

Eurocode 2 — Design of concrete structures —

Part 2: Concrete bridges — Design and detailing rules

ICS 93.040; 91.010.30; 91.080.40

National foreword

This British Standard is the UK implementation of EN 1992-2:2005, incorporating corrigendum July 2008. It supersedes DD ENV 1992-2:2001 which is withdrawn.

The start and finish of text introduced or altered by corrigendum is indicated in the text by tags. Text altered by CEN corrigendum July 2008 is indicated in the text by **AC1** ~~AC1~~.

The structural Eurocodes are divided into packages by grouping Eurocodes for each of the main materials, concrete, steel, composite concrete and steel, timber, masonry and aluminium. This is to enable a common date of withdrawal (DOW) for all the relevant parts that are needed for a particular design. The conflicting national standards will be withdrawn at the end of the coexistence period, after all the EN Eurocodes of a package are available.

Following publication of the EN, there is a period allowed for national calibration during which the National Annex is issued, followed by a coexistence period of a maximum three years. During the coexistence period Member States will be encouraged to adapt their national provisions.

At the end of this coexistence period, the conflicting parts of national standards will be withdrawn.

In the UK, the corresponding national standards are:

- BS 5400-4:1990, *Steel, concrete and composite bridges — Code of practice for design of concrete bridges*
- BS 5400-7:1978, *Steel, concrete and composite bridges — Specification for materials and workmanship, concrete, reinforcement and prestressing tendons*
- BS 5400-8:1978, *Steel, concrete and composite bridges — Recommendations for materials and workmanship, concrete, reinforcement and prestressing tendons*

and based on this transition period, these standards will be withdrawn/revised on a date to be announced but at the latest by March 2010.

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The UK participation in its preparation was entrusted by Technical Committee B/525, Building and civil engineering structures, to Subcommittee B/525/2, Structural use of concrete, and Subcommittee B/525/10, Bridges.

A list of organizations represented on these subcommittees can be obtained on request to its secretary.

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

To enable EN 1992-2 to be used in the UK, the NDPs will be published in a National Annex, which will be made available by BSI in due course, after public consultation has taken place.

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

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

English Version

Eurocode 2 - Design of concrete structures - Concrete bridges - Design and detailing rules

Eurocode 2 - Calcul des structures en béton - Partie 2:
Ponts en béton - Calcul et dispositions constructives

Eurocode 2 - Planung von Stahlbeton- und
Spannbetontragwerken - Teil 2: Betonbrücken - Planungs-
und Ausführungsregeln

This European Standard was approved by CEN on 25 April 2005.

CEN members are bound to comply with the CEN/CENELEC Internal Regulations which stipulate the conditions for giving this European Standard the status of a national standard without any alteration. Up-to-date lists and bibliographical references concerning such national standards may be obtained on application to the Central Secretariat or to any CEN member.

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

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



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NOTE This contents list includes sections, clauses and annexes that have been introduced or modified in EN 1992-2.

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Foreword

This European Standard (EN 1992-2:2005) 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.

This European Standard shall be given the status of a national standard, either by publication of an identical text or by endorsement, at the latest by April 2006, and conflicting national standards shall be withdrawn at the latest by March 2010.

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

This Eurocode supersedes ENV 1992-2.

Background to the Eurocode programme

See EN 1992-1-1.

Status and field of application of Eurocodes

See EN 1992-1-1.

National Standards implementing Eurocodes

See EN 1992-1-1.

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

See EN 1992-1-1.

Additional information specific to EN 1992-2 and link to EN 1992-1-1

EN 1992-2 describes the principles and requirements for safety, serviceability and durability of concrete structures, together with specific provisions for bridges. It is based on the limit state concept used in conjunction with a partial factor method.

- EN 1992-2 gives Principles and Application Rules for the design of bridges in addition to those stated in EN 1992-1-1. All relevant clauses of EN 1992-1-1 are applicable to the design of bridges unless specifically deleted or varied by EN 1992-2. It has been appropriate to introduce in EN 1992-2 some material, in the form of new clauses or amplifications of clauses in EN 1992-1-1, which is not bridge specific and which strictly belongs to EN 1992-1-1. These new clauses and amplifications are deemed valid interpretations of EN 1992-1-1 and designs complying with the requirements of EN 1992-2 are deemed to comply with the Principles of EN 1992-1-1.

- clauses in EN 1992-2 that modify those in EN 1992-1-1 are numbered by adding '100' to the corresponding clause number in EN 1992-1-1.
- when additional clauses or sub-clauses are introduced in EN 1992-2, these are numbered by adding '101' to the last relevant clause or sub-clause in EN 1992-1-1.

For the design of new structures, EN 1992-2 is intended to be used, for direct application, together with other parts of EN 1992, Eurocodes EN 1990, 1991, 1997 and 1998.

EN 1992-2 also serves as a reference document for other CEN/TCs concerning structural matters.

EN 1992-2 is intended for use by:

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

Numerical values for partial factors and other reliability parameters are recommended as basic values that provide an acceptable level of reliability. They have been selected assuming that an appropriate level of workmanship and of quality management applies. When EN 1992-2 is used as a base document by other CEN/TCs the same values need to be taken.

National Annex for EN 1992-2

This standard gives values with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1992-2 should have a National Annex containing all Nationally Determined Parameters to be used for the design of bridges to be constructed in the relevant country.

National choice is allowed in EN 1992-2 through the following clauses:

3.1.2 (102)P	5.3.2.2 (104)	6.8.1 (102)	9.1 (103)
3.1.6 (101)P	5.5 (104)	6.8.7 (101)	9.2.2 (101)
3.1.6 (102)P	5.7 (105)	7.2 (102)	9.5.3 (101)
3.2.4 (101)P	6.1 (109)	7.3.1 (105)	9.7 (102)
4.2 (105)	6.1 (110)	7.3.3 (101)	9.8.1 (103)
4.2 (106)	6.2.2 (101)	7.3.4 (101)	11.9 (101)
4.4.1.2 (109)	6.2.3 (103)	8.9.1 (101)	113.2 (102)
5.1.3 (101)P	6.2.3 (107)	8.10.4 (105)	113.3.2 (103)
5.2 (105)	6.2.3 (109)	8.10.4 (107)	

Where references to National Authorities is made in this standard, the term should be defined in a Country's National Annex.

