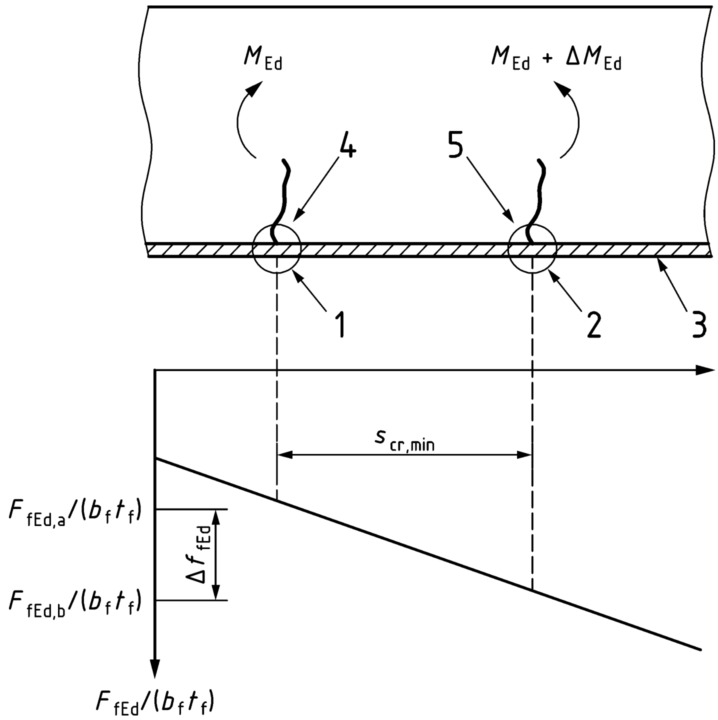
With the bond strength per length:

$$F_{bsm} = \sum_{i=1}^{n} n_{s,i} \cdot \phi_i \cdot \pi \cdot f_{bsm} \tag{J.45}$$

The mean bond stress of the reinforcing steel may be determined by:

$$f_{bsm} = \begin{cases} \kappa_{vb1} \cdot 0{,}43 \cdot f_{cm}{}^{\frac{2}{3}} & \text{for ribbed bars} \\ \kappa_{vb2} \cdot 0{,}28 \cdot f_{cm}{}^{\frac{1}{2}} & \text{for plane bars} \end{cases} \tag{J.46}$$

The following should be used for good bond conditions $\kappa_{vb1} = \kappa_{vb2} = 1$ and for medium bond conditions $\kappa_{vb1} = 0{,}7$ and $\kappa_{vb2} = 0{,}5$.



Key

1 $F_{fEd,a}$ force in CFRP at Crack A

2 $F_{fEd,b}$ force in CFRP at Crack B

3 CFRP strip

4 Crack A

5 Crack B

Figure J.5 — CFRP between flexural cracks

(5) The bond resistance between adjacent cracks, Δf_{fRd} may be determined using Formula (J.47) by taking account of the beneficial effects of bond friction, $\Delta f_{fk,F}$, and clamping from curvature of the beam, $\Delta f_{fk,C}$, in addition to the adhesive bond resistance between the cracks, $\Delta f_{fk,B}$ as follows:

$$\Delta f_{fRd} = {}^{1}/_{\gamma_{BA}} \cdot \left((\eta_{cc} \cdot k_{tc} \cdot k_{tt})^{0,5} \cdot \Delta f_{fk,B} + \Delta f_{fk,F} + \Delta f_{fk,C} \right) \tag{J.47}$$

$$\Delta f_{fk,B} = 0{,}84 \cdot k_{sys,b1} \cdot \sqrt{f_{cm} \cdot f_{ctm,surf}} \cdot \frac{\sqrt{s_{cr,min}}}{t_f} \tag{J.48}$$

$$\Delta f_{fk,F} = f_{cm}{}^{-0,9} \cdot \frac{s_{cr,min}{}^{4/3}}{t_f} \tag{J.49}$$

$$\Delta f_{fk,C} = \frac{\kappa_h}{h_f} \cdot \frac{s_{cr,min}{}^{1/3}}{t_f} \tag{J.50}$$

where

$\kappa_h = 2\,000$ for reinforced concrete members;

$\kappa_h = 0$ for prestressed concrete members;

$h_f = \min\{100 \text{ mm}, h\}$.

(6) Unless a more accurate analysis is undertaken, Δf_{fEd} may be calculated by ignoring slip of the CFRP.

(7) In the region where stirrups are required in accordance with J.11.1.2.5, verification of intermediate crack debonding is not required.

J.11.1.2.4 End cover separation

(1) The maximum design shear force (V_{Ed}) at the end of the CFRP reinforcement shall be less than the design resistance value against concrete cover separation $V_{\mathrm{Rd,cfE}}$ calculated in accordance to Formula (J.51):

$$V_{\mathrm{Ed}} \leq V_{\mathrm{Rd,cfE}} = \left(0{,}11 + 2{,}2 \cdot \frac{(100\rho_{\mathrm{l}})^{0{,}15}}{a_{\mathrm{fE}}{}^{0{,}36}}\right) \cdot (100 \cdot \rho_{\mathrm{l}} \cdot f_{\mathrm{ck}})^{1/3} \cdot b_{\mathrm{w}} \cdot d \tag{J.51}$$

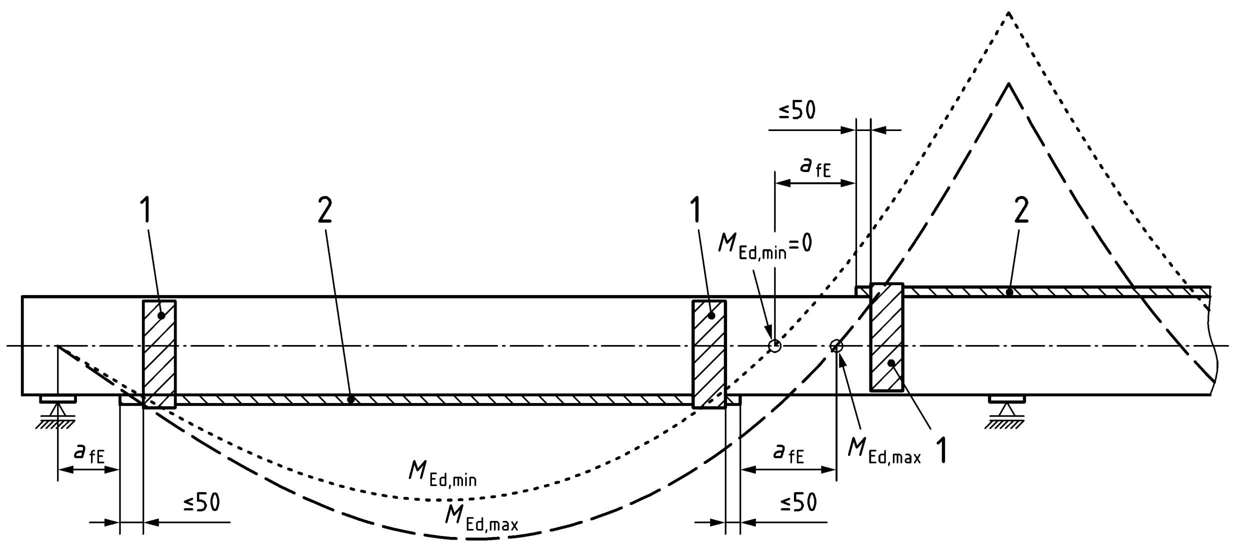
Where the condition in Formula (J.51) is not met, shear strengthening at the end of the longitudinal strengthening shall be provided in accordance with Formula (J.52) with anchorage satisfying the provisions of this Eurocode.

$$f_{\mathrm{fwd}} \leq f_{\mathrm{bfRd}} \cdot \tan\theta \tag{J.52}$$

where f_{bfRd} is the CFRP anchorage capacity for the flexural strengthening system being designed.

Stirrups for end cover separation should be detailed as shown in Figure J.6.

Dimensions in millimetres



Key

1 end stirrup

2 flexural strengthening

Figure J.6 — CFRP shear stirrup arrangement to avoid end cover separation

J.11.1.2.5 Shear induced separation

(1) Where the limits in Formula (J.53) or (J.54) are exceeded for members strengthened with EBR in flexure, additional EBR stirrups shall be provided.

$$\frac{\tau_{\mathrm{Ed}} \cdot \sigma_{\mathrm{swd}}}{\tau_{\mathrm{Rd}}} \leq \begin{cases} 75\ \mathrm{MPa} & \text{for ribbed steel bars} \\ 25\ \mathrm{MPa} & \text{for plain round steel bars} \end{cases} \tag{J.53}$$

where τ_{Rd} is calculated according to Formula (8.62) and σ_{swd} according to Formula (8.63).

$$\tau_{\mathrm{Ed}} \leq 0{,}33 \cdot f_{\mathrm{ck}}^{\frac{2}{3}} \tag{J.54}$$

Where stirrups are required in accordance with Formulae (J.53) or (J.54), additional transverse EBR stirrups should be provided to resist the shear force in Formula (J.55) with adequate anchorage.

$$\tau_{\mathrm{Ed,f}} = \max \begin{cases} \dfrac{E_{\mathrm{f}} A_{\mathrm{f}}}{E_{\mathrm{f}} A_{\mathrm{f}} + E_{\mathrm{s}} A_{\mathrm{s}}} \cdot \tau_{\mathrm{Ed}} \\ \tau_{\mathrm{Ed}} - \tau_{\mathrm{Rd,sy}} \end{cases} \tag{J.55}$$

J.11.1.3 Basic anchorage resistance — CFRP strips to concrete for NSM CFRP strengthening

(1) The design bond capacity per strip should be determined according to Formula (J.56) or (J.57).

For $l_{\mathrm{bf}} \leq 115\mathrm{mm}$:

$$F_{\mathrm{bfRd}} = 0{,}95 \cdot b_{\mathrm{f}} \cdot \tau_{\mathrm{bAd}} \cdot {a_{\mathrm{r}}}^{1/4} \cdot l_{\mathrm{bf}} \cdot (0{,}4 - 0{,}0015 \cdot l_{\mathrm{bf}}) \tag{J.56}$$

For $l_{\mathrm{bf}} > 115\mathrm{mm}$:

$$F_{\mathrm{bfRd}} = 0{,}95 \cdot b_{\mathrm{f}} \cdot \tau_{\mathrm{bAd}} \cdot {a_{\mathrm{r}}}^{1/4} \cdot \left(26{,}2 + 0{,}065 \cdot \tan h\left(\frac{a_{\mathrm{r}}}{70}\right) \cdot (l_{\mathrm{bf}} - 115)\right) \tag{J.57}$$

where a_{r} is the distance from the longitudinal axis of the strip to the free edge

$$a_{\mathrm{r}} \leq 150\ \mathrm{mm} \tag{J.58}$$

The maximum design strength of the adhesive according for NSM CFRP systems may be obtained from Formula (J.59):

$$\tau_{\mathrm{bAd}} = \frac{1}{\gamma_{\mathrm{BA}}} \cdot \min \begin{cases} \tau_{\mathrm{bAk}} \cdot \alpha_{\mathrm{bA}} \\ \tau_{\mathrm{bck}} \cdot \alpha_{\mathrm{bc}} \end{cases} \tag{J.59}$$

where the maximum characteristic bond strength of the adhesive may be obtained from Formula (J.60), where the definitions of f_{Ack} and f_{Atk} are given in J.5.

$$\tau_{\mathrm{bAk}} = 0{,}6 \cdot \sqrt{\left(2 \cdot f_{\mathrm{Atk}} - 2 \cdot \sqrt{f_{\mathrm{Atk}}^2 + f_{\mathrm{Ack}} \cdot f_{\mathrm{Atk}}} + f_{\mathrm{Ack}}\right) \cdot f_{\mathrm{Atk}}} \tag{J.60}$$

$$\tau_{\mathrm{bck}} = 4{,}5 \cdot \sqrt{f_{\mathrm{cm}}} \tag{J.61}$$

The value of α_{bA} may be taken as 0,5 unless more accurate information is available based on production data of NSM CFRP strips.

The value of α_{bc} may be taken as $(\eta_{\mathrm{cc}} \cdot k_{\mathrm{tc}} \cdot k_{\mathrm{tt}})^{0,5}$ unless more accurate information is available based on production data of NSM CFRP strips, with η_{cc}, k_{tc} and k_{tt} in accordance with 5.1.6.

NOTE For round and square CFRP bars further considerations for F_{fbrd} (e.g. bond tests) are necessary.

(2) The NSM anchorage should comply with provisions of J.11.1.2.4 for end cover separation.

(3) Where the limit in Formula (J.54) in J.11.1.2.5 is exceeded additional CFRP stirrups shall be provided for NSM.

J.12 Detailing of members and particular rules

J.12.1 Flexural strengthening with externally bonded CFRP

(1) The following should be applied to the centre-to-centre spacing s_{f}, of EBR CFRP strips:

$s_{\mathrm{f,max}} \leq 0{,}2$ times distance between points of zero moments;

$s_{\mathrm{f,max}} \leq 3$ times slab thickness;

$s_{\mathrm{f,max}} \leq 0{,}4$ times cantilever length;

$s_{\mathrm{f,max}} \leq 400$ mm.

(2) The distance of the longitudinal edge of the strip from the member edge should be at least equivalent to the nominal concrete cover c_{nom} of the internal reinforcement.

J.12.2 Flexural strengthening with NSM CFRP

(1) Where slots are cut into the cover concrete for bonding of NSM CFRP systems, they should be located such that the cover is not adversely compromised when considering the accuracy of installation equipment along with adequate tolerance for installation.

(2) The geometrical limits for slots and edge distances and spacing, for NSM CFRP bars and strips should be in accordance with Table (J.2).

Table J.2 — Geometrical limits for slots and edge distances and spacing, in mm.

Geometrical limits	Square NSM CFRP bars	Round NSM CFRP bars	NSM CFRP strips
The slot width b_{slot}	$t_{\mathrm{f}} + 2 \leq b_{\mathrm{slot}} \leq t_{\mathrm{f}} + 6$	$\emptyset_{\mathrm{f}} + 2 \leq b_{\mathrm{slot}} \leq \emptyset_{\mathrm{f}} + 6$	$t_{\mathrm{f}} + 2 \leq b_{\mathrm{slot}} \leq t_{\mathrm{f}} + 4$
The slot depth t_{slot}	$b_{\mathrm{f}} + 1 \leq t_{\mathrm{slot}} \leq b_{\mathrm{f}} + 3$ $t_{\mathrm{f}} \leq b_{\mathrm{f}}$	$\emptyset_{f} + 1 \leq t_{\mathrm{slot}} \leq \emptyset_{\mathrm{f}} + 3$	$b_{\mathrm{f}} \leq t_{\mathrm{slot}} \leq b_{\mathrm{f}} + 2$
Distance from slot to edge a_{r} in accordance with Figure J.7	$a_{\mathrm{r}} \geq 4 \cdot b_{\mathrm{f}}$	$a_{\mathrm{r}} \geq 4 \cdot b_{\mathrm{f}}$	$a_{\mathrm{r}} \geq 4 \cdot b_{\mathrm{f}}$
The centre-to-centre spacing of adhesively bonded CFRP reinforcement s_{f}	$\max\{3 \cdot b_{\mathrm{slot}}; D_{\mathrm{upper}}\} \leq s_{\mathrm{f}} \leq \min\{0{,}2 \cdot l_{0\mathrm{b}}; 3 \cdot h\}$	$\max\{3 \cdot b_{\mathrm{slot}}; D_{\mathrm{upper}}\} \leq s_{\mathrm{f}} \leq \min\{0{,}2 \cdot l_{0\mathrm{b}}; 3 \cdot h\}$	$\max\{b_{\mathrm{f}}; D_{\mathrm{upper}}\} \leq s_{\mathrm{f}} \leq \min\{0{,}2 \cdot l_{0\mathrm{b}}; 3 \cdot h\}$

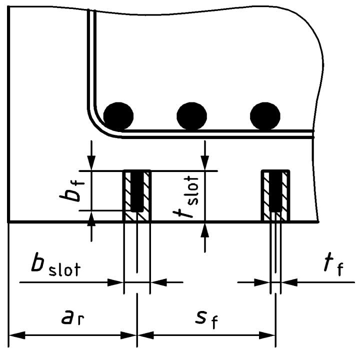


Figure J.7 — Geometrical limits for slots and edge distances and spacings of strips

(3) Near curved edges of slabs and beams with NSM CFRP bars, a minimum edge distance of 150 mm in the direction of centre of curvature should be maintained.

(4) The radius of curvature of the NSM CFRP strips shall be at least 2 m.

J.12.3 Permissible radius for bending of CFRP

(1) Straight prefabricated CFRP bars should not be designed to be arranged at a radius that is less than 1 000 times their thickness unless stresses that arise from the bending process are considered in determining the tensile strength f_{fuk}.

J.12.4 Permissible layers of CF sheets and CFRP strips

(1) CF sheets should be bonded in no more than five layers for flexural or shear strengthening and a maximum of ten layers for strengthening columns.

(2) CFRP strips should be bonded in no more than two layers. The maximum thickness of the CFRP strip cross section excluding the adhesive should not exceed 3 mm.

(3) No more than a single NSM strip or bar should be bonded into one slot.

J.12.5 Lapping of closed wrapped strengthening systems

(1) For strengthening with closed wrapped sheets and strips required lap length for the strengthening system has to be considered.

J.13 Additional rules for precast concrete elements and structures

NOTE There are no additional requirements.

J.14 Lightly reinforced concrete structures

NOTE There are no additional requirements.

J.15 Material requirements for ABR strengthening systems

(1) ABR strengthening systems used for structures in accordance with this Eurocode shall comply with the relevant product standards.

(2) The properties of CFRP strips, except η_{f}, may be determined according to ISO 10406-3. The properties of CF sheets, except η_{f}, may be determined according to ISO 10406-2.

(3) Recommended values for the following parameters are provided in this Annex. These values may alternatively be obtained using production data where available.

— η_{f}

— k_{cc}

— $k_{\mathrm{sys,b1}}$, $k_{\mathrm{sys,b2}}$ and $k_{\mathrm{sys,b3}}$

— α_{bA}

— α_{bc}

Annex K
(normative)

Bridges

K.1 Use of this annex

(1) This Normative Annex contains additional provisions for design of bridges.

NOTE 1 NDPs in Clauses 1 to 14 and Annexes A to S can be given different values in the National Annex for bridge applications.

NOTE 2 More restrictive provisions to Clauses 1 to 14 and Annexes A to S can be given in the National Annex as set out in this annex, or as specified by the relevant authority.

K.2 Scope and field of application

(1) This Normative Annex applies to bridges. The provisions of Clauses 1 to 14 and all normative annexes apply to bridges unless otherwise stated in this Normative Annex. The adoption of Informative Annexes for bridges is indicated in the country's National Annex.

K.3 Terms, definitions and symbols

NOTE There are no additional requirements.

K.4 Basis of design

NOTE There are no additional requirements.

K.5 Materials

(1) In addition to Clause 5, the following paragraphs shall be applied.

(2) In addition to 5.2, the types of reinforcing steel, the maximum bar diameters, the strength classes and the ductility classes may be further restricted.

NOTE The content of 5.2 applies unless the National Annex gives more restrictive provisions.

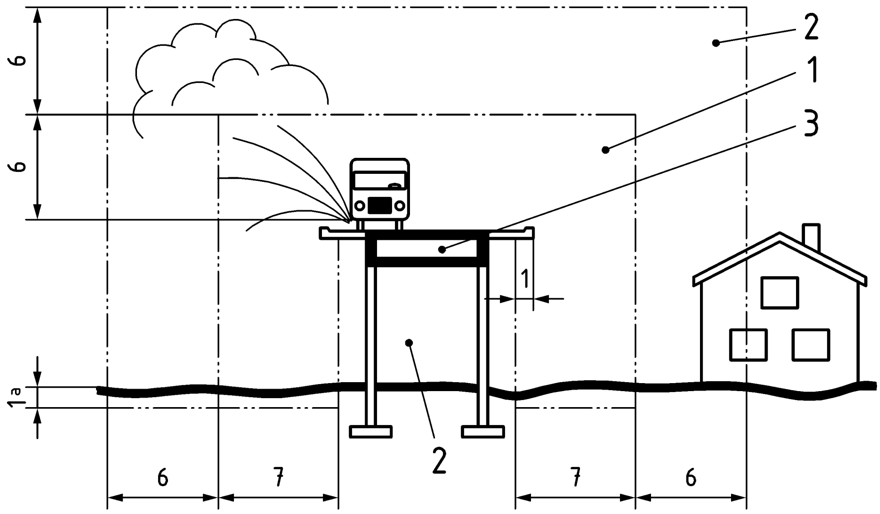
K.6 Durability and concrete cover

(1) In addition to Clause 6, the following paragraphs shall be applied.

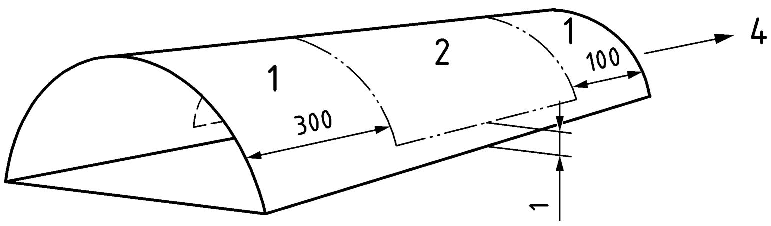
(2) Where de-icing salts are used, exposed concrete surfaces above, adjacent to and in the ground below the carriageway should be considered as being affected by de-icing salts according to Figure K.1.

NOTE 1 The exposure classes, dimensions and distances of Zones I, II and III are given in Figure K.1 unless the National Annex uses a different figure or gives different values.

Dimensions in metres



a) Road bridges



b) Inside of tunnel

Key

1	Zone I:	XD3 + XF4
2	Zone II:	XD1 + XF2
3	Zone III:	XC3
4		Traffic direction
a		Distance to ground surface in a)

Figure K.1 — Exposure classes for structures along roads

(3) Protection by waterproofing systems may be considered when determining the exposure classes for bridge decks.

NOTE 2 Exposure classes for bridge decks protected by waterproofing systems are: XD1 for bridge deck top surface; XD3 inside, top side and outside of curbs; and XD3 for surfaces under expansion joints directly affected by water containing de-icing salts unless the National Annex gives different values.