SECTION 1 General

The following clauses of EN 1992-1-1 apply.

1.1.1 (1)P	1.1.2 (3)P	1.2.2	1.5.2.1
1.1.1 (2)P	1.1.2 (4)P	1.3 (1)P	1.5.2.2
1.1.1 (3)P	1.2 (1)P	1.4 (1)P	1.5.2.3
1.1.1 (4)P	1.2.1	1.5.1 (1)P	1.5.2.4

1.1 Scope

1.1.2 Scope of Part 2 of Eurocode 2

(101)P Part 2 of Eurocode 2 gives a basis for the design of bridges and parts of bridges in plain, reinforced and prestressed concrete made with normal and light weight aggregates.

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

Section 1:	General
Section 2:	Basis of design
Section 3:	Materials
Section 4:	Durability and cover to reinforcement
Section 5:	Structural analysis
Section 6:	Ultimate limit states
Section 7:	Serviceability limit states
Section 8:	Detailing of reinforcement and prestressing tendons — General
Section 9:	Detailing of members and particular rules
Section 10:	Additional rules for precast concrete elements and structures
Section 11:	Lightweight aggregate concrete structures
Section 12:	Plain and lightly reinforced concrete structures
Section 113:	Design for the execution stages

1.106 Symbols

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

NOTE The notation used is based on ISO 3898:1987. Symbols with unique meanings have been used as far as possible. However, in some instances a symbol may have more than one meaning depending on the context.

Latin upper case letters

A	Accidental action
A	Cross sectional area
A_c	Cross sectional area of concrete
A_{ct}	Area of concrete in tensile zone
A_p	Area of a prestressing tendon or tendons
A_s	Cross sectional area of reinforcement
$A_{s,min}$	minimum cross sectional area of reinforcement

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

T_{Ed}	Design value of the applied torsional moment
ULS	Ultimate limit state
V	Shear force
V_{Ed}	Design value of the applied shear force
Vol	Volume of traffic
X	Advisory limit on percentage of coupled tendons at a section

Latin lower case letters

a	Distance
a	Geometrical data
Δa	Deviation for geometrical data
b	Overall width of a cross-section, or actual flange width in a T or L beam
b_w	Width of the web on T, I or L beams
c_{min}	Minimum cover
d	Diameter; Depth
d	Effective depth of a cross-section
d_g	Largest nominal maximum aggregate size
e	Eccentricity
f	Frequency
f_c	Compressive strength of concrete
f_{cd}	Design value of concrete compressive strength
f_{ck}	Characteristic compressive cylinder strength of concrete at 28 days
f_{cm}	Mean value of concrete cylinder compressive strength
f_{ctb}	Tensile strength prior to cracking in biaxial state of stress
f_{ctk}	Characteristic axial tensile strength of concrete
f_{ctm}	Mean value of axial tensile strength of concrete
f_{ctx}	Appropriate tensile strength for evaluation of cracking bending moment
f_p	Tensile strength of prestressing steel
f_{pk}	Characteristic tensile strength of prestressing steel
$f_{p0,1}$	0,1% proof-stress of prestressing steel
$f_{p0,1k}$	Characteristic 0,1 % proof-stress of prestressing steel
$f_{0,2k}$	Characteristic 0,2 % proof-stress of reinforcement
f_t	Tensile strength of reinforcement
f_{tk}	Characteristic tensile strength of reinforcement
f_y	Yield strength of reinforcement
f_{yd}	Design yield strength of reinforcement

f_{yk}	Characteristic yield strength of reinforcement
f_{ywd}	Design yield of shear reinforcement
h	Height
h	Overall depth of a cross-section
i	Radius of gyration
k	Coefficient; Factor
l	Length, span or height
m	Mass or slab components
n	Plate components
q_{ud}	Maximum value of combination reached in non linear analysis
r	Radius or correcting factor for prestress
$1/r$	Curvature at a particular section
s	Spacing between cracks
t	Thickness
t	Time being considered
t_0	The age of concrete at the time of loading
u	Perimeter of concrete cross-section, having area A_c
u	Component of the displacement of a point
v	Component of the displacement of a point or transverse shear
w	Component of the displacement of a point or crack width
x	Neutral axis depth
x, y, z	Coordinates
x_u	Neutral axis depth at ULS after redistribution
z	Lever arm of internal forces

Greek upper case letters

Φ	Dynamic factor according to EN 1991-2
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Greek lower case letters

α	Angle; Ratio; Long term effects coefficient or ratio between principal stresses
α_e	E_s/E_{cm} ratio
α_h	Reduction factor for θ_1
β	Angle ; Ratio; Coefficient
γ	Partial factor
γ_A	Partial factor for accidental actions A
γ_C	Partial factor for concrete
γ_F	Partial factor for actions, F
$\gamma_{F, fat}$	Partial factor for fatigue actions

$\gamma_{C,fat}$	Partial factor for fatigue of concrete
γ_O	Overall factor
γ_G	Partial factor for permanent actions, G
γ_M	Partial factor for a material property, taking account of uncertainties in the material property itself, in geometric deviation and in the design model used
γ_P	Partial factor for actions associated with prestressing, P
γ_Q	Partial factor for variable actions, Q
γ_S	Partial factor for reinforcing or prestressing steel
$\gamma_{S,fat}$	Partial factor for reinforcing or prestressing steel under fatigue loading
γ_f	Partial factor for actions without taking account of model uncertainties
γ_g	Partial factor for permanent actions without taking account of model uncertainties
γ_m	Partial factors for a material property, taking account only of uncertainties in the material property
δ	Increment/redistribution ratio
ξ	Creep redistribution function or bond strength ratio
ζ	Reduction factor/distribution coefficient
ε_c	Compressive strain in the concrete
ε_{ca}	Autogeneous shrinkage
ε_{cc}	Creep strain
ε_{cd}	Desiccation shrinkage
ε_{c1}	Compressive strain in the concrete at the peak stress f_c
ε_{cu}	Ultimate compressive strain in the concrete
ε_u	Strain of reinforcement or prestressing steel at maximum load
ε_{uk}	Characteristic strain of reinforcement or prestressing steel at maximum load
θ	Angle
θ_1	Inclination for geometric imperfections
λ	Slenderness ratio or damage equivalent factors in fatigue
μ	Coefficient of friction between the tendons and their ducts
ν	Poisson's ratio
ν	Strength reduction factor for concrete cracked in shear
ρ	Oven-dry density of concrete in kg/m^3
$\rho_{1\,000}$	Value of relaxation loss (in %), at 1 000 hours after tensioning and at a mean temperature of 20 °C
ρ_l	Reinforcement ratio for longitudinal reinforcement
ρ_w	Reinforcement ratio for shear reinforcement
σ_c	Compressive stress in the concrete

σ_{cp}	Compressive stress in the concrete from axial load or prestressing
σ_{cu}	Compressive stress in the concrete at the ultimate compressive strain ϵ_{cu}
τ	Torsional shear stress
ϕ	Diameter of a reinforcing bar or of a prestressing duct
ϕ_n	Equivalent diameter of a bundle of reinforcing bars
$\phi(t, t_0)$	Creep coefficient, defining creep between times t and t_0 , related to elastic deformation at 28 days
ϕ_{fat}	Damage equivalent impact factor in fatigue
$\phi(\infty, t_0)$	Final value of creep coefficient
ψ	Factors defining representative values of variable actions
ψ_0	for combination values
ψ_1	for frequent values
ψ_2	for quasi-permanent values
χ	Ageing coefficient

SECTION 2 Basis of Design

All the clauses of EN 1992-1-1 apply.

SECTION 3 Materials

The following clauses of EN 1992-1-1 apply.

3.1.1 (1)P	3.1.8 (1)	3.3.1 (1)P	3.3.4 (5)
3.1.1 (2)	3.1.9 (1)	3.3.1 (2)P	3.3.5 (1)P
3.1.2 (1)P	3.1.9 (2)	3.3.1 (3)	3.3.5 (2)P
3.1.2 (3)	3.2.1 (1)P	3.3.1 (4)	3.3.6 (1)P
3.1.2 (4)	3.2.1 (2)P	3.3.1 (5)P	3.3.6 (2)
3.1.2 (5)	3.2.1 (3)P	3.3.1 (6)	3.3.6 (3)
3.1.2 (6)	3.2.1 (4)P	3.3.1 (7)P	3.3.6 (4)
3.1.2 (7)P	3.2.1 (5)	3.3.1 (8)P	3.3.6 (5)
3.1.2 (8)	3.2.2 (1)P	3.3.1 (9)P	3.3.6 (6)
3.1.2 (9)	3.2.2 (2)P	3.3.1 (10)P	3.3.6 (7)
3.1.3 (1)	3.2.2 (3)P	3.3.1 (11)P	3.3.7 (1)P
3.1.3 (2)	3.2.2 (4)P	3.3.2 (1)P	3.3.7 (2)P
3.1.3 (3)	3.2.2 (5)	3.3.2 (2)P	3.4.1.1 (1)P
3.1.3 (4)	3.2.2 (6)P	3.3.2 (3)P	3.4.1.1 (2)P
3.1.3 (5)	3.2.3 (1)P	3.3.2 (4)P	3.4.1.1 (3)P
3.1.4 (1)P	3.2.4 (2)	3.3.2 (5)	3.4.1.2.1 (1)P
3.1.4 (2)	3.2.5 (1)P	3.3.2 (6)	3.4.1.2.1 (2)
3.1.4 (3)	3.2.5 (2)P	3.3.2 (7)	3.4.1.2.2 (1)P
3.1.4 (4)	3.2.5 (3)P	3.3.2 (8)	3.4.2.1 (1)P
3.1.4 (5)	3.2.5 (4)	3.3.2 (9)	3.4.2.1 (2)P
3.1.4 (6)	3.2.6 (1)P	3.3.3 (1)P	3.4.2.1 (3)
3.1.5 (1)	3.2.7 (1)	3.3.4 (1)P	3.4.2.2 (1)
3.1.7 (1)	3.2.7 (2)	3.3.4 (2)	
3.1.7 (2)	3.2.7 (3)	3.3.4 (3)	
3.1.7 (3)	3.2.7 (4)	3.3.4 (4)	

3.1 Concrete

3.1.2 Strength

(102)P The strength classes (C) in this code are denoted by the characteristic cylinder strength f_{ck} determined at 28 days with a minimum value of C_{min} and a maximum value of C_{max} .

NOTE The values of C_{min} and C_{max} for use in a Country may be found in its National Annex. The recommended values are C30/37 and C70/85 respectively.

3.1.6 Design compressive and tensile strengths

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

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

where:

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

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

NOTE The value of α_{cc} for use in a Country should lie between 0,80 and 1,00 and may be found in its National Annex. The recommended value of α_{cc} is 0,85.

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

$$f_{ctd} = \alpha_{ct} f_{ctk,0,05} / \gamma_C$$

where:

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

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

NOTE The value of α_{ct} for use in a Country should lie between 0,80 and 1,00 and may be found in its National Annex. The recommended value of α_{ct} is 1,0.

3.2 Reinforcing steel

3.2.4 Ductility characteristics

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

NOTE The classes of reinforcement to be used in bridges in a Country may be found in its National Annex. The recommended classes are Class B and Class C.

SECTION 4 Durability and cover to reinforcement

The following clauses of EN 1992-1-1 apply.