(4) In addition to 6.5.2.3 and Table 6.6, more restrictive rules for minimum concrete cover for post-tensioning ducts may be given.

NOTE 3 The contents of 6.5.2.3 and Table 6.6 apply unless the National Annex gives more restrictive provisions.

K.7 Structural analysis

(1) In addition to Clause 7, the following paragraphs shall be applied.

(2) Instead of 7.1(6), more restrictive rules regarding what stiffnesses may be neglected in analysis and design may be specified.

NOTE 1 The contents of 7.1(6) apply unless the National Annex gives more restrictive provisions.

(3) Instead of 7.2.1.1(3) and 7.2.1.1(5) more restrictive provisions may be given for geometric imperfections by a country in the National Annex within the limits given in Clause 7.

NOTE 2 The contents of 7.2.1.1(3) and 7.2.1.1(5) apply unless the National Annex gives more restrictive provisions.

(4) 7.2.2(4) and 7.2.3(6) shall not be applied.

(5) Stress increase in tendons according to 7.6.5(3) without detailed calculation shall not be applied.

K.8 Ultimate limit states (ULS)

(1) In addition to Clause 8, the following paragraphs shall be applied.

(2) In addition to 8.1.4, more restrictive provisions may be given by a country in the National Annex for specific design situations within the limits of 8.1.4.

NOTE 1 The contents of 8.1.4 apply unless the National Annex gives more restrictive provisions.

(3) Instead of 8.2.3(3) and 8.2.3(5) more restrictive values for cotθ and ν may be given by a country in the National Annex within the limits of 8.2.3.

NOTE 2 The values given in 8.2.3(3) and 8.2.3(5) apply unless the National Annex gives more restrictive values.

(4) Instead of 8.2.5(3) more restrictive values for cotθf may be given by a country in the National Annex within the limits of 8.2.5.

NOTE 3 The values given in 8.2.3(3) and 8.2.3(5) apply unless the National Annex gives more restrictive values.

(5) Instead of 8.2.6(6), more restrictive values Dlower > 8 mm may be specified by a country in the National Annex within the limits of 8.2.6.

NOTE 4 The values given in 8.2.6(6) apply unless the National Annex gives more restrictive values.

(6) Instead of 8.6(2), more restrictive values for νpart may be given by a country in the National Annex within the limits of 8.6.

NOTE 5 The values given in 8.6(2) apply unless the National Annex gives more restrictive values.

K.9 Serviceability limit states (SLS)

(1) In addition to Clause 9, the following paragraphs shall be applied.

(2) In addition to Clause 9, more restrictive provisions for serviceability limit state design (SLS) may be given.

NOTE 1 The contents of Clause 9 apply unless the National Annex gives more restrictive provisions.

— (3) In addition to 9.2.1(6) and Table 9.2 (NDP), in the transient design situation (e.g. construction or maintenance) crack widths should be limited to $w_{\mathrm{lim,cal}}$ for quasi-permanent load combination.

NOTE 2 The value $w_{\mathrm{lim,cal}}$ = 0,5 mm applies unless the National Annex gives different values.

NOTE 3 The decompression criteria for frequent combination of action can be substituted by a stress limit of f_{ctm} or $f_{\mathrm{ctm}}(t)$ for t < 28 days at the extreme fibre in the tensile zone unless the National Annex gives a different value.

(4) 9.3.2 and 9.3.3 shall not be applied.

(5) In addition to 9.4(2) and 9.4(4), additional damping effected by pavement, railings and other non-structural components may be considered on the basis of experimental evidence.

NOTE 4 Values for structural damping of railway bridges can be found in EN 1991-2:2021, Table 8.6 (NDP), and for footbridges in EN 1991-2:2021, Annex G.

K.10 Fatigue verification

(1) In addition to Clause 10 and E.4, the following paragraphs shall be applied when using damage equivalent stress range.

NOTE 1 A fatigue verification can generally be omitted for the following structures and members:

a) footbridges, with the exception of structural components with wind and rain induced vibrations;
b) buried arch and frame structures with a minimum earth cover of 1,00 m for road bridges and 1,50 m for railway bridges, respectively;
c) foundations of bridges except those of frame and arch structures in railway bridges;
d) piers and columns which do not have moment connection to superstructures;
e) abutments of road and railway bridges which do not have moment connection to superstructures, except for deck slabs of abutments.

(2) In addition to 10.7, more restrictive provisions for shear at interfaces may be given.

NOTE 2 The contents of 10.7 apply unless the National Annex gives more restrictive provisions.

K.10.1 General rules using damage equivalent stress range

(1) The relevant fatigue load models for fatigue verification of bridges using the method of damage equivalent stress range (FLM3 for road bridges, LM71 for railway bridges) given in EN 1991-2 and procedures for the calculation of the equivalent stress range $\Delta\sigma_{\mathrm{S,equ}}$ for superstructures of road and railway bridges given in EN 1990 should be used.

K.10.2 Verification for reinforcement using damage equivalent stress range

K.10.2.1 General

(1) For road and railway bridges the damage equivalent stress range for reinforcement verification in Formula (E.1) should be calculated according to Formula (K.1):

$$\Delta\sigma_{\mathrm{s,equ}} = \Delta\sigma_{\mathrm{s,Ec}} \cdot \lambda_{\mathrm{s}} \tag{K.1}$$

where

$\Delta\sigma_{\mathrm{s,Ec}}$ is the reinforcement stress range caused by the following fatigue load models:

For road bridges: based on the load combination given in 10.2 with fatigue load model 3 (FLM3) according to EN 1991-2 with the axle loads increased by the following factors:

— 1,75 for verification at intermediate supports in continuous bridges,

— 1,40 for verification in other areas.

For railway bridges: load model 71 (and where required SW/0) according to EN 1991-2;

λ_s is the damage equivalent factor for fatigue.

(2) The correction factor λ_s reflects the influence of span, annual traffic volume, design service life, multiple lanes/tracks, and may be calculated by Formula (K.2):

$$\lambda_s = \varphi \cdot \lambda_{s,1} \cdot \lambda_{s,2} \cdot \lambda_{s,3} \cdot \lambda_{s,4} \tag{K.2}$$

where

$\lambda_{s,1}$ is a factor accounting for member type (e.g. continuous beam) and takes into account the damaging effect of traffic depending on the critical length of the influence line or area;

$\lambda_{s,2}$ is a factor that takes into account the traffic volume;

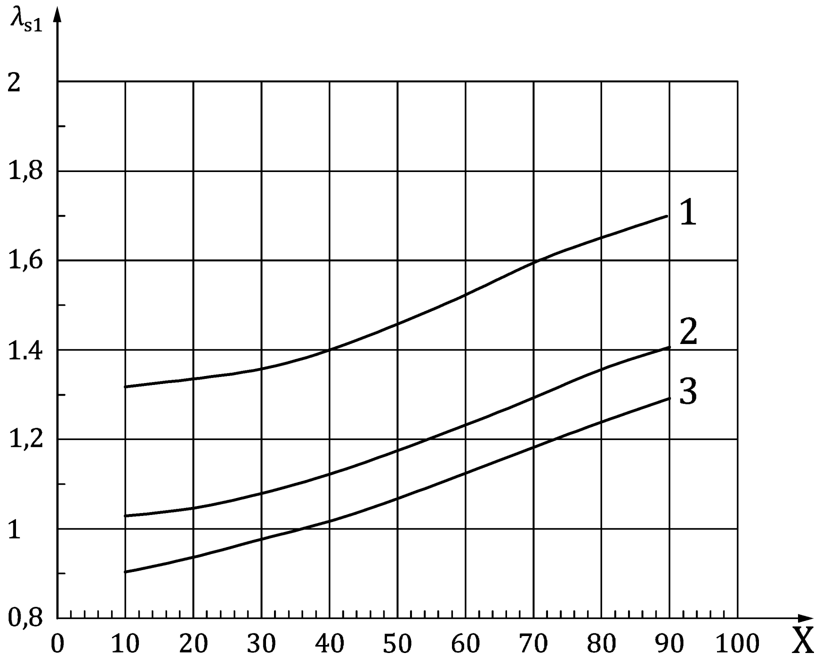
$\lambda_{s,3}$ is a factor that takes into account the design service life of the bridge;

$\lambda_{s,4}$ is a factor to be applied when the structural element is loaded by more than one lane/track;

φ equals the damage equivalent impact factor φ_{fat} controlled by the surface roughness according to EN 1991-2, Annex B for road bridges and the dynamic factor Φ according to EN 1991-2 for railway bridges, respectively.

K.10.2.2 Road bridges

(1) For road bridges $\lambda_{s,1}$ should be determined from Figures K.2 and K.3 to take into account the critical length of the influence line and the shape of S-N-curve.



Key

1 couplers for reinforcing steel

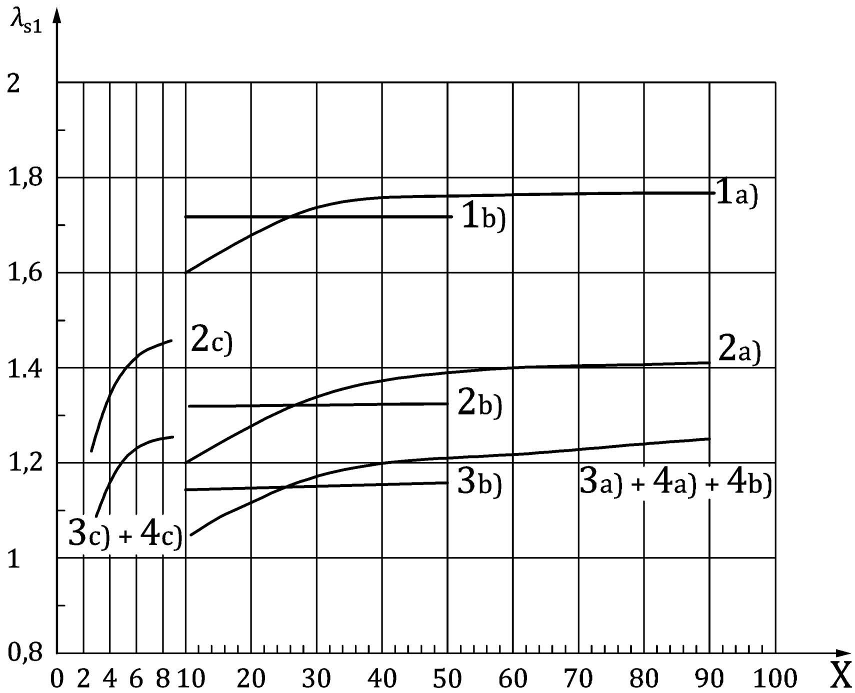
2 curved tendons in steel ducts

3 reinforcing steel (straight and bent); pre-tensioning (all); post-tensioning (strand in plastic ducts or straight tendons in steel ducts)

X critical length of influence line [m]

Figure K.2 — $\lambda_{s,1}$ value for fatigue verification in the intermediate support area of road bridges

Verification in span and for carriageway slab



Key

1)	couplers for reinforcing steel	a)	continuous member
2)	curved tendons in steel ducts	b)	single span member
3)	reinforcing steel (straight and bent); pre-tensioning (all); post-tensioning (strand in plastic ducts or straight tendons in steel ducts)	c)	deck slab
4)	shear reinforcement		
X	critical length of influence line [m]		

Figure K.3 — $\lambda_{s,1}$ value for fatigue verification in span and for local elements of road bridges

(2) For road bridges $\lambda_{s,2}$ reflects the influence of the annual traffic volume and may be calculated by Formula (K.3) also depending on the traffic type:

$$\lambda_{s,2} = \overline{Q} \cdot \left(\frac{N_{obs}}{2{,}0 \cdot 10^6}\right)^{1/k_{f2}} \tag{K.3}$$

where

N_{obs} is the number of lorries per year according to EN 1991-2:2018, Table 6.5;

k_{f2} is the slope of the appropriate *S*-*N*-Line to be taken from Tables E.1 (NDP) and E.2 (NDP);

$\overline{Q}$ is a factor for traffic type according to Table K.1.

Table K.1 — Factors for traffic type of road bridges

slope of *S-N* curve	$\overline{Q}$-factor for traffic type (see EN 1991-2)		
	long distance	medium distance	local traffic
$k_{f2} = 5$	1,0	0,90	0,73
$k_{f2} = 7$	1,0	0,92	0,78
$k_{f2} = 9$	1,0	0,94	0,82

(3) $\lambda_{s,3}$ reflects the influence of the service life and may be calculated from Formula (K.4):

$$\lambda_{s,3} = \left(\frac{N_{years}}{100}\right)^{1/k_{f2}} \tag{K.4}$$

where

N_{years} is the design service life of the bridge;

k_{f2} is the slope of appropriate *S-N* line to be taken from Tables E.1 (NDP) and E.2 (NDP).

(4) $\lambda_{s,4}$ reflects the influence of multiple lanes/tracks and may be calculated by Formula (K.5):

$$\lambda_{s,4} = \left(\frac{\sum N_{obs,i}}{N_{obs,1}}\right)^{1/k_{f2}} \tag{K.5}$$

where

$N_{obs,i}$ is the number of lorries expected on lane *i* per year;

$N_{obs,1}$ is the number of lorries on the slow lane per year.

K.10.2.3 Railway bridges

(1) For railway bridges, $\lambda_{s,1}$ accounts for the critical length of the influence line and the traffic mix. The values of $\lambda_{s,1}$ for standard traffic mix and heavy traffic mix may be taken from Table K.2. The values have been calculated on the basis of a constant ratio of bending moments to stress ranges.

The values given for traffic mix correspond to the combination of train types given in Annex F of EN 1991-2.

Values of $\lambda_{s,1}$ for a critical length of influence line between 2 m and 20 m may be obtained from the Formula (K.6):

$$\lambda_{s,1}(L) = \lambda_{s,1}(2\text{ m}) + [\lambda_{s,1}(20\text{ m}) - \lambda_{s,1}(2\text{ m})] \cdot (\lg L - 0{,}3) \tag{K.6}$$

where

L is the critical length of the influence line [m];

$\lambda_{s,1}(2\text{ m})$ is the $\lambda_{s,1}$ value for $L = 2$ m;

$\lambda_{s,1}(20\text{ m})$ is the $\lambda_{s,1}$ value for $L = 20$ m.

Table K.2 — $\lambda_{s,1}$ values for simply supported and continuous members of railway bridges

a) simply supported members				b) continuous members (interior span)			
	L [m]	*STM*	*HTM*		L [m]	*STM*	*HTM*
(1)	≤ 2	0,90	0,95	(1)	≤ 2	0,95	1,05
	≥ 20	0,65	0,70		≥ 20	0,50	0,55
(2)	≤ 2	1,00	1,05	(2)	≤ 2	1,00	1,15
	≥ 20	0,70	0,70		≥ 20	0,55	0,55
(3)	≤ 2	1,25	1,35	(3)	≤ 2	1,25	1,40
	≥ 20	0,75	0,75		≥ 20	0,55	0,55
(4)	≤ 2	0,80	0,85	(4)	≤ 2	0,75	0,90
	≥ 20	0,40	0,40		≥ 20	0,35	0,30
c) continuous members (end span)				**d) continuous members (intermediate support area)**			
	L [m]	*STM*	*HTM*		L [m]	*STM*	*HTM*
(1)	≤ 2	0,90	1,00	(1)	≤ 2	0,85	0,85
	≥ 20	0,65	0,65		≥ 20	0,70	0,75
(2)	≤ 2	1,05	1,15	(2)	≤ 2	0,90	0,95
	≥ 20	0,65	0,65		≥ 20	0,70	0,75
(3)	≤ 2	1,30	1,45	(3)	≤ 2	1,10	1,10
	≥ 20	0,65	0,70		≥ 20	0,75	0,80
(4)	≤ 2	0,80	0,90	(4)	≤ 2	0,70	0,70
	≥ 20	0,35	0,35		≥ 20	0,35	0,40

STM standard traffic mix

HTM heavy traffic mix

(1) reinforcing steel, pre-tensioning (all), post-tensioning (tendons in plastic ducts and straight tendons in steel ducts); *S-N* curve with $k_{f1} = 5$, $k_{f2} = 9$ and $N^* = 10^6$ (values may be changed due to changes in k_{f1} and k_{f2} or N^*);

(2) post-tensioning (curved tendons in steel ducts); *S-N* curve with $k_{f1} = 3$, $k_{f2} = 7$ and $N^* = 10^6$ (values may be changed due to changes in k_{f1} and k_{f2} or N^*);

(3) couplers (prestressing steel); *S-N* curve with $k_{f1} = 5$, $k_{f2} = 5$ and $N^* = 10^6$ (values may be changed due to changes in k_{f1} and k_{f2} or N^*);

(4) couplers (reinforcing steel); welded bars including tack welding and butt joints; *S-N* curve with $k_{f1} = 3$, $k_{f2} = 5$ and $N^* = 10^7$ (values may be changed due to changes in k_{f1} and k_{f2} or N^*).

NOTE 1 Interpolation between the given L-values according to Formula (K.6) may be carried out.

NOTE 2 Different N^* values can be considered as follows: $\lambda_{s,1,N^*new} = \lambda_{s,1,N^*old}\,(N^*_{old}/N^*_{new})^{1/k_{f1}}$

(2) No values of $\lambda_{s,1}$ for a light traffic mix are given in Table K.2. For bridges designed to carry a light traffic mix the values for $\lambda_{s,1}$ to be used may be based either on the values given in Table K.2 for standard traffic mix or on values determined from detailed calculations.

(3) $\lambda_{s,2}$ reflects the influence of the annual traffic volume. $\lambda_{s,2}$ may be calculated by Formula (K.4).

(4) $\lambda_{s,3}$ reflects the influence of the service life and may be calculated from Formula (K.7):

$$\lambda_{s,3} = \left(\frac{N_{years}}{100}\right)^{1/k_{f2}} \tag{K.7}$$

where

N_{years} is the design service life of the bridge;

k_{f2} is the slope of appropriate *S*-*N* curve to be taken from Tables E.1 (NDP) and E.2 (NDP).

(5) For railway structures carrying multiple tracks, the fatigue loading should be applied to a maximum of two tracks in the most unfavourable positions (see prEN 1991-2). The effect of loading from two tracks may be calculated by Formula (K.8):

$$\lambda_{s,4} = \left(n_{st} + (1-n_{st}) \cdot s_1^{k_{f2}} + (1 - n_{st}) \cdot s_2^{k_{f2}}\right)^{1/k_{f2}} \tag{K.8}$$

where

$$s_1 = \frac{\Delta\sigma_1}{\Delta\sigma_{1+2}} \tag{K.9}$$

$$s_2 = \frac{\Delta\sigma_2}{\Delta\sigma_{1+2}} \tag{K.10}$$

where

n_{st} is the proportion of traffic that crosses the bridge simultaneously (suggested value $n_{st} = 0{,}12$);

$\Delta\sigma_1$, $\Delta\sigma_2$ is the stress range due to load model 71 on one track at the section to be checked;

$\Delta\sigma_{1+2}$ is the stress range at the same section due to the load model LM71 on any two tracks, according to EN 1991-2;

k_{f2} is the slope of the appropriate *S*-*N* curve from Tables E.1 (NDP) and E.2 (NDP).

If only compressive stresses occur under traffic loads on a track, the corresponding value should be $s_j = 0$.

K.10.3 Verification for concrete using damage equivalent stress range

K.10.3.1 Road bridges

(1) Road bridges should be verified for the combination of actions in Formula (10.1) with the load model 3 (FLM3) according to EN 1991-2.

NOTE Fatigue verification of members of road bridges in which the maximum concrete stress under characteristic load combination is limited to $\sigma_c \leq 0{,}6\, f_{ck}$ can be omitted.

K.10.3.2 Railway bridges

(1) For railway bridges, the upper and lower stresses of the damage equivalent stress amplitude used in Formula (E. 2) should be calculated according to Formula (K.11)

$$\begin{aligned} \sigma_{cd,max,equ} &= \gamma_{Ff} \cdot (\sigma_{c,perm} + \lambda_c\,(\sigma_{c,max,71} - \sigma_{c,perm})) \\ \sigma_{cd,min,equ} &= \gamma_{Ff} \cdot (\sigma_{c,perm} - \lambda_c\,(\sigma_{c,perm} - \sigma_{c,min,71})) \end{aligned} \tag{K.11}$$

where

$\sigma_{c,perm}$ is the compressive concrete stress caused by the combination of actions in Formula (10.1), without load model 71;

$\sigma_{c,max,71}$ is the maximum compressive stress caused by the combination of actions in Formula (10.1) including load model 71 and the dynamic factor Φ according to EN 1991-2;

$\sigma_{c,min,71}$ is the minimum compressive stress under the combination of actions in Formula (10.1) including load model 71 and the dynamic factor Φ according to EN 1991-2;

λ_c is a correction factor to calculate the upper and lower stresses of the damage equivalent stress spectrum from the stresses caused by load model 71.

(2) The correction factor λ_c accounts for the permanent stress, the span, annual traffic volume, design service life and multiple tracks. It should be calculated from Formula (K.12):

$$\lambda_c = \lambda_{c,0} \cdot \lambda_{c,1} \cdot \lambda_{c,2,3} \cdot \lambda_{c,4} \tag{K.12}$$

where

$\lambda_{c,0}$ is a factor to take account of the permanent stress;

$\lambda_{c,1}$ is a factor accounting for member type (e.g. continuous beam) that takes into account the damaging effect of traffic depending on the critical length of the influence line or area;

$\lambda_{c,2,3}$ is a factor to account for the traffic volume and the design service life of the bridge;

$\lambda_{c,4}$ is a factor to be applied when the structural member is loaded by more than one track.

(3) $\lambda_{c,0}$ reflects the influence of the permanent stress and may be calculated from Formula (K.13)

$$\lambda_{c,0} = 0{,}94 + 0{,}2\frac{\sigma_{c,perm}}{f_{cd,fat}} \geq 1 \quad \text{for the compression zone} \tag{K.13}$$

$\lambda_{c,0} = 1$ for the precompressed tensile zone (including prestressing effect)

(4) $\lambda_{c,1}$ is a function of the critical length of the influence line and the traffic. For standard traffic mix and heavy traffic mix it may be taken from Table K.3.

Values of $\lambda_{c,1}$ for critical lengths of influence lines between 2 m and 20 m may be obtained by applying Formula (K.6) with $\lambda_{s,1}$ replaced by $\lambda_{c,1}$.

(5) $\lambda_{c,2,3}$ reflects the influence of annual traffic volume and design service life and may be calculated from Formula (K.14):

$$\lambda_{c,2,3} = 1 + \frac{1}{8}\lg\left[\frac{\mathrm{Vol}}{25 \cdot 10^6}\right] + \frac{1}{8}\lg\left[\frac{N_{years}}{100}\right] \tag{K.14}$$

where

Vol is the volume of traffic (tonnes/years/track);

N_{years} is the design service life of the bridge.

(6) $\lambda_{c,4}$ reflects the effect of loading from more than one track. For structures carrying multiple tracks, the fatigue loading should be applied to a maximum of two tracks in the most unfavourable positions (see EN 1991-2). The effect of loading from two tracks may be calculated from Formulae (K.15) and (K.16):

$$\lambda_{c,4} = 1 + (1/8)\lg n_{st} \geq 0{,}54 \quad \text{for } a \leq 0{,}8 \tag{K.15}$$

$$\lambda_{c,4} = 1 \quad \text{for } a > 0{,}8 \tag{K.16}$$

$$\text{with } a = \frac{\max\{|\sigma_{c1}|;|\sigma_{c2}|\}}{|\sigma_{c1+2}|} \tag{K.17}$$

where

n_{st} is the proportion of traffic crossing the bridge simultaneously (recommended value $n_{st} = 0{,}12$);

σ_{c1}, σ_{c2} is the compressive stress caused by load model 71 on one track, including the dynamic factor for load model 71 according to EN 1991-2;

σ_{c1+2} is the compressive stress caused by load model 71 on two tracks, including the dynamic factor for load model 71 according to EN 1991-2.

(7) No values of $\lambda_{c,1}$ are given in Table K.3 for a light traffic mix. For bridges designed to carry a light traffic mix the values for $\lambda_{c,1}$ to be used may be based either on the values given in Table K.3 for standard traffic mix or on values derived from detailed calculations.

Table K.3 — $\lambda_{c,1}$ values for simply supported and continuous members of railway bridges

a) simply supported members				**b) continuous members (interior span)**			
	L [m]	*STM*	*HTM*		*L* [m]	*STM*	*HTM*
(1)	≤ 2	0,70	0,70	(1)	≤ 2	0,75	0,90
	≥ 20	0,75	0,75		≥ 20	0,55	0,55
(2)	≤ 2	0,95	1,00	(2)	≤ 2	1,05	1,15
	≥ 20	0,90	0,90		≥ 20	0,65	0,70
c) Continous members (end span)				**d) Continuous members (intermediate support area)**			
	L [m]	*STM*	*HTM*		*L* [m]	*STM*	*HTM*
(1)	≤ 2	0,75	0,80	(1)	≤ 2	0,70	0,75
	≥ 20	0,70	0,70		≥ 20	0,85	0,85
(2)	≤ 2	1,10	1,20	(2)	≤ 2	1,10	1,15
	≥ 20	0,70	0,70		≥ 20	0,80	0,85
STM – standard traffic mix, *HTM* – heavy traffic mix (1) compression zone (2) precompressed tensile zone							
NOTE Interpolation between the given *L*-values according to Formula (K.6) is allowed, with $\lambda_{s,1}$ replaced by $\lambda_{c,1}$.							

K.11 Detailing of reinforcement and post-tensioning tendons

(1) In addition to Clause 11, the following paragraphs shall be applied.

(2) In addition to 11.2, more restrictive provisions for bundles may be given by a country in the National Annex within the limits of Clause 11.

NOTE 1 The contents of 11.2 apply unless the National Annex gives more restrictive provisions.

(3) In addition to 11.4, more restrictive provisions for methods of anchorage may be given by a country in the National Annex within the limits of 11.4.

NOTE 2 The contents of 11.4 apply unless the National Annex gives more restrictive provisions.

(4) In addition to 11.5, more restrictive provisions for types of laps may be given by a country in the National Annex within the limits given in 11.5.

NOTE 3 The contents of 11.5 apply unless the National Annex gives more restrictive provisions.

(5) In addition to 11.6, more restrictive provisions for post-tensioning tendons may be given by a country in the National Annex within the limits given in 11.6.

NOTE 4 The contents of 11.6 apply unless the National Annex gives more restrictive provisions.

K.12 Detailing of members and particular rules

K.12.1 General

(1) In addition to Clause 12, the following paragraphs shall be applied.

(2) 12.9 shall not be applied.

(3) Voids in structural members should be drained.

K.12.2 Minimum reinforcement rules

(1) Brittle failure of bridges due loss of cross section of prestressing tendons (e.g. due to corrosion) should be prevented. Methods a) or b) below may be applied:

a) Check that the structure with the remaining prestressing has sufficient strength for the traffic load combination. The requirement may be verified by the following procedure:

- (i) Calculate the maximum sagging and hogging bending moments in each span due to the relevant combination of actions, M_{Ed}.
- (ii) Determine the reduced cross sectional area of prestressing, $A_{\mathrm{p,red}}$, and corresponding prestressing load resultants that make the tensile stress reach the relevant tensile strength at the extreme tension fibre when the section is subject to M_{Ed}. Immediate and time-dependent prestressing losses should be taken into consideration.
- (iii) Calculate the ultimate flexural capacity, $M_{\mathrm{Rd,red}}$, of the section with reduced cross sectional area of prestressing, $A_{\mathrm{p,red}}$, and ordinary reinforcement using material partial safety factors for accidental design situations.
- (iv) Check that $M_{\mathrm{Rd,red}} \geq M_{\mathrm{Ed}}$.

b) Ensure that there is sufficient longitudinal reinforcement to compensate for the loss of resistance when the tensile strength of the concrete is exceeded and the section cracks.

If tensile stresses occur anywhere in the section for the relevant load combination (statically determinated effects of prestressing ignored, but indeterminated effects included, see 7.6.1(1)), longitudinal reinforcement should be at least:

$$A_{\mathrm{s,min}} = \frac{M_{\mathrm{rep}}}{z \cdot f_{\mathrm{yk}}} \tag{K.18}$$

where

M_{rep} cracking moment with extreme fibre tension reaching the relevant tensile strength for sections without prestressing. At the joint of segmental precast elements $M_{\text{rep}} = 0$;

z lever arm at the ultimate limit state related to the reinforcing steel.

NOTE The relevant load combinations for determining M_{Ed} and for checking whether tensile stresses occur are the frequent combination and the characteristic combination, respectively, and f_{ctm} is the relevant tensile strength of concrete unless the National Annex gives different combinations and tensile strengths.

K.12.3 Bridges with external or unbonded internal tendons

(1) External prestressing tendons should be inspectable and replaceable. For internal unbonded tendons, the anchorages should be inspectable.

(2) Bridges should be verified to ensure that any one tendon of the external and, if specified in the project specification, any one of the unbonded tendons can be replaced. A project specific load combination with permissible live loads (traffic) should be defined for the replacement.

(3) When designing members for anchorage and deviation of external tendons, deviation forces accounting for an angular tolerance of $\pm 3°$ should be taken into account.

(4) The risk of vibration of external tendons induced between tendon fix points by traffic loads should be controlled by adequate detailing of tendon supports. Without more detailed analysis, spacing of tendon support points should be limited to 12 m for railway bridges and 18 m for road bridges.

K.12.4 Cable stayed, extradosed and suspension bridges

(1) For stay cables, extradosed cables and hangers for cable supported members and structures, the general provisions of EN 1990, A.2, should be applied.

(2) For ultimate limit state verifications, the design value of tension resistance should be assumed in accordance with EN 1993-1-11.

(3) In addition to 10.4(1), for prestressing cables which extend above or below the bridge deck and in which

— the variations of tensile stress under frequent traffic loads are less than 100 MPa;

— the vibrations caused by wind are negligible;

the maximum tensile stress under the characteristic combinations at SLS should be:

$$\max \sigma_{\text{cable}} \leq ,60\sigma_{\text{uk}} \qquad \text{if } \Delta\sigma_{\text{freq}} \leq 50 \text{ MPa} \tag{K.19}$$

$$\max \sigma_{\text{cable}} \leq 0,46 \cdot \left(\frac{\Delta\sigma_{\text{freq}}}{140 \text{ MPa}}\right)^{-0,25} \cdot \sigma_{\text{uk}} \qquad \text{if } 50 \text{ MPa} < \Delta\sigma_{\text{freq}} \leq 100 \text{ MPa} \tag{K.20}$$

where

σ_{uk} is the characteristic breaking strength of the cable: $\sigma_{\text{uk}} = F_{\text{uk}}/A_{\text{m}}$;

$\Delta\sigma_{\text{freq}}$ is the variation of the tensile stress under frequent traffic loads.

(4) When cables are used as hangers to suspend the deck of bow-string bridges, the sequence of tensioning and the level of pretensioning after losses should ensure that under the characteristic combination at SLS a residual stress of at least 100 MPa is present in all hangers.

K.13 Additional rules for precast concrete elements and structures

(1) In addition to Clause 13, the following paragraphs shall be applied.

(2) In addition to 13.5.1 and Table 13.1, more restrictive rules for minimum concrete cover for and spacing of pre-tensioning tendons may be given.

NOTE 1 The contents of 13.5.1 and Table 13.1 apply unless the National Annex gives more restrictive provisions.

(3) 13.6, 13.7.1(9), (11), (12), (13), 13.6.3 and 13.8 shall not be applied. .

(4) In the case of segmental construction with precast elements and without internal bonded tendons in the tension chord, the effect of opening of the joint at ULS shall be considered. In these conditions, in the absence of a more detailed analysis, the force in the tension chord should be assumed to remain unchanged after the joints have opened. In consequence, as the applied load increases and the joints open (Figure K.4), the concrete stress field inclination θ within the web increases. The depth of concrete section available for the flow of the web compression field decreases to a value of z_{red} (see Formula (K.21)). The shear capacity may be evaluated in accordance with 8.2.3 by assuming a value of θ derived from the minimum value of residual depth z_{red}.

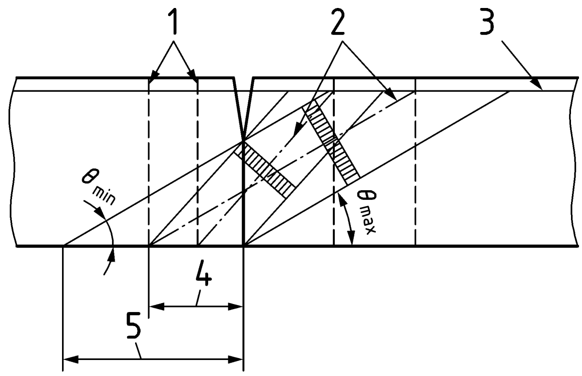
$$z_{\text{red}} = \frac{V_{\text{Ed}}}{b_{\text{w}} \cdot \nu \cdot f_{\text{cd}}} (\cot\theta + \tan\theta) \tag{K.21}$$

The prestressing force should be increased, if necessary, so that, at ULS under the combination of bending moment and shear, the joint opening is limited to the value $h - z_{\text{red}}$ as calculated above.

NOTE 2 The minimum value of z_{red} is 0,5h unless the National Annex gives a different value.

(5) Shear reinforcement should be provided within a distance $z_{\text{red}} \cdot \cot\theta$, but not greater than the segment length, from both edges of the joint, having the area per unit length according to Formula (K.22):

$$\frac{A_{\text{sw}}}{s} = \frac{V_{\text{Ed}}}{z_{\text{red}} \cdot f_{\text{ywd}} \cdot \cot\theta} \tag{K.22}$$



Key

1 axes of theoretical tie

2 axes of theoretical struts

3 Tension chord of truss (external tendon)

4 Field A: arrangement of stirrups with θ_{max} ($\cot\theta = 1{,}0$)

5 Field B: arrangement of stirrups with θ_{min} ($\cot\theta = 2{,}5$)

Figure K.4 — Diagonal stress fields in the web across a joint between segments

(6) In the case of segmental construction with precast box elements subject to torsion and no internal bonded prestressing in the tension region, the opening of a joint to an extent greater than the thickness of the corresponding flange should not be permitted unless a sufficient strength can be demonstrated with refined design models accounting for this opening of the joint.

NOTE 3 Such amount of joint opening induces a substantial modification of the torsional resisting mechanism.

K.14 Plain and lightly reinforced concrete structures

NOTE There are no additional requirements.

K.15 Amendments to annex G

(1) In addition to Annex G, more restrictive provisions for design of membrane-, shell- and slab-elements may be given.

NOTE The contents of Annex G apply unless the National Annex gives more restrictive provisions.

Annex L
(informative)

Steel Fibre Reinforced Concrete Structures

L.1 Use of this annex

(1) This Informative Annex provides supplementary rules for structures which comprise steel fibre reinforced concrete (SFRC).

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this Informative Annex, it can be used.

L.2 Scope and field of application

(1) This Informative Annex covers the design of concrete structures which comprise steel fibre reinforced concrete with or without reinforcing steel, pretensioning or post-tensioning tendons.

(2) This Informative Annex applies to normal weight and heavy weight, precast or cast in place concrete in accordance with 5.1.1.

NOTE Slabs on ground that are not required for the structural stability (e.g. industrial floors) are not intended to be designed with these provisions, and can be designed based on alternative provisions due to the specific requirements and conditions of such applications.

L.3 General

(1) All provisions of this Eurocode in Clauses 1 to 14 apply unless specifically omitted or supplemented in Annex L.

L.4 Basis of design - Partial factors for materials

(1) Partial factors for SFRC in tension γ_{SF} shall be applied for the limit states in accordance with Table L.1 (NDP). For SFRC in compression, the values γ_C given in Table 4.3 (NDP) apply.

NOTE The values of γ_{SF} given in Table L.1 (NDP) apply unless the National Annex gives different values

Table L.1 (NDP) — Partial factors for SFRC in tension

Design situations – Limit states	γ_{SF}
Persistent and transient design situations	1,50
Accidental design situations	1,20
Serviceability limit states	1,00

L.5 Materials

L.5.1 Properties

(1) Characteristic residual strengths $f_{R,1k}$ and $f_{R,3k}$ for concrete reinforced with steel fibres shall be determined according to EN 14651. They shall be classified into both the strength classes SC (1,0; 1,5; 2,0; 2,5; 3,0; 3.5; 4,0; 4.5; 5,0; 6,0; 7,0; 8,0) and the ductility classes a) to e) according to Table L.2.

NOTE 1 The classification is denominated according to $f_{R,1k}$ and $f_{R,3k}$ values. The selection of the strength class is obtained by comparing the experimentally determined $f_{R,1k}$ with the limits defining the strength classes, specified in brackets. The ductility class, denominated by a letter, is selected by comparing the experimentally determined $f_{R,3k}$ with the values of Table L.2.

NOTE 2 $f_{R,1k}$ and $f_{R,3k}$ to use for design correspond to the values obtained from classifying the strength class and ductility class according to Table L.2, respectively.

NOTE 3 Values of characteristic residual flexural strength used in this Eurocode correspond to those determined using EN 14651 at the age t_{ref} defined in 5.1.3(2).

Table L.2: Performance classes for SFRC [MPa]

Ductility classes	Strength classes SC ($f_{R,1k} \geq SC$)												Analytical formulae
	1,0	**1,5**	**2,0**	**2,5**	**3,0**	**3,5**	**4,0**	**4,5**	**5,0**	**6,0**	**7,0**	**8,0**	
a	0,5	0,8	1,0	1,3	1,5	1,8	2,0	2,3	2,5	3,0	3,5	4,0	$f_{R,3k} \geq 0{,}5SC$
b	0,7	1,1	1,4	1,8	2,1	2,5	2,8	3,2	3,5	4,2	4,9	5,6	$f_{R,3k} \geq 0{,}7SC$
c	0,9	1,4	1,8	2,3	2,7	3,2	3,6	4,1	4,5	5,4	6,3	7,2	$f_{R,3k} \geq 0{,}9SC$
d	1,1	1,7	2,2	2,8	3,3	3,9	4,4	5,0	5,5	6,6	7,7	8,8	$f_{R,3k} \geq 1{,}1SC$
e	1,3	2,0	2,6	3,3	3,9	4,6	5,2	5,9	6,5	7,8	9,1	10,4	$f_{R,3k} \geq 1{,}3SC$

NOTE 4 All strength classes apply unless the National Annex excludes specific classes.

NOTE 5 Intermediate classes can be used, if included in the National Annex.

L.5.2 Strength

(1) The characteristic residual flexural strengths $f_{R,1k}$ and $f_{R,3k}$ necessary for design correspond to the classification values defined in L.5.1.

(2) If required, the strength of SFRC should be specified for times t that can be before or after t_{ref} for a number of stages (e.g. demoulding, removal of propping, transfer of prestress).

L.5.3 Elastic deformation

(1) The values of the modulus of elasticity, the Poisson's ratio and the thermal expansion coefficient may be taken according to 5.1.

(2) Values of the modulus of elasticity and Poisson's ratio may be subject to higher variation than for ordinary concrete when steel fibres are introduced into the mix. Where they are a significant component of action effects, they should be determined by testing.

L.5.4 Creep and shrinkage

(1) Creep and shrinkage properties may be taken according to 5.1.

NOTE Creep in uniaxial tension affects only elastic strain, but fibres work in cracked condition and therefore the effect on structural behaviour can be usually neglected.

(2) Values of creep and shrinkage may be subject to higher variation than for ordinary concrete when steel fibres are introduced into the mix. Where they are a significant component of action effects, they should be determined by testing.

L.5.5 Design assumptions

L.5.5.1 Design residual tensile strengths

(1) For design according to Annex L, Formula (L. 1) should be satisfied:

$$f_{\mathrm{R,1k}}/f_{\mathrm{ctk;0,05}} \geq 0{,}5 \tag{L. 1}$$

(2) The values of the design residual tensile strength, f_{Ftsd} and f_{Ftud} should be taken as follows according to the rigid-plastic model (Figure L.2 a)):

$$f_{\mathrm{Fts,ef}} = \kappa_{\mathrm{O}} \cdot \kappa_{\mathrm{G}} \cdot 0{,}37 \cdot f_{\mathrm{R,1k}} \tag{L. 2}$$

$$f_{\mathrm{Ftu,ef}} = \kappa_{\mathrm{O}} \cdot \kappa_{\mathrm{G}} \cdot 0{,}33 \cdot f_{\mathrm{R,3k}} \tag{L. 3}$$

$$f_{\mathrm{Ftsd}} = f_{\mathrm{Fts,ef}}/\gamma_{\mathrm{SF}} \tag{L. 4}$$

$$f_{\mathrm{Ftud}} = f_{\mathrm{Ftu,ef}}/\gamma_{\mathrm{SF}} \tag{L. 5}$$

(3) The factor accounting for fibre orientation should be taken as $\kappa_0 = 0{,}5$ unless otherwise specified in Annex L or verified by testing.

(4) For bending moments, shear and punching shear forces, torsion, crack width control and deflections in slabs and beams made of concrete with consistency classes S2–S5 in accordance with EN 206, $\kappa_0 = 1{,}0$ may be used. Note that $\kappa_0 = 1{,}0$ corresponds to the orientation factor expected for test specimens according to EN 14651.

NOTE 1 For concrete consistency S5, the provision given in (6) applies in case of SFRC without additional longitudinal reinforcement having a depth under 250 mm.

(5) More accurate information based on production of SFRC and well-founded theoretical approaches may be used to establish more accurate values for κ_0. For favourable effects, an orientation factor $\kappa_0 > 1{,}0$ may be applied if experimentally verified. When $\kappa_0 > 1{,}0$ is applied in one direction, the κ_0 in the orthogonal direction must be considered lower than 1,0.

(6) For self compacting concrete, κ_0 in every direction should be experimentally determined by using specimens and casting methods representative for the real structure. For design the value of κ_0 shall not exceed 1.7.

(7) The factor accounting for the effect of member size on the coefficient of variation κ_{G} shall be calculated according to (L. 6). When κ_{kmax} is set equal to 0,7 based on L.15(5), a maximum value of 1,3 shall be considered for κ_{G}. For torsion, $\kappa_{\mathrm{G}} = 1{,}0$ shall be used.

$$\kappa_{\mathrm{G}} = 1{,}0 + A_{\mathrm{ct}} \cdot 0{,}5 \leq 1{,}5 \tag{L. 6}$$

where

A_{ct} area of the tension zone (in m^2) of the cross section involved in the failure of an equilibrium system.

NOTE 2 For shear, A_{ct} should be calculated along the surface of the shear crack, using the angle of the shear crack calculated in accordance with 8.2.

(8) Formula (5.4) in 5.1.6 may be used for determination of η_{cc} unless more accurate values are obtained from testing.

L.5.5.2 Stress-strain relation for structural analysis

(1) For structural analysis the constitutive law given in Figure L.1 may be used. The parameters should be determined as:

$$f_{\mathrm{Ft1,ef}} = \kappa_{\mathrm{O}} \cdot \kappa_{\mathrm{G}} \cdot 0{,}37 f_{\mathrm{R,1k}} \quad \text{(L. 7)}$$

$$f_{\mathrm{Ft3,ef}} = \kappa_{\mathrm{O}} \cdot \kappa_{\mathrm{G}} \cdot \left(0{,}57 f_{\mathrm{R,3k}} - 0{,}26 f_{\mathrm{R,1k}}\right) \quad \text{(L. 8)}$$

$$\varepsilon_{\mathrm{Ftu}} = w_{\mathrm{u}}/l_{\mathrm{cs}} \leq 2{,}5\ \mathrm{mm}/l_{\mathrm{cs}} \leq \varepsilon_{\mathrm{Ftud}} \quad \text{(L. 9)}$$

$$l_{\mathrm{cs}} = \begin{cases} \min\left\{h; s_{\mathrm{r,m,cal,F}}\right\} & \text{for members subjected to combined axial and bending} \\ s_{\mathrm{r,m,cal,F}} & \text{for members subjected to uniaxial tension} \end{cases} \quad \text{(L. 10)}$$

$$\varepsilon_{\mathrm{ctm}} = f_{\mathrm{ctm}}/E_{\mathrm{cm}} \text{ and } \varepsilon_{\mathrm{F,0}} = 2 \cdot \varepsilon_{\mathrm{ctm}}\text{, see figure L.1} \quad \text{(L. 11)}$$

Where $s_{\mathrm{r,m,cal,F}}$ is the mean crack spacing defined in L. 9.3, Formula (L.26).