4.1 (1) <i>P</i>	4.2 (3)	4.4.1.2 (4)	4.4.1.2 (13)
4.1 (2) <i>P</i>	4.3 (1) <i>P</i>	4.4.1.2 (5)	4.4.1.3 (1) <i>P</i>
4.1 (3) <i>P</i>	4.3 (2) <i>P</i>	4.4.1.2 (6)	4.4.1.3 (2)
4.1 (4)	4.4.1.1 (1) <i>P</i>	4.4.1.2 (7)	4.4.1.3 (3)
4.1 (5)	4.4.1.1 (2) <i>P</i>	4.4.1.2 (8)	4.4.1.3 (4)
4.1 (6)	4.4.1.2 (1) <i>P</i>	4.4.1.2 (10)	
4.2 (1) <i>P</i>	4.4.1.2 (2) <i>P</i>	4.4.1.2 (11)	
4.2 (2)	4.4.1.2 (3)	4.4.1.2 (12)	

4.2 Environmental conditions

(104) Water penetration or the possibility of leakage from the carriageway into the inside of voided structures should be considered in the design.

(105) For a concrete surface protected by waterproofing the exposure class should be given in a Country's National Annex.

NOTE For surfaces protected by waterproofing the exposure class for use in a Country may be found in its National Annex. The recommended exposure class for surfaces protected by waterproofing is XC3.

(106) Where de-icing salt is used, all exposed concrete surfaces within x m of the carriageway horizontally or within y m above the carriageway should be considered as being directly affected by de-icing salts. Top surfaces of supports under expansion joints should also be considered as being directly affected by de-icing salts.

NOTE 1 The distances x and y for use in a Country may be found in its National Annex. The recommended value for x is 6m and the recommended value for y is 6m.

NOTE 2 The exposure classes for surfaces directly affected by de-icing salts for use in a Country may be found in its National Annex. The recommended classes for surfaces directly affected by de-icing salts are XD3 and XF2 or XF4, as appropriate, with covers given in Tables 4.4N and 4.5N for XD classes.

4.3 Requirements for durability

(103) External prestressing tendons should comply with the requirements of National Authorities.

4.4 Methods of verifications

4.4.1 Concrete cover

4.4.1.2 Minimum cover, $c_{\min}$

(109) Where in-situ concrete is placed against an existing concrete surface (precast or in-situ) the requirements for cover to the reinforcement from the interface may be modified.

NOTE The requirements for use in a Country may be found in its National Annex.

The recommended requirement is that, provided the following conditions are met, the cover needs only satisfy the requirements for bond (see 4.4.1.2 (3) of EN 1992-1-1):

- the existing concrete surface has not been subject to an outdoor environment for more than 28 days.
- the existing concrete surface is rough.
- the strength class of the existing concrete is at least C25/30.

(114) Bare concrete decks of road bridges, without waterproofing or surfacing, should be classified as Abrasion Class XM2.

(115) Where a concrete surface is subject to abrasion caused by ice or solid transportation in running water the cover should be increased by a minimum of 10 mm.

SECTION 5 Structural analysis

The following clauses of EN 1992-1-1 apply.

5.1.1 (1) <i>P</i>	5.6.1 (3) <i>P</i>	5.8.5 (2)	5.10.1 (3)
5.1.1 (2)	5.6.1 (4)	5.8.5 (3)	5.10.1 (4)
5.1.1 (3)	5.6.2 (1) <i>P</i>	5.8.5 (4)	5.10.1 (5) <i>P</i>
5.1.1 (4) <i>P</i>	5.6.2 (3)	5.8.6 (1) <i>P</i>	5.10.2.1 (1) <i>P</i>
5.1.1 (5)	5.6.2 (4)	5.8.6 (2) <i>P</i>	5.10.2.1 (2)
5.1.1 (6) <i>P</i>	5.6.2 (5)	5.8.6 (3)	5.10.2.2 (1) <i>P</i>
5.1.1 (7)	5.6.3 (1)	5.8.6 (4)	5.10.2.2 (2) <i>P</i>
5.1.2 (1) <i>P</i>	5.6.3 (3)	5.8.6 (5)	5.10.2.2 (3) <i>P</i>
5.1.2 (2)	5.6.3 (4)	5.8.6 (6)	5.10.2.2 (4)
5.1.2 (3)	5.6.4 (1)	5.8.7.1 (1)	5.10.2.2 (5)
5.1.2 (4)	5.6.4 (2)	5.8.7.1 (2)	5.10.2.3 (1) <i>P</i>
5.1.2 (5)	5.6.4 (3)	5.8.7.2 (1)	5.10.3 (1) <i>P</i>
5.1.4 (1) <i>P</i>	5.6.4 (4)	5.8.7.2 (2)	5.10.3 (2)
5.1.4 (2)	5.6.4 (5)	5.8.7.2 (3)	5.10.3 (3)
5.1.4 (3)	5.7 (1)	5.8.7.2 (4)	5.10.3 (4)
5.2 (1) <i>P</i>	5.7 (2)	5.8.7.3 (1)	5.10.4 (1)
5.2 (2) <i>P</i>	5.7 (3)	5.8.7.3 (2)	5.10.5.1 (1)
5.2 (3)	5.7 (4) <i>P</i>	5.8.7.3 (3)	5.10.5.1 (2)
5.2 (7)	5.8.1	5.8.7.3 (4)	5.10.5.2 (1)
5.3.1 (1) <i>P</i>	5.8.2 (1) <i>P</i>	5.8.8.1 (1)	5.10.5.2 (2)
5.3.1 (3)	5.8.2 (2) <i>P</i>	5.8.8.1 (2)	5.10.5.2 (3)
5.3.1 (4)	5.8.2 (3) <i>P</i>	5.8.8.2 (1)	5.10.5.2 (4)
5.3.1 (5)	5.8.2 (4) <i>P</i>	5.8.8.2 (2)	5.10.5.3 (1)
5.3.1 (7)	5.8.2 (5) <i>P</i>	5.8.8.2 (3)	5.10.5.3 (2)
5.3.2.1 (1) <i>P</i>	5.8.2 (6)	5.8.8.2 (4)	5.10.6 (1)
5.3.2.1 (2)	5.8.3.1 (1)	5.8.8.3 (1)	5.10.6 (2)
5.3.2.1 (3)	5.8.3.1 (2)	5.8.8.3 (2)	5.10.6 (3)
5.3.2.1 (4)	5.8.3.2 (1)	5.8.8.3 (3)	5.10.7 (1)
5.3.2.2 (1)	5.8.3.2 (2)	5.8.8.3 (4)	5.10.7 (2)
5.3.2.2 (2)	5.8.3.2 (3)	5.8.9 (1)	5.10.7 (3)
5.3.2.2 (3)	5.8.3.2 (4)	5.8.9 (2)	5.10.7 (4)
5.4 (1)	5.8.3.2 (5)	5.8.9 (3)	5.10.7 (5)
5.4 (2)	5.8.3.2 (6)	5.8.9 (4)	5.10.7 (6)
5.4 (3)	5.8.3.2 (7)	5.9 (1) <i>P</i>	5.10.8 (1)
5.5 (1) <i>P</i>	5.8.4 (1) <i>P</i>	5.9 (2)	5.10.8 (2)
5.5 (2)	5.8.4 (2)	5.9 (3)	5.10.9 (1) <i>P</i>
5.5 (3)	5.8.4 (3)	5.9 (4)	5.11 (1) <i>P</i>
5.5 (6)	5.8.4 (4)	5.10.1 (1) <i>P</i>	5.11 (2) <i>P</i>
5.6.1 (2) <i>P</i>	5.8.5 (1)	5.10.1 (2)	