As a simplification a constant value of l_{cs} = 125 mm may be used unless explicitly precluded in this standard.

NOTE 1 The value of $\varepsilon_{\mathrm{Ftud}}$ is 0,02 unless the National Annex gives a different value.

NOTE 2 Simplified stress distributions in cross sections used to determine the resistance to bending with axial force at the ultimate limit states are provided in L.8.1.

NOTE 3 For box girders or u shaped sections without longitudinal reinforcement in a flange or slab, subjected to predominately uniaxial tension, $s_{\mathrm{r,m,cal,F}}$ should be taken as the distance between adjacent webs.

NOTE 4 The structural characteristic length modifies the localized crack opening in an equivalent average strain between two following cracks, or, in case of a single crack, in the strain accumulated in an equivalent segment, treated as an elasto-softening hinge.

In millimetres

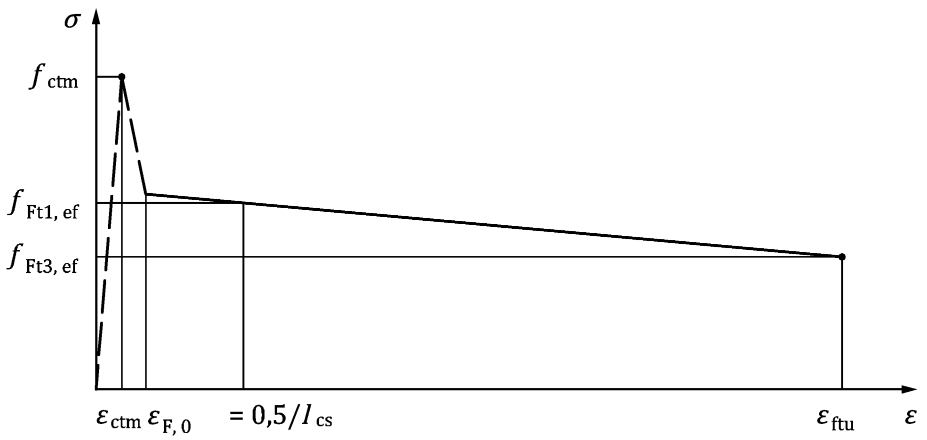


Figure L.1 — Constitutive law of SFRC in uniaxial tension for structural analysis

(2) The relation between σ_c and ε_c in compression in Formula (5.6) may be used to model the response of SFRC to short-term uniaxial compression provided the following modifications in the parameters are made:

$$\varepsilon_{\mathrm{c1}}(‰) = 0{,}7 f_{\mathrm{cm}}{}^{1/3}\left(1 + 0{,}03 f_{\mathrm{R,1k}}\right) \quad \text{(L. 12)}$$

and, for $\varepsilon_{c1} \leq \varepsilon \leq \varepsilon_{cu1}$:

$$k = 1 + \frac{20}{\sqrt{82 - 2{,}2 f_{R,1k}}} \quad \text{(L. 13)}$$

and

$$\varepsilon_{cu1} = k \cdot \varepsilon_{c1} \quad \text{(L. 14)}$$

L.6 Durability - Minimum cover

(1) For SFRC, the concrete cover due to durability requirements $c_{min,dur}$ according to 6.5.2.2 shall only apply to the embedded reinforcement, not to the steel fibres.

(2) To avoid fibre accumulation, a minimum cover of c_{min} = 20 mm to embedded reinforcement shall be used for all SFRC members.

(3) For ULS design of SFRC in exposure classes XC2-XC4, XD1-XD3, and XS1-XS3, designed to be uncracked at SLS, the tensile strength of SFRC at the greatest distance from the neutral axis shall be disregarded within a layer of $c_{f,dur}$ = 10 mm from the exposed surface.

(4) For ULS and SLS design of SFRC in exposure classes XC2-XC4, XD1-XD3, and XS1-XS3 designed to be cracked at SLS, the residual tensile strength of SFRC at the greatest distance from the neutral axis shall be disregarded with a distance of $c_{f,dur}=k_{dur}\cdot c_{min,dur}$ from the exposed surface unless provisions (5) or (6) are satisfied.

If the characteristic crack width calculated in accordance with 9.2 is less than the crack width limit $w_{lim,cal}$ stated in Table 9.2 (NDP) and L.9, the values of $c_{f,dur}$ may be reduced as follows:

$$c_{f,dur} = k_{dur}\, c_{min,dur} \frac{w_{k,cal}}{w_{lim,cal}} \geq 10 \text{ mm} \quad \text{(L. 15)}$$

NOTE The value of k_{dur} is 0,50 unless the National Annex gives a different value for use in a Country.

(5) If stainless steel fibres are used, the residual tensile strength of SFRC through the full cross-section may be used.

(6) The residual tensile strength of SFRC throughout the full cross-section may be used during the construction phase.

L.7 Structural analysis - Plastic analysis

(1) Plastic analysis of SFRC structures without any direct check of rotation capacity may be used for the ultimate limit state analysis of the following structure types:

— Foundations and slabs supported directly on ground even without any ordinary reinforcement.

— For statically indeterminate foundations elements on piles (rafts and slabs) without reinforcing steel, subject to the following conditions:

  — the ductility class is at least c; and

  — if the member is needed for structural stability $\kappa_G\, f_{R,3k}/f_{ctm,fl} \geq 1{,}0$.

— For statically indeterminate foundation elements on piles (rafts and slabs) and elevated slabs with reinforcing steel, subject to the following conditions:

  — two-way systems with $L_x/L_y \leq 1{,}5$ and $A_s \geq A_{s,min}$ as defined in Clause L.12 ($L_x > L_y$);

  — one-way systems and two-way systems with $L_x/L_y > 1{,}5$ and $A_s \geq \alpha_{duct}\cdot A_{s,min}$ with $A_{s,min}$ as defined in Clause L.12 ($L_x > L_y$).

NOTE 1 The value α_{duct} = 2,0 applies unless the National Annex gives a different value for use in a Country.

(2) For members not fulfilling the requirements of (1), methods based on plastic analysis, or linear analysis with redistribution, shall only be applied where the deformation capacity of the critical sections is demonstrated to be sufficient by calculation for the envisaged failure mechanisms to be formed.

NOTE 2 Due to possible localisation effects occurring at yielding of longitudinal reinforcement, minimum reinforcement as prescribed in L.12.1 is not sufficient to generally allow plastic analysis without verification of the deformation capacity.

(3) Methods to be used for verification of plastic deformation capacity shall take local variations in residual tensile strength into account.

L.8 Ultimate Limit States (ULS)

L.8.1 Bending with or without axial force

(1) A simplified rigid plastic approach for the residual tensile strength according to Figure L.2 a) may be used for ultimate limit state design of a member subjected to bending with or without axial compression, for ductility classes a, b and c.

For classes d and e this approach should only be used to determine the ULS moment capacity at the design tensile strain limit.

(2) The tensile strain limit relevant to the simplified rigid plastic approach should be calculated from Formula (L. 16) with $\varepsilon_{\mathrm{Ftud}}$ as defined in L.5.5.2(1):

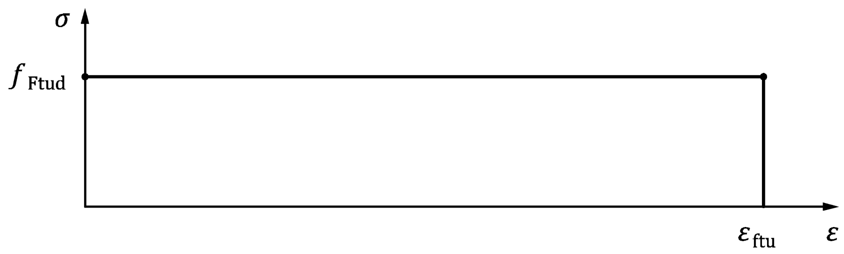
$$\varepsilon_{\mathrm{Ftu}} = \varepsilon_{\mathrm{Ftud}} \tag{L. 16}$$

(3) For the design of steel fibre reinforced cross sections a bi-linear residual tensile stress distribution according to Figure L.2 b) may be used with parameters as defined in L.5.5.2(1) and:

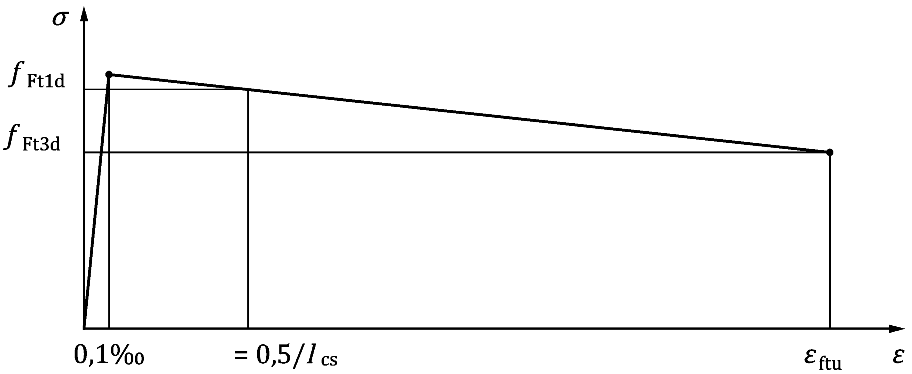
$$f_{\mathrm{Ft1d}} = f_{\mathrm{Ft1,ef}}/\gamma_{\mathrm{SF}} \tag{L. 17}$$

$$f_{\mathrm{Ft3d}} = f_{\mathrm{Ft3,ef}}/\gamma_{\mathrm{SF}} \tag{L. 18}$$

(4) The stress distribution according to Formula (8.4) for concrete in compression may be modified for SFRC by applying $\varepsilon_{\mathrm{c2}} = 0{,}0025$ and $\varepsilon_{\mathrm{cu}} = 0{,}006$.



a) Plastic distribution



b) bi-linear distribution

Figure L.2 — Simplified stress distributions for SFRC

L.8.2 Shear

L.8.2.1 General verification procedure

(1) Shear reinforcement may be omitted in regions of the SFRC member where $\tau_{Ed} \leq \tau_{Rd,cF}$ according to L.8.2.2.

(2) If (1) is not met, shear reinforcement shall be designed according to L.8.2.3 to satisfy $\tau_{Ed} \leq \tau_{Rd,sF}$.

(3) Steel fibres shall not be taken into account for members resisting shear in combination with axial tension arising from externally applied loads. For compressive axial forces (e.g. prestressing), L.8.2.2 and L.8.2.3 apply in combination with the specific provisions in 8.4.3 and 8.4.4.

L.8.2.2 Detailed verification for members without shear reinforcement

(1) For SFRC with longitudinal reinforcement bars in the tensile zone, the design value of the shear strength should be taken as:

$$\tau_{Rd,cF} = \eta_{cF} \cdot \tau_{Rd,c} + \eta_F \cdot f_{Ftud} \geq \eta_{cF} \cdot \tau_{Rdc,min} + \eta_F \cdot f_{Ftud} \quad \text{(L. 19)}$$

where

$$\eta_{cF} = \max\{1{,}2 - 0{,}5 f_{Ftuk}; 0{,}4\} \leq 1{,}0 \quad \text{(L. 20)}$$

$\eta_F = 1{,}0$;

$\tau_{Rdc,min}$ is determined in accordance with 8.2.1(4).

$\tau_{Rd,c}$ is defined acc. (8.27), or (8.32) if compressive normal forces shall be accounted for.

(2) When, on the basis of the design shear calculation, no shear reinforcement is required, minimum shear reinforcement may be needed according to L.12.

L.8.2.3 Members with shear reinforcement

(1) For members with SFRC that require design shear reinforcement, and have steel fibres and longitudinal reinforcement bars in the tensile zone, the design value of the shear strength should be taken as:

$$\tau_{\mathrm{Rd,sF}} = (\eta_{\mathrm{sw}} \cdot \rho_{\mathrm{w}} \cdot f_{\mathrm{ywd}} + \eta_{\mathrm{F}} \cdot f_{\mathrm{Ftud}}) \cdot \cot\theta \geq \rho_{\mathrm{w}} \cdot f_{\mathrm{ywd}} \cdot \cot\theta \qquad \text{(L. 21)}$$

where

η_{sw} = 0,75 and η_F = 1,0.

L.8.3 Torsion - Torsional resistance of compact or closed sections

(1) The torsional resistance in the transverse and longitudinal directions of SFRC members should be taken as:

$$\tau_{\mathrm{t,Rd,swF}} = \eta_{\mathrm{sw}} \tau_{\mathrm{t,Rd,sw}} + \eta_F f_{\mathrm{Ftud}} \geq \tau_{\mathrm{t,Rd,sw}} \qquad \text{(L. 22)}$$

$$\tau_{\mathrm{t,Rd,slF}} = \eta_{\mathrm{sw}} \tau_{\mathrm{t,Rd,sl}} + \eta_F f_{\mathrm{Ftud}} \geq \tau_{\mathrm{t,Rd,sl}} \qquad \text{(L. 23)}$$

where

$\tau_{\mathrm{t,Rd,swF}}$ and $\tau_{\mathrm{t,Rd,slF}}$ replace $\tau_{\mathrm{t,Rd,sw}}$ and $\tau_{\mathrm{t,Rd,sl}}$ respectively in Formula (8.81).

η_{sw} = 0,75 and η_F = 1,0.

(2) Where a member is subject to torsion in combination with shear and bending, fibre contribution to design resistances should only be as one of the two approaches:

— the fibre contribution is used to resist torsional action effects only;

— the fibre contribution is used to resist tensile forces arising from shear and bending action effects, with fibre contribution to sectional resistance disregarded in determining axial tension and torsional resistances.

(3) Minimum reinforcement according to L.12.1 shall always be provided for beams.

L.8.4 Punching

L.8.4.1 Punching shear resistance of SFRC slabs without shear reinforcement

(1) The design punching shear stress resistance of SFRC slabs with flexural reinforcement complying with L.12.1 should be calculated as follows:

$$\tau_{\mathrm{Rd,cF}} = \eta_{\mathrm{c}} \cdot \tau_{\mathrm{Rd,c}} + \eta_{\mathrm{F}} \cdot f_{\mathrm{Ftud}} \geq \eta_{\mathrm{c}} \cdot \tau_{\mathrm{Rdc,min}} + f_{\mathrm{Ftud}} \qquad \text{(L. 24)}$$

where

η_c $= \tau_{Rd,c} / \tau_{Ed} \leq 1{,}0$;

$\tau_{Rd,c}$ according to 8.4.3(1);

η_F $= 0{,}4$;

$\tau_{Rdc,min}$ according to 8.2.1(4);

$\tau_{RD,cf}$ replaces $\tau_{Rd,c}$ in 8.4.1(ii) and (iii).

(2) Unless more rigorous analysis is undertaken, Formula (L. 24) shall not be applied to members subject to punching shear in combination with axial tension arising from externally applied loads.

(3) Formula (L.24) may be used in combination with 8.4.3(4) to cater for applied axial compressive action effects.

L.8.4.2 Punching shear resistance of SFRC slabs with shear reinforcement

(1) Where shear reinforcement is required in SFRC slabs with flexural reinforcement complying with L.12.1, Formula (8.104) may be replaced by Formula (L. 25):

$$\tau_{\mathrm{Rd,csF}} = \eta_{\mathrm{c}} \cdot \tau_{\mathrm{Rd,c}} + \eta_{\mathrm{s}} \cdot \rho_{\mathrm{w}} \cdot f_{\mathrm{ywd}} + \eta_{\mathrm{F}} \cdot f_{\mathrm{Ftud}} \geq \rho_{\mathrm{w}} \cdot f_{\mathrm{ywd}} + \eta_{\mathrm{F}} \cdot f_{\mathrm{Ftud}} \tag{L. 25}$$

where

Compliance with 8.4.1 shall be achieved for punching shear where $\tau_{\mathrm{Rd,csF}}$ replaces $\tau_{\mathrm{Rd,cs}}$ in 8.4.1(iv)

η_{c} and η_{sw} are defined in L.8.4.1(1) and in L.8.3(1);

η_{F} = 0,4.

(2) Formula (L.25) may be used in combination with 8.4.3(4) to cater for applied axial compressive action effects.

L.8.5 Design with strut-and-tie models - Ties

(1) Fibres may be used in ties in place of the transverse reinforcement.

(2) Fibres may substitute up to 30 % of the total required main longitudinal reinforcement in ties. A maximum stress of $f_{\mathrm{Fts,ef}}$ and f_{Ftud} may be assumed for serviceability and ultimate limit state respectively in the concrete areas involved.

L.8.6 Partially loaded areas

(1) Fibres may be used to replace the transverse reinforcement required to resist transverse tensile stresses arising from action effects in partially loaded areas. For the design, the idealised tensile post-cracking behaviour (see L.5.5.2 and L.8.1) may be used.

L.9 Serviceability Limit States (SLS) - Crack control

L.9.1 General considerations

(1) The stress and crack width limits in Table 9.1 (NDP) and Table 9.2 (NDP) may be applied to members reinforced with steel fibres.

(2) In determining stresses and crack widths for use of Table 9.1 (NDP) and Table 9.2 (NDP), $c_{\mathrm{f,dur}}$ shall be disregarded from the concrete section in accordance with L.6(3) and (4).

L.9.2 Minimum reinforcement areas to avoid yielding

(1) The reinforcement areas calculated by Formulae (9.2) to (9.4) to avoid yielding may be adjusted for the fibre contribution in accordance with L.8.1. The stress in the tensile reinforcement σ_{s} shall be considered according to L.9.3(2).

L.9.3 Refined control of cracking

(1) The mean crack spacing for members subjected to bending reinforced with steel fibres and longitudinal bars should be calculated as the minimum between these two values:

a) the multi-cracking pattern associated typically to the presence of conventional reinforcement when the spacing of bonded reinforcement in the tension zone is $s \leq 10\phi$:

$$s_{\mathrm{r,m,cal,F}} = 1{,}5 \cdot c + \frac{k_{\mathrm{fl}} \cdot k_{\mathrm{b}}}{7{,}2} \cdot \frac{\phi}{\rho_{\mathrm{p,eff}}} \cdot (1 - \alpha_{\mathrm{f}}) \tag{L. 26}$$

where:

$$\alpha_{\mathrm{f}} = \frac{0{,}37 \cdot \kappa_0 \cdot \kappa_{\mathrm{G}} \cdot f_{\mathrm{R1k}}}{f_{\mathrm{ctm}}} \leq 1{,}0 \tag{L. 27}$$

and the remaining parameters are as defined in 9.2.3.

b) the single crack or the crack distance typically associated to the spacing of bonded reinforcement in the tension zone when s > 10ϕ:

$$s_{\mathrm{r,m,cal,F}} = h - x \tag{L. 28}$$

Note that for the conversion into maximum crack width ($s_{\mathrm{r,max,cal,F}}$), k_{w} equal to 1,7 and 1,3 applies for cases a) and b), respectively.

NOTE 1 If the reinforcement is not homogeneously distributed in the cross-section, more than one $s_{\mathrm{r,m,cal,F}}$ can be adopted in the same cross-section.

NOTE 2 For cross sections or parts of a cross section without longitudinal reinforcement subjected to predominately uniaxial tension, $s_{\mathrm{r,m,cal,F}}$ is the distance between adjacent webs.

(2) The stress in the tensile reinforcement σ_{s} in Formula (9.11), should be calculated using the provisions of L.8.1 with partial material factors for serviceability limit state.

L.10 Fatigue

(1) For fatigue verification of SFRC members, the contribution of fibres should be neglected unless the fibre effect can be demonstrated from sufficient and specific testing.

L.11 Detailing of reinforcement and post-tensioning tendons

L.11.1 General

(1) Steel fibres shall not be used to replace reinforcement across a construction joint.

(2) The residual tensile strength of SFRC shall be disregarded at construction joints.

L.11.2 Spacing of bars

(1) A clear bar distance ≥ k_{F} times the fibre length should be used.

NOTE 1 The value of k_{F} is 2,0 unless the National Annex gives a different value.

NOTE 2 Smaller distances are possible if sufficient practical experience is available or special measures are foreseen.

L.12 Detailing of members and particular rules

L.12.1 Minimum reinforcement rules

(1) In members reinforced with steel fibres with or without axial force, minimum reinforcement shall be provided so that:

$$M_{\mathrm{R,min}}(N_{\mathrm{Ed}}) \geq M_{\mathrm{cr}}(N_{\mathrm{Ed}}) \tag{L. 29}$$

where

$M_{\mathrm{R,min}}$ is the bending strength of the section with $A_{\mathrm{s,min}}$ in presence of the co-existing axial force N_{Ed}, the effects of the fibres included by the effective residual tensile strength $f_{\mathrm{Ftu,ef}}$ and the stress distributions in L.8.1 used. The reduction in $A_{s,min}$ due to the fibre contribution should fulfil the limits described in the subsequent clauses.

(2) In members subjected to axial tension, $A_{s,min}$, shall meet the following requirement:

$$N_{R,min} \geq N_{cr} \tag{L. 30}$$

Similarly as above the effects of the fibres are included by the residual tensile strength, and the reduction $A_{s,min}$ due to the fibre contribution should fulfil limits described in the subsequent clauses.

(3) The minimum shear reinforcement ratio $\rho_{Fw,min}$ for members reinforced with steel fibres requiring shear or torsion reinforcement may be taken as:

$$\rho_{Fw,min} = \rho_{w,min} - f_{Ftu,ef}/f_{yk} \geq 0 \tag{L. 31}$$

where

$$f_{Ftu,ef} \geq 0,08 \cdot \sqrt{f_{ck}} \tag{L. 32}$$

(4) The minimum torsion reinforcement ratio $\rho_{Fw,min}$ for members reinforced with steel fibres requiring longitudinal and transverse reinforcement may be taken as follows for both types of reinforcement:

$$\rho_{Fw,min} = \rho_{w,min} - f_{Ftu,ef}/f_{yk} \geq 0.3 \frac{f_{ctm}}{f_{yk}} \tag{L. 33}$$

L.12.2 Beams

L.12.2.1 Longitudinal reinforcement

(1) The minimum longitudinal reinforcement in beams should not be replaced by steel fibres.

L.12.2.2 Shear and torsion reinforcement

(1) The shear and torsion reinforcement in beams may be fully replaced by steel fibres if the provisions in L.12.1 are fulfilled.

L.12.3 Slabs

L.12.3.1 General

(1) The longitudinal reinforcement in slabs may be partly replaced by steel fibres subject to the provisions in L.12.1.

(2) The replacement of minimum longitudinal tensile reinforcement should be limited to a fraction k_{AS} of $A_{s,min}$.

NOTE The value of k_{AS} is 0,50 unless the National Annex gives a different value for use in a Country.

(3) The minimum secondary tensile reinforcement in one-way slabs may be fully replaced by steel fibres.

L.12.3.2 Shear reinforcement

(1) The shear reinforcement in slabs may be fully replaced by steel fibres if Formula (L. 34) is fulfilled:

$$f_{Ftu,ef}/f_{yk} \geq \rho_{w,min} \tag{L. 34}$$

where $f_{Ftu,ef}$ according to Formula (L. 3).

(2) The minimum depth given in 12.4.2(2) does not apply for SFRC slabs with fibres as the only shear reinforcement.

L.12.4 Walls and deep beams

(1) Vertical and horizontal minimum reinforcement in walls $A_{s,min,v}$ and $A_{s,min,h}$ may be fully replaced by the steel fibres, taking into account the post cracking tensile strength $f_{Ftu,ef}$ according to Formula (L. 3).

L.12.5 Tying systems for robustness of buildings

(1) Fibre reinforcement shall not be considered in the calculation of tie reinforcement.

L.13 Additional rules for precast concrete elements and structures

L.13.1 Concrete - Strength of SFRC

(1) Where quality control measures in accordance with the relevant execution standard are applied, the characteristic values $f_{R,1k}$ and $f_{R,3k}$ may be determined using $\kappa_{k,max}$ = 1,0 if κ_G = 1,0. In this case residual strength and ductility classes according to Table L.2 may be neglected.

L.13.2 Connections and supports

(1) 8.5.5 or 8.6 should be used to determine the capacity of connection in compression without considering steel fibres of SFRC.

L.14 Lightly reinforced SFRC structures

L.14.1 General

(1) This clause provides additional rules for steel fibre reinforced structural members where the minimum reinforcement provided is less than the requirements in L.12.

To avoid brittle failure, Clause L.14 may only be applied for statically indeterminate structures such as: elastically supported structures, anchored underwater concrete slabs, piled slabs, shell-type components and monolithic, prefabricated containers, segmental lining.

(2) Linear elastic, plastic and non-linear analysis can be used.

(3) For members constructed with joints to avoid uncontrolled cracking, it should be ensured that brittle failure of these members does not lead to collapse of the structure.

(4) Members using SFRC and designed in accordance with this clause do not preclude the provision of steel reinforcement needed to satisfy serviceability, nor reinforcement in certain parts of the members. Any reinforcement may be taken into account for the local verification of ultimate limit states as well as for the checks of the serviceability limit states.

(5) Members subject to imposed deformations should comply with provisions of L.12.

L.14.2 Concrete

(1) When tensile stresses are considered for the design resistance of steel fibre reinforced concrete members, linear elastic analysis may be applied with the post cracking tensile design strength using Formula (L. 5) in L.5.5.1 as stress limit.

(2) For design of steel fibre reinforced concrete members according to L.14, tensile stresses according to the stress strain diagrams in L.8.1 may be considered in the design.

L.14.3 Ultimate limit states (ULS) - Shear resistance of lightly reinforced SFRC members without longitudinal reinforcement

(1) Unless more rigorous analysis is undertaken, the design value of shear strength of lightly reinforced SFRC members may be taken as:

$$\tau_{\mathrm{Rd,cF}} = f_{\mathrm{Ftud}} \tag{L. 35}$$

NOTE Punching shear for concrete structures with reinforcement quantities less than the minimum is not covered by this standard, and can be assessed by rigorous analysis.

L.14.4 Serviceability limit states (SLS)

(1) For SFRC members without longitudinal bars, and structural hardening behaviour under bending with or without axial compression, the mean crack spacing can be determined as defined in Formula (L. 28).

(2) ($\varepsilon_{\mathrm{sm}}$ - $\varepsilon_{\mathrm{cm}}$) in Formula (9.11) should be replaced with the strain in the SFRC member calculated using the provisions of L.8.1 with partial material factors for the serviceability limit state.

L.14.5 Detailing of members and particular rules

L.14.5.1 SFRC Column footing on rock

(1) For design of SFRC, f_{Ftsd} and f_{Ftud} may be taken into account in accordance with L.5.5.1(2).

L.14.5.2 Foundations directly on ground

(1) For continuously ground supported rafts and foundation beams a minimum residual strength and ductility class of 1b according to Table L.2 should be applied.

L.14.5.3 Foundations on piles

(1) For rafts and slabs on piles a minimum residual strength and ductility class of 2c according to Table L.2 or Formula (L. 29), should be applied.

L.14.5.4 Tunnel lining segments

(1) For tunnel lining segments without additional longitudinal reinforcement, a minimum performance according to class 4c should replace Formula (L. 29).

L.15 Requirements for Materials: SFRC

(1) The test report of a SFRC shall contain the load-CMOD curve or the load-deflection curve and the limit of proportionality LOP according to EN 14651.

(2) If a new steel fibre concrete mix is used, an initial test shall be carried out by the producer in order to obtain a mixture design that meets the specified characteristic residual flexural strength according to L.5.1(1).

(3) The initial assessment shall demonstrate, that the documented procedures guarantee a uniform fibre distribution in the batch.

(4) The test series shall comprise of at least six specimens of dimensions 150 mm/150 mm/550-700 mm according to EN 14651.

(5) Determination of $f_{\mathrm{R,1k}}$ and $f_{\mathrm{R,3k}}$ shall be based on a log-normal distribution according to EN 1990 (5% quantile, 75% confidence level). Unless explicitly agreed otherwise, the coefficient of variation shall be assumed unknown. The value of $\kappa_{\mathrm{k,max}}$ in EN 206 shall be taken as 0,6. In the case the COV on the material

extracted by the structure should be also checked, a higher value equal to 0,7 could be used if the COV ≤ 0,15.

(6) Linear interpolation of the residual strength $f_{R,1k}$ and $f_{R,3k}$ between two quantified fibre dosages with no more than 20 kg/m^3 difference in dosage is admitted for concretes with the same concrete mix design, fibre type and consistency. The amount of liquefying admixtures can be adjusted to achieve the required consistency.

In identity tests, an increase of one class of only $f_{R,1k}$ or $f_{R,3k}$ may be accepted if both $f_{R,ik}$-values are higher than those of the specified performance classes.

Annex M
(normative)

Lightweight aggregate concrete structures

M.1 Use of this annex

(1) This Normative Annex contains additional provisions for all concretes made with natural or artificial mineral lightweight aggregates with closed structure. The provisions of this Eurocode apply for concrete members with lightweight aggregates unless modified in this Annex M.

M.2 Scope and field of application

(1) This Normative Annex applies to all concretes made with natural or artificial mineral lightweight aggregates with closed structure unless reliable experience indicates that provisions different from those given can be adopted safely.

(2) This Normative Annex does not apply to aerated concrete either autoclaved or normally cured nor to lightweight aggregate concrete with an open structure.

M.3 General

(1) Lightweight aggregate concrete should be classified according to EN 206 by:

— its oven-dry density as shown in Table M.1. Alternatively, the oven-dry density may be specified as a target value;

— its compressive strength f_{ck} = 12 MPa to 80 MPa according to LC12/13 to LC80/88.

(2) All clauses of this standard are generally applicable unless substituted by special provisions given in Table M.2.

Table M.1 — Density classes and corresponding design densities of LWAC according to EN 206

	Density class						Analytical formula[a]
	D1,0	**D1,2**	**D1,4**	**D1,6**	**D1,8**	**D2,0**	
Oven-dry density (kg/m^3)	801 −1 000	1 001 −1 200	1 201 −1 400	1 401 −1 600	1 601 −1 800	1 801 −2 000	–
Coefficient $\eta_{lw,fc}$	0,45	0,55	0,64	0,73	0,82	0,91	$\eta_{lw,fc} = \frac{\rho_c}{2\,200}$
Coefficient $\eta_{lw,fct}$	0,67	0,73	0,78	0,84	0,89	0,95	$\eta_{lw,fct} = 0{,}40 + 0{,}60\frac{\rho_c}{2\,200}$
Coefficient $\eta_{lw,Ec}$	0,21	0,30	0,40	0,53	0,67	0,83	$\eta_{lw,Ec} = \left(\frac{\rho_c}{2\,200}\right)^2$

[a] The upper value of the density ρ_c should be used in Formulae $\eta_{lw,fc}$, $\eta_{lw,fct}$ and $\eta_{lw,Ec}$.

Table M.2 — Special provisions for LWAC

Reference to original clause	Values and terms to be modified for lightweight aggregate concrete	Provisions and formulae for lightweight aggregate concrete
5.1.3(3)	Maximum compressive strength	$f_{ck} \leq 80$ MPa
Table 5.1	Mean value of concrete cylinder compressive strength f_{cm}	$f_{cm} = 17$ MPa for $f_{ck} = 12$ MPa; $f_{cm} = 22$ MPa for $f_{ck} = 16$ MPa; values given in Table 5.1 for $f_{ck} \geq 20$ MPa.
Table 5.1	Concrete tensile strength f_{ctm}, $f_{ctk,0,05}$, $f_{ctk,0,95}$	The tensile strength may be obtained by multiplying the values given in Table 5.1 by coefficient $\eta_{lw,fct}$ given in Table M.1.
5.1.4	Modulus of elasticity E_{cm}	An estimate of the mean values of the secant modulus E_{cm} may be obtained by multiplying the values for normal density concrete according to 5.1.4 by coefficient $\eta_{lw,Ec}$ given in Table M.1.
5.1.6(6)	Linear coefficient of thermal expansion	Unless more accurate information is available, the linear coefficient of thermal expansion may be taken equal to $8 \cdot 10^{-6}$ °C^{-1}
5.1.5(2), Table 5.2	Creep coefficient	The creep coefficient φ may be assumed equal to the value of normal density concrete multiplied by: — $1{,}3 \cdot \eta_{lw,Ec}$ for $f_{ck} \leq 16$ MPa; — $\eta_{lw,Ec}$ for $f_{ck} \geq 20$ MPa.
5.1.5(4), Table 5.3	Nominal total shrinkage values $\varepsilon_{cs,50y}$	Shrinkage values may be obtained by multiplying the value for normal density concrete in Table 5.3 by: — 1,5 for $f_{ck} \leq 16$ MPa; — 1,2 for $f_{ck} \geq 20$ MPa.
5.1.6(1)	Design value of concrete compressive strength f_{cd}	The influence of the increased brittleness of lightweight concrete on the design strength f_{cd} shall be accounted by replacing Formula (5.4) by : $\eta_{cc} = \left(\eta_{lw,fc}\dfrac{f_{ck,ref}}{f_{ck}}\right)^{1/3} \leq 1$ where coefficient $\eta_{lw,fc}$ is given in Table M.1
Table 6.3 (NDP), Table 6.4 (NDP)	Minimum clear cover $c_{min,dur}$ due to durability requirement	The values of $c_{min,dur}$ given in Tables 6.3 (NDP) and 6.4 (NDP) should be increased by 5 mm.
Table 6.5 (NDP)	Minimum clear cover $c_{min,b}$ for bond	The value of $c_{min,b}$ given in Table 6.5 (NDP) should be increased by 5 mm.
8.1.4(2)	Confined concrete	Formulae (8.9) and (8.10) shall be replaced by: $\Delta f_{cd} = \sigma_{c2d}$ $\Delta f_{cd} = 0{,}88\sigma_{c2d}^{3/4} \cdot f_{cd}^{1/4}$

Reference to original clause	Values and terms to be modified for lightweight aggregate concrete	Provisions and formulae for lightweight aggregate concrete
8.1.4(2), 8.2.1(4), 8.2.2(2) 8.4.3(1)	Parameter d_{dg} affecting the resistance of confined concrete, the shear resistance of members not requiring design shear reinforcement and the punching shear resistance	The parameter d_{dg} taking account of concrete type and its aggregate properties shall be assumed as $d_{dg} = 16$ mm.
8.6(2)	Partially loaded areas	Formula (8.126) shall be replaced by: $\sigma_{Rdu} = f_{cd} \cdot [A_{c1}/A_{c0}]^{\frac{\eta_{lw,fc}}{2}} \leq \eta_{lw,fc} \cdot \nu_{part} \cdot f_{cd}$
9.3.2(1), Table 9.3	Deflection control	The basic ratios of span/effective depth for reinforced concrete members given in Table 9.3, should be multipied by $\eta_{lw,Ec}{}^{0,15}$.
11.3(4) and 11.3(5)	Permissible mandrel diameter for bent bars	The required minimum mandrel diameter to avoid concrete failures according to 11.3(4) and 11.3(5) shall be increased by factor $1/\eta_{lw,fct}$ (coefficient $\eta_{lw,fct}$ according to Table M.1)
11.4.2, 11.5.2	Anchorages and laps of bars	The design anchorage lengths ℓ_{bd} according to 11.4.2 and the design lengths of lap splices ℓ_{sd} according to 11.5.2 shall be increased by factor $1/\eta_{lw,fct}$ (coefficient $\eta_{lw,fct}$ according to Table M.1)
11,4.7	Anchorage of headed bars	In Formula (11.8), ν_{part} should be replaced by $\nu_{part}\,\eta_{lw,fc}$.
B.5(1)	Creep coefficient	The creep coefficient $\varphi(t,t_0)$ may be assumed equal to the value of normal density concrete according to Formula (B.5) multiplied by: — $1,3 \cdot \eta_{lw,Ec}$ for $f_{ck} \leq 16$ MPa; — $\eta_{lw,Ec}$ for $f_{ck} \geq 20$ MPa
B.6(1)	Shrinkage strain	The total shrinkage values may be obtained by multiplying the value for normal density concrete in Formula (B.23) by: — 1,5 for $f_{ck} \leq 16$ MPa; — 1,2 for $f_{ck} \geq 20$ MPa.

Annex N
(informative)

Recycled aggregates concrete structures

N.1 Use of this annex

(1) This Informative Annex provides supplementary rules for structures made of recycled aggregates concrete. The provisions of this Eurocode apply for concrete members with recycled aggregates concrete unless modified in this Annex N.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this Informative Annex, it can be used.

N.2 Scope and field of application

(1) This Informative Annex applies to structures made of recycled aggregates concrete.

N.3 General

(1) Concrete with recycled aggregates may be used where the use of recycled aggregates will not impair load-bearing capacity and durability performance of the structures as well as service performance like appearance or wear, or represents a risk of polluting ground water or air. Recycled aggregates may be used in normal concrete production without any particular consent if done in accordance with the provisions of EN 206.

(2) If the percentage of recycled aggregates exceeds the limits given in EN 206 and if the properties listed in 5.1.2(2) for concrete with recycled aggregates are relevant to the design in accordance with this Eurocode, these properties should be determined by testing in accordance with EN 206. The exposure resistance class should be determined based on durability performance testing.

(3) In order to facilitate recycling, the following may be assumed for type A recycled aggregate as defined in EN 206. For type B recycled aggregate, the substitution rate limits provided below should be decreased by 50 %.

a) For reinforced concrete:

- when the substitution rate of recycled aggregates (quantity of fine and coarse recycled aggregates/total quantity of aggregates) $\alpha_{RA} \leq 0{,}20$ there is no change in the mechanical properties;
- when $0{,}20 < \alpha_{RA} \leq 0{,}40$ the values of properties in Table N.1 should be used or values should be determined by testing;
- when $\alpha_{RA} > 0{,}40$ the properties in Table N.1 should be determined by testing using an identified batch of aggregates.

b) For prestressed concrete:

- when $0 < \alpha_{RA} \leq 0{,}20$ the values of properties in Table N.1 should be used or values should be determined by testing;
- when $\alpha_{RA} > 0{,}20$ the properties in Table N.1 should be determined by testing using an identified batch of aggregate.

Table N.1 — Special provisions for recycled aggregates concrete

Reference to original clause	Values and terms to be modified for recycled aggregates concrete	Provisions and formulae for recycled aggregates concrete [a)]
5.1.6(5)	Density	$\rho_{\mathrm{c}} = 2500 - 220\alpha_{\mathrm{RA}}$
5.1.3(3)	Maximum compressive strength	$f_{\mathrm{ck}} \leq 50$ MPa
Table 5.1	Concrete tensile strength $f_{\mathrm{ctm}}, f_{\mathrm{ctk,0,05}}, f_{\mathrm{ctk,0,95}}$	Determine by testing if relevant. Alternatively may be used: $f_{\mathrm{ctm}} = 0{,}3 \cdot f_{\mathrm{ck}}^{2/3}$
5.1.4	Modulus of elasticity E_{cm}	Determine by testing if relevant. Alternatively may be used the following: $E_{\mathrm{cm}} = \eta_{\mathrm{ERA}} \cdot f_{\mathrm{cm}}^{1/3}$ where $\eta_{\mathrm{ERA}} = k_{\mathrm{E}} \cdot (1 - 0{,}25 \cdot \alpha_{\mathrm{RA}})$
5.1.5(2), Table 5.2	Creep coefficient	Determine by testing if relevant. Alternatively the creep coefficient for basic and drying creep should be multiplied by a factor $\eta_{\mathrm{cRA}} = 1 + 0{,}6 \cdot \alpha_{\mathrm{RA}}$.
5.1.5(4), Table 5.3	Shrinkage strain	Determine by testing if relevant. Alternatively the basic and drying shrinkages should be multiplied by a factor $\eta_{\mathrm{shRA}} = 1 + 0{,}8 \cdot \alpha_{\mathrm{RA}}$.
Table 6.3 (NDP), Table 6.4 (NDP)	Minimum clear cover $c_{\mathrm{min,dur}}$ due to durability requirement	Determine ERC by testing if relevant. For concrete including recycled aggregate, the same minimum cover depth for durability $c_{\mathrm{min,dur}}$ applies provided the material pertains the same exposure resistance class (ERC) as concrete including natural aggregate only. Adaptation of the limiting values and/or performance thresholds ensuring compliance with ERCs for concrete including recycled aggregate are given in EN 206 complemented by the provisions valid in the place of use. If the ERC is not determined, the values of $c_{\mathrm{min,dur}}$ given in 6.5.2.2 should be increased by +5 mm in case of exposure classes XC2, XC3 and XC4, and by +10 mm in case of all XD/XS-exposure classes.
5.1.6(3)	Stress-strain relationship	Multiply in 5.1.6(3), Formulae (5.9) and (5.10) the values $\varepsilon_{\mathrm{c1}}$ and $\varepsilon_{\mathrm{cu1}}$ respectively, by $\eta_{\varepsilon\mathrm{c}} = 1 + 0{,}33 \cdot \alpha_{\mathrm{RA}}$: $\varepsilon_{\mathrm{c1}}(‰) = \eta_{\varepsilon\mathrm{c}} \left(0{,}7 f_{\mathrm{cm}}^{\frac{1}{3}}\right) \leq 2{,}8\ ‰$ $\varepsilon_{\mathrm{cu1}}(‰) = \eta_{\varepsilon\mathrm{c}} \left(2{,}8 + 14\left(1 - \frac{f_{\mathrm{cm}}}{108}\right)^4\right) \leq 3{,}5\ ‰$
8.2.1(4) 8.2.2(2)	Shear resistance of members not requiring design shear reinforcement	Shear strength without shear reinforcement: Multiply in 8.2.1(4), Formula (8.20) and in 8.2.2(2), Formula (8.27) by $\eta_{\tau} = 1 - 0{,}2 \cdot \alpha_{\mathrm{RA}}$.

Reference to original clause	Values and terms to be modified for recycled aggregates concrete	Provisions and formulae for recycled aggregates concrete a)
Table 9.3	Simplified deflection control by span/depth-ratio	Span/depth ratios should be multiplied by the coefficient $1/(1 + 0.12 \cdot \alpha_{RA})$.
9.3.4(3)	General method for deflection calculations	$\zeta = 1 - \beta_{tRA} \left(\frac{\sigma_{sr}}{\sigma_{s}} \right)^{2} \geq 0$ with $\beta_{tRA} = 1{,}0$ for a single short term loading and $\beta_{tRA} = 0{,}25$ for sustained loads or many cycles of repeated loading.
B.5(1)	Creep coefficient	Determine by testing if relevant. Alternatively the creep coefficient for basic and drying creep should be multiplied by a factor $\eta_{ccRA} = 1 + 0{,}6 \cdot \alpha_{RA}$ (high dispersion of the results).
B.6(1)	Shrinkage strain	Determine by testing if relevant. Alternatively the basic and drying shrinkages should be multiplied by a factor $\eta_{shRA} = 1 + 0{,}8 \cdot \alpha_{RA}$ (high dispersion of the results).

a In Formulae is α_{RA} the substitution rate of recycled concrete aggregates (varying from 0 to 1):

$$\alpha_{RA} = \frac{\text{quantity of fine and coarse recycled aggregates}}{\text{total quantity of aggregates}}$$

Annex O
(informative)

Simplified approaches for second order effects

O.1 Use of this Informative Annex

(1) Annex O provides complementary guidance to 7.4 for second order structural analysis of members and systems.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this Informative Annex, it can be used.

O.2 Scope and field of application

(1) Annex O provides conservative simplified methods for design against second order effects, including:

— a formulation for the determination of the buckling load for reasonably symmetrical building structures (see O.3) and isolated members (see O.4);

— a slenderness criterion for isolated members to below which second order effects may be neglected (see O.6);

— a conservative simplified method to evaluate the second order effects of isolated members based on a nominal curvature (see O.7); and

— simplified criteria to determine second order effects by the second order elastic method or by a moment magnification factor (see O.8).