5.1 General

5.1.1 General requirements

(108) For the analysis of time dependent effects in bridges, recognised design methods may be applied.

NOTE Further information may be found in Annex KK.

5.1.3 Load cases and combinations

(101)P In considering the combinations of actions (see Section 6 and Annex A2 of EN 1990) the relevant load cases shall be considered to enable the critical design conditions to be established at all sections, within the structure or part of the structure considered.

NOTE Simplifications to the load arrangements for use in a Country may be found in its National Annex. Recommendations on simplifications are not given in this standard.

5.2 Geometric imperfections

(104) The provisions of (105) and (106) of this Part and (7) of EN 1992-1-1 apply to members with axial compression and structures with vertical load. Numerical values are related to normal execution deviations (Class 1 in EN 13670). Where other execution deviations apply numerical values should be adjusted accordingly.

(105) Imperfections may be represented by an inclination, θ_1 , given by

$$\theta_1 = \theta_0 \alpha_h \quad (5.101)$$

where

θ_0 is the basic value

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

l is the length or height [m]

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

(106) For arch bridges, the shape of imperfections in the horizontal and vertical planes should be based on the shape of the first horizontal and vertical buckling mode shape respectively. Each mode shape may be idealised by a sinusoidal profile. The amplitude should be taken as $a = \theta_1 \frac{l}{2}$, where l is the half wavelength.

(8) and (9) of EN 1992-1-1 do not apply.

5.3 Idealisation of the structure

5.3.1 Structural models for overall analysis

(2) and (6) of EN 1992-1-1 do not apply

5.3.2 Geometric data

5.3.2.2 Effective span of beams and slabs

NOTE (1), (2) and (3) of EN 1992-1-1 apply despite the fact that the title of the clause refers to buildings.

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

$$\Delta M_{Ed} = F_{Ed,sup} t/8 \quad (5.9)$$

where:

$F_{Ed,sup}$ is the design support reaction

NOTE The value of t for use in a Country may be found in its National Annex. The recommended value is the breadth of the bearing.

5.5 Linear elastic analysis with limited redistribution

(104) In continuous beams or slabs which:

- a) are predominantly subject to flexure and
- b) have the ratio of the lengths of adjacent spans in the range of 0,5 to 2

redistribution of bending moments may be carried out without explicit check on the rotation capacity, provided that:

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

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

$$\delta \geq k_5 \text{ where Class B and Class C reinforcement is used (see Annex C)}$$

No redistribution is allowed for Class A steel (see Annex C)

where:

δ is the ratio of the redistributed moment to the elastic bending moment

x_u is the depth of the neutral axis at the ultimate limit state after redistribution

d is the effective depth of the section

NOTE 1 The values of k_1 , k_2 , k_3 , k_4 and k_5 for use in a Country may be found in its National Annex. The recommended value for k_1 is 0,44, for k_2 is $1,25(0,6+0,0014/\varepsilon_{cu2})$, for k_3 is 0,54, for k_4 is $1,25(0,6+0,0014/\varepsilon_{cu2})$ and for k_5 is 0,85.

NOTE 2 The limits of EN 1992-1-1 may be used for the design of solid slabs.

(105) Redistribution should not be carried out in circumstances where the rotation capacity cannot be defined with confidence (e.g. in curved and or skewed bridges).

5.6 Plastic analysis

5.6.1 General

(101)P Methods based on plastic analysis should only be used for the check at ULS and only when permitted by National Authorities.

5.6.2 Plastic analysis for beams, frames and slabs

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

- i) the area of tensile reinforcement is limited such that, at any section

$$x_u/d \leq 0,15 \text{ for concrete strength classes } \leq C50/60$$

$$\leq 0,10 \text{ for concrete strength classes } \geq C55/67$$
- ii) reinforcing steel is either Class B or C
- iii) the ratio of the moments at intermediate supports to the moments in the span is between 0,5 and 2.

NOTE The limits of EN 1992-1-1 may be used for the design of solid slabs.

5.6.3 Rotation capacity

(102) In regions of yield hinges, x_u/d should not exceed 0,30 for concrete strength classes less than or equal to C50/60, and 0,23 for concrete strength classes greater than or equal to C55/67.

5.7 Non-linear analysis

(105) Non-linear analysis may be used provided that the model can appropriately cover all failure modes (e.g. bending, axial force, shear, compression failure affected by reduced effective concrete strength, etc.) and that the concrete tensile strength is not utilised as a primary load resisting mechanism.

If one analysis is not sufficient to verify all the failure mechanisms, separate additional analyses should be carried out.