O.3 Critical load of building structures

(1) For multilevel, reasonably symmetrical building structures having distinct bracing members (typically shear walls) with reasonably constant bending stiffness along the height, and with approximately equal loading on each level and negligible global torsional action, the global buckling load, F_{VB}, may be taken as:

When shear deformations may be neglected:

$$F_{\mathrm{VB}} = F_{\mathrm{VBB}} \tag{O.1}$$

When shear deformations may not be neglected:

$$F_{\mathrm{VB}} = \left(\frac{1}{F_{\mathrm{VBB}}} + \frac{1}{F_{\mathrm{VBS}}}\right)^{-1} \tag{O.2}$$

where

F_{VBB} is the buckling load of a cantilever, (without shear deformation) with a rotational restraint at the base, that may be taken as:

$$F_{\mathrm{VBB}} = \frac{7{,}8 n_{\mathrm{s}}}{n_{\mathrm{s}} + 1{,}6} \sum \left(\frac{1}{1 + 3{,}9 f_{\mathrm{r_i}}} \frac{E I_{\mathrm{i}}}{L^2}\right) \tag{O.3}$$

F_{VBS} is the buckling load due to localised lateral storey deformations to be included when shear deformation is significant. Considering the solid portion of shear walls it may be taken as the cracked shear stiffness:

$$F_{VBS} = k_c \cdot G_{cd} \cdot A_c \tag{O.4}$$

where

n_s is the number of storeys;

L is the total height of the building above the base (foundation or top of a rigid basement);

EI $= (k_c E_{cd} I_c)/(1+\varphi_{eff,s})$ is the sum of the bending stiffnesses of all bracing members;

I_c is the second moment of area of the gross concrete cross-section;

E_{cd} is the design value of the modulus of elasticity, see 7.4.3.3(3);

$\varphi_{eff,s}$ is the effective creep coefficient for global second order effects, see 7.4.2(2);

G_{cd} $\approx 0{,}4E_{cd}$ is the design value of the elastic shear modulus;

k_c reflects the extent of cracking and the effect of non-linear material properties and may be taken as:

$k_c = 0{,}4$ in the general case;

$k_c = 0{,}8$ if it can be shown that the tensile stress in the bracing members under the effect of the characteristic combination of actions at the critical sections is less than f_{ctd}.

f_r is the sum of rotational restraint stiffnesses at the base of the bracing members;

$$f_r = \frac{\theta}{M} \cdot \frac{EI}{L} \tag{O.5}$$

θ is the rotation for bending moment M.

(2) In cases where the global buckling load F_{VB} is not well defined, the Formula (O.6) may be used:

$$F_{VB} = \frac{F_{H,0Ed}}{F_{H,1Ed}} F_{V,Ed} \tag{O.6}$$

where

$F_{H,1Ed}$ is a fictitious horizontal force, giving the same bending moments as the vertical load $F_{V,Ed}$ acting on the deformed structure, with deformation caused by $F_{H,0Ed}$ and calculated with an effective stiffness according to 7.4.3.2(1);

$F_{H,0Ed}$ is the first order horizontal force due to wind, imperfections, etc.

O.4 Critical load of isolated members

(1) For use in simplified second order analyses, the elastic buckling load (N_B) and effective length l_0 of a compression member are defined by:

$$N_B = \frac{\pi^2 EI}{l_0^2} \tag{O.7}$$

where

EI is a representative effective stiffness, in the considered plane of bending;

l_0 is the effective length that may be determined using Formulae (O.9) or (O.10).

O.5 Slenderness ratio and effective length of isolated members

(1) The slenderness ratio is defined as follows:

$$\lambda = l_0/i \tag{O.8}$$

where

$i = \sqrt{\dfrac{I_c}{A_c}}$ is the radius of gyration of the uncracked concrete section.

(2) In the definition of the effective length l_0, the stiffness of restraining members should include the effect of cracking unless they can be shown to be uncracked in ULS.

(3) For compression members in regular frames the effective length l_0 may be determined in the following way:

For braced members:

$$l_0 = 0{,}5l\sqrt{\left(1+\frac{f_{r1}}{0{,}45+f_{r1}}\right)\cdot\left(1+\frac{f_{r2}}{0{,}45+f_{r2}}\right)} \tag{O.9}$$

For unbraced members:

$$l_0 = l\frac{\sqrt{(1+2{,}4f_{r1}+2{,}4f_{r2})\cdot(1+2{,}4f_{r1})\cdot(1+2{,}4f_{r2})}}{1+1{,}2f_{r1}+1{,}2f_{r2}} \tag{O.10}$$

where

f_{r1}, f_{r2} are the relative flexibilities of rotational restraints at ends 1 and 2 respectively; since fully rigid restraint is rare in practice, a minimum value of 0,1 should be taken for f_{r1} and f_{r2}.

$$f_r = \frac{\theta}{M}\cdot\frac{EI}{l} \tag{O.11}$$

θ is the rotation of restraining members for bending moment M, see also (2);

EI is the bending stiffness of the compression member, see also (4);

l is the clear height of the compression member between end restraints.

NOTE $f_r = 0$ is the theoretical limit for rigid rotational restraint and $f_r = \infty$ represents the limit for no rotational restraint at all.

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

O.6 Slenderness criteria for isolated members

(1) As an alternative to 7.4.1(3) second order effects may be ignored if the slenderness λ is smaller than λ_{lim} (see Formula (O.12)):

$$\lambda_{lim} = 20\frac{A\ B\ C}{\sqrt{n}} \tag{O.12}$$

where

A $= 1/(1 + 0{,}2\varphi_{\mathrm{eff,b}})$ (if φ_{eff} 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 $= 1{,}7 - r_{\mathrm{m}}$ (if r_{m} is not known, $C = 0{,}7$ may be used);

$\varphi_{\mathrm{eff,b}}$ is the effective creep ratio for local effects, see 7.4.2(2);

ω $= A_{\mathrm{s}} f_{\mathrm{yd}}/(A_{\mathrm{c}} f_{\mathrm{cd}})$ is the mechanical reinforcement ratio;

A_{s} is the total area of longitudinal reinforcement;

n $= |N_{\mathrm{Ed}}|/(A_{\mathrm{c}} f_{\mathrm{cd}})$ is the non-dimensional axial force;

r_{m} $= 1{,}0$ for unbraced members and for braced members in which the first order moments arise only from or predominantly from imperfections or transverse loading, otherwise $= M_{01}/M_{02}$; moment ratio (see Figure O.1);

(2) In cases with biaxial bending, the slenderness criterion may be checked separately in each principal plane of bending.

O.7 Simplified analysis of isolated members based on nominal curvature

O.7.1 General

(1) This method is primarily suitable for isolated members with constant axial force and a well defined effective length l_0 (see O.5). 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 Figure 7.4).

O.7.2 Design moments

(1) The design moment should be taken as:

$$M_{\mathrm{Ed}} = \max\{M_{0\mathrm{Ed}} + M_2;\ M_{02};|M_{01} - 0{,}5M_2 - 2|N_{\mathrm{Ed}}| \cdot e_{\mathrm{i}}|\} \tag{O.13}$$

where

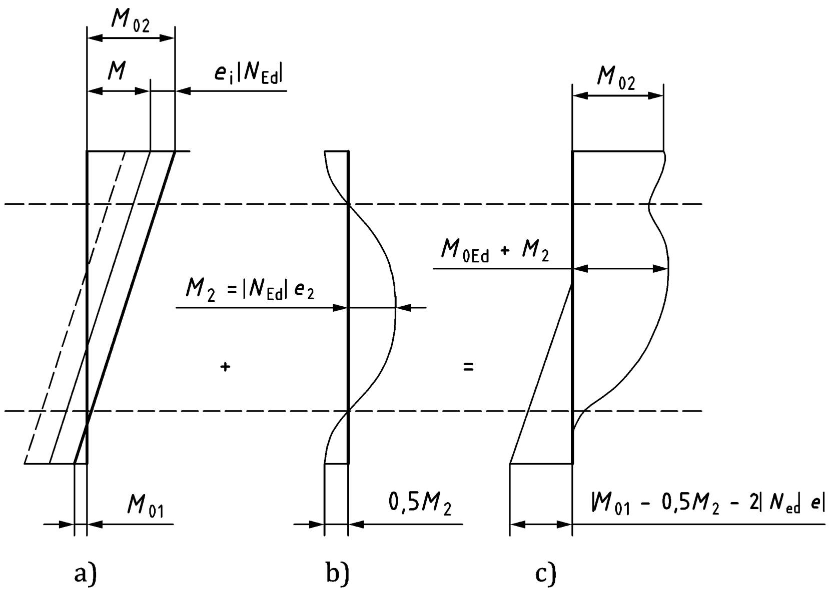
$M_{0\mathrm{Ed}}$ is the 1st order moment, including the effect of imperfections, see (2);

M_2 is the nominal 2nd order moment, see (3).

e_{i} should be determined according to 7.2.1.2(5)

M_{01}, M_{02} are the first order moments at both supports 1 and 2, see Figure O.1.

The maximum value of M_{Ed} depends on the distributions of $M_{0\mathrm{Ed}}$ and M_2 over the member length; the latter may be taken as parabolic or sinusoidal over the effective length.



a) First order moments b) Additional second order moments c) Envelope of design moments

Figure O.1 — First order moments, additional second order moments and moment envelope in a braced column

(2) For unbraced single compression members and for compression members in braced frame systems, with significant transverse loads on the member M_{0Ed} is equal to the absolute largest value of the first order moment over the height of the member.

For compression members in braced frame systems, without significant transverse loads on the member:

$$M_{0Ed} = C_m \cdot M_{02} \tag{O.14}$$

where

$$C_m = 0{,}6 + 0{,}4 r_m \geq 0{,}4 \tag{O.15}$$

$$r_m = 1 \text{ in case } |M_{02}| < 0{,}05\ N_{Ed} \cdot h, \text{ otherwise } r_m = M_{01}/M_{02} \tag{O.16}$$

$C_m M_{02}$ is an equivalent moment assumed to be constant over the length and therefore $c_{1/r} = 8$ (corresponding to constant moment).

(3) The nominal second order moment M_2 at the critical section should be taken as:

$$M_2 = |N_{Ed}| \cdot e_2 \tag{O.17}$$

where

$$e_2 = \frac{l_0^2}{c_{1/r}} \frac{1}{r} \tag{O.18}$$

$c_{1/r}$ is a total curvature distribution factor;

$1/r$ is the member's equilibrium curvature including second order effects ($1/r = (\varepsilon_c + \varepsilon_s)/d$). In the absence of more accurate design, $1/r$ may be taken equal to the nominal curvature in O.7.3.

(4) The $c_{1/\mathrm{r}}$ factor depends on the total curvature distribution along the member (due to first plus second order moments and the non-linear moment-curvature relationship of the cross-sections). For single unbraced members with constant cross-section, $c_{1/\mathrm{r}} = 10$ may be used, if more accurate values are not justified. For braced members, $c_{1/\mathrm{r}} = 8$ may be adopted.

O.7.3 Nominal curvature

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

$$\frac{1}{r} = k_\mathrm{r} \cdot k_\varphi \cdot \frac{1}{r_0} \tag{O.19}$$

where

k_r is a coefficient depending on the axial force, see O.7.3(3);

k_φ is a coefficient accounting for creep, see O.7.3(4);

$1/r_0$ $= 2\,\varepsilon_\mathrm{yd}/(d - d')$;

ε_yd $= f_\mathrm{yd}/E_\mathrm{s}$;

d is the effective depth; see also O.7.3(2);

d' is the cover measured to the centroid of the compression reinforcement.

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

$$d - d' = 2i_\mathrm{s} \text{ for general sections and } (h/2) + i_\mathrm{s} \text{ for rectangular sections;} \tag{O.20}$$

where

i_s is the radius of gyration of the total group of longitudinal reinforcement.

(3) k_r in Formula (O.19) may be taken as 1,0 as a first approximation. If a more refined calculation is needed, k_r may be taken as:

$$k_\mathrm{r} = \frac{n_\mathrm{u} - n}{n_\mathrm{u} - n_\mathrm{bal}} \leq 1 \tag{O.21}$$

where

n $= |N_\mathrm{Ed}|/(A_\mathrm{c}\,f_\mathrm{cd})$, is the non dimensional axial force;

n_u $= 1+\omega$;

n_bal is the value of n at maximum moment resistance; the value $n_\mathrm{bal} = 0{,}4$ may be used;

ω $= A_\mathrm{s} f_\mathrm{yd}/(A_\mathrm{c} f_\mathrm{cd})$;

A_s is the total area of the longitudinal reinforcement;

(4) k_φ in Formula (O.19) should be taken as:

$$k_\varphi = 1 + \beta_{f_\mathrm{ck}} \varphi_\mathrm{eff,b} \geq 1 \tag{O.22}$$

where

$\varphi_\mathrm{eff,b}$ is the effective creep ratio for local effects, see 7.4.2(2);

$\beta_{f_{ck}} = 0{,}35 + f_{ck}/200 - \lambda/150.$

O.8 Second order elastic method

O.8.1 General

(1) The determination of the second order effects may be carried out using either a second order (geometric non-linear) elastic analysis or a theoretical solution based on the magnification factor, both satisfying the general principles of 7.4.1. Non-linear material behaviour (cracking, non-linear materials, creep and tension stiffening) shall be taken into account by means of an effective stiffness.

(2) Member stiffnesses may in principle be chosen within relatively wide limits provided equilibrium and compatibility are satisfied as described in (3) and (4).

NOTE Optimal design will require several iterations.

(3) For compression members with approximately constant axial force, constant section and reinforcement along the member length, it suffices to satisfy equilibrium and curvature compatibility at the most critical section.

(4) In other cases, equilibrium and curvature compatibility shall be satisfied at a sufficient number of sections to ensure a safe design, i.e., the member sections shall be designed for corresponding values of N_{Ed}, M_{Ed} and curvature ($1/r = M_{Ed}/EI$) obtained with the assumed bending stiffness values EI at these sections.

(5) For global analysis as a simplification, the value of the stiffnesses may be taken as:

$EI = 0{,}4E_{cd}I_c$ for walls and columns;

$EI = 0{,}3E_{cd}I_c$ for reinforced concrete beams and slabs;

$EI = E_{cd}I_c$ for uncracked concrete beams and slabs due to prestressing.

When using such an approximation, and in order to prevent false force redistributions, the stiffness of restraining elements should be reduced in the same proportion.

Where simplified properties are used for global analysis, local 2nd order effects in slender members shall be accounted for either by a local analysis using O.8.2, by the simplified method in O.7, or with an effective stiffness that fulfils (2).

O.8.2 Moment magnification method

(1) In case of a global or a local analysis the total design moment, including the second order moment, may be expressed as a magnification of the bending moment from a first order linear analysis, namely:

$$M_{Ed} = M_{0Ed}\left[1 + \frac{\beta_c}{|N_B/N_{Ed}| - 1}\right] \tag{O.23}$$

where

M_{0Ed} is the first order moment;

β_c is a coefficient which depends on the distribution of first and second order moments, see (3);

N_B is the buckling load based on the effective stiffness according to 7.4.3.2 or O.8.1.

NOTE 1 Magnification factors applicable to specific situations can be found in literature.

(2) Alternatively, for global analysis, second order effects may be obtained by fictitious magnification of the horizontal forces according to Formula (O.24)

$$F_{\mathrm{Ed},2} = F_{\mathrm{H,0Ed}} \left[\frac{1}{1 - |N_{\mathrm{Ed}}/N_{\mathrm{B}}|} \right] \tag{O.24}$$

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

$$\beta_{\mathrm{c}} = \pi^2 / c_{1/\mathrm{r}} \tag{O.25}$$

where

$c_{1/\mathrm{r}}$ is a coefficient which depends on the distribution of first and second order moments (for instance, $c_{1/\mathrm{r}} = 8$ for a constant first order moment distribution, $c_{1/\mathrm{r}} = 9{,}6$ for a parabolic distribution and 12 for a triangular distribution etc.).

(4) 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 according to O.7.2(2). Consistent with the assumption of a constant first order moment, $c_{1/\mathrm{r}} = 8$ should be used.

NOTE 2 The value of $c_{1/\mathrm{r}} = 8$ also applies to members bent in double curvature. In some cases, depending on slenderness and axial force, the end moments can be greater than the magnified equivalent moment.

(5) Where O.8.2(3) or (4) is not applicable, $\beta_{\mathrm{c}} = 1$ may be used.

NOTE 3 This 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 in bracing members.

Annex P
(informative)

Alternative cover approach for durability

P.1 Use of this Informative Annex

(1) This Informative Annex provides an alternative approach to design cover for durability without use of Exposure Resistance Classes (ERC) as defined in 6.4. In this case, the resistance of the cover depends on the value of concrete cover and the concrete mix requirements based on Deemed-to-Satisfy values for the limiting values of concrete composition according to EN 206:2013, Annex F (see P.3).

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this Informative Annex, it can be used.

P.2 Scope and field of application

(1) This Informative Annex applies to the design of cover for durability of concrete without the use of Exposure Resistance Classes (ERC) as defined in 6.4.

(2) Except for 6.4, 6.5.2.1(1) and 6.5.2.2, all provisions of Clause 6 apply.

P.3 Minimum cover

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

$$c_{min} = \max \{c_{min,b};\ c_{min,dur} + \Delta c_{dur,\gamma} - \Delta c_{dur,st} - \Delta c_{dur,add};\ 10\ \text{mm}\} \tag{P.1}$$

where

$c_{min,b}$ is the minimum cover due to bond requirement, see 6.5.2.3;

$c_{min,dur}$ is the minimum cover due to environmental conditions, see P.3(2);

$\Delta c_{dur,\gamma}$ is the additive safety element, see P.3(3);

$\Delta c_{dur,st}$ is the reduction of minimum cover for use of stainless steel, see P.3(4);

$\Delta c_{dur,add}$ is the reduction of minimum cover for use of additional protection, see P.3(5).

(2) The minimum cover values for reinforcing steel and prestressing steel in normal weight concrete taking account of the exposure classes and the structural classes shall be taken as $c_{min,dur}$.

NOTE The recommended Structural Class (design working life of 50 years) is S4 for the indicative concrete strengths given in Table P.4 and the recommended modifications to the structural class are given in Table P.1. The recommended values of $c_{min,dur}$ are given in Table P.2 (carbon reinforcing steel) and Table P.3 (prestressing steel).

Table P.1— Recommended structural classification

Criterion	Exposure Class according to Table 6.1						
	X0	XC1	XC2/XC3	XC4	XD1	XD2/XS1	XD3/XS2/XS3
Design Working 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[a,b]	≥ 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 (position of reinforcement not affected by construction process)	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 of the concrete production 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

[a] The strength class and w/c ratio are considered to be related values. A special composition (type of cement, w/c value, fine fillers) with the intent to produce low permeability may be considered.

[b] The limit may be reduced by one strength class if air entrainment of more than 4 % is applied.

Table P.2— Values of minimum cover $c_{\text{min,dur}}$ [mm] for carbon reinforcing steel

Structural Class	Exposure Class according to Table 6.1						
	X0	XC1	XC2/XC3	XC4	XD1/XS1	XD2/XS2	XD3/XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

Table P.3— Values of minimum cover $c_{\text{min,dur}}$ [mm] for prestressing steel

Structural Class	Exposure Class according to Table 6.1						
	X0	XC1	XC2/XC3	XC4	XD1/XS1	XD2/XS2	XD3/XS3
S1	10	15	20	25	30	35	40
S2	10	15	25	30	35	40	45
S3	10	20	30	35	40	45	50
S4	10	25	35	40	45	50	55
S5	15	30	40	45	50	55	60
S6	20	35	45	50	55	60	65

(3) The concrete cover may be increased by an additive safety element $\Delta c_{dur,\gamma}$. The recommended value is 0 mm.

(4) Where stainless steel is used or where other special measures have been taken, the minimum cover may be reduced by $\Delta c_{dur,st}$. For such situations the effects on all relevant material properties should be considered, including bond. The recommended value, without further specification, is 0 mm.

(5) For concrete with additional protection (e.g. coating) the minimum cover may be reduced by $\Delta c_{dur,add}$. The recommended value, without further specification, is 0 mm.

P.4 Indicative strength classes for durability

(1) The choice of adequately durable concrete for corrosion protection of reinforcement and protection from concrete attack, requires consideration of the composition of concrete. This may result in a higher compressive strength of the concrete than is required for structural design. The relationship between concrete strength classes and exposure classes (see Table 6.1) may be described by indicative strength classes. The recommended values are given in Table P.4.

Table P.4— Indicative minimum strength classes

Corrosion										
	Carbonation-induced				**Chloride-induced**			**Chloride-induced from sea-water**		
1	XC1	XC2	XC3	XC4	XD1	XD2	XD3	XS1	XS2	XS3
2	C20/25	C25/30	C30/37		C30/37		C35/45	C30/37	C35/45	
Concrete damage										
	No risk	**Freeze/Thaw Attack**						**Chemical Attack**		
3	X0	XF1	XF2		XF3			XA1	XA2	XA3
4	C12/15	C30/37	C25/30		C30/37			C30/37		C35/45
NOTE For Exposure Classes see Table 6.1.										

Annex Q
(normative)

Stainless reinforcing steel

Q.1 Use of this annex

(1) This Normative Annex contains additional provisions for stainless reinforcing steel.

Q.2 Scope and field of application

(1) This Normative Annex applies to ribbed and indented stainless reinforcing steel. The provisions of this Eurocode apply to stainless reinforcing steel unless modified in this Annex Q.

(2) This Normative Annex does not cover cast-in fastenings used to anchor steel plates or members to the surface of concrete structures.

NOTE Fastenings can be designed according to EN 1992-4, and exposed steel members in accordance with prEN 1993-1-1.

Q.3 General

(1) Special provisions for stainless reinforcing steel are given in Table Q.1.

Table Q.1 — Special provisions for stainless reinforcing steel

Reference to original clause	Values and terms to be modified for stainless reinforcing steel	Provisions and formulae for stainless reinforcing steel
5.2.1(5)	New clause	Stainless steels used for structures in accordance with this Eurocode shall comply with the relevant standards for stainless steel. NOTE 3 The National Annex can specify relevant standards for stainless reinforcing steel. NOTE 4 The harmonised product standard EN10370 for stainless steel is currently under development.
5.2.2(2)	New term $f_{0,2k}$ in Tables 5.4 and 5.5 introduced.	Characteristic value of $k = (f_t/f_{0,2})_k$
5.2.3(2)	New note added.	NOTE Further guidance for welding of stainless steel can be found in the relevant standard for stainless steel.
5.2.4(2)	New terms $f_{0,2k}$ introduced.	(2) For design either of the following assumptions may be made (see Figure 5.2): a) an inclined post-elastic branch with a strain limit of $\varepsilon_{ud} \leq \varepsilon_{uk}/\gamma_S$ and a maximum stress of $k \cdot f_{0,2k}/\gamma_S$ at ε_{uk}, where $k = (f_t/f_{0,2})_k$.