NOTE 1 The details of acceptable methods for non-linear analysis and safety format to be used in a Country may be found in its National Annex. The recommended details are as follows:

When using non-linear analysis the following assumptions should be made:

- For reinforcing steel, the stress-strain diagram to be used should be based on Figure 3.8, curve A. In this diagram, f_{yk} and kf_{yk} should be replaced by $1,1f_{yk}$ and $1,1kf_{yk}$
- For prestressing steel, the idealised stress-strain diagram given in 3.3.6 (Figure 3.10, curve A) should be used. In this diagram f_{pk} should be replaced with $1,1f_{pk}$
- For concrete, the stress-strain diagram should be based on expression (3.14) in 3.1.5. In this expression, and in the k -value, f_{cm} should be replaced by $\gamma_{cf}f_{ck}$ with $\gamma_{cf} = 1,1 \cdot \gamma_s/\gamma_c$

The following design format should be used:

- The resistance should be evaluated for different levels of appropriate actions which should be increased from their serviceability values by incremental steps, such that the value of $\gamma_G G_k$ and $\gamma_Q Q_k$ are reached in the same step. The incrementing process should be continued until one region of the structure attains the ultimate strength, evaluated taking account of α_{cc} , or there is global failure of the structure. The corresponding load is referred to as q_{ud} .
- Apply an overall safety factor γ_O and obtain the corresponding strength $R\left(\frac{q_{ud}}{\gamma_O}\right)$,
- One of the following inequalities should be satisfied:

$$\gamma_{Rd}E(\gamma_G G + \gamma_Q Q) \leq R\left(\frac{q_{ud}}{\gamma_O}\right) \quad (5.102 \text{ aN})$$

or

$$E(\gamma_G G + \gamma_Q Q) \leq R \left(\frac{q_{ud}}{\gamma_{Rd} \cdot \gamma_O} \right) \quad (5.102 \text{ bN})$$

$$(\text{i.e.}) \quad R \left(\frac{q_{ud}}{\gamma_O} \right)$$

or

$$\gamma_{Rd} \gamma_{Sd} E(\gamma_g G + \gamma_q Q) \leq R \left(\frac{q_{ud}}{\gamma_O} \right) \quad (5.102 \text{ cN})$$

where:

γ_{Rd} is the partial factor for model uncertainty for resistance, $\gamma_{Rd} = 1,06$,

γ_{Sd} is the partial factor for model uncertainty for action/action effort, $\gamma_{Sd} = 1,15$,

γ_O is the overall safety factor, $\gamma_O = 1,20$.

Refer to Annex PP for further details.

When model uncertainties γ_{Rd} and γ_{Sd} are not considered explicitly in the analysis (i.e. $\gamma_{Rd} = \gamma_{Sd} = 1$), $\gamma_O = 1,27$ should be used.

NOTE 2 If design properties of materials (e.g. as 5.8.6 of EN 1992-1-1) are used for non-linear analysis particular care should be exercised to allow for the effects of indirect actions (e.g. imposed deformations).

5.8 Analysis of second order effects with axial load

5.8.3 Simplified criteria for second order effects

5.8.3.3 Global second order effects in buildings

This clause does not apply

5.8.4 Creep

(105) A more refined approach to the evaluation of creep may be applied.

NOTE Further information may be found in Annex KK

5.10 Prestressed members and structures

5.10.1 General

(106) Brittle failure should be avoided using the method described in 6.1 (109).

5.10.8 Effects of prestressing at ultimate limit state

(103) If the stress increase in external tendons is calculated using the deformation state of the overall member non-linear analysis should be used. See 5.7.

SECTION 6 Ultimate Limit States (ULS)

The following clauses of EN 1992-1-1 apply.

6.1 (1) <i>P</i>	6.2.4 (6)	6.4.3 (1) <i>P</i>	6.5.4 (9)
6.1 (2) <i>P</i>	6.2.4 (7)	6.4.3 (2)	6.6 (1) <i>P</i>
6.1 (3) <i>P</i>	6.2.5 (1)	6.4.3 (3)	6.6 (2)
6.1 (4)	6.2.5 (2)	6.4.3 (4)	6.6 (3)
6.1 (5)	6.2.5 (3)	6.4.3 (5)	6.7 (1) <i>P</i>
6.1 (6)	6.2.5 (4)	6.4.3 (6)	6.7 (2)
6.1 (7)	6.3.1 (1) <i>P</i>	6.4.3 (7)	6.7 (3)
6.2.1 (1) <i>P</i>	6.3.1 (2)	6.4.3 (8)	6.7 (4)
6.2.1 (2)	6.3.1 (3)	6.4.3 (9)	6.8.1 (1) <i>P</i>
6.2.1 (3)	6.3.1 (4)	6.4.4 (1)	6.8.2 (1) <i>P</i>
6.2.1 (4)	6.3.1 (5)	6.4.4 (2)	6.8.2 (2) <i>P</i>
6.2.1 (5)	6.3.2 (1)	6.4.5 (1)	6.8.2 (3)
6.2.1 (6)	6.3.2 (5)	6.4.5 (2)	6.8.3 (1) <i>P</i>
6.2.1 (7)	6.3.3 (1)	6.4.5 (3)	6.8.3 (2) <i>P</i>
6.2.1 (8)	6.3.3 (2)	6.4.5 (4)	6.8.3 (3) <i>P</i>
6.2.1 (9)	6.4.1 (1) <i>P</i>	6.4.5 (5)	6.8.4 (1)
6.2.2 (2)	6.4.1 (2) <i>P</i>	6.5.1 (1) <i>P</i>	6.8.4 (2)
6.2.2 (3)	6.4.1 (3)	6.5.2 (1)	6.8.4 (3) <i>P</i>
6.2.2 (4)	6.4.1 (4)	6.5.2 (2)	6.8.4 (4)
6.2.2 (5)	6.4.1 (5)	6.5.2 (3)	6.8.4 (5)
6.2.2 (6)	6.4.2 (1)	6.5.3 (1)	6.8.4 (6) <i>P</i>
6.2.2 (7)	6.4.2 (2)	6.5.3 (2)	6.8.5 (1) <i>P</i>
6.2.3 (1)	6.4.2 (3)	6.5.3 (3)	6.8.5 (2)
6.2.3 (2)	6.4.2 (4)	6.5.4 (1) <i>P</i>	6.8.5 (3)
6.2.3 (4)	6.4.2 (5)	6.5.4 (2) <i>P</i>	6.8.6 (1)
6.2.3 (5)	6.4.2 (6)	6.5.4 (3)	6.8.6 (2)
6.2.3 (6)	6.4.2 (7)	6.5.4 (4)	6.8.7 (2)
6.2.3 (8)	6.4.2 (8)	6.5.4 (5)	6.8.7 (3)
6.2.4 (1)	6.4.2 (9)	6.5.4 (6)	6.8.7 (4)
6.2.4 (2)	6.4.2 (10)	6.5.4 (7)	
6.2.4 (4)	6.4.2 (11)	6.5.4 (8)	

6.1 Bending with or without axial force

(108) For external prestressing tendons the strain in the prestressing steel between two consecutive fixed points is assumed to be constant. The strain in the prestressing steel is then equal to the remaining strain, after losses, increased by the strain resulting from the structural deformation between the fixed points considered.