Reference to original clause	Values and terms to be modified for stainless reinforcing steel	Provisions and formulae for stainless reinforcing steel
5.2.4(3)	New terms added.	(3) The design value of the modulus of elasticity E_s for stainless steel may be assumed to be 200 000 MPa unless a more accurate value is obtained from production data. NOTE The E-moduli of stainless steel depend on the alloy and can be between 150 000 MPa and 200 000 MPa.
5.2.4(5)	New terms added.	(5) The coefficient of thermal expansion may be taken as $\alpha_{s,th} = 10 \cdot 10^{-6}\ K^{-1}$ for stainless reinforcing steel unless a more accurate value is obtained from production data.
6.5.2.2	New rules added.	See Q.4.

Q.4 Minimum cover for durability

(1) Unless more refined methods are used, the durability design with stainless reinforcing steel may be based on Stainless Steel Resistance Classes SSRC as defined in Table Q.2.

NOTE 1 For an alternative approach to design cover for durability without use of Exposure Resistance Classes (ERC) see Annex P.

Table Q.2 — Classification of corrosion resistance of stainless reinforcing steel dependent on the Pitting Resistance Equivalent PRE

Stainless Steel Resistance Class	Pitting Resistance Equivalent PRE[a]	Description	Informative examples EN 10088-1		
			Ferritic	Duplex	Austenitic
SSRC0	0 to 9	Carbon steel reinforcement	—	—	—
SSRC1	10 to 16	Chromium steels	1.4003	—	—
SSRC2	17 to 22	Chromium Nickel steels	—	1.4482	1.4301 1.4307
SSRC3	23 to 30	Chromium Nickel steels with Molybdenum	—	1.4362	1.4401 1.4404 1.4571
SSRC4	≥ 31	Steels with increased content of Chromium and Molybdenum	—	1.4462	1.4529

[a] Calculation of the Pitting Resistance Equivalent: PRE = Cr + 3,3 · Mo + n · N; Cr, Mo and N in M.- %.
With: $n = 0$ for ferritic steels, $n = 16$ for Duplex steels and $n = 30$ for austenitic steels.
The calculation of the Pitting Resistance Equivalent is a useful / practical indication when classifying stainless steels. Other factors can negatively influence the corrosion resistance. The results of the test could be used as input parameters for the service life design.

(2) Where stainless reinforcing steel is used, the minimum cover $c_{min,dur}$ in Table Q.3 (NDP) may be used.

NOTE 2 The values given in Table Q.3 (NDP) apply unless the National Annex gives different values.

Table Q.3 (NDP) — Minimum concrete cover $c_{min,dur}$ to stainless steel reinforcement

Exposure Class	Exposure Resistance Class ERC	Stainless Steel Resistance Class[a]			
		SSRC1	SSRC2	SSRC3	SSRC4
XC1	≤ XRC7	0	0	0	0
XC2		0	0	0	0
XC3	≤ XRC4	0	0	0	0
	≤ XRC7	15	0	0	0
XC4	≤ XRC4	15	0	0	0
	≤ XRC7	20	0	0	0
XD1, XS1	≤ XRDS0,5	10	0	0	0
	≤ XRDS1,5	20	10	0	0
	≤ XRDS3	25	15	10	0
	≤ XRDS6	35	25	15	0
	≤ XRDS10	45	35	25	15
XD2, XD3, XS2, XS3	≤ XRDS0,5	15	10	10	0
	≤ XRDS1,5	25	20	15	0
	≤ XRDS3	35	30	20	10
	≤ XRDS6	50	40	30	20
	≤ XRDS10	65	50	40	30

NOTE 1 The tabulated cover values apply for a design service life of 50 years unless the National Annex excludes some classes or gives other values.

NOTE 2 For a design service life of 100 years $c_{min,dur}$ in Table Q.3 (NDP) should be increased by +10 mm for all ERC classes unless the National Annex excludes some classes or gives other values.

NOTE 3 In case of combined action of carbonation and chloride induced corrosion, $c_{min,dur}$ in Table Q.3 (NDP) should be increased by 20 mm or a higher stainless steel resistance class should be chosen unless the National Annex gives other values.

NOTE 4 As alternative to the class system of Table Q.3 (NDP) a performance-oriented service life design may be applied if the input parameters out of technical product specifications are available.

[a] For stainless steel corrosion resistance classes see Table Q.2.

(3) If welding of stainless reinforcing steel is necessary, the sensitivity of intercrystalline corrosion (stress corrosion cracking) shall be taken into account for the selection of the appropriate stainless steel.

Q.5 Fatigue verification

(1) The values given in 10.4(1) may be used for stainless reinforcing steel complying with the provisions of C.4.2.

(2) The values given in Table E.1 (NDP) may be used for stainless reinforcing steel complying with the provisions of C.4.2.

Annex R
(informative)

Embedded FRP reinforcement

R.1 Use of this annex

(1) This Informative Annex contains supplementary guidance for the design of new structures reinforced with non-prestressed glass and carbon fibre reinforcement.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this Informative Annex, it can be used.

R.2 Scope and field of application

(1) This Informative Annex applies only to profiled or roughened glass and carbon fibre reinforced polymer (FRP) reinforcement bars and mesh. Prestressed FRP is not covered.

(2) This Informative Annex does not apply for use of FRP reinforcement in lightweight concrete and in concrete made with recycled aggregate, i.e. Annexes M and N do not apply.

(3) This Informative Annex applies to members with FRP reinforcement subjected to predominantly static loads, i.e. Clause 10 and Annex E do not apply to members with FRP reinforcement.

R.3 General

(1) The provisions of this Eurocode apply for concrete members with FRP reinforcement unless modified in this Annex R.

NOTE The provisions of this Eurocode apply for concrete members with FRP reinforcement unless modified in this Annex R .

R.4 Verification- Partial factors for FRP reinforcement

(1) The partial factor for material γ_{FRP} shall be used for FRP reinforcement.

NOTE The values of the partial factors for FRP reinforcement are those in Table R.1 (NDP) unless the National Annex gives different values.

Table R.1 (NDP) — Partial factors for FRP reinforcement

Design Situation	γ_{FRP}
Persistent and transient design situation	1,50
Accidental design situation	1,10
Serviceability limit state	1,00

R.5 Materials

R.5.1 General

(1) Embedded FRP reinforcement materials suitable for design in accordance with this Eurocode shall satisfy the requirements of R.15.

(2) Embedded FRP reinforcement materials used for structures in accordance with this Eurocode shall comply with the relevant product standards for FRP.

NOTE 1 As long as harmonized product standards are not available, the National Annex can specify the relevant standards for embedded FRP reinforcement materials.

NOTE 2 ISO 10406 can be used for determination of selected properties.

(3) Annex R provides design rules for member reinforced with embedded FRP reinforcement within the following limits of applicability:

— Minimum modulus of elasticity of $E_{\mathrm{fR}} \geq 40\ 000$ MPa;

— Ratio of $f_{\mathrm{ftk,100a}}/E_{\mathrm{fR}} \geq 0{,}005$;

— characteristic compressive strength of concrete $f_{\mathrm{ck}} \geq 20$ MPa;

— members with $\rho_{\mathrm{lf}} \leq 0{,}05$.

R.5.2 Properties

(1) Specified properties and related conditions of fibre reinforced polymer systems that are required for design to this Eurocode shall include at least the following:

— f_{ftk0} determined in accordance with ISO 10406-1;

— E_{fR} determined in accordance with ISO 10406-1;

— nominal diameter.

(2) The following properties of the FRP reinforcement should be available to ensure a performance as assumed in design:

— section sizes and tolerance on size;

— minimum characteristic short-term tensile strength;

— minimum long-term characteristic tensile strength;

— modulus of elasticity;

— long term bond strength $f_{\mathrm{bd,100a}}$;

— Strain of FRP shear reinforcement $\varepsilon_{\mathrm{fwRd}}$ at design tensile strength of FRP shear reinforcement f_{fwRd};

— installation temperature;

— maximum temperature of the FRP reinforcement for the design life of the structure;

— minimum temperature of the FRP reinforcement for the design life of the structure;

— exposure classification, in accordance with Table 6.1;

— durability requirements.

R.5.3 Design assumptions

(1) Design should be based on the nominal cross section area of the reinforcement.

(2) The value of the design tensile strength of embedded FRP reinforcement shall be taken as:

$$f_{\mathrm{ftd}} = \frac{f_{\mathrm{ftk,100a}}}{\gamma_{\mathrm{FRP}}} \tag{R.1}$$

where

$f_{\mathrm{ftk,100a}}$ design long-term strength. When not directly determined from production data, it can be obtained with Formula (R.2),

$$f_{\mathrm{ftk,100a}} = C_{\mathrm{t}} \cdot C_{\mathrm{C}} \cdot C_{\mathrm{e}} \cdot f_{\mathrm{ftk0}} \tag{R.2}$$

C_{t} is the factor considering temperature effects. The following values may be used for C_{t}:

— $C_{\mathrm{t}} = 1{,}0$ for indoor and underground environments;

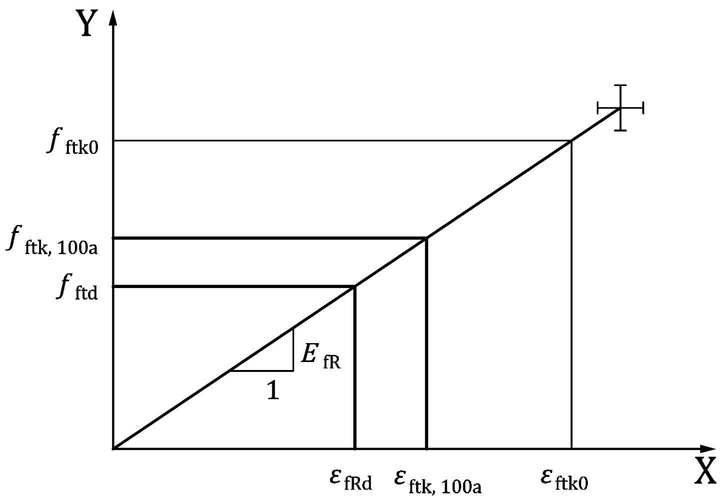
— $C_{\mathrm{t}} = 0{,}8$ for outdoor members if heating through solar radiation cannot be excluded;

C_{c} is the coefficient between the strength under sustained load and the strength under short-term load. The factor may be determined according ISO 10406-1, Clause 12. The value shall be taken as 0,35 for Glass FRP reinforcement and 0,8 for Carbon FRP reinforcement unless more accurate values are determined.

C_{e} is the coefficient between the strength after ageing and before ageing. The factor may be determined according to the test concept in ISO 10406-1 with exposure to 60°C for a duration of 3000 h. The value shall be taken as 0,7 unless more accurate values are determined.

NOTE Further guidance for the estimation of C_{t} can be found in *fib* Bulletin 40.

(3) The stress-strain relationship should be taken as illustrated in Figure R.1.



Key

Y σ_{f}

X ε_{f}

Figure R.1 — Design stress-strain-diagram for FRP reinforcement

(3) FRP reinforcement shall be used as tension reinforcement only.

(4) The mean density of FRP for the purposes of design may be taken as 2 000 kg/m^3 for GFRP reinforcement and 1650 kg/m^3 for CFRP reinforcement.

(5) Unless production data gives more accurate values, the coefficient of thermal expansion in longitudinal direction may be taken as $\alpha_{\mathrm{FRP,th}} = 5 \cdot 10^{-6}$ K^{-1} for GFRP bars and $\alpha_{\mathrm{FRP,th}} = 0$ for CFRP bars.

R.6 Durability - Concrete cover

(1) The value $c_{\mathrm{min,dur}} = 0$ may be taken for FRP reinforcement in Formula (6.2).

(2) Unless more accurate information is available based on test data, the cover for bond for FRP reinforcement should be taken as $c_{\mathrm{min,b}} \geq 2\phi$. But at least the minimum cover for FRP reinforcement shall be taken as $c_{\mathrm{min,b}} \geq 1{,}5\phi$ and $c_{\mathrm{min,b}} \geq 10$ mm.

(3) Direct contact of carbon FRP reinforcement bars with steel reinforcement should be avoided.

R.7 Structural analysis

(1) For straight FRP-bars installed in a curved shape the bending stress shall be taken into account as permanent action.

(2) Linear elastic analysis with redistribution according to 7.3.2 shall not be undertaken for members with FRP reinforcement.

(3) Plastic analysis according to 7.3.3 shall not be undertaken for members with FRP reinforcement.

(4) Non-linear analysis according to 7.3.4 may be undertaken using the model outlined in Figure R.1 with design strength, f_{ftd}, and corresponding design rupture strain, $\varepsilon_{\mathrm{fRd}}$.

R.8 Ultimate Limit States (ULS)

R.8.1 Bending with or without axial forces

(1) The tensile strain in FRP reinforcement shall be limited to the design rupture strain, $\varepsilon_{\mathrm{fRd}}$.

(2) FRP reinforcement shall not be considered as compression reinforcement.

(2) Unless more rigorous analysis is undertaken the benefit of the confining effect of FRP reinforcement should be reduced by the ratio $E_{\mathrm{fR}}/E_{\mathrm{s}}$ in any direction that confinement is considered.

R.8.2 Shear

(1) For members reinforced with longitudinal FRP reinforcement without requiring shear reinforcement, the minimum shear resistance in 8.2.1(4) may be calculated as:

$$\tau_{\mathrm{Rdc,min}} = \frac{11}{\gamma_{\mathrm{v}}}\sqrt{\frac{f_{\mathrm{ck}}}{f_{\mathrm{ftk0}}} \cdot \frac{E_{\mathrm{fR}}}{E_{\mathrm{s}}} \cdot \frac{d_{\mathrm{dg}}}{d}} \tag{R.3}$$

(2) The provisions in 8.2.2 may be used provided that the ρ_{l} of longitudinal FRP reinforcement in Formula (8.28) is reduced by the ratio $E_{\mathrm{fR}}/E_{\mathrm{s}}$. The provisions in 8.2.2 for determining shear resistance in presence of tension axial forces should not be applied if the height of the compression zone in the cracked state of the section is less than 0,1d.

(3) The provisions outlined in 8.2.3 may be used subject to the following modifications:

— The angle of inclination of the compression field should be replaced by:

$$\cot\theta = 0{,}8 \tag{R.4}$$

— The shear resistance for members requiring shear reinforcement may be determined by:

$$\tau_{\mathrm{Rd,f}} = \tau_{\mathrm{Rd,c}} + \rho_{\mathrm{w}} \cdot f_{\mathrm{fwRd}} \cdot \cot\theta \leq 0{,}17 \cdot f_{\mathrm{cd}} \tag{R.5}$$

where

$$f_{\mathrm{fwRd}} = f_{\mathrm{fwk,100a}} / \gamma_{\mathrm{FRP}} \leq \varepsilon_{\mathrm{fwRd}} \cdot E_{\mathrm{fwR}} \tag{R.6}$$

where

$$\varepsilon_{\mathrm{fwRd}} = 0{,}0023 + 1/15 \cdot E_{\mathrm{fR}} \cdot A_{\mathrm{fl}} \cdot (0{,}8 \cdot d)^2 \cdot 10^{-15} \leq 0{,}007 \tag{R.7}$$

A_{fl} longitudinal FRP reinforcement, considering for slabs a width of 1m

(4) The provision outlined in 8.2.5 may be used subject to the following alterations:

— f_{yd} is replaced by f_{ftd};

— $\cot\theta$ = 1,0 is used;

— ν = 0,35 is used.

(5) The provisions outlined in 8.2.6 for determining the shear resistance at an interface may be used subject to the following modifications:

— Formula (8.76) shall be replaced by

$$\tau_{\mathrm{Rdi}} = c_{\mathrm{v1}} \cdot \frac{\sqrt{f_{\mathrm{ck}}}}{\gamma_{\mathrm{c}}} + \mu_{\mathrm{v}} \cdot \sigma_{\mathrm{n}} \leq 0{,}17 \cdot f_{\mathrm{cd}} \tag{R.8}$$

— 8.2.6(7) and Formula (8.77) do not apply for FRP reinforcement.

R.8.3 Torsion

(1) The provisions of 8.3.4 may be used for members with FRP reinforcement subject to the following alterations:

— f_{yd} is replaced by f_{ftd}, where f_{ftd} should be limited by $f_{\mathrm{ftd}} \leq 0{,}004 \cdot E_{\mathrm{fR}}$;

— f_{ywd} is replaced by f_{fwRd}, according to Formula (R.6), where f_{fwRd} should be limited by $f_{\mathrm{fwRd}} \leq 0{,}004 \cdot E_{\mathrm{fwR}}$;

— A_{sl} is replaced by A_{fl};

— $\cot\theta$ = 1,0 is used;

— ν = 0,35 is used,

(2) For combined shear and torsion the compatibility of strains has to be ensured due to the different approaches for shear and for torsion. The required shear reinforcement shall be the sum of that due to shear and due to torsion.

R.8.4 Punching

(1) The provisions in 8.4.3 shall not be applied to concrete members with longitudinal FRP reinforcement. The design punching shear stress resistance $\tau_{\mathrm{Rd,c}}$ may be calculated in accordance to the shear resistance in R.8.2.

(2) The provisions in 8.4.4 shall not be applied to concrete members with longitudinal FRP reinforcement requiring punching shear reinforcement.

R.8.5 Design with strut-and-tie models and stress fields

(1) Design with strut-and-tie models and stress fields for concrete structure reinforced with FRP reinforcement are not covered by this Eurocode.

R.9 Serviceability Limit States (SLS) - Special rules for FRP reinforcement

R.9.1 General

(1) Provision 9.1(5) shall not be applied.

R.9.2 Stress limitation and crack control

(1) Provision 9.2.1(2) does not apply for FRP reinforcement.

(2) In contrast to 9.2.1(4), by applying a sufficient web surface reinforcement according to S.5, a verification of the crack width by calculation may not be omitted for FRP reinforcement.

(3) Tables 9.1 (NDP) and 9.2 (NDP) do not apply for members reinforced only with FRP reinforcement. Tables R.2 and R.3 should be applied.

Table R.2 — Verifications, stress and crack width limits for appearance

Verification	**Calculation of minimum reinforcement according to 9.2.2**	**Verification of crack width according to 9.2.3**	**Verification of reinforcement stresses to avoid failure at SLS**
Combination of actions for calculating σ_f	Cracking forces according to 9.2.2	Quasi-permanent combination of actions	Characteristic combination of actions
Limiting value of crack width $w_{\text{lim,cal}}$ or stress σ_f	$\sigma_f \leq f_{\text{ftd}}$	$w_{\text{lim,cal}} = 0{,}4$ mm $\sigma_f \leq f_{\text{ftd}}$	$\sigma_f \leq 0{,}8 f_{\text{ftd}}$

Table R.3 — Verifications, stress and crack width limits for durability

Exposure Class	**Combination of actions**	
	quasi-permanent	**characteristic**
XC, XF, XD	$w_{\text{lim,cal}} = 0{,}4$ mm[c]	$\sigma_c \leq 0{,}6 f_{\text{ck}}$[a,b]

[a] No limitation in serviceability conditions is necessary for stresses under bearings, partially loaded areas and plates of headed bars.

[b] The compressive stress σ_c may be increased to $0{,}66 f_{\text{ck}}$ if the cover is increased by 10 mm or confinement by transverse reinforcement is provided.

[c] In absence of appearance conditions, fasteners, punctual wheel pressure, lap splice or freeze thaw this limit may be relaxed to values up to 0,7 mm.

(4) The provisions relevant to steel reinforcement in 9.2.2, 9.2.3 and Annex S may be applied to concrete with FRP reinforcement with the following modifications under the assumption that the bond behaviour of the FRP reinforcement is similar as for steel reinforcement:

— σ_s is replaced by σ_f resp. $\sigma_{s,lim}$ by $\sigma_{f,lim}$;

— f_{yk} is replaced by f_{ftd};

— E_s is replaced by E_{fR};

— A_s is replaced by $A_f \cdot (E_{fR}/E_s)$ in all relevant formulae in these clauses;

— 9.2.2(3) dealing with prestressing does not apply.

R.9.3 Deflection control

(1) Table 9.3 in 9.3.2(1) should not be used for structures with FRP reinforcement.

(2) 9.3.3 does not apply for structures with FRP reinforcement.

R.10 Fatigue

(1) This Eurocode does not provide rules for fatigue utilising FRP reinforcement.

R.11 Detailing of FRP reinforcement

R.11.1 General

(1) Without further evidence, the provisions in Clause 11 and Clause R.11 should only be used for straight FRP reinforcement.

R.11.2 Spacing of bars

NOTE See 11.2.

R.11.3 Permissible mandrel diameters for bent bars

(1) The minimum diameter to which a bar may be bent shall be such as to avoid:

— damaging the FRP reinforcement (see (2)); and

— failure of the concrete inside the bend of the bar (crushing, splitting or spalling of reinforcement cover), see (3) and (4).

(2) The mandrel diameter of FRP reinforcement may be found in the Technical Product Specification. Bending on site and re-bending (straightening) is not permitted.

The mandrel diameter should be at least:

— $\phi_{mand,min} = 4\phi$ for $\phi \leq 16$ mm;

— $\phi_{mand,min} = 7\phi$ for $\phi > 16$ mm.

NOTE Bending of FRP bars is done under controlled factory and warm conditions only.

(3) Provided that $f_{ftd} \leq 25 f_{cd}$ and $\gamma_C \leq 1,5$, verification of the concrete inside the bend may be omitted for:

— stirrups in compliance with 12.3.3(4);

— standard hook and bend anchorages complying with Figure 11.6 at a clear distance;

— $c_x \geq 1,5\phi$ from an edge parallel to the bent and a clear distance between bars $c_s \geq 3\phi$ according to Figure 11.6c; and

— all bends with an angle $\alpha_{bend} \leq 45°$ at a clear distance $c_x \geq 2{,}5\phi$ from an edge parallel to the bent, a clear distance between bars $c_s \geq 5\phi$ and a length $\geq 2\phi$ of the straight segments between multiple bends.

(4) In cases not complying with (3) the design value of the stress in the FRP bar σ_{ftd} should be verified to avoid concrete failures inside the bend according to Formula (R.9):

$$\sigma_{ftd} \leq 25 \cdot f_{cd} \tag{R.9}$$

R.11.4 Anchorage of FRP reinforcement in tension and compression

(1) Provisions for anchorage in 11.4 may be applied to determine the anchorage lengths of FRP reinforcement only where additions and modifications in Annex R are used in the determination of relevant parameters.

(2) Only the methods of anchorage according to Figure 11.2 a), b) and c) in 11.4.1(6) may be used for FRP reinforcement.

(3) Formula (R.10) may be applied to determine the anchorage length of FRP reinforcement:

$$l_{bd} = k_{lb} \cdot k_{cp} \cdot \emptyset \cdot \left(\frac{\sigma_{ftd}}{217}\right)^{n_\sigma} \cdot \left(\frac{25}{f_{ck}}\right)^{\frac{1}{2}} \cdot \left(\frac{\phi}{20}\right)^{\frac{1}{3}} \cdot \left(\frac{1{,}5.\phi}{c_d}\right)^{\frac{1}{2}} \geq 10\phi \tag{R.10}$$

Where

n_σ = 1,0 for $\sigma_{ftd} \leq 217$ MPa;

n_σ = 1,5 for $\sigma_{ftd} > 217$ MPa;

l_{bd} shall be limited also to

$$l_{bd} \geq (\phi/4) \cdot (\sigma_{ftd}/f_{bd,100a}) \tag{R.11}$$

NOTE The value of $f_{bd,100a}$ = 1,5 MPa applies unless the more accurate information is available based on production data.

(4) Where the clear distance between FRP reinforcement bars $c_s < 7{,}5 \cdot \phi_f$, concrete cover spalling shall be prevented by limiting the design strain to $\varepsilon_{fRd} \leq 0{,}0035$ in straight bars or with by confining the anchorage zone.

(5) Unless more rigorous analysis is undertaken the provisions in 11.4.3 for bundles should not be used for FRP bars.

(6) The provisions in 11.4.4 may be applied with the assumption, that only the straight part is considered for determining the anchorage length and design long-term tensile strength f_{fwRd} is considered.

(7) The provisions in 11.4.5 shall not be applied for FRP reinforcement.

(8) The provisions in 11.4.6 may be applied with the assumption, that only the straight part is considered for determining the anchorage length and design long-term tensile strength f_{fwRd} is considered.

(9) The provisions in 11.4.7 and 11.4.8 shall not be applied for FRP reinforcement.

R.11.5 Laps of FRP reinforcement in tension

(1) Provisions for laps in 11.5 should only be used for FRP reinforcement within the provisions stated in Annex R.

(2) Laps of FRP reinforcement to FRP reinforcement or other reinforcement types shall be situated in zones where the stress in the reinforcement at ultimate limit state is less than 80 % of the design strength.

(3) The provisions in 11.5.3 for bundles should not be used, if no further proof is done.

(4) The provisions in 11.5.4 may be applied with the assumption, that only the straight part is considered for determining the anchorage length and design long-term tensile strength f_{fwRd} is considered.

(5) The provisions in 11.5.5, 11.5.6 and 11.5.7 shall not be used for FRP reinforcement.

R.11.6 Post-tensioning tendons

(1) The provisions in 11.6 do not apply for FRP reinforcement.

R.11.7 Deviation forces due to curved tensile and compressive chords

NOTE See 11.7

R.12 Detailing of members and particular rules

R.12.1 General

(1) Annex R does not provide rules for FRP reinforcement utilized for robustness of concrete structures in case of progressive collapse as punching or concrete shear failure.

(2) Detailing of members shall be consistent with the design models adopted considering the rules given in Annex R.

(3) All provisions of Clause 12 apply unless specifically omitted or supplemented in Clause R.12. The bond behaviour and properties of the embedded FRP reinforcement should comply with Clause R.15.

(4) If not mentioned otherwise, the following modifications apply to all relevant provisions in Clause 12:

— f_{yk} is replaced by f_{ftd};

— E_{s} is replaced by E_{fR};

— A_{s} is replaced by A_{f}.

— (5) Annex R does not provide rules for bent-up FRP bars.

R.12.2 Minimum reinforcement rules

(1) The minimum FRP reinforcement $A_{\text{f,min}}$ in members with pure tension may be determined as follows:

$$A_{\text{fl,min}} = A_{\text{c}} \cdot f_{\text{ctm}} / f_{\text{ftd}} \tag{R.12}$$

(2) Provision 12.2(3) does not apply for FRP reinforcement.

(3) Minimum FRP reinforcement shall generally be anchored and lapped according to Clause R.11 for a stress level f_{ftd}.

R.12.3 Beams

(1) FRP reinforcement in beams, longitudinal and transverse, should be detailed in accordance with the requirements of Table 12.1 (NDP) using $s_{\text{l,max}}$ < 250 mm.

(2) The minimum reinforcement should be distributed over the width and proportionally over the height of the tension zone.

(3) The minimum reinforcement required should be fully provided between the supports.

(4) The minimum reinforcement required for cantilevers shall be provided for the total length of the cantilevers.

(5) For members with and without shear reinforcement, it may be assumed $a_{\mathrm{l}} = d$.

(6) Provision 12.3.2(6) does not apply for FRP reinforcement.

(7) The shear reinforcement shall only consist of a combination of:

— stirrups/links enclosing the longitudinal tension reinforcement and the compression zone;

— cages, ladders, etc. which are cast in without enclosing the longitudinal reinforcement but are properly anchored in the compression and tension zones.

(8) Anchorages with headed bars or welded / connected transverse reinforcement are not covered.

(9) Laps on legs of stirrups in shear reinforcement may be used and designed according to R.11.3 for a stress level f_{ftd}.

(10) Provisions in 12.3.4 do not apply for FRP reinforcement.

R.12.4 Slabs

(1) Reinforcement in slabs should be detailed in accordance with the requirements of Table 12.2 (NDP) using $s_{\mathrm{max,slab}}$, $s_{\mathrm{l,max}}$, $s_{\mathrm{bu,max}}$ and $s_{\mathrm{tr,max}}$< 250 mmm.

(2) The minimum height of the concrete slab is 200 mm if shear reinforcement is provided.

(3) This Eurocode does not provide rules for $A_{\mathrm{f,min}}$ for robustness in case of progressive collapse utilising FRP reinforcement in slabs.

(4) Maximum longitudinal spacing $s_{\mathrm{l,max}}$ of shear stirrups is 0,3d instead of 0,75d.

(5) Provision 12.4.2(5) does not apply for FRP reinforcement.

R.12.5 Slab-column connections and column bases

(1) Annex R does not provide rules for utilising FRP reinforcement for punching.

R.12.6 Columns

(1) Annex R does not provide rules for utilising FRP reinforcement for reinforcement under compression.

R.12.7 Walls and deep beams

(1) Reinforcement in walls and deep beams should be detailed in accordance with the requirements of Table 12.4 (NDP) using $s_{\mathrm{l,max}}$ < 250 mmm.

R.12.8 Foundations

(1) Annex R does not provide rules for FRP reinforcement in foundations according to 12.8.

R.12.9 Tying systems for robustness of buildings

(1) The provisions of 12.9 may be applied for members with FRP reinforcement.

R.12.10 Supports, bearings and expansion joints

(1) The provisions of 12.10 may be applied for members with FRP reinforcement.

R.13 Additional rules for precast concrete elements and structures

(1) Additional provisions for precast concrete elements in Clause 13 may be applied for members with FRP reinforcement with the following restrictions and alterations.

(2) Provisions in 13.3, 13.4.2 and 13.5 do not apply as heat-curing and prestressing are not covered by Annex R.

(3) In 13.7.1(9), Formula (13.14) shall be replaced by:

$$A_{\mathrm{f}} = 0{,}25 \cdot t/h \cdot F_{\mathrm{Ed}}/f_{\mathrm{ftd}} \tag{R.13}$$

with $f_{\mathrm{ftd}} \leq 0{,}004 \cdot E_{\mathrm{fR}}$.

(4) Provisions in 13.7.1(13) does not apply.

(5) Provisions in 13.7.3 for pocket foundations do not apply.

R.14 Lightly reinforced concrete structures

NOTE There are no additional requirements.

R.15 Material requirements for FRP reinforcement

(1) For the design to this Eurocode, embedded FRP reinforcement shall comply with the relevant product standards.

(2) Annex R provides design rules for member reinforced with embedded FRP reinforcement within the following limits of declared properties determined in accordance with ISO 10406 or the relevant product standards:

— Minimum strength retention after alkali-resistance test without stress according to ISO 10406-1 or to the relevant product standards shall be greater than $0{,}7f_{\mathrm{ftk0}}$

— Minimum value of $f_{\mathrm{bd,100a}} \geq 1{,}5$ MPa.

NOTE All limitations and required properties provided in Clause R.5 also apply.

R.16 Surface reinforcement for large diameter bars

(1) When determining the surface reinforcement in accordance with S.5, the different moduli of elasticity, elongation at fracture and bond properties of the main reinforcement and the surface reinforcement shall be taken into account.

(2) The surface reinforcement shall not fail before the ULS is reached.

(3) The surface reinforcement shall be:

$$A_{\mathrm{f,surf}} = 0{,}01 \cdot A_{\mathrm{ct}} \cdot E_{\mathrm{s}}/E_{\mathrm{fR}} \tag{R.14}$$

Annex S
(informative)

Minimum reinforcement for crack control and simplified control of cracking

S.1 Use of this informative annex

(1) This Informative Annex provides additional information for crack width control, covering both minimum reinforcement for crack control and simplified control of cracking.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this Informative Annex, it can be used.

S.2 Scope and field of application

(1) This Informative Annex includes:

— A formulation to determine the minimum reinforcement required to limit the crack width when, even though the characteristic service forces acting on a given section are not expected to produce cracking, cracking can nevertheless occur due to action effects not explicitly accounted for in design. This formulation can be used, for instance (but not only), to determine the base reinforcement of slabs or beams (see Clause S.3);

— a simplified and conservative formulation to determine the crack width based of the general formulation of 9.2.3. (see Clause S.4); and

— empirical rules for surface reinforcement to control crack widths in members reinforced with large bar diameters (see Clause S.5).

S.3 Minimum reinforcement areas for crack width control

(1) Unless a more refined calculation shows lesser areas to be adequate, the required minimum areas of reinforcement to control crack width may be calculated as follows. In profiled cross sections like T-beams and box girders, minimum reinforcement should be determined for the individual parts of the section (webs, flanges). In addition to the requirements in 9.2.2, the reinforcement necessary in the part of the cross section under consideration may be determined by Formulae (S.1), (S.2) or (S.3), as appropriate.

(i) For pure bending:

$$A_{\mathrm{s,min,w1}} = \frac{0{,}8\,\dfrac{h - h_{\mathrm{c,eff}}}{h}\,f_{\mathrm{ct,eff}}A_{\mathrm{c,eff}}}{\sigma_{\mathrm{s,lim}}} \qquad A_{\mathrm{s,min,w2}} = 0 \tag{S.1}$$

NOTE 1 For minimum reinforcement to avoid yielding, see 9.2.2(2).

(ii) For pure tension:

$$A_{\mathrm{s,min,w1}} = A_{\mathrm{s,min,w2}} = \frac{f_{\mathrm{ct,eff}}A_{\mathrm{c,eff}}}{\sigma_{\mathrm{s,lim}}} \tag{S.2}$$

(iii) For a combination of bending and axial force:

$$A_{\mathrm{s,min,w1}} = \frac{\left(0{,}5 - 0{,}4\dfrac{h - h_{\mathrm{c,eff}}}{h}\right) N_{\mathrm{Ed}} + 0{,}8\dfrac{h - h_{\mathrm{c,eff}}}{h} f_{\mathrm{ct,eff}} A_{\mathrm{c,eff}}}{\sigma_{\mathrm{s,lim}}} \begin{cases} \geq 0 \\ \leq \dfrac{f_{\mathrm{ct,eff}} A_{\mathrm{c,eff}}}{\sigma_{\mathrm{s,lim}}} \end{cases} \tag{S.3}$$

$$A_{\mathrm{s,min,w2}} = \frac{N_{\mathrm{Ed}}}{\sigma_{\mathrm{s,lim}}} - A_{\mathrm{s,min,w1}} \begin{cases} \leq A_{\mathrm{s,min,w1}} \\ \geq 0 \end{cases}$$

where

A_c is the area of the part of the section under consideration;

$A_{s,min,w1}$ is the area of minimum reinforcing steel to be placed at the most tensioned face of the part of the section under consideration to control cracking;

$A_{s,min,w2}$ is the area of minimum reinforcing steel to be placed at the least tensioned face of the part of the section under consideration to control cracking;

N_{Ed} is the design axial force at the serviceability limit state acting on the part of the section under consideration (tensile force positive). N_{Ed} should be determined considering the characteristic values of prestress and axial forces under the relevant combination of actions;

$\sigma_{s,lim}$ is the maximum stress permitted in the reinforcement immediately after formation of the crack. $\sigma_{s,lim}$ may be taken as the yield stress or a lower stress according to Formula (S.4), if necessary, to limit the crack width. In absence of other methods, for ribbed or indented reinforcing steel, $\sigma_{s,lim}$ may be taken as:

$$\sigma_{\mathrm{s,lim}} \leq k_{\sigma 1} f_{\mathrm{ct,eff}} \cdot \left(\frac{1}{\phi}\right)\left(-c + \sqrt{c^2 + k_{\sigma 2}\frac{E_{\mathrm{s}} \cdot w_{\mathrm{lim,cal}} \cdot \phi}{k_{\mathrm{w}} \cdot k_{\frac{1}{r},\mathrm{simpl}} \cdot f_{\mathrm{ct,eff}}}}\right) \tag{S.4}$$

$$k_{\frac{1}{r},\mathrm{simpl}} = \begin{cases} (1{,}2\dfrac{h}{d} - 0{,}1) \text{ for bending} \\ 1{,}0 \text{ for pure tension} \end{cases}$$

$$k_{\sigma 1} = \begin{cases} 4{,}8 \text{ for bending} \\ 6{,}0 \text{ for tension} \end{cases}$$

$$k_{\sigma 2} = \begin{cases} 0{,}4 \text{ for bending} \\ 0{,}3 \text{ for tension} \end{cases}$$

k_w is a factor converting the mean crack width into a calculated crack width, see 9.2.3(2);

$f_{ct,eff}$ is the mean value of the tensile strength of the concrete effective at the time when the cracks may first be expected to occur: $f_{ct,eff} = f_{ctm}$ for $t \geq t_{ref}$ or $f_{ct,eff} = f_{ctm}(t)$, if cracking is expected at $t < t_{ref}$;

NOTE 2 For more information, see Annex D.

k_h is a coefficient which allows for the effect of non-uniform self-equilibrating stresses, which lead to a reduction of the apparent tensile strength which may be taken as:

$$k_{\mathrm{h}} = 0{,}8 - 0{,}6(\min\{b; h\} - 0{,}3) \begin{cases} \leq 0{,}8 \\ \geq 0{,}5 \end{cases} \tag{S.5}$$

where h and b [m] are the dimensions of the part of the section under consideration.

NOTE 3 For consideration of thick members, see 9.2.2.

S.4 Simplified control of cracking

NOTE Clause S.4 is a simplification of 9.2.3. A conservative value is assumed for the effective tension area and tension stiffening effects are estimated as a 10 % reduction in the steel stress obtained considering a fully cracked section.

(1) The rules given in 9.2.3 may be complied with by restricting either the bar diameter ϕ according to Formula (S.6) or the bar spacing s_l according to Formula (S.7).

$$\phi \leq \frac{2{,}1 \cdot \rho_p}{\frac{a}{d} \cdot k_{fl,simpl} \cdot k_{b,simpl}} \left(\frac{w_{lim,cal}}{k_w \cdot k_{\frac{1}{r},simpl} \cdot 0{,}9 \frac{\sigma_s}{E_s}} - 1{,}5 \cdot c \right) \quad \text{(S.6)}$$

where

$$k_{fl,simpl} = \begin{cases} \left(1 - 3{,}5 \frac{a_y}{h}\right) & \text{if one face is in compresion} \\ 1{,}00 & \text{if both faces are in tension} \end{cases}$$

$$k_{1/r,simpl} = \begin{cases} 25\left(\frac{h}{d} - 1\right) \cdot \rho_p + 1{,}15 \cdot \frac{h}{d} - 0{,}15 & \text{for bending} \\ 1{,}00 & \text{for tension} \end{cases}$$

$$k_{b,simpl} = \begin{cases} 0{,}9 & \text{for good bond conditions} \\ 1{,}2 & \text{for poor bond conditions} \end{cases}$$

$$s_l \leq \frac{3{,}45 \cdot \rho_p}{\frac{a^2}{d} \cdot k_{fl,simpl}^2 \cdot k_{b,simpl}^2} \left(\frac{w_{lim,cal}}{k_w \cdot k_{1/r,simpl} \cdot 0{,}9 \cdot \frac{\sigma_s}{E_s}} - 1{,}5 \cdot c \right)^2 \quad \text{(S.7)}$$

where

$\rho_p = \frac{(A_s + \xi_1 A_p)}{b \cdot d}$	is the reinforcement ratio corresponding to the tensioned face under consideration. When considering the least tensioned face of an element with both faces in tension, $k_{1/r,simpl}$ may be set equal to 1,0 conservatively;
a	is the distance from the concrete surface to the centre of the first layer of bars;
s_l	is the bar spacing;
σ_s	is the stress permitted in the reinforcement closest to the most tensioned concrete surface after formation of all cracks. σ_s may be taken as the calculated stress according to loads or a lower stress $\sigma_{s,lim}$ according to Formula (S.4);
a_y	cover to the centre of the external bar in the y -direction

(2) Cracking due to shear and torsion may be assumed to be controlled if the detailing rules given in Clause 12 are observed. For cases where verification of cracking due to shear and torsion is considered relevant, G.5 should be used.

S.5 Surface reinforcement for large diameter bars

(1) In lieu of a verification of the crack width for large diameter bars by calculation, a constructive web surface reinforcement may be used.

(2) This surface reinforcement to limit crack widths or to resist spalling should be used where the main reinforcement is made up of:

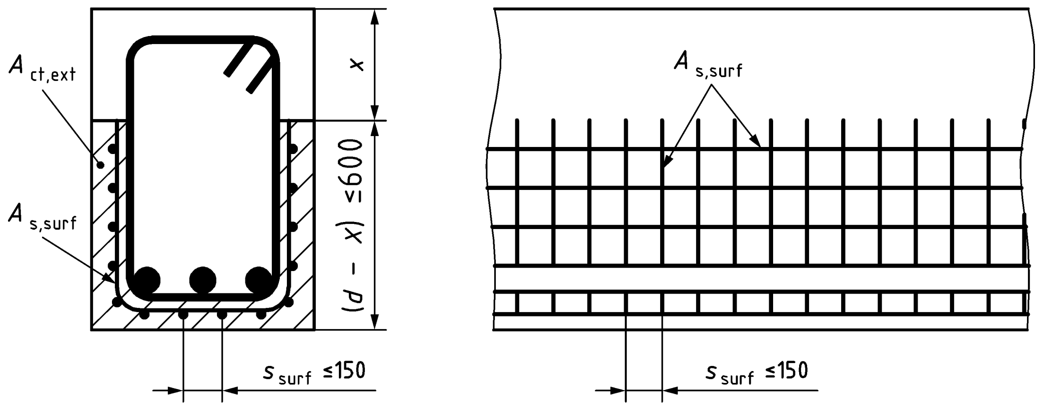
— bars with diameter $\phi > 32$ mm; or

— bundled bars with equivalent diameter $\phi_b > 32$ mm.

(3) This surface reinforcement $A_{s,surf}$ should consist of wire mesh or small diameter bars in the two directions parallel and orthogonal to the tension reinforcement in the beam and be placed outside the links in the cover as indicated in Figure S.1.

$$A_{s,surf} \geq 0{,}01 A_{ct,ext} \text{ (in each direction)} \tag{S.8}$$

Dimensions in millimetres



NOTE x is the depth of the neutral axis at ULS.

Figure S.1 — Surface reinforcement for large diameter bars and bundles

(4) Where the cover to reinforcement is greater than 70 mm, similar surface reinforcement should be used, with an area of $0{,}005A_{ct,ext}$ in each direction.

(5) The longitudinal bars of the surface reinforcement may be taken into account as longitudinal bending reinforcement and the transverse bars as shear reinforcement provided that they meet the requirements for the arrangement and anchorage of these types of reinforcement.

Bibliography

References contained in recommendations (i.e. “should” clauses)

The following documents are referred to in the text in such a way that some or all of their content constitutes highly recommended choices or course of action of this document. Subject to national regulation and/or any relevant contractual provisions, alternative documents could be used/adopted where technically justified. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

[1] EN 12390-13, *Testing hardened concrete — Part 13: Determination of secant modulus of elasticity in compression*

[2] EN 12390-16, *Testing hardened concrete — Part 16: Determination of the shrinkage of concrete*

[3] EN ISO 15630 (all parts), *Steel for the reinforcement and prestressing of concrete — Test methods*

[4] EN 1536, *Execution of special geotechnical work — Bored piles*

[5] EN 1538, *Execution of special geotechnical work — Diaphragm walls*

[6] EN 1993-1-11, *Eurocode 3: Design of steel structures — Part 1-11: Design of structures with tension components*

[7] EN 12390-17, *Testing hardened concrete — Part 17: Determination of creep of concrete in compression*

[8] EN 13369, *Common rules for precast concrete products*

References contained in permissions (i.e. “may” clauses)

The following documents are referred to in the text in such a way that some or all of their content expresses a course of action permissible within the limits of the Eurocodes. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

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[11] EN 12390-14, *Testing hardened concrete — Part 14: Semi-adiabatic method for the determination of heat released by concrete during its hardening process*

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The following documents are cited informatively in the document, for example in "can" clauses and in notes.

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