(109) For prestressed structures, 5(P) of 5.10.1 may be satisfied by any of the following methods:

- a) Verifying the load capacity using a reduced area of prestress. This verification should be undertaken as follows:
 - i) Calculate the applied bending moment due to the frequent combination of actions.

- ii) Determine the reduced area of prestress that results in the tensile stress reaching f_{ctm} at the extreme tension fibre when the section is subject to the bending moment calculated in i) above.
 - iii) Using this reduced area of prestress, calculate the ultimate flexural capacity. It should be ensured that this exceeds the bending moment due to the frequent combination. Redistribution of internal actions within the structure may be taken into account for this verification and the ultimate bending resistance should be calculated using the material partial safety factors for accidental design situations given in Table 2.1N of 2.4.2.4.
- b) Providing a minimum reinforcing steel area according to the Expression (6.101a). Reinforcing steel provided for other purposes may be included in $A_{s,min}$.

$$A_{s,min} = \frac{M_{rep}}{z_s f_{yk}} \quad (6.101a)$$

where:

M_{rep} is the cracking bending moment calculated using an appropriate tensile strength, f_{ctx} at the extreme tension fibre of the section, ignoring any effect of prestressing. At the joint of segmental precast elements M_{rep} should be assumed to be zero.

z_s is the lever arm at the ultimate limit state related to the reinforcing steel.

NOTE The value of f_{ctx} for use in a Country may be found in its National Annex. The recommended value for f_{ctx} is f_{ctm} .

- c) Agreeing an appropriate inspection regime with the relevant National Authority on the basis of satisfactory evidence.

NOTE The applicable method or methods (selected from a, b and c) for use in a Country may be given in its National Annex.

(110) In cases where method b) in (109) above is chosen, the following rules apply:

- i) The minimum reinforcement steel area should be provided in regions where tensile stresses occur in the concrete under the characteristic combination of actions. In this check the parasitic effects of prestressing should be considered and the primary effects should be ignored.
- ii) For pretensioned members Expression (6.101a) should be applied using one of the alternative approaches a) or b) described below:
 - a) Tendons with concrete cover at least k_{cm} times the minimum specified in 4.4.1.2 are considered as effective in $A_{s,min}$. A value of z_s based on the effective strands is used in the expression and f_{yk} is replaced with $f_{p0,1k}$.
 - b) Tendons subject to stresses lower than $0,6 f_{pk}$ after losses under the characteristic combination of actions are considered as fully active. In this case Expression (6.101a) is replaced by:

$$A_{s,min} f_{yk} + A_p \Delta \sigma_p \geq \frac{M_{rep}}{z} \quad (6.101b)$$

where $\Delta \sigma_p$ is the smaller of $0,4 f_{ptk}$ and 500 MPa.

NOTE The value of k_{cm} for use in a Country may be found in its National Annex. The recommended value for k_{cm} is 2,0.

- iii) To ensure adequate ductility the minimum reinforcing steel area $A_{s,min}$, defined in Expressions (6.101), in continuous beams should extend to the intermediate support of the span considered.

However, this extension is not necessary if, at the ultimate limit state, the resisting tensile capacity provided by reinforcing and prestressing steel above the supports, calculated with the characteristic strength f_{yk} and $f_{p0,1k}$ respectively, is less than the resisting compressive capacity of the bottom flange, which means that the failure of the compressive zone is not likely to occur:

$$A_s f_{yk} + k_p A_p f_{p0,1k} < t_{inf} b_0 \alpha_{cc} f_{ck} \quad (6.102)$$

where:

t_{inf} , b_0 are, respectively, the thickness and the width of the bottom flange of the section. In case of T sections, t_{inf} is taken as equal to b_0 .

A_s , A_p denote respectively the area of reinforced and prestressing steel in the tensile zone at the ultimate limit state.

NOTE The value of k_p for use in a Country may be found in it's National Annex. The recommended value for k_p is 1,0.

6.2 Shear

6.2.2 Members not requiring design shear reinforcement

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

$$V_{Rd,c} = [C_{Rd,c} k (100 \rho_1 f_{ck})^{1/3} + k_1 \sigma_{cp}] b_w d \quad (6.2.a)$$

with a minimum of

$$V_{Rd,c} = (v_{min} + k_1 \sigma_{cp}) b_w d \quad (6.2.b)$$

where:

f_{ck} is in MPa

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

$$\rho_1 = \frac{A_{sl}}{b_w d} \leq 0,02$$

A_{sl} is the area of the tensile reinforcement, which extends $\geq (l_{bd} + d)$ beyond the section considered (see Figure 6.3); the area of bonded prestressing steel may be included in the calculation of A_{sl} . In this case a weighted mean value of d may be used.

b_w is the smallest width of the cross-section in the tensile area [mm]

$$\sigma_{cp} = N_{Ed}/A_c < 0,2 f_{cd} \text{ [MPa]}$$

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

A_C is the area of concrete cross section [mm²]

$V_{Rd,c}$ is Newtons.

NOTE The values of $C_{Rd,c}$, v_{min} and k_1 for use in a Country may be found in its National Annex. The recommended value for $C_{Rd,c}$ is $0,18/\gamma_c$, that for v_{min} is given by Expression (6.3N) and that for k_1 is 0,15.

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

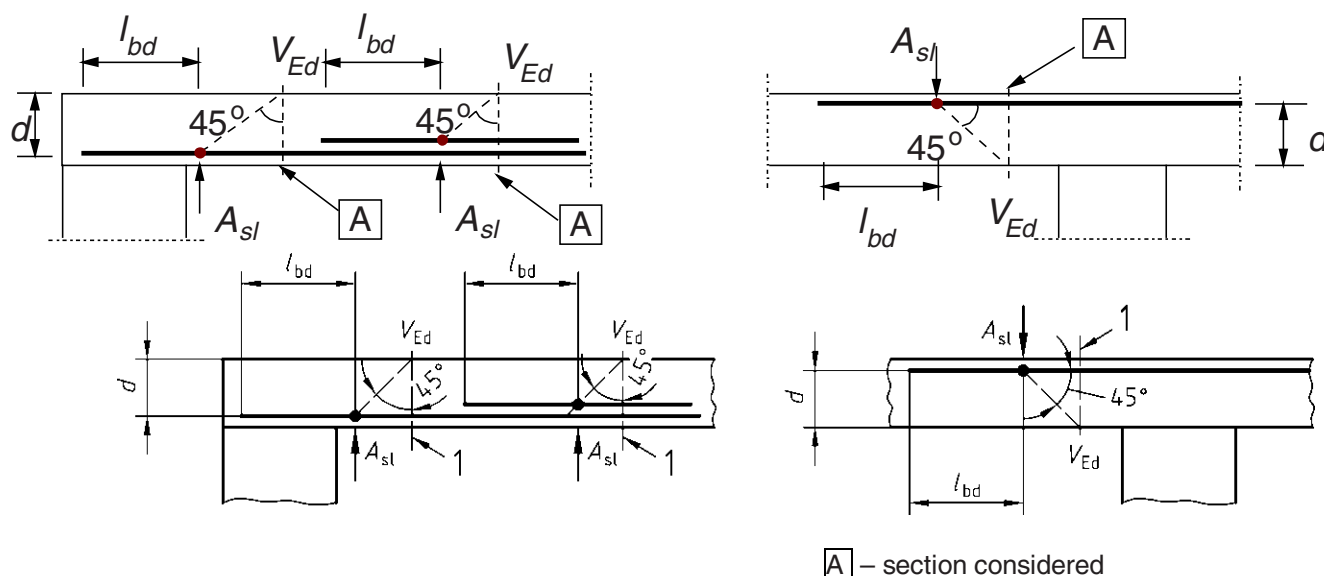


Figure 6.3 — Definition of A_{sl} in Expression (6.2)

6.2.3 Members requiring design shear reinforcement

(103) For members with vertical shear reinforcement, the shear resistance, V_{Rd} is the smaller value of:

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

NOTE 1 If Expression (6.10) is used the value of f_{ywd} should be reduced to $0,8 f_{ywk}$ in Expression (6.8)

and

$$V_{Rd,max} = \alpha_{cw} b_w z v_1 f_{cd} / (\cot \theta + \tan \theta) \quad (6.9)$$

where:

A_{sw} is the cross-sectional area of the shear reinforcement

s is the spacing of the stirrups

f_{ywd} is the design yield strength of the shear reinforcement

v_1 is a strength reduction factor for concrete cracked in shear

α_{cw} is a coefficient taking account of the state of the stress in the compression chord

NOTE 2 The value of ν_1 and α_{cw} for use in a Country may be found in its National Annex. The recommended value of ν_1 is ν . (See Expression (6.6N)).

NOTE 3 If the design stress of the shear reinforcement is below 80% of the characteristic yield stress f_{yk} , ν_1 may be taken as:

$$\nu_1 = 0,6 \quad \text{for } f_{ck} \leq 60 \text{ MPa} \quad (6.10.aN)$$

$$\nu_1 = 0,9 - f_{ck}/200 > 0,5 \quad \text{for } f_{ck} \geq 60 \text{ MPa} \quad (6.10.bN)$$

NOTE 4 The recommended value of α_{cw} is as follows:

1 for non-prestressed structures

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

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

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

where:

σ_{cp} is the mean compressive stress, measured positive, in the concrete due to the design axial force. This should be obtained by averaging it over the concrete section taking account of the reinforcement. The value of σ_{cp} need not be calculated at a distance less than $0,5d \cot \theta$ from the edge of the support.

In the case of straight tendons, a high level of prestress ($\sigma_{cp}/f_{cd} > 0,5$) and thin webs, if the tension and the compression chords are able to carry the whole prestressing force and blocks are provided at the extremity of beams to disperse the prestressing force (see fig. 6.101), it may be assumed that the prestressing force is distributed between the chords. In these circumstances, the compression field due to shear only should be considered in the web ($\alpha_{cw} = 1$).

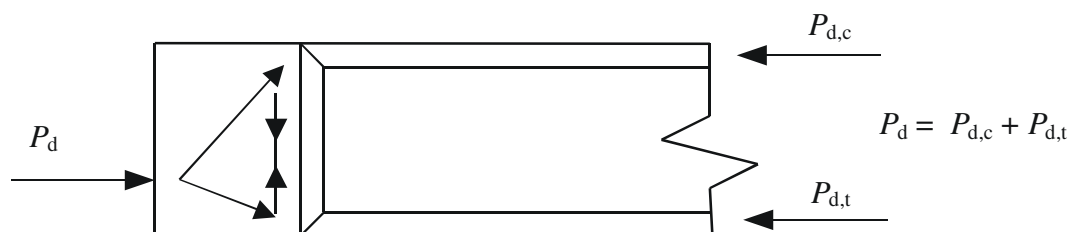


Figure 6.101 — Dispersion of prestressing by end blocks within the chords

NOTE 5 The maximum effective cross-sectional area of the shear reinforcement $A_{sw,max}$ for $\cot \theta = 1$ is given by:

$$\frac{A_{sw,max} f_{ywd}}{b_w s} \leq \frac{1}{2} \alpha_{cw} \nu_1 f_{cd} \quad (6.12)$$

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

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

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

NOTE Guidance on the superposition of different truss models for use in a Country may be found in its National Annex. The recommended guidance is as follows: