BS EN 1992-2:2005
Incorporating corrigendum July 2008

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 It supersedes DD ENV 1992-2:2001 which is withdrawn.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 The conflicting national standards will be withdrawn at the end of the coexistence period, after all the EN Eurocodes of a package are available.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At the end of this coexistence period, the conflicting parts of national standards will be withdrawn.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and based on this transition period, these standards will be withdrawn/revised on a date to be announced but at the latest by March 2010.

This British Standard was published under the authority of the Standards Policy and Strategy Committee on 2 December 2005

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Date	Comments
28 February 2010	Implementation of CEN corrigendum July 2008

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.

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EUROPEAN STANDARD

NORME EUROPÉENNE

EUROPÄISCHE NORM

EN 1992-2

October 2005

ICS 93.040; 91.010.30; 91.080.40

Supersedes ENV 1992-2:1996
Incorporating corrigendum July 2008

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
$f_{\mathrm{p0,1}}$	0,1% proof-stress of prestressing steel
$f_{\mathrm{p0,1k}}$	Characteristic 0,1 % proof-stress of prestressing steel
$f_{\mathrm{0,2k}}$	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

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_l
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_l	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
$\varphi(t,t_0)$	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
$\varphi(\infty,t_0)$	Final value of creep coefficient
ψ	Factors defining representative values of variable actions
ψ_0	for combination values
ψ_1	for frequent values
ψ_2	for quasi-permanent values
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
3.1.1 (2)
3.1.2 (1)P
3.1.2 (3)
3.1.2 (4)
3.1.2 (5)
3.1.2 (6)
3.1.2 (7)P
3.1.2 (8)
3.1.2 (9)
3.1.3 (1)
3.1.3 (2)
3.1.3 (3)
3.1.3 (4)
3.1.3 (5)
3.1.4 (1)P
3.1.4 (2)
3.1.4 (3)
3.1.4 (4)
3.1.4 (5)
3.1.4 (6)
3.1.5 (1)
3.1.7 (1)
3.1.7 (2)
3.1.7 (3)
3.1.8 (1)
3.1.9 (1)
3.1.9 (2)
3.2.1 (1)P
3.2.1 (2)P
3.2.1 (3)P
3.2.1 (4)P
3.2.1 (5)
3.2.2 (1)P
3.2.2 (2)P
3.2.2 (3)P
3.2.2 (4)P
3.2.2 (5)
3.2.2 (6)P
3.2.3 (1)P
3.2.4 (2)
3.2.5 (1)P
3.2.5 (2)P
3.2.5 (3)P
3.2.5 (4)
3.2.6 (1)P
3.2.7 (1)
3.2.7 (2)
3.2.7 (3)
3.2.7 (4)
3.3.1 (1)P
3.3.1 (2)P
3.3.1 (3)
3.3.1 (4)
3.3.1 (5)P
3.3.1 (6)
3.3.1 (7)P
3.3.1 (8)P
3.3.1 (9)P
3.3.1 (10)P
3.3.1 (11)P
3.3.2 (1)P
3.3.2 (2)P
3.3.2 (3)P
3.3.2 (4)P
3.3.2 (5)
3.3.2 (6)
3.3.2 (7)
3.3.2 (8)
3.3.2 (9)
3.3.3 (1)P
3.3.4 (1)P
3.3.4 (2)
3.3.4 (3)
3.3.4 (4)
3.3.4 (5)
3.3.5 (1)P
3.3.5 (2)P
3.3.6 (1)P
3.3.6 (2)
3.3.6 (3)
3.3.6 (4)
3.3.6 (5)
3.3.6 (6)
3.3.6 (7)
3.3.7 (1)P
3.3.7 (2)P
3.4.1.1 (1)P
3.4.1.1 (2)P
3.4.1.1 (3)P
3.4.1.2.1 (1)P
3.4.1.2.1 (2)
3.4.1.2.2 (1)P
3.4.2.1 (1)P
3.4.2.1 (2)P
3.4.2.1 (3)
3.4.2.2 (1)

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

$$f_{cd} = \alpha_{cc} f_{ck}/\gamma_C \tag{3.15}$$

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)P	*4.2 (3)*	*4.4.1.2 (4)*	*4.4.1.2 (13)*
4.1 (2)P	*4.3 (1)P*	*4.4.1.2 (5)*	*4.4.1.3 (1)P*
4.1 (3)P	*4.3 (2)P*	*4.4.1.2 (6)*	*4.4.1.3 (2)*
4.1 (4)	*4.4.1.1 (1)P*	*4.4.1.2 (7)*	*4.4.1.3 (3)*
4.1 (5)	*4.4.1.1 (2)P*	*4.4.1.2 (8)*	*4.4.1.3 (4)*
4.1 (6)	*4.4.1.2 (1)P*	*4.4.1.2 (10)*	
4.2 (1)P	*4.4.1.2 (2)P*	*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)P
5.1.1 (2)
5.1.1 (3)
5.1.1 (4)P
5.1.1 (5)
5.1.1 (6)P
5.1.1 (7)
5.1.2 (1)P
5.1.2 (2)
5.1.2 (3)
5.1.2 (4)
5.1.2 (5)
5.1.4 (1)P
5.1.4 (2)
5.1.4 (3)
5.2 (1)P
5.2 (2)P
5.2 (3)
5.2 (7)
5.3.1 (1)P
5.3.1 (3)
5.3.1 (4)
5.3.1 (5)
5.3.1 (7)
5.3.2.1 (1)P
5.3.2.1 (2)
5.3.2.1 (3)
5.3.2.1 (4)
5.3.2.2 (1)
5.3.2.2 (2)
5.3.2.2 (3)
5.4 (1)
5.4 (2)
5.4 (3)
5.5 (1)P
5.5 (2)
5.5 (3)
5.5 (6)
5.6.1 (2)P
5.6.1 (3)P
5.6.1 (4)
5.6.2 (1)P
5.6.2 (3)
5.6.2 (4)
5.6.2 (5)
5.6.3 (1)
5.6.3 (3)
5.6.3 (4)
5.6.4 (1)
5.6.4 (2)
5.6.4 (3)
5.6.4 (4)
5.6.4 (5)
5.7 (1)
5.7 (2)
5.7 (3)
5.7 (4)P
5.8.1
5.8.2 (1)P
5.8.2 (2)P
5.8.2 (3)P
5.8.2 (4)P
5.8.2 (5)P
5.8.2 (6)
5.8.3.1 (1)
5.8.3.1 (2)
5.8.3.2 (1)
5.8.3.2 (2)
5.8.3.2 (3)
5.8.3.2 (4)
5.8.3.2 (5)
5.8.3.2 (6)
5.8.3.2 (7)
5.8.4 (1)P
5.8.4 (2)
5.8.4 (3)
5.8.4 (4)
5.8.5 (1)
5.8.5 (2)
5.8.5 (3)
5.8.5 (4)
5.8.6 (1)P
5.8.6 (2)P
5.8.6 (3)
5.8.6 (4)
5.8.6 (5)
5.8.6 (6)
5.8.7.1 (1)
5.8.7.1 (2)
5.8.7.2 (1)
5.8.7.2 (2)
5.8.7.2 (3)
5.8.7.2 (4)
5.8.7.3 (1)
5.8.7.3 (2)
5.8.7.3 (3)
5.8.7.3 (4)
5.8.8.1 (1)
5.8.8.1 (2)
5.8.8.2 (1)
5.8.8.2 (2)
5.8.8.2 (3)
5.8.8.2 (4)
5.8.8.3 (1)
5.8.8.3 (2)
5.8.8.3 (3)
5.8.8.3 (4)
5.8.9 (1)
5.8.9 (2)
5.8.9 (3)
5.8.9 (4)
5.9 (1)P
5.9 (2)
5.9 (3)
5.9 (4)
5.10.1 (1)P
5.10.1 (2)
5.10.1 (3)
5.10.1 (4)
5.10.1 (5)P
5.10.2.1 (1)P
5.10.2.1 (2)
5.10.2.2 (1)P
5.10.2.2 (2)P
5.10.2.2 (3)P
5.10.2.2 (4)
5.10.2.2 (5)
5.10.2.3 (1)P
5.10.3 (1)P
5.10.3 (2)
5.10.3 (3)
5.10.3 (4)
5.10.4 (1)
5.10.5.1 (1)
5.10.5.1 (2)
5.10.5.2 (1)
5.10.5.2 (2)
5.10.5.2 (3)
5.10.5.2 (4)
5.10.5.3 (1)
5.10.5.3 (2)
5.10.6 (1)
5.10.6 (2)
5.10.6 (3)
5.10.7 (1)
5.10.7 (2)
5.10.7 (3)
5.10.7 (4)
5.10.7 (5)
5.10.7 (6)
5.10.8 (1)
5.10.8 (2)
5.10.9 (1)P
5.11 (1)P
5.11 (2)P

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_l, given by

$$\theta_l = \theta_0 \, \alpha_h \tag{5.101}$$

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 $a = \theta_l \dfrac{l}{2}$, 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:

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

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:

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

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

$\delta \geq k_5$ 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/ε_{cu2}), for k_3 is 0,54, for k_4 is 1,25(0,6+0,0014/ε_{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 ≤ 0,15 for concrete strength classes ≤ C50/60

≤ 0,10 for concrete strength classes ≥ 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 1.1 f_{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:

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

or

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

(i.e.) $R\left(\frac{q_{ud}}{\gamma_{O'}}\right)$

or

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

where:

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

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

γ_O is the overall safety factor, γ_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)P	*6.2.4 (6)*	*6.4.3 (1)P*	*6.5.4 (9)*
6.1 (2)P	*6.2.4 (7)*	*6.4.3 (2)*	*6.6 (1)P*
6.1 (3)P	*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)P*
6.1 (6)	*6.2.5 (4)*	*6.4.3 (6)*	*6.7 (2)*
6.1 (7)	*6.3.1 (1)P*	*6.4.3 (7)*	*6.7 (3)*
6.2.1 (1)P	*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)P*
6.2.1 (3)	*6.3.1 (4)*	*6.4.4 (1)*	*6.8.2 (1)P*
6.2.1 (4)	*6.3.1 (5)*	*6.4.4 (2)*	*6.8.2 (2)P*
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)P*
6.2.1 (7)	*6.3.3 (1)*	*6.4.5 (3)*	*6.8.3 (2)P*
6.2.1 (8)	*6.3.3 (2)*	*6.4.5 (4)*	*6.8.3 (3)P*
6.2.1 (9)	*6.4.1 (1)P*	*6.4.5 (5)*	*6.8.4 (1)*
6.2.2 (2)	*6.4.1 (2)P*	*6.5.1 (1)P*	*6.8.4 (2)*
6.2.2 (3)	*6.4.1 (3)*	*6.5.2 (1)*	*6.8.4 (3)P*
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)P*
6.2.2 (7)	*6.4.2 (2)*	*6.5.3 (2)*	*6.8.5 (1)P*
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)P*	*6.8.5 (3)*
6.2.3 (4)	*6.4.2 (5)*	*6.5.4 (2)P*	*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 $A_{s,min}$.

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

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:

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

where $\Delta\sigma_p$ is the smaller of 0,4f_{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:

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

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:

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

with a minimum of

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

where:

f_{ck} is in MPa

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

ρ_1 $= \dfrac{A_{sl}}{b_w d} \leq 0{,}02$

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

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

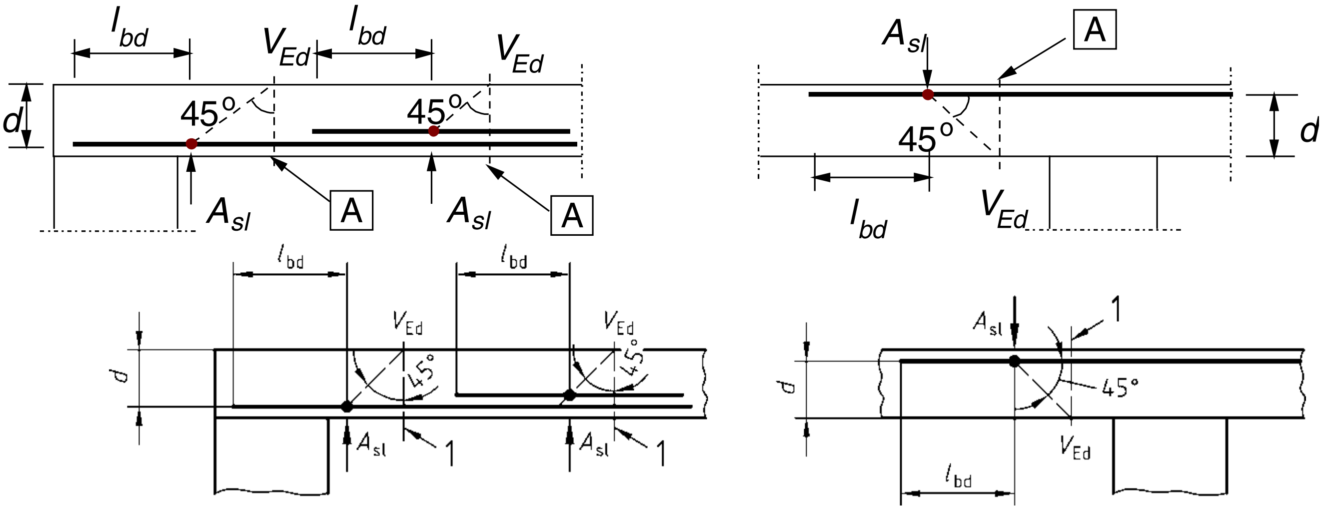
N_{Ed} is the axial force in the cross-section due to loading or to the acting effect of prestressing in Newtons ($N_{Ed} > 0$ for compression). The influence of imposed deformations on N_{Ed} may be ignored.

A_C is the area of concrete cross section [mm^2]

$V_{Rd,c}$ 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.

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



A – section considered

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:

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

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

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

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:

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

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

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

1 for non-prestressed structures

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

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

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

where:

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

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 ($\alpha_{cw} = 1$).

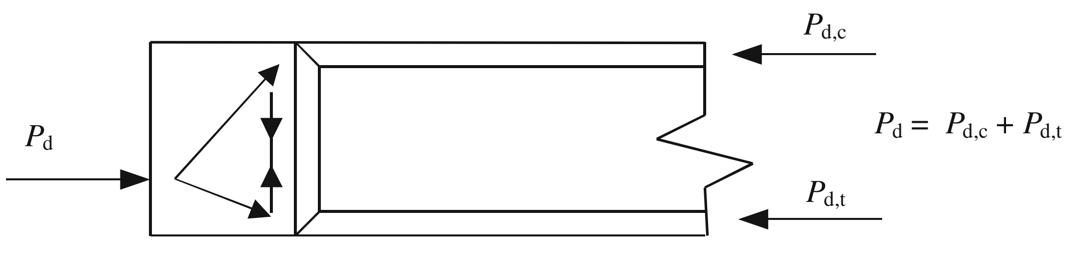


Figure 6.101 — Dispersion of prestressing by end blocks within the chords

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

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

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

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

$(M_{Ed}/z) + \Delta F_{td}$ 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:

In the case of bonded prestressing, located within the tensile chord, the resisting effect of prestressing may be taken into account for carrying the total longitudinal tensile force. In the case of inclined bonded prestressing tendons in combination with other longitudinal reinforcement/tendons the shear strength may be evaluated, by a simplification, superimposing two different truss models with different geometry (Figure. 6.102N); a weighted mean value between θ_1 and θ_2 may be used for concrete stress field verification with Expression (6.9).

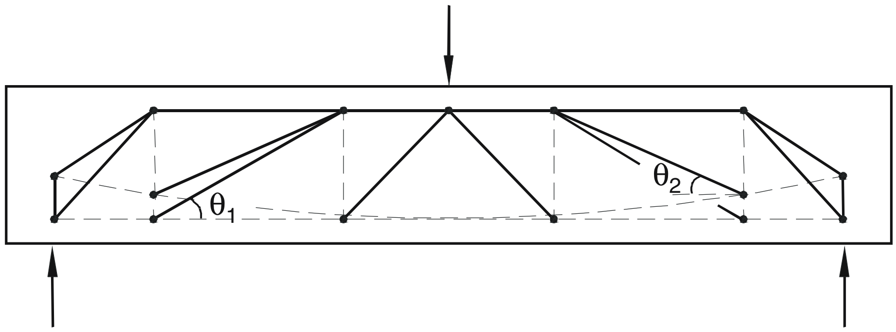
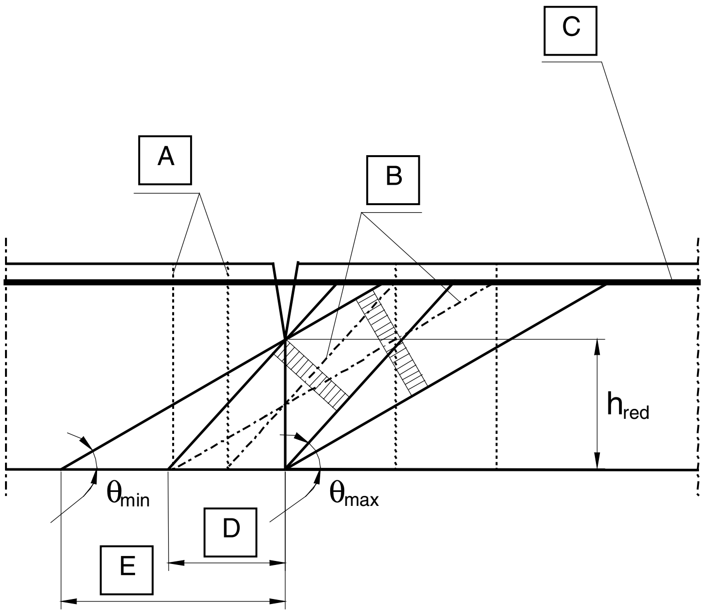


Figure 6.102N: Superimposed resisting model for shear

(109) In the case of segmental construction with precast elements and no bonded prestressing in the tension chord, the effect of opening of the joint should be considered. In these conditions, in the absence of a 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 6.103), the concrete stress field inclination within the web increases. The depth of concrete section available for the flow of the web compression field decreases to a value of h_{red}. The shear capacity can be evaluated in accordance with Expression 6.8 by assuming a value of θ derived from the minimum value of residual depth h_{red}.



A Axes of theoretical tension tie

B Axes of theoretical compression struts

AC1 C Tension chord of truss (external or internal unbonded tendon) AC1

D Field A : arrangement of stirrups with θ_{max} (cot θ = 1,0)

E Field B : arrangement of stirrups with θ_{min} (cot θ = 2,5)

Figure 6.103 — Diagonal stress fields across the joint in the web

$$h_{\mathrm{red}} = \frac{V_{\mathrm{Ed}}}{b_{\mathrm{w}}\, \nu\, f_{\mathrm{cd}}} (\cot\theta + \tan\theta) \tag{6.103}$$

Shear reinforcement stirrups, having the following area per unit length:

$$\frac{A_{\mathrm{sw}}}{s} = \frac{V_{\mathrm{Ed}}}{h_{\mathrm{red}}\, f_{\mathrm{ywd}} \cot\theta} \tag{6.104}$$

should be provided within a distance $h_{\mathrm{red}} \cot\theta$, but not greater than the segment length, from both edges of the joint.

The prestressing force should be increased if necessary such that, at the ultimate limit state, under the combination of bending moment and shear, the joint opening is limited to the value $h - h_{\mathrm{red}}$ as calculated above.

NOTE The absolute minimum value of h_{red} to be used in a Country may be found in its National Annex. The recommended absolute minimum value for h_{red} is 0,5 h.

6.2.4 Shear between web and flanges of T-sections

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

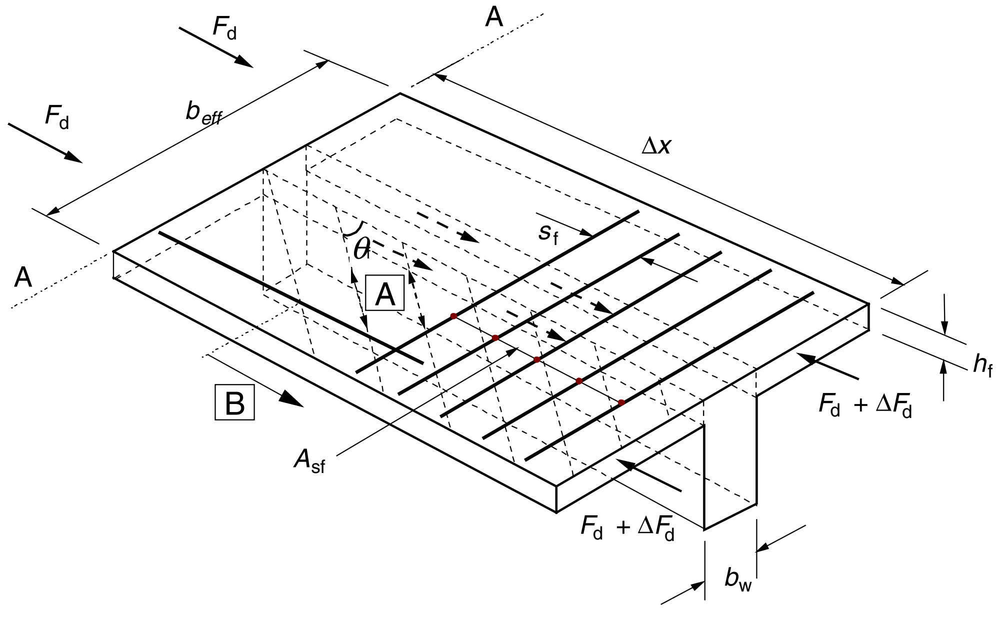
$$v_{\mathrm{Ed}} = \Delta F_{\mathrm{d}}/(h_{\mathrm{f}} \cdot \Delta x) \tag{6.20}$$

where:

h_{f} is the thickness of flange at the junctions

Δx is the length under consideration, see Figure 6.7

ΔF_{d} is the change of the normal force in the flange over the length Δx.



A – compressive struts B – longitudinal bar anchored beyond this projected point (see 6.2.4 (7))

Figure 6.7 — Notations for the connection between flange and web

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

Alternatively, considering a length Δx of the beam, the shear transmitted from the web to the flange is $V_{Ed}\Delta x/z$ and is divided into three parts: one remaining within the web breadth and the other two going out to the flange outstands. It should be generally assumed that the proportion of the force remaining within the web is the fraction b_w/b_{eff} of the total force. A greater proportion may be assumed if the full effective flange breadth is not required to resist the bending moment. In this case a check for cracks opening at SLS may be necessary.

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

For the verification of concrete compression crushing according to Expression (6.22) of EN 1992-1-1 h_f should be reduced by the depth of compression considered in the bending assessment.

NOTE If this verification is not satisfied the refined method given in Annex MM may be used.

6.2.5 Shear at the interface between concrete cast at different times

(105) For fatigue or dynamic verifications, the values for c in 6.2.5 (1) in EN 1992-1-1 should be taken as zero.

6.2.106 Shear and transverse bending

(101) Due to the presence of compressive stress fields arising from shear and bending, the interaction between longitudinal shear and transverse bending in the webs of box girder sections should be considered in the design.

When $V_{Ed}/V_{Rd,max} < 0,2$ or $M_{Ed}/M_{Rd,max} < 0,1$ this interaction can be disregarded; where $V_{Rd,max}$ and $M_{Rd,max}$ represent respectively the maximum web capacity for longitudinal shear and transverse bending.

NOTE Further information on the interaction between shear and transverse bending may be found in Annex MM.

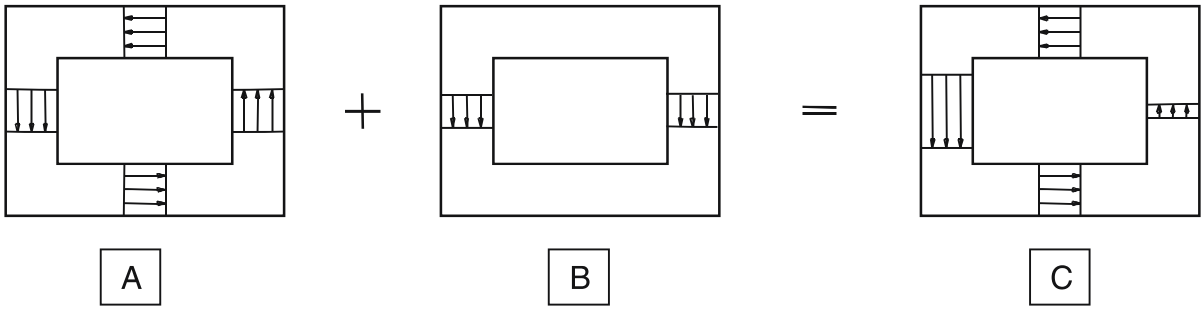
6.3 Torsion

6.3.2 Design procedure

(102) The effects of torsion and shear for both hollow and solid members may be superimposed, assuming the same value for the strut inclination θ. The limits for θ given in 6.2.3 (2) are also fully applicable for the case of combined shear and torsion.

[AC1) The maximum bearing capacity of a member loaded in shear and torsion follows from 6.3.2 (104). (AC1]

For box sections, each wall should be verified separately, for the combination of shear forces derived from shear and torsion (Figure 6.104).



A Torsion

B Shear

C Combination

Figure 6.104 — Internal actions combination within the different walls of a box section

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

$$\frac{\sum A_{sl} f_{yd}}{u_k} = \frac{T_{Ed}}{2A_k} \cot\theta \tag{6.28}$$

where

u_k is the perimeter of the area A_k

f_{yd} is the design yield stress of the longitudinal reinforcement A_{sl}

θ is the angle of compression struts (see Figure 6.5).

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

Bonded prestressing tendons can be taken into account limiting their stress increase to $\Delta\sigma_p \leq 500$ MPa. In that case, $\sum A_{sl} f_{yd}$ in Expression (6.28) is replaced by $\sum A_{sl} f_{yd} + A_p \Delta\sigma_p$.

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

— for solid cross-sections:

$$T_{Ed}/T_{Rd,max} + V_{Ed}/V_{Rd,max} \leq 1{,}0 \tag{6.29}$$

where:

T_{Ed} is the design torsional moment

V_{Ed} is the design transverse force

$T_{Rd,max}$ is the design torsional resistance moment according to

$$T_{Rd,max} = 2\nu\alpha_{cw} f_{cd} A_k t_{ef,i} \sin\theta \cos\theta \qquad (6.30)$$

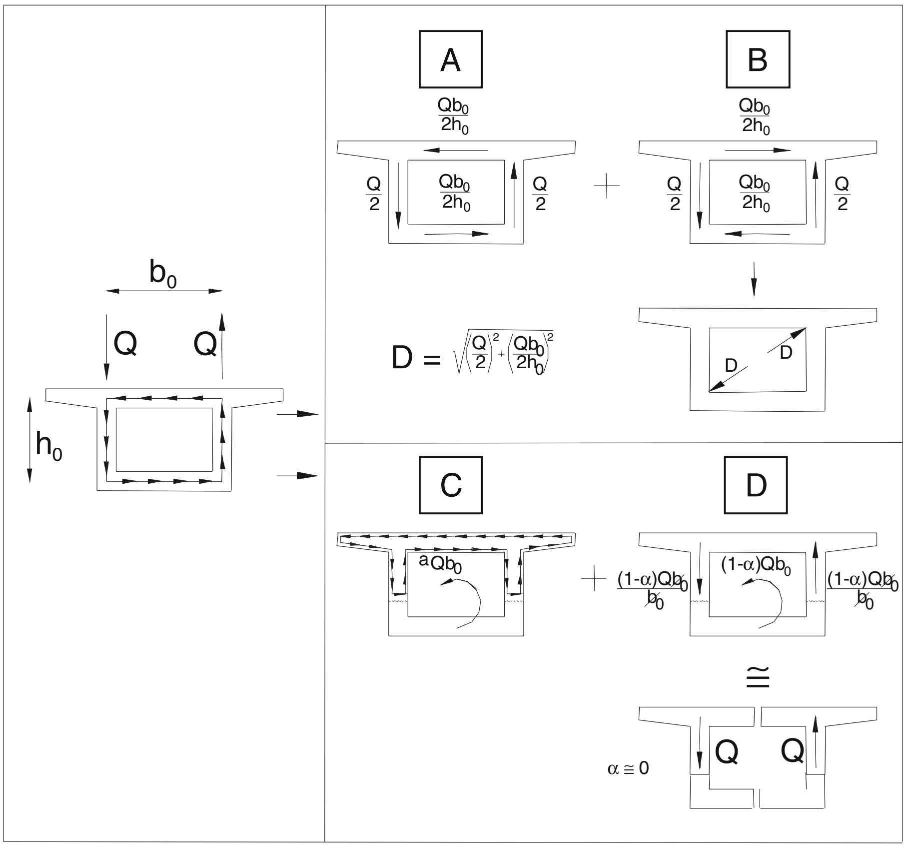
AC1 where ν follows from 6.2.2 (6.6N) of EN 1992-1-1 and α_{cw} from Expression (6.9) AC1

$V_{Rd,max}$ is the maximum design shear resistance according to Expressions (6.9) or (6.14). In solid cross-sections the full width of the web may be used to determine $V_{Rd,max}$

— for box sections:

Each wall should be designed separately for combined effects of shear and torsion. The ultimate limit state for concrete should be checked with reference to the design shear resistance $V_{Rd,max}$.

(106) In the case of segmental construction with precast box elements and no internal bonded prestressing in the tension region, the opening of a joint to an extension greater than the thickness of the corresponding flange entails a substantial modification of the torsional resisting mechanism if the relevant shear keys are not able to transfer the local shear due to torsion. It changes from Bredt circulatory torsion to a combination of warping torsion and De Saint Venant torsion, with the first mechanism prevailing over the second (Figure 6.105). As a consequence, the web shear due to torsion is practically doubled and significant distortion of the section takes place. In these circumstances, the capacity at the ultimate limit state should be verified in the most heavily stressed web according to the procedure in Annex MM taking into account the combination of bending, shear and torsion.



A Bredt

B Self balanced

C De Saint Venant

D Warping

Figure 6.105 — Variation in torsional behaviour from closed to opened joint

6.7 Partially loaded areas

(105) The design of bearing zones for bridges should be carried out using recognised methods.

NOTE Further information may be found in Annex J

6.8 Fatigue

6.8.1 Verification conditions

(102) A fatigue verification should be carried out for structures and structural components which are subjected to regular load cycles.

NOTE A fatigue verification is generally not necessary for the following structures and structural elements:

a) footbridges, with the exception of structural components very sensitive to wind action;

b) buried arch and frame structures with a minimum earth cover of 1.00 m and 1.50 m respectively for road and railway bridges;

c) foundations;

d) piers and columns which are not rigidly connected to superstructures;

e) retaining walls of embankments for roads and railways;

f) abutments of road and railway bridges which are not rigidly connected to superstructures, except the slabs of hollow abutments;

g) prestressing and reinforcing steel, in regions where, under the frequent combination of actions and P_k, only compressive stresses occur at the extreme concrete fibres.

The National Annex may define additional rules.

6.8.4 Verification procedure for reinforcing and prestressing steel

(107) Fatigue verification for external and unbonded tendons, lying within the depth of the concrete section, is not necessary.

6.8.7 Verification of concrete under compression or shear

(101) The verification should be carried out using traffic data, S-N curves and load models specified by the National Authorities. A simplified approach based on λ values may be used for the verification for railway bridges; see Annex NN.

Miner's rule should be applied for the verification of concrete; accordingly $\sum_{i=1}^{m} \frac{n_i}{N_i} \leq 1$ where:

m = number of intervals with constant amplitude

n_i = actual number of constant amplitude cycles in interval "i"

N_i = ultimate number of constant amplitude cycles in interval "i" that can be carried before failure. N_i may be given by National Authorities (S-N curves) or calculated on a simplified basis using Expression 6.72 of EN 1992-1-1 substituting the coefficient 0,43 with ($\log N_i$)/14 and transforming the inequality in the expression.

Then a satisfactory fatigue resistance may be assumed for concrete under compression, if the following condition is fulfilled:

$$\sum_{i=1}^{m} \frac{n_i}{N_i} \leq 1 \tag{6.105}$$

where:

AC1 $$N_i = 10^{\left(14 \frac{1-E_{\text{cd,max},i}}{\sqrt{1-R_i}}\right)} \tag{6.106}$$ AC1

$$R_i = \frac{E_{\text{cd, min},i}}{E_{\text{cd, max},i}} \tag{6.107}$$

$$E_{\text{cd, min},i} = \frac{\sigma_{\text{cd, min},i}}{f_{\text{cd, fat}}} \tag{6.108}$$

$$E_{\text{cd, max},i} = \frac{\sigma_{\text{cd, max},i}}{f_{\text{cd, fat}}} \tag{6.109}$$

where:

R_i is the stress ratio

$E_{cd,min,i}$ is the minimum compressive stress level

$E_{cd,max,i}$ is the maximum compressive stress level

$f_{cd,fat}$ is the design fatigue strength of concrete according to (6.76)

$\sigma_{cd,max,i}$ is the upper stress in a cycle

$\sigma_{cd,min,i}$ is the lower stress in a cycle

$$f_{cd,fat} = k_1 \beta_{cc}(t_0) f_{cd} \left(1 - \frac{f_{ck}}{250}\right) \quad (6.76)$$

where:

$\beta_{cc}(t_0)$ is a coefficient for concrete strength at first load application (see 3.1.2 (6) of EN 1992-1-1)

t_0 is the time of the start of the cyclic loading on concrete in days

NOTE 1 The value of k_1 for use in a Country may be found in its National Annex. The recommended value is 0,85.

NOTE 2 See also Annex NN for further information.

6.109 Membrane elements

(101) Membrane elements may be used for the design of two-dimensional concrete elements subject to a combination of internal forces evaluated by means of a linear finite element analysis. Membrane elements are subjected only to in plane forces, namely σ_{Edx}, σ_{Edy}, τ_{Edxy} as shown in Figure 6.106.

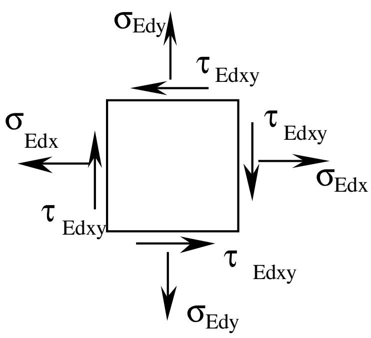


Figure 6.106 — Membrane element

(102) Membrane elements may be designed by application of the theory of plasticity using a lower bound solution.

(103) The maximum value for compressive stress field strength should be defined as a function of the principal stress values:

i) If the principal stresses are both compressive, the maximum compression in the concrete stress field is:

$$\sigma_{cd\,max} = 0{,}85 f_{cd} \frac{1 + 3{,}80\alpha}{(1+\alpha)^2} \quad (6.110)$$

where $\alpha \leq 1$ is the ratio between the two principal stresses.

ii) Where a plastic analysis has been carried out with $\theta = \theta_{el}$ and at least one principal stress is in tension and no reinforcement yields, the maximum compression in the concrete stress field is given by:

$$\sigma_{cd\,max} = f_{cd}\left[0{,}85 - \frac{\sigma_s}{f_{yd}}(0{,}85 - \nu)\right] \tag{6.111}$$

where σ_s is the maximum tensile stress in the reinforcement, and ν is defined in 6.2.2 (6) of EN 1992-1-1.

iii) Where a plastic analysis has been carried out and there is yielding of any reinforcement, the maximum compression in the concrete stress field is:

$$\sigma_{cd\,max} = \nu f_{cd}\,(1 - 0{,}032\,|\theta - \theta_{el}|) \tag{6.112}$$

where:

θ_{el} (in degrees) is the inclination to the x axis of the principal compression stress in the elastic analysis.

θ (in degrees) is the angle of the plastic compression field (principal compressive stress) at ULS, to the x axis.

In Expression (6.112) $|\theta - \theta_{el}|$ should be limited to 15 degrees.

SECTION 7 Serviceability Limit States (SLS)

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

7.1 (1)P	*7.3.1 (4)*	*7.3.3 (3)*	*7.4.3 (1)P*
7.1 (2)	*7.3.1 (6)*	*7.3.3 (4)*	*7.4.3 (2)P*
7.2 (1)P	*7.3.1 (7)*	*7.3.4 (2)*	*7.4.3 (3)*
7.2 (3)	*7.3.1 (8)*	*7.3.4 (3)*	*7.4.3 (4)*
7.2 (4)P	*7.3.1 (9)*	*7.3.4 (4)*	*7.4.3 (5)*
7.2 (5)	*7.3.2 (1)P*	*7.3.4 (5)*	*7.4.3 (6)*
7.3.1 (1)P	*7.3.2 (3)*	*7.4.1 (1)P*	*7.4.3 (7)*
7.3.1 (2)P	*7.3.2 (4)*	*7.4.1 (2)*	
7.3.1 (3)	*7.3.3 (2)*		

7.2 Stresses

(102) Longitudinal cracks may occur if the stress level under the characteristic combination of loads exceeds a critical value. Such cracking may lead to a reduction of durability. In the absence of other measures, such as an increase in the cover to reinforcement in the compressive zone or confinement by transverse reinforcement, it may be appropriate to limit the compressive stress to a value $k_1 f_{ck}$ in areas exposed to environments of exposure classes XD, XF and XS (see Table 4.1 of EN1992-1-1).

NOTE The value of k_1 for use in a Country may be found in its National Annex. The recommended value is 0,6. The maximum increase in the stress limit above $k_1 f_{ck}$ in the presence of confinement may also be found in a country's National Annex. The recommended maximum increase is 10 %.

7.3 Crack control

7.3.1 General considerations

(105) A limiting calculated crack width w_{max}, taking account of the proposed function and nature of the structure and the costs of limiting cracking, should be established. Due to the random nature of the cracking phenomenon, actual crack widths cannot be predicted. However, if the crack widths calculated in accordance with the models given in this Standard are limited to the values given in Table 7.101N, the performance of the structure is unlikely to be impaired.

NOTE The value of w_{max} and the definition of decompression and its application for use in a country may be found in its National Annex. The recommended value for w_{max} and the application of the decompression limit are given in Table 7.101N. The recommended definition of decompression is noted in the text under the Table.

Table 7.101N — Recommended values of w_{max} and relevant combination rules

Exposure Class	Reinforced members and prestressed members without bonded tendons	Prestressed members with bonded tendons
	Quasi-permanent load combination	Frequent load combination
X0, XC1	0,3[a]	0,2
XC2, XC3, XC4	0,3	0,2[b]
XD1, XD2, XD3 XS1, XS2, XS3		Decompression

[a] For X0, XC1 exposure classes, crack width has no influence on durability and this limit is set to guarantee acceptable appearance. In the absence of appearance conditions this limit may be relaxed.

[b] For these exposure classes, in addition, decompression should be checked under the quasi-permanent combination of loads.

The decompression limit requires that all concrete within a certain distance of bonded tendons or their ducts should remain in compression under the specified loading.

NOTE The value of the distance considered to be used in a Country may be found in its National Annex. The recommended value is 100 mm.

(110) In some cases it may be necessary to check and control shear cracking in webs.

NOTE Further information may be found in Annex QQ.

7.3.2 Minimum reinforcement areas

(102) Unless a more rigorous 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).

$$A_{s,min}\sigma_s = k_c\, k\, f_{ct,eff}\, A_{ct} \qquad (7.1)$$

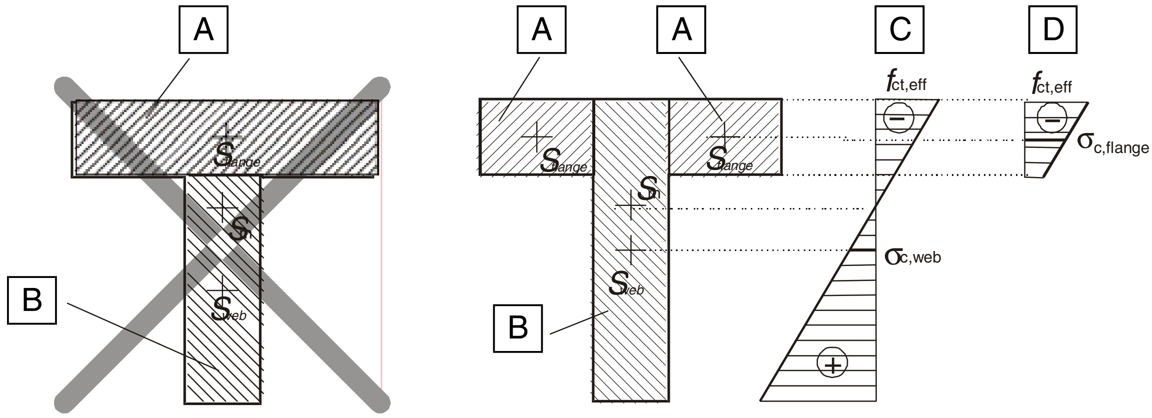
where:

$A_{s,min}$ is the minimum area of reinforcing steel within the tensile zone

A_{ct} is the area of concrete within tensile zone. The tensile zone is that part of the section which is calculated to be in tension just before formation of the first crack

In flanged cross sections such as T-beams and box girders the division into parts should be as indicated in Figure 7.101.

Stress distribution for plain bending: Cross sectional stresses



A Component section "Flange"

B Component section "Web"

C "Web"

D "Flange"

Figure 7.101 — Example for a division of a flanged cross-section for analysis of cracking

σ_s is the absolute value of the maximum stress permitted in the reinforcement immediately after formation of the crack. This may be taken as the yield strength of the reinforcement, f_{yk}. A lower value may, however, be needed to satisfy the crack width limits according to the maximum bar size or the maximum bar spacing (see 7.3.3 (2) of EN 1992-1-1)

$f_{ct,eff}$ is the mean value of the tensile strength of the concrete effective at the time when the cracks may first be expected to occur:

$f_{ct,eff} = f_{ctm}$ or lower, ($f_{ctm}(t)$), if cracking is expected earlier than 28 days

k is the coefficient which allows for the effect of non-uniform self-equilibrating stresses, which lead to a reduction of restraint forces

= 1,0 for webs with $h \leq 300$ mm or flanges with widths less than 300 mm

= 0,65 for webs with $h \geq 800$ mm or flanges with widths greater than 800 mm

intermediate values may be interpolated

k_c is a coefficient which takes account of the stress distribution within the section immediately prior to cracking and of the change of the lever arm:

For pure tension k_c = 1,0

For bending or bending combined with axial forces:

— For rectangular sections and webs of box sections and T-sections:

$$k_c = 0{,}4 \cdot \left[1 - \frac{\sigma_c}{k_1 (h/h^*) f_{ct,eff}}\right] \leq 1 \qquad (7.2)$$

— For flanges of box sections and T-sections:

$$k_c = 0{,}9 \frac{F_{cr}}{A_{ct} f_{ct,eff}} \geq 0{,}5 \qquad (7.3)$$

where

σ_c is the mean stress of the concrete acting on the part of the section under consideration:

$$\sigma_c = \frac{N_{Ed}}{bh} \qquad (7.4)$$

N_{Ed} is the axial force at the serviceability limit state acting on the part of the cross-section under consideration (compressive force positive). N_{Ed} should be determined considering the characteristic values of prestress and axial forces under the relevant combination of actions

h^* $h^* = h$ for $h < 1{,}0$ m

$h^* = 1{,}0$ m for $h \geq 1{,}0$ m

k_1 is a coefficient considering the effects of axial forces on the stress distribution:

$k_1 = 1{,}5$ if N_{Ed} is a compressive force

$k_1 = \frac{2h^*}{3h}$ if N_{Ed} is a tensile force

F_{cr} is the absolute value of the tensile force within the flange immediately prior to cracking due to the cracking moment calculated with $f_{ct,eff}$

(105) For bridges, in calculating the minimum reinforcement AC1 to cater for shrinkage, $f_{ct,eff}$ in Expression (7.1) should be taken as AC1 the greater of 2,9 MPa or $f_{ctm}(t)$.

7.3.3 Control of cracking without direct calculation

(101) The control of cracking without direct calculation may be performed by means of simplified methods.

NOTE Details of a simplified method for control of cracking without calculation may be found in a Country's National Annex. The recommended method is given in EN 1992-1-1 7.3.3 (2) to (4).

7.3.4 Calculation of crack widths

(101) The evaluation of crack width may be performed using recognised methods.

NOTE Details of recognised methods for crack width control may be found in a Country's National Annex. The recommended method is that in EN 1992-1-1, 7.3.4.

7.4 Deflection control

7.4.1 General considerations

(3), (4), (5) and (6) of EN 1992-1-1 do not apply.

AC1 *Text deleted* AC1

SECTION 8 Detailing of reinforcement and prestressing tendons — General

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

8.1 (1)P
8.1 (2)P
8.1 (3)
8.1 (4)
8.2 (1)P
8.2 (2)
8.2 (3)
8.2 (4)
8.3 (1)P
8.3 (2)
8.3 (3)
8.4.1 (1)P
8.4.1 (2)
8.4.1 (3)
8.4.1 (4)
8.4.1 (5)
8.4.1 (6)
8.4.2 (1)P
8.4.2 (2)
8.4.3 (1)P
8.4.3 (2)
8.4.3 (3)
8.4.3 (4)
8.4.4 (1)
8.4.4 (2)
8.5 (1)
8.5 (2)
8.6 (1)
8.6 (2)
8.6 (3)
8.6 (4)
8.6 (5)
8.7.1 (1)P
8.7.2 (1)P
8.7.2 (2)
8.7.2 (3)
8.7.2 (4)
8.7.3 (1)
8.7.4.1 (1)
8.7.4.1 (2)
8.7.4.1 (3)
8.7.4.1 (4)
8.7.4.2 (1)
8.7.5.1 (1)
8.7.5.1 (2)
8.7.5.1 (3)
8.7.5.1 (4)
8.7.5.1 (5)
8.7.5.1 (6)
8.7.5.1 (7)
8.7.5.2 (1)
8.8 (1)
8.8 (2)
8.8 (3)
8.8 (4)
8.8 (5)
8.8 (6)
8.8 (7)
8.8 (8)
8.9.1 (2)
8.9.1 (3)
8.9.1 (4)
8.9.2 (1)
8.9.2 (2)
8.9.2 (3)
8.9.3 (1)
8.9.3 (2)
8.9.3 (3)
8.10.1.1 (1)P
8.10.1.2 (1)
8.10.1.2 (2)
8.10.1.3 (1)P
8.10.1.3 (2)
8.10.1.3 (3)
8.10.2.1 (1)
8.10.2.2 (1)
8.10.2.2 (2)
8.10.2.2 (3)
8.10.2.2 (4)
8.10.2.2 (5)
8.10.2.3 (1)
8.10.2.3 (2)
8.10.2.3 (3)
8.10.2.3 (4)
8.10.2.3 (5)
8.10.2.3 (6)
8.10.3 (1)
8.10.3 (2)
8.10.3 (3)
8.10.3 (5)
8.10.4 (1)P
8.10.4 (2)P
8.10.4 (3)
8.10.4 (4)
8.10.5 (1)P
8.10.5 (2)P
8.10.5 (3)P
8.10.5 (4)

8.9 Bundled bars

8.9.1 General

(101) Unless otherwise stated, the rules for individual bars also apply for bundles of bars. In a bundle, all the bars should be of the same characteristics (type and grade). Bars of different sizes may be bundled provided that the ratio of diameters does not exceed 1,7.

NOTE Details of restrictions on the use of bundled bars for use in a Country may be found in its National Annex. No additional restrictions are recommended in this standard.

8.10 Prestressing tendons

8.10.3 Anchorage zones of post-tensioned members

(104) Tensile forces due to concentrated forces should be assessed by a strut and tie model, or other appropriate representation (see 6.5). Reinforcement should be detailed assuming that it acts at its design strength. If the stress in this reinforcement is limited to 250 MPa no check of crackwidths is necessary.

(106) Particular consideration should be given to the design of anchorage zones where two or more tendons are anchored.

NOTE Further information may be found in Annex J.

8.10.4 Anchorages and couplers for prestressing tendons

(105) The placing of couplers on more than X % of the tendons at one cross-section should be avoided unless:

— continuous minimum reinforcement according to Expression 7.1 of EN 1992-1-1 (Section 7.3.2) is provided, or

— there is a minimum residual compressive stress of 3 MPa at the cross-section under the characteristic combination of actions.

NOTE The value of X and the maximum percentage of tendons to be coupled at a section in a Country may be found in its National Annex. The recommended values are 50 % and 67 % respectively.

Where a proportion of tendons are joined with couplers at a particular cross section, remaining tendons may not be joined with couplers within distance '*a*' of the that cross section.

NOTE The distance "*a*" to be used in a Country may be found in its National Annex. The recommended value of a is given in Table 8.101N.

Table 8.101N — Minimum distance between sections at which tendons are joined with couplers

Construction depth h	Distance a
≤ 1,5 m	1,5 m
1,5 m < h < 3,0 m	$a = h$
≥ 3,0 m	3,0 m

(106) If slabs are transversely prestressed, special consideration should be given to the arrangement of prestressing, to achieve a reasonably uniform distribution of prestress.

(107) In an aggressive environment openings and pockets which are necessary to apply the prestress to the tendons should be avoided on the upper side of carriageway slabs. Where, in exceptional circumstances, openings and pockets are provided on the upper side of carriageway slabs appropriate precautions should be taken to ensure durability.

NOTE Additional rules relating to the provision of openings and pockets on the upper side of carriageway slabs for use in a Country may be found in its National Annex. No additional rules are recommended in this standard.

(108) If tendons are anchored at a construction joint or within a concrete member (whether on an external rib, within a pocket or entirely inside the member), it should be checked that a minimum residual compressive stress of at least 3 MPa is present in the direction of the anchored prestressing force, under the frequent load combination. If the minimum residual stress is not present, reinforcement should be provided to cater for the local tension behind the anchor. The check for residual stress is not required if the tendon is coupled at the anchorage considered.

SECTION 9 Detailing of members and particular rules

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

9.1 (1)P	*9.2.3 (1)*	*9.4.2 (1)*	*9.8.1 (1)*
9.1 (2)	*9.2.3 (2)*	*9.4.3 (1)*	*9.8.1 (2)*
9.2.1.1 (1)	*9.2.3 (3)*	*9.4.3 (2)*	*9.8.1 (4)*
9.2.1.1 (2)	*9.2.3 (4)*	*9.4.3 (3)*	*9.8.1 (5)*
9.2.1.1 (3)	*9.2.4 (1)*	*9.4.3 (4)*	*9.8.2.1 (1)*
9.2.1.1 (4)	*9.2.5 (1)*	*9.5.1 (1)*	*9.8.2.1 (2)*
9.2.1.2 (1)	*9.2.5 (2)*	*9.5.2 (1)*	*9.8.2.1 (3)*
9.2.1.2 (2)	*9.3 (1)*	*9.5.2 (2)*	*9.8.2.2 (1)*
9.2.1.2 (3)	*9.3.1.1 (1)*	*9.5.2 (3)*	*9.8.2.2 (2)*
9.2.1.3 (1)	*9.3.1.1 (2)*	*9.5.2 (4)*	*9.8.2.2 (3)*
9.2.1.3 (2)	*9.3.1.1 (3)*	*9.5.3 (2)*	*9.8.2.2 (4)*
9.2.1.3 (3)	*9.3.1.1 (4)*	*9.5.3 (3)*	*9.8.2.2 (5)*
9.2.1.3 (4)	*9.3.1.2 (1)*	*9.5.3 (4)*	*9.8.3 (1)*
9.2.1.4 (1)	*9.3.1.2 (2)*	*9.5.3 (5)*	*9.8.3 (2)*
9.2.1.4 (2)	*9.3.1.3 (1)*	*9.5.3 (6)*	*9.8.4 (1)*
9.2.1.4 (3)	*9.3.1.4 (1)*	*9.6.1 (1)*	*9.8.4 (2)*
9.2.1.5 (1)	*9.3.1.4 (2)*	*9.6.2 (1)*	*9.8.5 (1)*
9.2.1.5 (2)	*9.3.2 (1)*	*9.6.2 (2)*	*9.8.5 (2)*
9.2.1.5 (3)	*9.3.2 (2)*	*9.6.2 (3)*	*9.8.5 (3)*
9.2.2 (3)	*9.3.2 (3)*	*9.6.3 (1)*	*9.8.5 (4)*
9.2.2 (4)	*9.3.2 (4)*	*9.6.3 (2)*	*9.9 (1)*
9.2.2 (5)	*9.3.2 (5)*	*9.6.4 (1)*	*9.9 (2)P*
9.2.2 (6)	*9.4.1 (1)*	*9.6.4 (2)*	
9.2.2 (7)	*9.4.1 (2)*	*9.7 (1)*	
9.2.2 (8)	*9.4.1 (3)*	*9.7 (3)*	

9.1 General

(103) Minimum areas of reinforcement are given in order to prevent a brittle failure, wide cracks and also to resist forces arising from restrained actions.

NOTE Additional rules concerning the minimum thickness of structural elements and the minimum reinforcement for all surfaces of members in bridges, with minimum bar diameter and maximum bar spacing for use in a Country may be found in its National Annex. No additional rules are recommended in this standard.

9.2 Beams

9.2.2 Shear reinforcement

(101) The shear reinforcement should form an angle α of between 45° and 90° to the longitudinal axis of the structural element.

NOTE Details of the form of shear reinforcement permitted for use in a Country may be found in its National Annex. The recommended forms are:

— links enclosing the longitudinal tension reinforcement and the compression zone (see Figure 9.5 of EN 1992-1-1);

— bent-up bars;

— or a combination of the two.

(2) of EN 1992-1-1 does not apply.

9.5 Columns

9.5.3 Transverse reinforcement

(101) The diameter of the transverse reinforcement (links, loops or helical spiral reinforcement) should not be less than ϕ_{min} or one quarter of the maximum diameter of the longitudinal bars, whichever is the greater. The diameter of the wires of welded mesh fabric for transverse reinforcement should not be less than $\phi_{min,mesh}$.

NOTE The minimum diameter of transverse reinforcement for use in a Country may be found in its National Annex. The recommended values are ϕ_{min} = 6 mm and $\phi_{min,mesh}$ = 5 mm.

9.7 Deep beams

(102) The distance between two adjacent bars of the mesh should not exceed s_{mesh}.

NOTE The maximum spacing of adjacent bars for use in a Country may be found in its National Annex. The recommended value of s_{mesh} is the lesser of the web thickness or 300 mm.

9.8 Foundations

9.8.1 Pile caps

(103) The main tensile reinforcement to resist the action effects should be concentrated in the stress zones between the tops of the piles. A minimum bar diameter d_{min} should be provided. If the area of this reinforcement is at least equal to the minimum reinforcement, evenly distributed bars along the bottom surface of the member may be omitted.

NOTE The value of d_{min} for use in a Country may be found in its National Annex. The recommended value is 12 mm.

9.10 Tying systems

This clause does not apply.

SECTION 10 Additional rules for precast concrete elements and structures

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

10.1.1	*10.9.2 (1)*	*10.9.4.2 (1)P*	*10.9.5.1 (5)P*
10.2 (1)P	*10.9.2 (2)*	*10.9.4.2 (2)P*	*10.9.5.2 (1)*
10.2 (2)	*10.9.3 (1)P*	*10.9.4.2 (3)*	*10.9.5.2 (2)*
10.2 (3)	*10.9.3 (2)P*	*10.9.4.3 (1)*	*10.9.5.2 (3)*
10.3.1.1 (1)	*10.9.3 (3)P*	*10.9.4.3 (2)*	*10.9.5.3 (1)P*
10.3.1.1 (2)	*10.9.3 (4)*	*10.9.4.3 (3)*	*10.9.5.3 (2)P*
10.3.1.1 (3)	*10.9.3 (5)*	*10.9.4.3 (4)*	*10.9.5.3 (3)P*
10.3.1.2 (1)	*10.9.3 (6)*	*10.9.4.3 (5)*	*10.9.6.1 (1)P*
10.3.1.2 (2)	*10.9.3 (7)*	*10.9.4.3 (6)*	*10.9.6.2 (1)*
10.3.1.2 (3)	*10.9.3 (8)*	*10.9.4.4 (1)*	*10.9.6.2 (2)*
10.3.2.2 (1)P	*10.9.3 (9)*	*10.9.4.5 (1)P*	*10.9.6.2 (3)*
10.3.2.2 (2)	*10.9.3 (10)*	*10.9.4.5 (2)*	*10.9.6.3 (1)*
10.5.1 (1)P	*10.9.3 (11)*	*10.9.4.6 (1)*	*10.9.6.3 (2)*
10.5.1 (2)	*10.9.3 (12)*	*10.9.4.7 (1)*	*10.9.6.3 (3)*
10.5.1 (3)	*10.9.4.1 (1)P*	*10.9.5.1 (1)P*	
10.5.2 (1)	*10.9.4.1 (2)P*	*10.9.5.1 (2)P*	
10.9.1 (1)	*10.9.4.1 (3)P*	*10.9.5.1 (3)*	
10.9.1 (2)	*10.9.4.1 (4)P*	*10.9.5.1 (4)P*	

10.1 General

(101)P The rules in this section apply to structures made partly or entirely of precast concrete elements, and are supplementary to the rules in other sections. Additional matters related to detailing, production and assembly are covered by specific product standards.

10.9 Particular rules for design and detailing

10.9.7 Tying systems

This clause does not apply.

SECTION 11 Lightweight aggregate concrete structures

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

11.1 (1)P	*11.3.2 (1)*	*11.3.7 (1)*	*11.6.4.2 (1)*
11.1.1 (1)P	*11.3.2 (2)*	*11.4.1 (1)*	*11.6.4.2 (2)*
11.1.1 (2)P	*11.3.3 (1)*	*11.4.2 (1)P*	*11.6.5 (1)*
11.1.1 (3)	*11.3.3 (2)*	*11.5.1*	*11.6.6 (1)*
11.1.1 (4)P	*11.3.3 (3)*	*11.6.1 (1)*	*11.7 (1)P*
11.1.2 (1)P	*11.3.4 (1)*	*11.6.1 (2)*	*11.8.1 (1)*
11.2 (1)P	*11.3.5 (1)P*	*11.6.2 (1)*	*11.8.2 (1)*
11.3.1 (1)P	*11.3.5 (2)P*	*11.6.3.1 (1)*	*11.10 (1)P*
11.3.1 (2)	*11.3.6 (1)*	*11.6.4.1 (1)*	
11.3.1 (3)	*11.3.6 (2)*	*11.6.4.1 (2)*	

11.9 Detailing of members and particular rules

(101) The diameter of bars embedded in LWAC should not normally exceed 32 mm. For LWAC bundles of bars should not consist of more than two bars and the equivalent diameter should not exceed 45 mm.

NOTE The use of bundled bars may be restricted by the National Annex.

SECTION 12 Plain and lightly reinforced concrete structures

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

SECTION 113 Design for the execution stages

113.1 General

(101) For bridges built in stages, the design should take account of the construction procedure in the following circumstances:

a) Where forces, other than those produced on the completed structure, occur in any structural section during the phases of construction (e.g. deck erection by incremental launching, piers of bridges built by balanced cantilever).

b) Where redistribution of forces due to rheological effects is originated by changes to the structural arrangement during the construction process (e.g. continuous bridges built span by span on falsework or by cantilever).

c) Where redistribution of stresses due to rheological effects is originated by changes to structural sections during the construction process (e.g. decks consisting of precast beams and an insitu slab).

d) Where the erection or casting sequence may have an influence on: the stability of the structure during construction, the forces in the completed structure, or the geometry of the completed structure.

(102) For structures in which any of the circumstances described in paragraphs (101) a) to d) apply, the serviceability limit states and ultimate limit states should be verified at construction stages.

(103) For structures in which the circumstances described in paragraphs (101) b) or c) apply, long term values of forces or stresses should determined from an analysis of redistribution effects. Step by step or approximate methods may be used in these calculations.

(104) For structures in which the circumstances described in paragraph (101) d) apply, erection and casting sequences/procedures should be indicated on drawings or detailed in a construction procedure document.

113.2 Actions during execution

(101) The actions to be taken into account during execution are given in EN 1991-1-6 and annexes.

(102) For the ultimate limit state verification of structural equilibrium for segmental bridges built by balanced cantilever, unbalanced wind pressure should be considered. An uplift or horizontal pressure of at least x N/m^2 acting on one of the cantilevers should be considered.

NOTE The x value to be used in a Country may be found in its National Annex. The recommended value of x is 200 N/m^2.

(103) For verification of ultimate limit states in bridges built by in-situ balanced cantilever, an accidental action arising from a fall of formwork should be considered. The action should include for dynamic effects. The fall may occur in any construction stage. (traveller movement, casting, etc.)

(104) For balanced cantilever construction with precast segments, an accidental fall of one segment should be taken into account.

(105) For incrementally launched decks imposed deformations should be taken into account.

113.3 Verification criteria

113.3.1 Ultimate limit states

(101) See EN 1992-2 section 6.

113.3.2 Serviceability limit states

(101) The verifications for the execution stage should be the same as those for the completed structure, with the following exceptions.

(102) Serviceability criteria for the completed structure need not be applied to intermediate execution stages, provided that durability and final appearance of the completed structure are not affected (e.g. deformations).

(103) Even for bridges or elements of bridges in which the limit-state of decompression is checked under the quasi-permanent or frequent combination of actions on the completed structure, tensile stresses less than $k.f_{ctm}(t)$ under the quasi-permanent combination of actions during execution are permitted.

NOTE The value of k to be used in a Country may be found in its National Annex. The recommended value of k is 1,0.

(104) For bridges or elements of bridges in which the limit-state of cracking is checked under frequent combination on the completed structure, the limit state of cracking should be verified under the quasi-permanent combination of actions during execution.

ANNEX A
(informative)

Modification of partial factors for materials

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

ANNEX B
(informative)

Creep and shrinkage strain

The following clauses of EN 1992-1-1 apply for ordinary concrete, except for particular thick sections (see below).

B.1(1)
B.1(2)
B.1(3)
B.2(1)

Section B.103 specifically applies to high performance concrete, made with Class R cements, of strength greater than C50/60 with or without silica fume. In general, the methods given in Section B.103 are preferred to those given in EN 1992-1-1 for the concretes referred to above and for thick members, in which the kinetics of basic creep and drying creep are quite different. It should be noted that the guidance in this Annex has been verified by site trials and measurements. For background information reference can be made to the following:

Le Roy, R., De Larrard, F., Pons, G. (1996) The AFREM code type model for creep and shrinkage of high performance concrete.

Toutlemonde, F., De Larrard, F., Brazillier, D. (2002) Structural application of HPC: a survey of recent research in France.

Le Roy, R., Cussac, J. M., Martin, O. (1999) Structures sensitive to creep :from laboratory experimentation to structural design - The case of the Avignon high-speed rail viaduct.

B.100 General

(101) This Annex may be used for calculating creep and shrinkage, including development with time. However, typical experimental values can exhibit a scatter of ± 30 % around the values of creep and shrinkage predicted in accordance with this Annex. Where greater accuracy is required due to the structural sensitivity to creep and/or shrinkage, an experimental assessment of these effects and of the development of delayed strains with time should be undertaken. Section B.104 includes guidelines for the experimental determination of creep and shrinkage coefficients.

(102) For High Strength Concrete (f_{ck} > 50MPa) an alternative approach to the evaluation of creep and shrinkage is given in Section B.103. The alternative approach takes account of the effect of adding silica fume and significantly improves the precision of the prediction.

(103) Furthermore, the expressions for creep in Sections B.1 and B.103 are valid when the the mean value of the concrete cylinder strength at the time of loading $f_{cm}(t_0)$ is greater that 0,6f_{cm} ($f_{cm}(t_0) > 0{,}6\, f_{cm}$).

When concrete is to be loaded at earlier ages, with significant strength development at the beginning of the loading period, specific determination of the creep coefficient should be undertaken. This should be based on an experimental approach and the determination of a mathematical expression for creep should be based on the guidelines included in Section B.104.

(104) Creep and shrinkage formulae and experimental determinations are based on data collected over limited time periods. Extrapolating such results for very long-term evaluations (e.g. one hundred years) results in the introduction of additional errors associated with the mathematical expressions used for the extrapolation. When safety would be increased by overestimation of delayed strains, and when it is relevant in the project, the creep and shrinkage predicted on the basis of the formulae or experimental determinations should be multiplied by a safety factor, as indicated in Section B.105.

B.103 High Strength Concrete

(101) In the case of high strength concrete (HSC), namely for concrete strength classes greater than or equal to C55/67, the model described in this clause should be used to obtain better consistency with experimental data when the information required to utilise the model is available. For HSC without silica fume, creep is generally greater than predicted in the average expressions of Section B.1. Formulae proposed in this section should not be used without verification when the aggregate fraction is lower than 67 %, which may be more frequently the case for self-consolidating concrete.

(102) The model makes a distinction between strains occurring in sealed concrete and additional deformation due to drying. Two expressions for shrinkage and two for creep, are given in this clause. The time-dependant strain components are:

— autogeneous shrinkage,

— drying shrinkage,

— basic creep,

— drying creep.

This distinguishes phenomena which are governed by different physical mechanisms. The autogeneous shrinkage is related to the hydration process whereas the drying shrinkage, due to humidity exchanges, is associated with the structure's environment.

(103) Specific formulae are given for silica-fume concrete (SFC). For the purpose of this clause, SFC is considered as concrete containing an amount of silica fume of at least 5 % of the cementitious content by weight.

B.103.1 Autogeneous shrinkage

(101) The hydration rate governs the kinetics of autogeneous shrinkage. Therefore the hardening rate controls the progress of the phenomenon. The ratio $f_{cm}(t)/f_{ck}$, known as the maturity of young concrete, is taken as the main variable before 28 days. Shrinkage appears to be negligible for maturity less than 0,1. For ages beyond 28 days, the variable governing the evolution of autogeneous shrinkage is time.

The model for evaluation of autogeneous shrinkage is as follows:

— for t < 28 days,

if $\frac{f_{cm}(t)}{f_{ck}} < 0{,}1$ $\varepsilon_{ca}(t) = 0$ (B.113)

if $\frac{f_{cm}(t)}{f_{ck}} \geq 0{,}1$ $\varepsilon_{ca}(t) = (f_{ck} - 20)\left(2{,}2\frac{f_{cm}(t)}{f_{ck}} - 0{,}2\right)10^{-6}$ (B.114)

where ε_{ca} is the autogeneous shrinkage occurring between setting and time t. In cases where this strength $f_{cm}(t)$ is not known, it can be evaluated in accordance with 3.1.2(6) of EN 1992-1-1.

— for $t \geq$ 28 days,

$\varepsilon_{ca}(t) = (f_{ck} - 20)\ [2{,}8 - 1{,}1 \exp(-t/96)]\ 10^{-6}$ (B.115)

Therefore, according to this model, 97 % of total autogeneous shrinkage has occurred after 3 months.

B.103.2 Drying shrinkage

The formulae in 103.2 apply to RH values of up to 80 %.

(101) The expression for drying shrinkage is as follows:

$$\varepsilon_{cd}(t) = \frac{K(f_{ck})\left[72 \exp(-0{,}046 f_{ck}) + 75 - RH\right](t - t_s)10^{-6}}{(t - t_s) + \beta_{cd} h_0^2} \quad \text{(B.116)}$$

with: $K(f_{ck}) = 18$ if $f_{ck} \leq 55$ MPa
$K(f_{ck}) = 30 - 0{,}21 f_{ck}$ if $f_{ck} > 55$ MPa.

$$\beta_{cd} = \begin{cases} 0{,}007 & \text{for silica-fume concrete} \\ 0{,}021 & \text{for non silica-fume concrete} \end{cases}$$

B.103.3 Creep

The formulae in 103.3 apply to RH values of up to 80%.

(101) The delayed stress dependent strain, $\varepsilon_{cc}(t,t_0)$, i.e. the sum of basic and drying creep, can be calculated by the following expression:

$$\varepsilon_{cc}(t,t_0) = \frac{\sigma(t_0)}{E_c}\left[\varphi_b(t,t_0) + \varphi_d(t,t_0)\right] \quad \text{(B.117)}$$

B.103.4 Basic creep

(101) The final basic creep coefficient of silica fume concrete has been found to depend on the strength at loading $f_{cm}(t_0)$. Furthermore, the younger the concrete at loading, the faster the deformation. However this tendency has not been observed for non silica-fume concrete. For this material, the creep coefficient is assumed to remain constant at a mean value of 1,4. The kinetics term is therefore a function of the maturity, expressed by the quantity $f_{cm}(t)/f_{ck}$. The equation is:

$$\varphi_b(t,t_0) = \varphi_{b0}\frac{\sqrt{t - t_0}}{\left[\sqrt{t - t_0} + \beta_{bc}\right]} \quad \text{(B.118)}$$

with:

$$\varphi_{b0} = \begin{cases} \dfrac{3{,}6}{f_{cm}(t_0)^{0{,}37}} & \text{for silica – fume concrete} \\ 1{,}4 & \text{for non silica – fume concrete} \end{cases} \tag{B.119}$$

and

$$\beta_{bc} = \begin{cases} 0{,}37\exp\left(2{,}8\dfrac{f_{cm}(t_0)}{f_{ck}}\right) & \text{for silica – fume concrete} \\ 0{,}4\exp\left(3{,}1\dfrac{f_{cm}(t_0)}{f_{ck}}\right) & \text{for non silica – fume concrete} \end{cases} \tag{B.120}$$

B.103.5 Drying creep

The formulae in 103.5 apply to RH values of up to 80%.

(101) The drying creep, which is very low for silica fume concrete, is evaluated with reference to the drying shrinkage occurring during the same period. The drying creep coefficient may be expressed by the following simplified equation:

$$\varphi_d(t, t_0) = \varphi_{d0}\,[\varepsilon_{cd}(t) - \varepsilon_{cd}(t_0)] \tag{B.121}$$

with:

$$\varphi_{d0} = \begin{cases} 1\,000 & \text{for silica - fume concrete} \\ 3\,200 & \text{for non silica - fume concrete} \end{cases}$$

B.104 Experimental identification procedure

(101) In order to evaluate delayed strains with greater precision, it may be necessary to identify the parameters included in the models describing creep and shrinkage from experimental measurements. The following procedure, based on the experimental determination of coefficients altering the formulae of Section B.103, may be used.

(102) Experimental data may be obtained from appropriate shrinkage and creep tests both in autogeneous and drying conditions. The measurements should be obtained under controlled conditions and recorded for at least 6 months.

B.104.1 Autogeneous shrinkage

(101) The autogeneous shrinkage model has to be separated in to two parts.

— for t < 28 days,

$$\text{if } \frac{f_{cm}(t)}{f_{ck}} \geq 0{,}1 \quad \varepsilon_{ca}(t) = \beta_{ca1}(f_{ck} - 20)\left(2{,}2\frac{f_{cm}(t)}{f_{ck}} - 0{,}2\right)10^{-6} \tag{B.122}$$

The parameter β_{ca1} has to be chosen in order to minimise the sum of the squares of the differences between the model estimation and the experimental results from the beginning of the measurement to 28 days.

— for $t \geq 28$ days,

$$\varepsilon_{ca}(t) = \beta_{ca1}(f_{ck} - 20)[\beta_{ca2} - \beta_{ca3}\exp(-t/\beta_{ca4})]\ 10^{-6} \qquad \text{(B.123)}$$

The other parameters β_{ca2}, β_{ca3}, β_{ca4} are then chosen using the same method.

B.104.2 Drying shrinkage

The formulae in 104.2 apply to RH values of up to 80%.

(101) The expression for drying shrinkage is as follows,

$$\varepsilon_{cd}(t) = \beta_{cd1}\frac{\mathrm{K}(f_{ck})[72\ \exp(-0{,}046 f_{ck}) + 75 - RH](t - t_s)10^{-6}}{(t - t_s) + \beta_{cd2}h_0^{\,2}} \qquad \text{(B.124)}$$

The parameters β_{cd1}, β_{cd2} have to be chosen in order to minimise the sum of the squares of the differences between the model estimation and the experimental results.

B.104.3 Basic creep

(101) Two parameters have to be identified, a global one β_{cd1} which is applied to the entire expression for basic creep,

$$\varphi_b\left(t, t_0, f_{ck}, f_{cm}(t_0)\right) = \beta_{cd1}\varphi_{b0}\frac{\sqrt{t - t_0}}{\left[\sqrt{t - t_0} + \beta_{bc}\right]} \qquad \text{(B.125)}$$

and β_{bc2} which is included in β_{bc}:

$$\beta_{bc} = \begin{cases} \beta_{bc2}\exp\left(2{,}8\dfrac{f_{cm}(t_0)}{f_{ck}}\right) & \text{for silica} - \text{fume concrete} \\ \beta_{bc2}\exp\left(3{,}1\dfrac{f_{cm}(t_0)}{f_{ck}}\right) & \text{for non silica} - \text{fume concrete} \end{cases} \qquad \text{(B.126)}$$

These two parameters have to be determined by minimising the sum of the square of the difference between experimental results and model estimation.

B.104.4 Drying creep

The formulae in 104.4 apply to RH values of up to 80%.

(101) Only the global parameter φ_{d0} has to be identified.

$$\varphi_d(t) = \varphi_{d0}[\varepsilon_{cd}(t) - \varepsilon_{cd}(t_0)] \qquad \text{(B.127)}$$

This parameter has to be determined by minimising the sum of the squares of the differences between experimental results and model estimation.

B.105 Long term delayed strain estimation

(101) Creep and shrinkage formulae and experimental determinations are based on data collected over limited periods of time. Extrapolating such results for very long-term evaluations (e.g. one hundred years) results in the introduction of additional errors associated with the mathematical expressions used for the extrapolation.

(102) The formulae given in Sections B.1, B.2 and B.103 of this Annex provide a satisfactory average estimation of delayed strains extrapolated to the long-term. However, when safety would be increased by overestimation of delayed strains, and when it is relevant in the project, the creep and shrinkage predicted on the basis of the formulae or experimental determinations should be multiplied by a safety factor.

(103) In order to take into account uncertainty regarding the real long term delayed strains in concrete (ie. uncertainty related to the validity of extrapolating mathematical formulae fitting creep and shrinkage measurements on a relatively short period), the following safety factor γ_{lt} can be included. Values for γ_{lt} are given in Table B.101

Table B.101 — Safety factor for long-term extrapolation of delayed strains, when relevant

t (age of concrete for estimating the delayed strains)	γ_{lt}
t < 1 year	1
t = 5 years	1,07
t = 10 years	1,1
t = 50 years	1,17
t = 100 years	1,20
t = 300 years	1,25

which corresponds to the following mathematical expression:

$$\begin{cases} t \leq 1\,\text{year} & \gamma_{\text{lt}} = 1 \\ t \geq 1\,\text{year} & \gamma_{\text{lt}} = 1 + 0{,}1 \log\left(\dfrac{t}{t_{\text{ref}}}\right) \quad \text{with} \quad t_{\text{ref}} = 1\,\text{year} \end{cases} \tag{B.128}$$

For concrete aged less than one year the B1, B2 and B103 expressions can be used directly, since they correspond to the duration of the experiments used for formulae calibration.

[AC1› For concrete aged 1 year or more, and thus especially for long-term evaluations of deformations, the values given in by Expressions (B.1) and (B.11) of EN 1991-1-1 and by Expressions (B.116) and (B.118) of EN 1991-2 (amplitude of delayed strains at time t) have to be multiplied by γ_{lt}. ‹AC1]

ANNEX C
(normative)

Properties of reinforcement suitable for use with this Eurocode

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

ANNEX D
(informative)

Detailed calculation method for prestressing steel relaxation losses

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

Annex E
(informative)

Indicative strength classes for durability

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

Annex F
(Informative)

Tension reinforcement expressions for in-plane stress conditions

NOTE The sign convention used in this Annex follows that in EN 1992-1-1 and is different to that used in Section 6.9, Annex LL and Annex MM of this standard.

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

F.1 (1)
F.1 (2)
F.1 (3)
F.1 (5)

F.1 General

(104) In locations where σ_{Edy} is tensile or $\sigma_{\mathrm{Edx}} \cdot \sigma_{\mathrm{Edy}} \leq \tau^2_{\mathrm{Edxy}}$, reinforcement is required.

The optimum reinforcement, corresponding to θ = 45°, is indicated by superscript ′, and related concrete stress are determined by:

For $\sigma_{\mathrm{Edx}} \leq |\tau_{\mathrm{Edxy}}|$

$$f'_{\mathrm{tdx}} = |\tau_{\mathrm{Edxy}}| - \sigma_{\mathrm{Edx}} \tag{F.2}$$

$$f'_{\mathrm{tdy}} = |\tau_{\mathrm{Edxy}}| - \sigma_{\mathrm{Edy}} \tag{F.3}$$

$$\sigma_{\mathrm{cd}} = 2|\tau_{\mathrm{Edxy}}| \tag{F.4}$$

For $\sigma_{\mathrm{Edx}} > |\tau_{\mathrm{Edxy}}|$

$$f'_{\mathrm{tdx}} = 0 \tag{F.5}$$

$$f'_{\mathrm{tdy}} = \frac{\tau^2_{\mathrm{Edxy}}}{\sigma_{\mathrm{Edx}}} - \sigma_{\mathrm{Edy}} \tag{F.6}$$

$$\sigma_{\mathrm{cd}} = \sigma_{\mathrm{Edx}}\left(1 + \left(\frac{\tau_{\mathrm{Edxy}}}{\sigma_{\mathrm{Edx}}}\right)^2\right) \tag{F.7}$$

The concrete stress, σ_{cd}, should be checked with a realistic model of cracked sections (see Section 6.109 ‘Membrane elements’ in EN 1992-2).

NOTE The minimum reinforcement is obtained if the directions of reinforcement are identical to the directions of the principal stresses.

Alternatively, for the general case the necessary reinforcement and the concrete stress may be determined by:

$$f_{tdx} = |\tau_{Edxy}| \cot\theta - \sigma_{Edx} \tag{F.8}$$

$$f_{tdy} = |\tau_{Edxy}| / \cot\theta - \sigma_{Edy} \tag{F.9}$$

$$\sigma_{cd} = |\tau_{Edxy}| \left(\cot\theta + \frac{1}{\cot\theta} \right) \tag{F.10}$$

where θ is the angle of the principal concrete compressive stress to the x-axis.

NOTE The value of $\cot\theta$ should be chosen to avoid compression values of f_{td}.

In order to avoid unacceptable cracks for the serviceability limit state, and to ensure the required deformation capacity for the ultimate limit state, the reinforcement derived from Expressions (F.8) and (F.9) for each direction should not be more than twice and not less than half the reinforcement determined by expressions (F.2) and (F.3) or (F.5) and (F.6). These limitations are expressed by $\frac{1}{2} f'_{tdx} \leq f_{tdx} \leq 2 f'_{tdx}$ and $\frac{1}{2} f'_{tdy} \leq f_{tdy} \leq 2 f'_{tdy}$.

Annex G
(informative)

Soil structure interaction

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

Annex H
(informative)

Global second order effects in structures

This Annex does not apply

Annex I
(informative)

Analysis of flat slabs and shear walls

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

I.1.1 (1)
I.1.1 (2)
I.1.2 (1)
I.1.2 (2)
I.1.2 (3)

I.1.2 Equivalent frame analysis

(4) and (5) of EN 1992-1-1 do not apply

I.1.3 Irregular column layout

This clause does not apply

I.2 Shear walls

This clause does not apply

Annex J
(informative)

Detailing rules for particular situations

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

J.1 (1)	*J.2.1 (1)*	*J.2.3 (1)*	*J.3 (4)*
J.1 (3)	*J.2.2 (1)*	*J.2.3 (2)*	*J.3 (5)*
J.1 (4)	*J.2.2 (2)*	*J.3 (1)*	
J.1 (5)	*J.2.2 (3)*	*J.3 (2)*	
J.1 (6)	*J.2.2 (4)*	*J.3 (3)*	

J.104 Partially loaded areas

J.104.1 Bearing zones of bridges

(101) The design of bearing zones of bridges should be in accordance with the rules given in this clause in addition to those in 6.5 and 6.7 of EN 1992-1-1.

(102) The distance from the edge of the loaded area to the free edge of the concrete section should not be less than 1/6 of the corresponding dimension of the loaded area measured in the same direction. In no case should the distance to the free edge be taken as less than 50 mm.

(103) For concrete classes equal to or higher than C55/67, f_{cd} in formula (6.63) of EN 1992-1-1 should be substituted by $\frac{0{,}46 \cdot f_{ck}^{2/3}}{1+0{,}1 \cdot f_{ck}} \cdot f_{cd}$.

(104) In order to avoid edge sliding, uniformly distributed reinforcement parallel to the loaded face should be provided to the point at which local compressive stresses are dispersed. This point is determined as follows: A line inclined at an angle θ (30°) to the direction of load application is drawn from the edge of the section to intersect with the opposite edge of the loaded surface, as shown in Figure J.107. [AC1⟩ The reinforcement provided to avoid edge sliding should be adequately anchored. ⟨AC1]

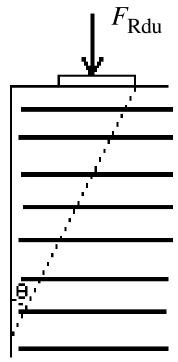


Figure J.107 — Edge sliding mechanism

(105) The reinforcement provided in order to avoid edge sliding (A_r) should be calculated in accordance with the expression $A_r \cdot f_{yd} \geq F_{Rdu}/2$.

J.104.2 Anchorage zones of post-tensioned members

(101) The following rules apply in addition to those in 8.10.3 of EN 1992-1-1 for the design of anchorage zones where two or more tendons are anchored.

(102) The bearing stress behind anchorage plates should be checked as follows:

— The minimum distance between the centreline of the anchorage and the edge of the concrete should not be less than that specified in the appropriate European Technical Approval. This minimum value depends on the strength of the concrete at the time of tensioning.

— The reinforcement required to prevent bursting and spalling in anchorage zones is determined in relation to a rectangular prism of concrete, known as the primary regularisation prism, located behind each anchorage. The cross section of the prism associated with each anchorage is known as the associate rectangle. The associate rectangle has the same centre and the same axes of symmetry as the anchorage plate (which should have two axes of symmetry) and should satisfy:

$$\frac{P_{max}}{c \cdot c'} \leq 0{,}6 \cdot f_{ck}(t) \tag{J.101}$$

where

P_{max} is the maximum force applied to the tendon according to 5.10.2.1 of EN 1992-1-1

c, c' are the dimensions of the associate rectangle

$f_{ck}(t)$ is the concrete strength at the time of tensioning

The associate rectangle should have approximately the same aspect ratio as the anchorage plate. This requirement is satisfied if c/a and c'/a' are not greater than $1{,}25\sqrt{\frac{c \cdot c'}{a \cdot a'}}$ where a and a' are the dimensions of the smallest rectangle including the anchorage plate.

— Rectangles associated with anchorages located in the same cross section should remain inside the concrete and should not overlap

— The “primary regularisation prism” represents very approximately the volume of concrete in which the stresses change from very high values just behind the anchorage plate to a reasonable value for concrete under uniaxial compression. The axis of the prism is taken as the axis of the tendon, its base is the associate rectangle and its depth behind the anchorage is taken as 1,2.max(c, c'). [AC1) The prisms associated with different anchorages may overlap when the tendons are not parallel, but should remain inside the concrete. (AC1]

(103) Reinforcement to prevent bursting and spalling of the concrete, in each regularisation prism (as defined in rule (102) above) should not be less than:

$$A_s = 0{,}15 \frac{P_{max}}{f_{yd}} \gamma_{p,unfav} \quad \text{with } \gamma_{p,unfav} \geq 1{,}20 \tag{J.102}$$

where P_{max} is the maximum force applied to the tendon according to 5.10.2.1 expression (5.41) of EN 1992-1-1 and f_{yd} is the design strength of the reinforcing steel.

This reinforcement should be distributed in each direction over the length of the prism. The area of the surface reinforcement at the loaded face should not be less than $0{,}03\dfrac{P_{\max}}{f_{\mathrm{yd}}}\gamma_{\mathrm{p,unfav}}$ in each direction.

(104) The minimum reinforcement derived from the appropriate European Technical Approval for the prestressing system should be provided. The arrangement of the reinforcement should be modified if it is utilised to withstand the tensile forces calculated according to 8.10.3 (4) of EN 1992-1-1.

Annex KK
(informative)

Structural effects of time dependent behaviour of concrete

KK.1 Introduction

This Annex describes different methods of evaluating the time dependent effects of concrete behaviour.

KK.2 General considerations

(101) Structural effects of time dependent behaviour of concrete, such as variation of deformation and/or [AC1) of internal actions, should be considered, in general, in serviceability conditions. (AC1]

NOTE In particular cases (e.g. structures or structural elements sensitive to second order effects or structures, in which action effects cannot be redistributed) time dependent effects may also have an influence at ULS.

(102) When the compressive stresses in concrete are less than 0,45 $f_{ck}(t)$ under the quasi permanent combination, a linear structural analysis and a linear ageing viscoelastic model is appropriate. The time dependent behaviour of concrete should be described by the creep coefficient $\varphi(t,t_0)$ or the creep function $J(t,t_0)$ or, alternatively, by the relaxation function $R(t,t_0)$. For higher compressive stresses, non-linear creep effects should be considered.

(103) Time dependent analysis for the evaluation of deformation and internal actions of rigid restrained reinforced and prestressed concrete structures may be carried out assuming them to be homogeneous and the limited variability of concrete properties in different regions of the structure may be ignored. Any variation in restraint conditions during the construction stages or the lifetime of the structure should be taken into account in the evaluation.

(104) Different types of analysis and their typical applications are shown in Table KK 101.

Table KK.101 — Type of analysis

Type of analysis	Comment and typical application
General and incremental step-by-step method	These are general methods and are applicable to all structures. Particularly useful for verification at intermediate stages of construction in structures where properties vary along the length (e.g. cantilever construction).
Methods based on the theorems of linear viscoelasticity	Applicable to homogeneous structures with rigid restraints.
The ageing coefficient method	This method will be useful when only the long -term distribution of forces and stresses are required. Applicable to bridges with composite sections (precast beams and in-situ concrete slabs).
Simplified ageing coefficient method	Applicable to structures that undergo changes in support conditions (e.g. span-to-span or free cantilever construction).

The following assumptions are made in all the methods noted above:

— Creep and shrinkage are considered independent of each other.

— For each type of concrete in a section, average creep and shrinkage properties are adopted ignoring any minor differences at different locations.

— The principle of superposition is valid for the assessment of total deformation due to actions applied at various ages.

Brief outline details of some of the methods are given in the following sections.

KK.3 General method

(101) The following assumptions are made:

a) The fundamental equation for time dependent concrete strain is:

$$\varepsilon_c(t) = \frac{\sigma_0}{E_c(t_0)} + \varphi(t,t_0)\frac{\sigma_0}{E_c(28)} + \sum_{i=1}^{n}\left(\frac{1}{E_c(t_i)} + \frac{\varphi(t,t_i)}{E_c(28)}\right)\Delta\sigma(t_i) + \varepsilon_{cs}(t,t_s) \qquad \text{(KK.101)}$$

In this equation, the first term represents the instantaneous deformations due to a stress applied at t_0. The second term represents the creep due to this stress. The third term represents the sum of the instantaneous and creep deformations due to the variation in stresses occurring at instant t_i. The fourth term represents the shrinkage deformation.

b) Reinforcing steel is assumed to behave linearly under instantaneous loads. When the stress in pre-stressing steel is greater than $0{,}5f_{pmax}$ relaxation and a variable state of deformation should be taken into account.

c) Perfect bond exists between concrete and the bonded steel.

d) In the case of linear elements, sections are assumed to be plane before and after deformation.

e) Equilibrium and compatibility are maintained.

(102) Concrete creep at each section depends on its stress history. This is accounted for by a step-by-step process. Structural analysis is carried out at successive time intervals maintaining conditions of equilibrium and compatibility and using the basic properties of materials relevant at the time under consideration. The deformation is computed at successive time intervals using the variation of concrete stress in the previous time interval.

KK.4 Incremental method

(101) At time t where the applied stress is σ, the creep strain $\varepsilon_{cc}(t)$, the potential creep strain $\varepsilon_{\infty cc}(t)$ (ie. the creep strain that would be reached at time $t = \infty$, if the stress applied at time t were kept constant) and the creep rate are theoretically derived from the whole loading history.

(102) The potential creep strain at time t may be evaluated using the principle of superposition (for notations, see formula (KK.101) and EN 1992-1-1 Annex B):

$$\frac{d\varepsilon_{\infty cc}(t)}{dt} = \frac{d\sigma}{dt}\frac{\varphi(\infty,t)}{E_c} \qquad \text{(KK.102)}$$

(103) At time t, it is possible to define an equivalent time t_e such that, under a constant stress applied from time t_e, the same creep strain and the same potential creep strain are obtained; t_e fulfils the equation:

$$\varepsilon_{\infty cc}(t) \cdot \beta_c(t,t_e) = \varepsilon_{cc}(t) \qquad \text{(KK.103)}$$

The creep rate at time t can thus be calculated using the creep curve corresponding to the equivalent time:

$$\frac{d\varepsilon_{cc}(t)}{dt} = \varepsilon_{\infty cc}(t)\frac{\partial\beta_c(t,t_e)}{\partial t} \qquad \text{(KK.104)}$$

(104) When $|\varepsilon_{cc}(t)| > |\varepsilon_{\infty cc}(t)|$, which particularly applies to the case of creep unloading, t_e is defined relative to the current phase and accounts for the sign change of the applied stress. It reads:

$$\varepsilon_{ccMax}(t) - \varepsilon_{cc}(t) = (\varepsilon_{ccMax}(t) - \varepsilon_{\infty cc}(t)) \cdot \beta_c(t,t_e) \qquad \text{(KK.105)}$$

$$\frac{d(\varepsilon_{ccMax}(t) - \varepsilon_{cc}(t))}{dt} = (\varepsilon_{ccMax}(t) - \varepsilon_{\infty cc}(t)) \cdot \frac{\partial\beta_c(t,t_e)}{\partial t} \qquad \text{(KK.106)}$$

where $\varepsilon_{ccMax}(t)$ is the last extreme creep strain reached before time t.

KK.5 Application of theorems of linear viscoelasticity

(101) In structures with rigid restraints, stresses and deformations may initially be evaluated by means of an elastic analysis of the structure in which the elastic modulus is assumed to be constant.

(102) Time dependent properties of concrete are fully characterised by the creep function $J(t,t_0)$ and the relaxation function $R(t,t_0)$, where:

$J(t,t_0)$ represents the total stress dependent strain per unit stress, i.e. the strain response at time "t" resulting from a sustained and constant imposed unit stress applied at time "t_0"

$R(t,t_0)$ represents the stress response at time "t" resulting from a sustained and constant imposed unit stress-dependent strain applied at time "t_0"

(103) Under direct actions (imposed loads) the elastic stresses are not modified by creep. The deformations $D(t)$ may be evaluated at time "t" by integration of elastic strain increments factored by the creep factor $J(t,\tau)\cdot E_C$

$$S(t) = S_{el}(t_0) \tag{KK.107}$$

$$D(t) = E_C \int_0^t J(t,\tau)\, d\, D_{el}(\tau) \tag{KK.108}$$

(104) Under indirect actions (imposed deformations) the elastic deformations are not modified by creep. The stresses may be evaluated at time "t" by integration of the elastic stress increments factored by the relaxation factor $R(t,\tau)/E_C$

AC1 $D(t) = D_{el}(t)$ AC1 (KK.109)

$$S(t) = \frac{1}{E_C} \int_0^t R(t,\tau)\, d\, S_{el}(\tau) \tag{KK.110}$$

(105) In a structure subjected to imposed constant loads, whose initial static scheme (101) is modified into a final scheme (102) by the introduction of additional restraint at time $t_1 \geq t_0$ (t_0 being the structure age at loading), the stress distribution evolves for $t > t_1$ and approaches that corresponding to the load application in the final static scheme

$$S_2(t) = S_{el,1} + \xi(t, t_0, t_1)\, \Delta S_{el,1} \tag{KK.111}$$

where:

$S_2(t)$ is the stress distribution for $t > t_1$ in the structure with modified restraints;

$S_{el,1}$ is the elastic stress distribution in the initial static scheme;

$\Delta S_{el,1}$ is the correction to be applied to the elastic solution $S_{el,1}$ to comply with the elastic solution related to the load application in the final static scheme.

$\xi(t, t_0, t_1)$ is the redistribution function

$$\xi(t,t_0,t_1) = \int_{t_1}^t R(t,\tau)\, dJ(\tau,t_0) \tag{KK.112}$$

with $0 \leq \xi(t,t_0,t_1) \leq 1$

and $\xi(t,t_0,t_0^+) = 1 - \frac{R(t,t_0)}{E_C(t_0)}$ (KK.113)

(106) In cases in which the transition from the initial static scheme to the final scheme is reached by means of several different restraint modifications applied at different times $t_i \geq t_0$, the stress variation induced by creep, by the effect of applying a group Δn_j of additional restraints at time t_j, is independent of the history of previous additional restraints introduced at times $t_i < t_j$ and depends only on the time t_j of application of Δn_j restraints

$$S_{j+1} = S_{\text{el},1} + \sum_{i=1}^{j} \xi(t, t_0, t_i)\Delta S_{\text{el},i} \qquad \text{(KK.114)}$$

KK.6 Ageing coefficient method

(101) The ageing coefficient method enables variations in stress, deformation, forces and movements due to the time-dependent behaviour of the concrete and the prestressing steel at infinite time to be calculated without discrete time related analysis. In particular, on a section level, the changes in axial deformation and curvature due to creep, shrinkage and relaxation may be determined using a relatively simple procedure.

(102) The deformation produced by stress variations with time in the concrete may be taken as that AC1 which would result from a variation in stress applied AC1 and maintained from an intermediate age.

AC1 $$\int_{\tau=t_0}^{t} \left[\frac{E_c(28)}{E_c(\tau)} + \varphi_{28}(t,\tau) \right] d\sigma(\tau) = \left[\frac{E_c(28)}{E_c(t_0)} + \chi(t,t_0)\varphi_{28}(t,\tau) \right] \Delta\sigma_{t_0 \to t} \qquad \text{(KK.118)}$$ AC1

where χ is the ageing coefficient. The value of χ may be determined at any given moment, by means of a step-by-step calculation or may be taken as being equal to 0,80 for $t = \infty$.

Relaxation at variable deformation may be evaluated in a simplified manner at infinite time as being the relaxation at constant length, multiplied by a reduction factor of 0,80.

KK.7 Simplified formulae

(101) Forces at time t_∞ may be calculated for those structures that undergo changes in support conditions (span-to-span construction, free cantilever construction, movements at supports, etc.) using a simplified approach. In these cases, as a first approximation, the internal force distribution at t_∞ may be taken as

AC1 $$S_\infty = S_0 + (S_1 - S_0)\frac{E_c(t_1)}{E_c(t_0)}\left[\frac{\varphi(\infty,t_0) - \varphi(t_1,t_0)}{1 + \chi\varphi(\infty,t_1)}\right] \qquad \text{(KK.119)}$$ AC1

where:

S_0 represents the internal forces at the end of the construction process.

AC1 S_1 represents the internal forces in the final static scheme.

t_0 is the concrete age at application of the constant permanent loads.

t_1 is the age of concrete when the restraint conditions are changed. AC1

Annex LL
(informative)

Concrete shell elements

(101) This section applies to shell elements, in which there are generally eight components of internal forces. The eight components of internal forces are listed below and shown in Figure LL.1 for an element of unit dimensions:

— 3 plate components n_{Edx}, n_{Edy}, $n_{Edxy} = n_{Edyx}$

— 3 slab components m_{Edx}, m_{Edy}, $m_{Edxy} = m_{Edyx}$

— 2 transverse shear forces v_{Edx}, v_{Edy}

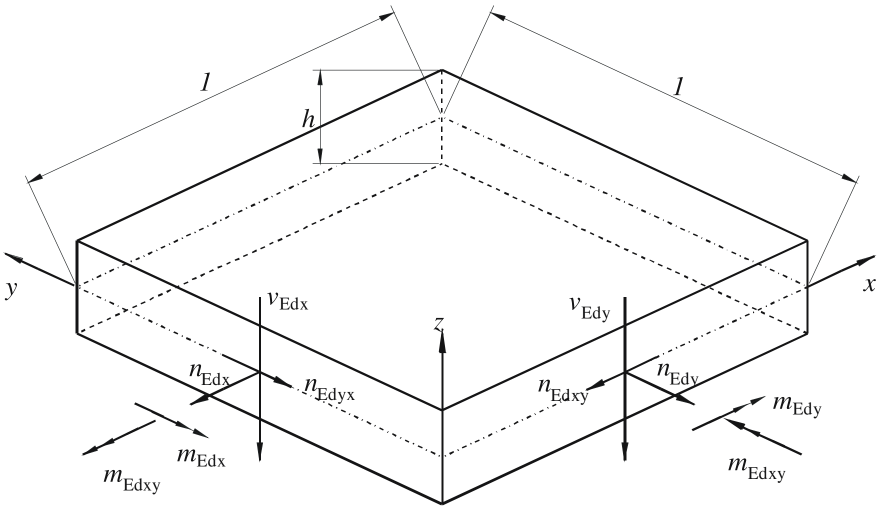


Figure LL.1 — Shell element

(102) The first stage in the verification procedure is to establish if the shell element is uncracked or cracked.

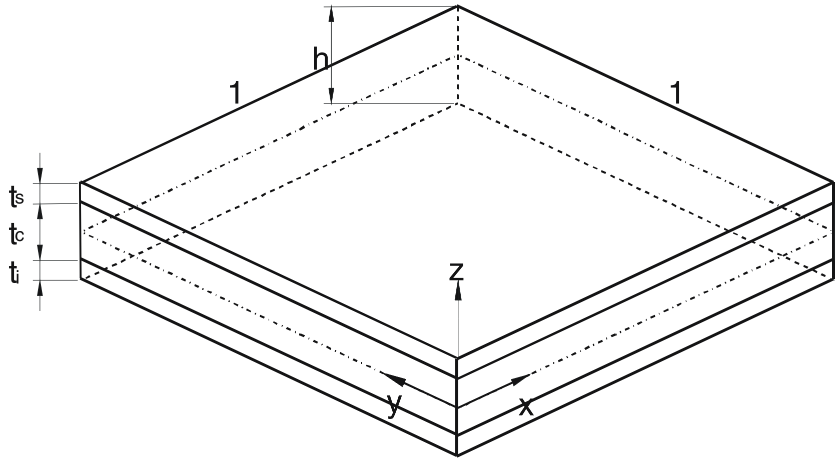


Figure LL.2 — The sandwich model

(103) In uncracked elements the only verification required is to check that the minimum principal stress is smaller than the design compressive strength f_{cd}. It may be appropriate to take into account the multiaxial compression state in the definition of f_{cd}.

(104) In cracked elements a sandwich model should be used for design or verification of the shell element.

(105) In the sandwich model three layers are identified (Figure LL.2): the two outer layers resist the membrane actions arising from n_{Edx}, n_{Edy}, n_{Edxy}, m_{Edx}, m_{Edy}, m_{Edxy}; and the inner layer carries the shear forces v_{Edx}, v_{Edy}. The thickness of the different layers should be established by means of an iterative procedure (see rules (113) to (115)).

(106) The inner layer should be designed according to 6.2, taking into account the principal shear, its principal direction and the longitudinal reinforcement components in that direction (see rules (113) to (115)).

(107) In order to establish whether shell elements are cracked, the principal stresses at different levels within the thickness of the element should be checked. In practice the following inequality should be verified:

$$\Phi = \alpha \frac{J_2}{f_{cm}{}^2} + \lambda \frac{\sqrt{J_2}}{f_{cm}} + \beta \frac{I_1}{f_{cm}} - 1 \le 0 \qquad \text{(LL.101)}$$

where:

$$J_2 = \frac{1}{6}[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2] \qquad \text{(LL.102)}$$

$$J_3 = (\sigma_1 - \sigma_m)\,(\sigma_2 - \sigma_m)\,(\sigma_3 - \sigma_m) \qquad \text{(LL.103)}$$

$$I_1 = \sigma_1 + \sigma_2 + \sigma_3 \qquad \text{(LL.104)}$$

$$\sigma_m = (\sigma_1 + \sigma_2 + \sigma_3)/3 \qquad \text{(LL.105)}$$

$$\alpha = \frac{1}{9k^{1,4}} \qquad \text{(LL.106)}$$

$$\lambda = c_1 \cos\left[\frac{1}{3}\operatorname{ar}\cos(C_2 \cos 3\theta)\right] \quad \text{for} \quad \cos 3\theta \geq 0$$
$$\lambda = c_1 \cos\left[\frac{\pi}{3} - \frac{1}{3}\operatorname{ar}\cos(-C_2 \cos 3\theta)\right] \quad \text{for} \quad \cos 3\theta < 0 \tag{LL.107}$$

$$\beta = \frac{1}{3{,}7k^{1{,}1}} \tag{LL.108}$$

$$\cos 3\theta = \frac{3\sqrt{3}}{2}\frac{J_3}{J_2^{3/2}} \tag{LL.109}$$

$$c_1 = \frac{1}{0{,}7k^{0{,}9}} \tag{LL.110}$$

$$c_2 = 1 - 6{,}8\,(k - 0{,}07)^2 \tag{LL.111}$$

$$k = \frac{f_{\text{ctm}}}{f_{\text{cm}}} \tag{LL.112}$$

If inequality (LL.101) is satisfied, then the element is considered to be uncracked; otherwise it should be considered as cracked.

(108) If the shell element is considered to be cracked, the forces within the outer layers of the sandwich model should be determined according to the following equations (figures LL.3a and LL.3b)

$$n_{\text{Edxs}} = n_{\text{Edx}}\frac{z_{\text{x}} - y_{\text{xs}}}{z_{\text{x}}} + \frac{m_{\text{Edx}}}{z_{\text{x}}} \tag{LL.113}$$

$$n_{\text{Edxi}} = n_{\text{Edx}}\frac{z_{\text{x}} - y_{\text{xi}}}{z_{\text{x}}} - \frac{m_{\text{Edx}}}{z_{\text{x}}} \tag{LL.114}$$

$$n_{\text{Edys}} = n_{\text{Edy}}\frac{z_{\text{y}} - y_{\text{ys}}}{z_{\text{y}}} + \frac{m_{\text{Edy}}}{z_{\text{y}}} \tag{LL.115}$$

$$n_{\text{Edyi}} = n_{\text{Edy}}\frac{z_{\text{y}} - y_{\text{yi}}}{z_{\text{y}}} - \frac{m_{\text{Edy}}}{z_{\text{y}}} \tag{LL.116}$$

$$n_{\text{Edyxs}} = n_{\text{Edyx}}\frac{z_{\text{yx}} - y_{\text{yxs}}}{z_{\text{yx}}} - \frac{m_{\text{Edyx}}}{z_{\text{yx}}} \tag{LL.117}$$

$$n_{\text{Edyxi}} = n_{\text{Edyx}}\frac{z_{\text{yx}} - y_{\text{yxi}}}{z_{\text{yx}}} + \frac{m_{\text{Edyx}}}{z_{\text{yx}}} \tag{LL.118}$$

$$n_{\text{Edxys}} = n_{\text{Edxy}}\frac{z_{\text{xy}} - y_{\text{xys}}}{z_{\text{xy}}} - \frac{m_{\text{Edxy}}}{z_{\text{xy}}} \tag{LL.119}$$

$$n_{\text{Edxyi}} = n_{\text{Edxy}}\frac{z_{\text{xy}} - y_{\text{xyi}}}{z_{\text{xy}}} + \frac{m_{\text{Edxy}}}{z_{\text{xy}}} \tag{LL.120}$$

where:

z_x and z_y are the lever arms for bending moments and membrane axial forces;

y_{xs}, y_{xi}, y_{ys}, y_{yi} are the distances from the centre of gravity of the reinforcement to mid-plane of the element in the x and y directions, in relation to bending and axial membrane forces; therefore $z_x = y_{xs} + y_{xi}$ and $z_y = y_{ys} + y_{yi}$;

y_{yxs}, y_{yxi}, y_{xys}, y_{xyi} are the distances from the centre of gravity of the reinforcement to the mid-plane of the element, in relation to torque moment and shear membrane forces; therefore $z_{yx} = y_{yxs} + y_{yxi}$ and $z_{xy} = y_{xys} + y_{xyi}$;

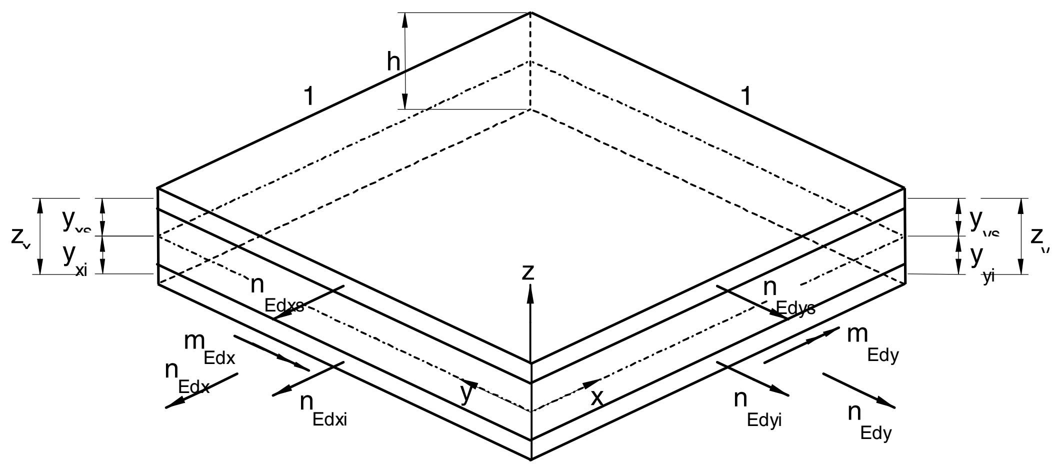


Figure LL.3a — Axial actions and bending moments in the outer layer

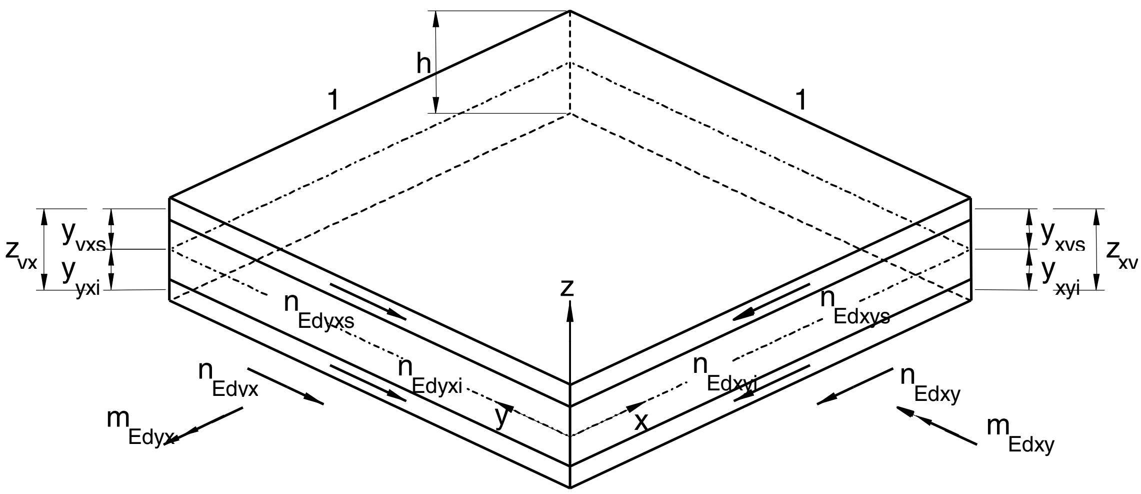


Figure LL.3b — Membrane shear actions and twisting moments in the outer layer

Out of plane shear forces v_{Edx} and v_{Edy} are applied to the inner layer with the lever arm z_c, determined with reference to the centroid of the appropriate layers of reinforcement.

(109) For the design of the inner layer the principal shear v_{Edo} and its direction φ_o should be evaluated as follows:

$$v_{\mathrm{Edo}} = \sqrt{v_{\mathrm{Edx}}^2 + v_{\mathrm{Edy}}^2} \quad \text{(LL.121)}$$

$$\tan \varphi_o = \frac{v_{\mathrm{Edy}}}{v_{\mathrm{Edx}}} \quad \text{(LL.122)}$$

(110) In the direction of principal shear the shell element behaves like a beam and the appropriate design rules should therefore be applied. In particular clause 6.2.2 should be applied for members not requiring shear reinforcement and clause 6.2.3 should be applied for members requiring shear reinforcement. In expression (6.2.a) ρ_l should be taken as:

$$\rho_l = \rho_x \cos^2 \varphi_o + \rho_y \sin^2 \varphi_o \quad \text{(LL.123)}$$

(111) When shear reinforcement is necessary, the longitudinal force resulting from the truss model $V_{\mathrm{Edo}} \cot\theta$ gives rise to the following membrane forces in x and y directions:

$$n_{\mathrm{Edyc}} = \frac{v_{\mathrm{Edy}}^2}{v_{\mathrm{Edo}}} \cot\theta \quad \text{(LL.124)}$$

$$n_{\mathrm{Edxyc}} = \frac{v_{\mathrm{Edx}}\, v_{\mathrm{Edy}}}{v_{\mathrm{Edo}}} \cot\theta \quad \text{(LL.125)}$$

$$n_{\mathrm{Edxc}} = \frac{v_{\mathrm{Edx}}^2}{v_{\mathrm{Edo}}} \cot\theta \quad \text{(LL.126)}$$

$$n_{\mathrm{Edyxc}} = n_{\mathrm{Edxyc}} = \frac{v_{\mathrm{Edx}}\, v_{\mathrm{Edy}}}{v_{\mathrm{Edo}}} \cot\theta \quad \text{(LL.127)}$$

(112) The outer layers should be designed as membrane [AC1) elements, using the design rules of 6.109 and Annex F. (AC1]

(113) The following simplified approach may generally be adopted with respect to figures LL.3a and LL.3b:

$$y_{\mathrm{ns}} = y_{\mathrm{xs}} = y_{\mathrm{ys}} \quad \text{(LL.128)}$$

$$y_{\mathrm{ni}} = y_{\mathrm{xi}} = y_{\mathrm{yi}} \quad \text{(LL.129)}$$

$$y_{\mathrm{ts}} = y_{\mathrm{xys}} = y_{\mathrm{yxs}} \quad \text{(LL.130)}$$

$$y_{\mathrm{ti}} = y_{\mathrm{xyi}} = y_{\mathrm{yxi}} \quad \text{(LL.131)}$$

$$z_x = z_y = z_n = y_{\mathrm{ns}} + y_{\mathrm{ni}} \quad \text{(LL.132)}$$

$$z_{xy} = z_{yx} = z_t = y_{\mathrm{ts}} + y_{\mathrm{ti}} \quad \text{(LL.133)}$$

The difference between z_n and z_t may generally be ignored, [AC1) assuming the thickness of the outer layers to be twice the edge distance to the gravity centre of reinforcement, therefore: (AC1]

$$y_{ns} = y_{ts} = y_s \tag{LL.134}$$

$$y_{ni} = y_{ti} = y_i \tag{LL.135}$$

$$z_n = z_t = z \tag{LL.136}$$

(114) Based on the above assumptions the forces in the outer layers can be evaluated as follows:

a) in the case for which no shear reinforcement is required to resist v_{Edx} and v_{Edy}

$$n_{Edxs} = n_{Edx}\frac{z - y_s}{z} + \frac{m_{Edx}}{z} \tag{LL.137}$$

$$n_{Edxi} = n_{Edx}\frac{z - y_i}{z} - \frac{m_{Edx}}{z} \tag{LL.138}$$

$$n_{Edys} = n_{Edy}\frac{z - y_s}{z} + \frac{m_{Edy}}{z} \tag{LL.139}$$

$$n_{Edyi} = n_{Edy}\frac{z - y_i}{z} - \frac{m_{Edy}}{z} \tag{LL.140}$$

$$n_{Edxys} = n_{Edxy}\frac{z - y_s}{z} - \frac{m_{Edxy}}{z} \tag{LL.141}$$

$$n_{Edxyi} = n_{Edxy}\frac{z - y_i}{z} + \frac{m_{Edxy}}{z} \tag{LL.142}$$

b) in the case for which shear reinforcement is required to resist v_{Edx} and v_{Edy}

$$n_{Edxs} = n_{Edx}\frac{z - y_s}{z} + \frac{m_{Edx}}{z} + \frac{1}{2}\frac{{v_{Edx}}^2}{v_{Edo}}\cot\theta \tag{LL.143}$$

$$n_{Edxi} = n_{Edx}\frac{z - y_i}{z} - \frac{m_{Edx}}{z} + \frac{1}{2}\frac{{v_{Edx}}^2}{v_{Edo}}\cot\theta \tag{LL.144}$$

$$n_{Edys} = n_{Edy}\frac{z - y_s}{z} + \frac{m_{Edy}}{z} + \frac{1}{2}\frac{{v_{Edy}}^2}{v_{Edo}}\cot\theta \tag{LL.145}$$

$$n_{Edyi} = n_{Edy}\frac{z - y_i}{z} - \frac{m_{Edy}}{z} + \frac{1}{2}\frac{{v_{Edy}}^2}{v_{Edo}}\cot\theta \tag{LL.146}$$

$$n_{Edxys} = n_{Edxy}\frac{z - y_s}{z} - \frac{m_{Edxy}}{z} + \frac{1}{2}\frac{v_{Edx}\ v_{Edy}}{v_{Edo}}\cot\theta \tag{LL.147}$$

$$v_{Edxyi} = n_{Edxy}\frac{z - y_i}{z} + \frac{m_{Edxy}}{z} + \frac{1}{2}\frac{v_{Edx}\ v_{Edy}}{v_{Edo}}\cot\theta \tag{LL.148}$$

(115) If the verification in (112) above is not satisfied, one of the following procedures should be followed:

a) increase the concrete cover and consequently reduce the internal lever arm;

b) use different values for z_n and z_t with $z_n > z_t$; internal concrete stresses should then be added vectorially;

c) increase the layer thickness to satisfy the concrete verification and leave the reinforcement position unchanged. This will cause the reinforcement to become eccentric in the layer; as a consequence two internal bending moments arise, and these should be in equilibrium within the shell element. In these circumstances, the internal forces in the reinforcement become:

$$n^*_{\mathrm{Eds}} = \left[n_{\mathrm{Eds}} \left(h - \frac{t_s}{2} - b'_i \right) + n_{\mathrm{Edi}} \left(\frac{t_i}{2} - b'_i \right) \right] \Big/ \left(h - b'_i - b'_s \right) \qquad \text{(LL.149)}$$

$$n^*_{\mathrm{Edi}} = n_{\mathrm{Eds}} + n_{\mathrm{Edi}} - n^*_{\mathrm{Eds}} \qquad \text{(LL.150)}$$

where:

t_s and t_i are the thickness of top and bottom layers, respectively;

$b'_{i,s}$ is the distance from the external surface of the layer to the axis of the reinforcement within the layer.

The internal layer should be checked for an additional out of plane shear corresponding to the force transferred between the layers of reinforcement.

Annex MM
(informative)

Shear and transverse bending

(101) Within the webs of box girders the interaction between longitudinal shear and transverse bending may be considered by means of the sandwich model (see Annex LL). The following simplifications to the general model may be introduced for the purpose of this application (Figure MM.1):

— The longitudinal shear per unit length should be considered as having a constant value along Δx : $v_{Ed} = V_{Ed}/\Delta y$.

— The transverse bending moment per unit length should be considered as having a constant value along Δy : $m_{Ed} = M_{Ed}/\Delta x$.

— The longitudinal force is assumed to have a constant value within the length Δy : $p_{Ed} = P_{Ed}/\Delta y$.

— The transverse shear within the web, due to variation in the corresponding bending moment, should be neglected within the length Δy.

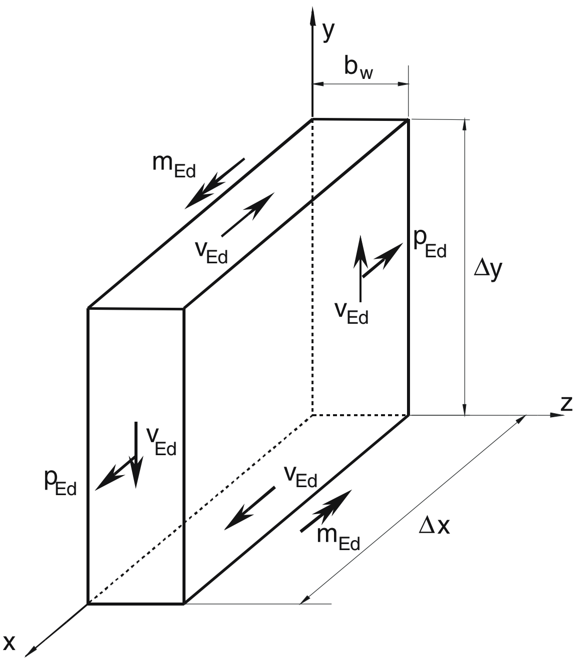


Figure MM.1 — Internal actions in a web element

(102) On the basis of the above assumptions, the sandwich model comprises only two plates in which the following stresses are acting (Figure MM.2)

$$\tau_{Ed1} = v_{Ed} \frac{b_w - z_2}{(2b_w - z_1 - z_2)z_1} \tag{MM.101}$$

$$\tau_{\mathrm{Ed}\,2} = v_{\mathrm{Ed}} \frac{b_{\mathrm{w}} - z_1}{(2b_{\mathrm{w}} - z_1 - z_2)z_2} \qquad \text{(MM.102)}$$

$$\sigma_{\mathrm{Edy}\,1} = \frac{m_{\mathrm{Edx}}}{(b_{\mathrm{w}} - (z_1 + z_2)/2)z_1} \qquad \text{(MM.103)}$$

$$\sigma_{\mathrm{Edy}\,2} = \frac{m_{\mathrm{Edx}}}{(b_{\mathrm{w}} - (z_1 + z_2)/2)z_2} \qquad \text{(MM.104)}$$

$$\sigma_{\mathrm{Edx}\,1} = p_d \frac{b_{\mathrm{w}} - z_2}{(2b_{\mathrm{w}} - z_1 - z_2)z_1} \qquad \text{(MM.105)}$$

$$\sigma_{\mathrm{Edx}\,2} = p_{\mathrm{d}} \frac{b_{\mathrm{w}} - z_1}{(2b_{\mathrm{w}} - z_1 - z_2)z_2} \qquad \text{(MM.106)}$$

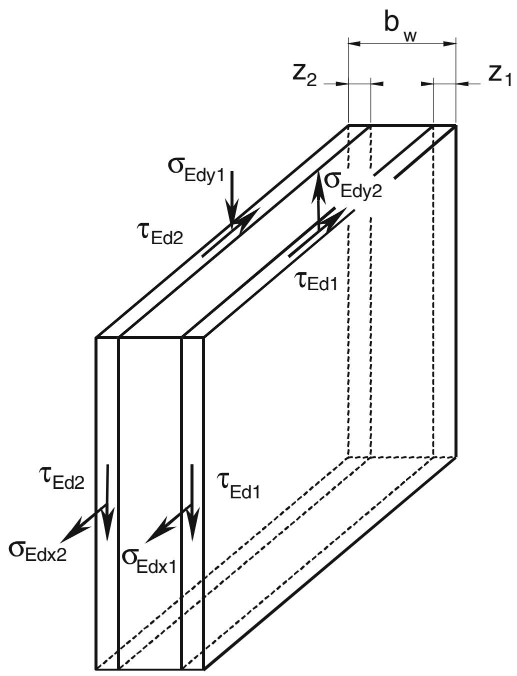


Figure MM.2 — Modified sandwich model

(103) The design of two plates should be based on an iterative approach, in order to optimise the thickness z_1 and z_2, using the procedure given in Section 6.109 and Annex F; different values for the θ_{el} angle and the θ angle may be assumed for the two plates, but they should have a constant value in each plate. If the resulting reinforcement is eccentric within the two plates, the Expressions (LL.149) and (LL.150) of Annex LL should be applied.

(104) If the calculated longitudinal force is tensile, this may be carried by reinforcement distributed along the web or alternatively, may be considered to be transferred to the tensile and compression chords; half to the tensile chord and half to the compression chord.

(105) In the case of there being no longitudinal force, the rules of 6.24 may be used as a simplification, but the shear reinforcement should be added to the bending reinforcement.

Annex NN
(informative)

Damage equivalent stresses for fatigue verification

NN.1 General

(101) This Annex gives a simplified procedure for calculating the damage equivalent stresses for fatigue verification of superstructures of road and railway bridges of concrete construction. The procedure is based on the fatigue load models given in EN 1991-2.

NN.2 Road bridges

NN.2.1 Reinforcing and prestressing steel

(101) The values given in this subclause are only applicable to the modified fatigue load model 3 in EN 1991-2.

For the calculation of damage equivalent stress ranges for steel verification, the axle loads of fatigue load model 3 shall be multiplied by the following factors:

1,75 for verification at intermediate supports in continuous bridges

1,40 for verification in other areas.

(102) The damage equivalent stress range for steel verification shall be calculated according to:

$$\Delta\sigma_{s,equ} = \Delta\sigma_{s,Ec} \cdot \lambda_s \tag{NN.101}$$

where:

$\Delta\sigma_{s,Ec}$ is the stress range caused by fatigue load model 3 (according to EN 1991-2) with the axle loads increased in accordance with (101), based on the load combination given in 6.8.3 of EN 1992-1-1.

λ_s is the damage equivalent factor for fatigue which takes account of site specific conditions including traffic volume on the bridge, design life and the span of the member.

(103) The correction factor λ_s includes the influence of span, annual traffic volume, design life, multiple lanes, traffic type and surface roughness and can be calculated by

$$\lambda_s = \varphi_{fat} \cdot \lambda_{s,1} \cdot \lambda_{s,2} \cdot \lambda_{s,3} \cdot \lambda_{s,4} \tag{NN.102}$$

where:

$\lambda_{s,1}$ is a factor accounting for element type (eg. continuous beam) and takes into account the damaging effect of traffic depending on the critical length of the influence line or area.

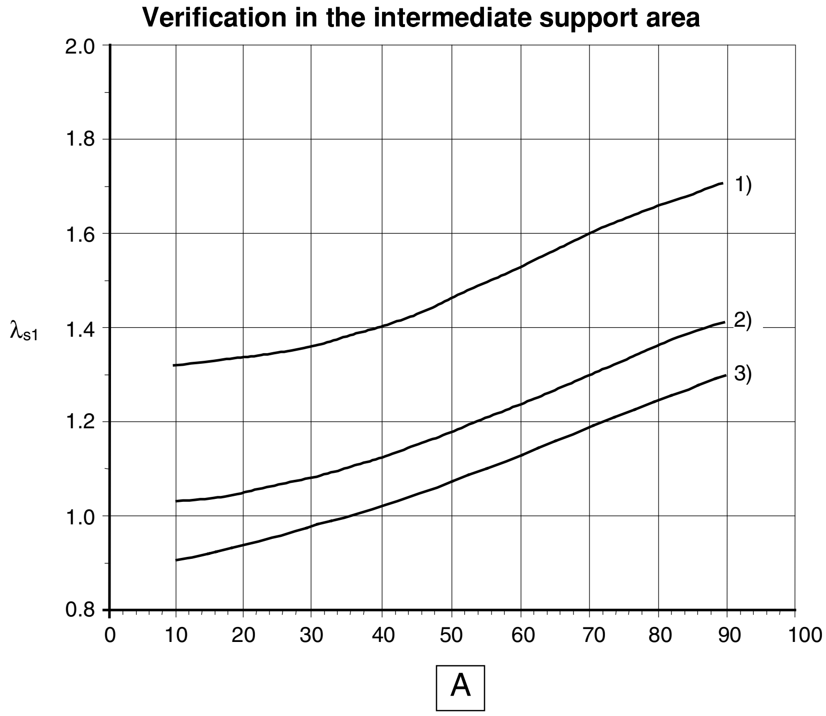
$\lambda_{s,2}$ is a factor that takes into account the traffic volume.

$\lambda_{s,3}$ is a factor that takes into account the design life of the bridge.

$\lambda_{s,4}$ is a factor to be applied when the structural element is loaded by more than one lane.

φ_{fat} is the damage equivalent impact factor controlled by the surface roughness.

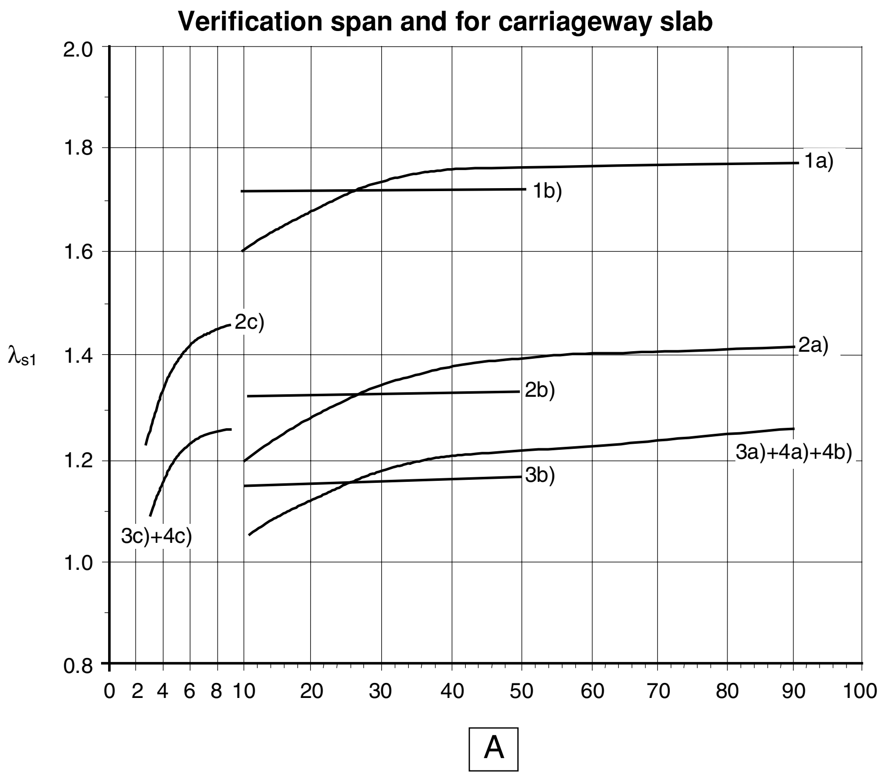
(104) The $\lambda_{s,1}$ value given in Figures NN.1 and NN.2 takes account of the critical length of the influence line and the shape of *S*-*N*-curve



1) splicing devices
2) curved tendons in steel ducts
3) reinforcing steel
pre-tensioning (all)
post-tensioning:
strand in plastic ducts
straight tendons in steel ducts

A Critical length of influence line [m]

Figure NN.1 — $\lambda_{s,1}$ value for fatigue verification in the intermediate support area



1) splicing devices
2) curved tendons in steel ducts
3) reinforcing steel
pre-tensioning (all)
post-tensioning:
strand in plastic ducts
straight tendons in steel ducts
4) shear reinforcement

a) continuous beam
b) single span beam
c) carriageway slab

A Critical length of influence line [m]

Figure NN.2 — $\lambda_{s,1}$ value for fatigue verification in span and for local elements

(105) The $\lambda_{s,2}$ value denotes the influence of the annual traffic volume and traffic type. It can be calculated by Equation (NN.103)

$$\lambda_{s,2} = \overline{Q}\sqrt[k_2]{\frac{N_{obs}}{2,0}} \tag{NN.103}$$

where:

N_{obs} is the number of lorries per year according to EN 1991-2, Table 4.5

k_2 is the slope of the appropriate S-N-Line to be taken from Tables 6.3N and 6.4N of EN 1992-1-1

$\overline{Q}$ is a factor for traffic type according to Table NN.1

Table NN.1 — Factors for traffic type

$\overline{Q}$ -factor for	Traffic type (see EN 1991-2 Table 4.7)		
	Long distance	Medium distance	Local traffic
$k_2 = 5$	1,0	0,90	0,73
$k_2 = 7$	1,0	0,92	0,78
$k_2 = 9$	1,0	0,94	0,82

(106) The $\lambda_{s,3}$ value denotes the influence of the service life and can be calculated from Equation (NN.104)

$$\lambda_{s,3} = \sqrt[k_2]{\frac{N_{Years}}{100}} \tag{NN.104}$$

where:

N_{Years} is the design life of the bridge

(107) The $\lambda_{s,4}$ value denotes the influence for multiple lanes and can be calculated from Equation (NN.105)

$$\lambda_{s,4} = \sqrt[k_2]{\frac{\sum N_{obs,i}}{N_{obs,1}}} \tag{NN.105}$$

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

(108) The φ_{fat} value is a damage equivalent impact factor according to EN 1991-2, Annex B.

NN.3 Railway bridges

NN.3.1 Reinforcing and prestressing steel

(101) The damage equivalent stress range for reinforcing and prestressing steel shall be calculated according to Equation (NN.106)

$$\Delta\sigma_{s,\,equ} = \lambda_s \cdot \Phi \cdot \Delta\sigma_{s,71} \tag{NN.106}$$

where:

$\Delta\sigma_{s,71}$ is the steel stress range due to load model 71 (and where required SW/0), but excluding α according to EN 1991.2, being placed in the most unfavourable position for the element under consideration. For structures carrying multiple tracks, load model 71 shall be applied to a maximum of two tracks

λ_s is a correction factor to calculate the damage equivalent stress range from the stress range caused by $\Phi \cdot \Delta\sigma_{s,71}$

Φ is a dynamic factor according to EN 1991-2

(102) The correction factor λ_s, takes account of the span, annual traffic volume, design life and multiple tracks. It is calculated from the following formula:

$$\lambda_s = \lambda_{s,1} \cdot \lambda_{s,2} \cdot \lambda_{s,3} \cdot \lambda_{s,4} \tag{NN.107}$$

where:

$\lambda_{s,1}$ is a factor accounting for element type (eg. continuous beam) and takes into account the damaging effect of traffic depending on the 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 life of the bridge.

$\lambda_{s,4}$ is a factor to be applied when the structural element is loaded by more than one track.

(103) The factor $\lambda_{s,1}$ is a function of the critical length of the influence line and the traffic. The values of $\lambda_{s,1}$ for standard traffic mix and heavy traffic mix may be taken from Table NN.2 of this Annex. The values have been calculated on the basis of a constant ratio of bending moments to stress ranges. The values given for mixed traffic 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 following equation:

$$\lambda_{s,1}(L) = \lambda_{s,1}(2\text{ m}) + [\lambda_{s,1}(20\text{ m}) - \lambda_{s,1}(2\text{ m})] \cdot (\log L - 0{,}3) \tag{NN.108}$$

where:

L is the critical length of the influence line in m

$\lambda_{s,1}$ (2 m) is the $\lambda_{s,1}$ value for L = 2 m

$\lambda_{s,1}$ (20 m) is the $\lambda_{s,1}$ value for L = 20 m

$\lambda_{s,1}(L)$ is the $\lambda_{s,1}$ value for 2 m < L < 20 m

(104) The $\lambda_{s,2}$ value denotes the influence of annual traffic volume and can be calculated from Equation (NN.109)

$$\lambda_{s,2} = \sqrt[k_2]{\frac{\mathrm{Vol}}{25 \cdot 10^6}} \tag{NN.109}$$

where:

Vol is the volume of traffic (tonnes/year/track)

k_2 is the slope of the appropriate S-N line to be taken from Tables 6.3N and 6.4N of EN 1992-1-1

(105) The $\lambda_{s,3}$ value denotes the influence of the service life and can be calculated from Equation (NN.110)

$$\lambda_{s,3} = \sqrt[k_2]{\frac{N_{\mathrm{Years}}}{100}} \tag{NN.110}$$

where:

N_{Years} is the design life of the bridge

k_2 is the slope of appropriate S-N line to be taken from Tables 6.3N and 6.4N of EN 1992-1-1

(106) The $\lambda_{s,4}$ value denotes the effect of loading from more than one track. For structures carrying multiple tracks, the fatigue loading shall be applied to a maximum of two tracks in the most unfavourable positions (see EN 1991-2). The effect of loading from two tracks can be calculated from Equation (NN.111).

$$\lambda_{s,4} = \sqrt[k_2]{n + (1-n) \cdot s_1^{k_2} + (1-n) \cdot s_2^{k_2}} \tag{NN.111}$$

$$s_1 = \frac{\Delta\sigma_1}{\Delta\sigma_{1+2}} \quad s_2 = \frac{\Delta\sigma_2}{\Delta\sigma_{1+2}}$$

where:

n is the proportion of traffic that crosses the bridge simultaneously (the suggested value of n is 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 71 on any two tracks, according to EN 1991-2

k_2 is the slope of appropriate S-N line to be taken from Tables 6.3N and 6.4N of EN 1992-1-1

If only compressive stresses occur under traffic loads on a track, set the corresponding value $s_j = 0$.

Table NN.2 — $\lambda_{s,1}$ values for simply supported and continuous beams

	L [m]	s^*	h^*
[1]	≤ 2 ≥ 20	0,90 0,65	0,95 0,70
[2]	≤ 2 ≥ 20	1,00 0,70	1,05 0,70
[3]	≤ 2 ≥ 20	1,25 0,75	1,35 0,75
[4]	≤ 2 ≥ 20	0,80 0,40	0,85 0,40

Simply supported beams

	L [m]	s^*	h^*
[1]	≤ 2 ≥ 20	0,95 0,50	1,05 0,55
[2]	≤ 2 ≥ 20	1,00 0,55	1,15 0,55
[3]	≤ 2 ≥ 20	1,25 0,55	1,40 0,55
[4]	≤ 2 ≥ 20	0,75 0,35	0,90 0,30

Continuous beams (mid span)

	L [m]	s^*	h^*
[1]	≤ 2 ≥ 20	0,90 0,65	1,00 0,65
[2]	≤ 2 ≥ 20	1,05 0,65	1,15 0,65
[3]	≤ 2 ≥ 20	1,30 0,65	1,45 0,70
[4]	≤ 2 ≥ 20	0,80 0,35	0,90 0,35

Continuous beams (end span)

	L [m]	s^*	h^*
[1]	≤ 2 ≥ 20	0,85 0,70	0,85 0,75
[2]	≤ 2 ≥ 20	0,90 0,70	0,95 0,75
[3]	≤ 2 ≥ 20	1,10 0,75	1,10 0,80
[4]	≤ 2 ≥ 20	0,70 0,35	0,70 0,40

Continuous beams
(intermediate support area)

s^* standard traffic mix

h^* heavy traffic mix

[1] reinforcing steel, pre-tensioning (all), post-tensioning (strands in plastic ducts and straight tendons in steel ducts)

[2] post-tensioning (curved tendons in steel ducts); S-N curve with $k_1 = 3$, $k_2 = 7$ and $N^* = 10^6$

[3] splice devices (prestressing steel); S-N curve with $k_1 = 3$, $k_2 = 5$ and $N^* = 10^6$

[4] splice devices (reinforcing steel); welded bars including tack welding and butt joints; S-N curve with $k_1 = 3$, $k_2 = 5$ and $N^* = 10^7$

Interpolation between the given L-values according to Expression NN.108 is allowed

NOTE No values of $\lambda_{s,1}$ are given in Table NN.2 for a light traffic mix. 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 NN.2 for standard traffic mix or on values determined from detailed calculations.

NN.3.2 Concrete subjected to compression

(101) For concrete subjected to compression adequate fatigue resistance may be assumed if the following expression is satisfied:

$$14 \cdot \frac{1 - E_{\mathrm{cd,max,equ}}}{\sqrt{1 - R_{\mathrm{equ}}}} \geq 6 \qquad \text{(NN.112)}$$

where:

$$R_{\text{equ}} = \frac{E_{\text{cd,min,equ}}}{E_{\text{cd,max,equ}}} \qquad E_{\text{cd,min,equ}} = \gamma_{\text{sd}} \frac{\sigma_{\text{cd,min,equ}}}{f_{\text{cd,fat}}} \qquad E_{\text{cd,max,equ}} = \gamma_{\text{sd}} \frac{\sigma_{\text{cd,max,equ}}}{f_{\text{cd,fat}}}$$

$\sigma_{\text{cd,max,equ}}$ and $\sigma_{\text{cd,min,equ}}$ are the upper and lower stresses of the damage equivalent stress spectrum with a number of cycles $N=10^6$

(102) The upper and lower stresses of the damage equivalent stress spectrum shall be calculated according to Equation (NN.113)

$$\sigma_{\text{cd,max,equ}} = \sigma_{\text{c,perm}} + \lambda_{\text{c}} \left(\sigma_{\text{c,max,71}} - \sigma_{\text{c,perm}}\right)$$

$$\sigma_{\text{cd,min,equ}} = \sigma_{\text{c,perm}} - \lambda_{\text{c}} \left(\sigma_{\text{c,perm}} - \sigma_{\text{c,min,71}}\right) \qquad \text{(NN.113)}$$

where:

$\sigma_{\text{c,perm}}$ is the compressive concrete stress caused by the characteristic combination of actions, without load model 71.

$\sigma_{\text{c,max,71}}$ is the maximum compressive stress caused by the characteristic combination including load model 71 and the dynamic factor Φ according to EN 1991-2.

$\sigma_{\text{c,min,71}}$ is the minimum compressive stress under the characteristic combination 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.

NOTE $\sigma_{\text{c,perm}}$, $\sigma_{\text{c,max,71}}$ and $\sigma_{\text{c,min,71}}$ do not include other variable actions (eg. wind, temperature etc.).

(103) The correction factor λ_{c} takes account of the permanent stress, the span, annual traffic volume, design life and multiple tracks. It is calculated from the following formula:

$$\lambda_{\text{c}} = \lambda_{\text{c,0}} \cdot \lambda_{\text{c,1}} \cdot \lambda_{\text{c,2,3}} \cdot \lambda_{\text{c,4}} \qquad \text{(NN.114)}$$

where:

$\lambda_{\text{c,0}}$ is a factor to take account of the permanent stress.

$\lambda_{\text{c,1}}$ is a factor accounting for element type (eg. continuous beam) that takes into account the damaging effect of traffic depending on the critical length of the influence line or area.

$\lambda_{\text{c,2,3}}$ is a factor to take account of the traffic volume and the design life of the bridge.

$\lambda_{\text{c,4}}$ is a factor to be applied when the structural element is loaded by more than one track.

(104) The $\lambda_{\text{c,0}}$ value denotes the influence of the permanent stress and can be calculated from Equation (NN.115)

$$\lambda_{\text{c,0}} = 0{,}94 + 0{,}2 \frac{\sigma_{\text{c,perm}}}{f_{\text{cd,fat}}} \geq 1 \qquad \text{for the compression zone} \qquad \text{(NN.115)}$$

$$\lambda_{\text{c,0}} = 1 \qquad \text{for the precompressed tensile zone (including prestressing effect)}$$

(105) The factor $\lambda_{c,1}$ is a function of the critical length of the influence line and the traffic. The values of $\lambda_{c,1}$ for standard traffic mix and heavy traffic mix may be taken from Table NN.2 of this Annex.

Values of $\lambda_{c,1}$ for critical lengths of influence lines between 2 m and 20 m may be obtained by applying Expression (NN.108) with $\lambda_{s,1}$ replaced by $\lambda_{c,1}$.

(106) The $\lambda_{c,2,3}$ value denotes the influence of annual traffic volume and service life and can be calculated from Equation (NN.116)

$$\lambda_{c,2,3} = 1 + \frac{1}{8}\log\left[\frac{\mathrm{Vol}}{25 \cdot 10^6}\right] + \frac{1}{8}\log\left[\frac{N_{\mathrm{Years}}}{100}\right] \qquad \text{(NN.116)}$$

where:

Vol is the volume of traffic (tonnes/years/track)

N_{Years} is the design life of the bridge

(107) The $\lambda_{c,4}$ value denotes the effect of loading from more than one track. For structures carrying multiple tracks, the fatigue loading shall 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 Equation (NN.117)

$$\lambda_{c,4} = 1 + \frac{1}{8}\log n \geq 0{,}54 \qquad \text{for } a \leq 0{,}8 \qquad \text{(NN.117)}$$

$\lambda_{c,4} = 1$ for $a > 0{,}8$

$$a = \frac{\max(\sigma_{c1}, \sigma_{c2})}{\sigma_{c1+2}} \qquad \text{(NN.118)}$$

where:

n is the proportion of traffic crossing the bridge simultaneously (the recommended value of n is 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

Table NN.3 — $\lambda_{c,1}$ values for simply supported and continuous beams

Simply supported beams

	L [m]	s^*	h^*
[1]	≤ 2	0,70	0,70
	≥ 20	0,75	0,75
[2]	≤ 2	0,95	1,00
	≥ 20	0,90	0,90

Continuous beams (mid span)

	L [m]	s^*	h^*
[1]	≤ 2	0,75	0,90
	≥ 20	0,55	0,55
[2]	≤ 2	1,05	1,15
	≥ 20	0,65	0,70

Continuous beams (end span)

	L [m]	s^*	h^*
[1]	≤ 2	0,75	0,80
	≥ 20	0,70	0,70
[2]	≤ 2	1,10	1,20
	≥ 20	0,70	0,70

Continuous beams
(intermediate support area)

	L [m]	s^*	h^*
[1]	≤ 2	0,70	0,75
	≥ 20	0,85	0,85
[2]	≤ 2	1,10	1,15
	≥ 20	0,80	0,85

s^* standard traffic mix

h^* heavy traffic mix

[1] compression zone

[2] precompressed tensile zone

Interpolation between the given L-values according to Expression NN.108 is allowed, with $\lambda_{s,1}$ replaced by $\lambda_{c,1}$.

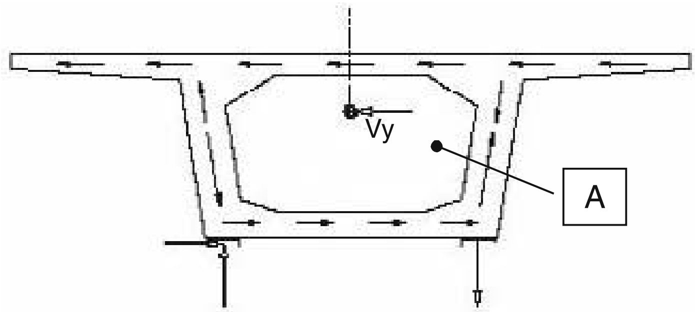
NOTE No values of $\lambda_{c,1}$ are given in Table NN.3 for a light traffic mix. For bridges designed to carry a light traffic mix the values for $\lambda_{c,1}$ to be used may be based either on the values given in Table NN.3 for standard traffic mix or on values derived from detailed calculations.

ANNEX OO
(informative)

Typical bridge discontinuity regions

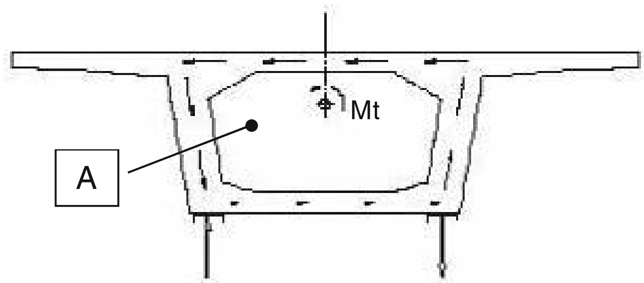
OO.1 Diaphragms with direct support of box section deck webs on bearings

(101) Diaphragms where the bearings are located directly below the webs of the box section will be subject to forces generated by the transmission of shear in the horizontal plane (Figure OO.1), or forces due to the transformation of the torsional moment in the deck into a pair of forces in cases where two bearings are present (Figure OO.2).



A Diaphragm

Figure OO.1 — Horizontal shear and reactions in bearings



A Diaphragm

Figure OO.2 — Torsion in the deck and reactions in bearings

(102) In general, from Figures OO.1 and OO.2 it can be seen that the flow of the forces from the lower flange and from the webs is channelled directly to the supports without any forces being induced in the central part of the diaphragm. The forces from the upper flange result in forces being applied to the diaphragm and these determine the design of the element. Figures OO.3 and OO.4 identify possible resistance mechanisms that can be used to determine the necessary reinforcement for elements of this type.

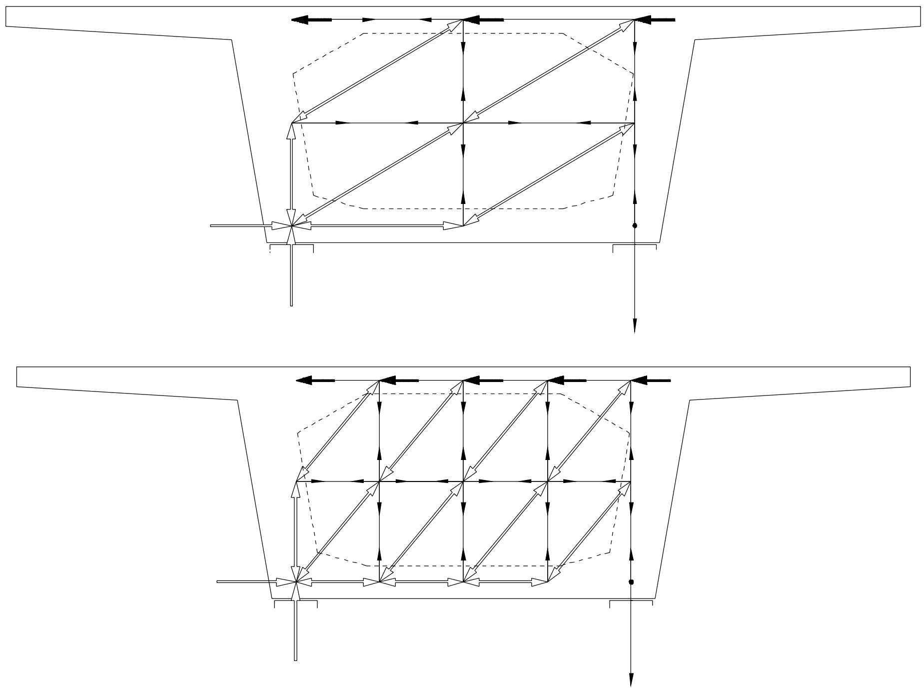


Figure OO.3 — Strut and tie model for a solid type diaphragm without manhole

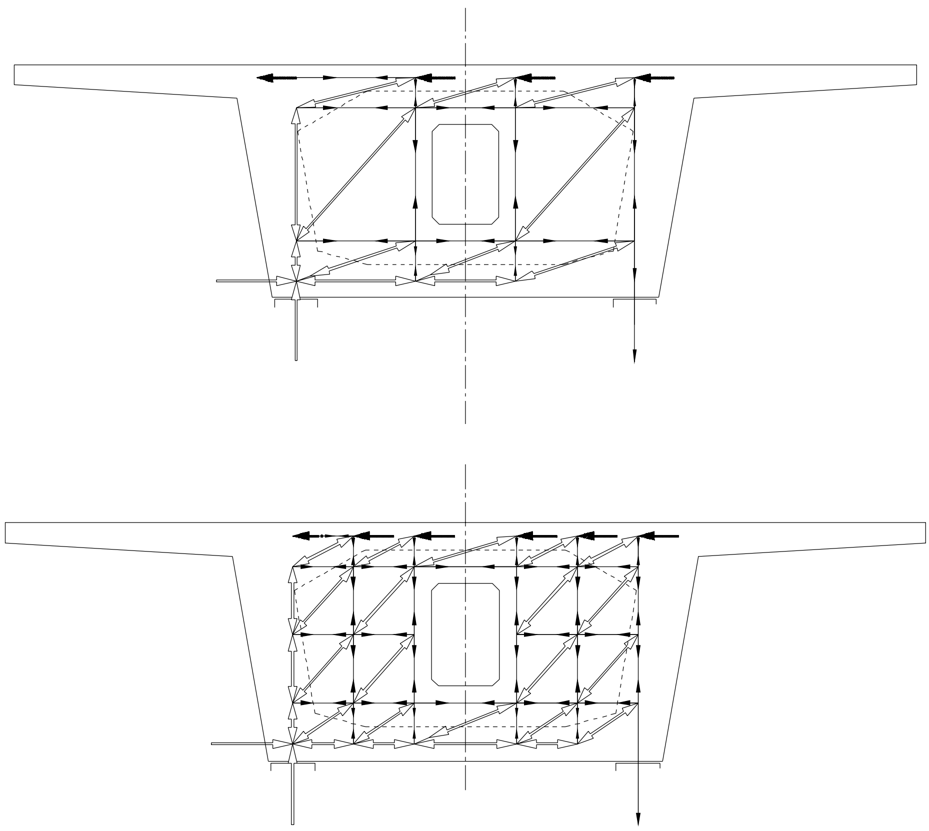


Figure OO.4 — Strut and tie model for a solid type diaphragm with manhole

(103) Generally, it is not necessary to check nodes or struts when the thickness of the diaphragm is equal to or greater than the dimension of the support area in the longitudinal direction of the bridge. In these circumstances, it is then only necessary to check the support nodes.

OO.2 Diaphragms for indirect support of deck webs on bearings

(101) In this case, in addition to the shear along the horizontal axis and, in the case of more than one support, the effect of the torsion, the diaphragm must transmit the vertical shear forces, transferred from the webs, to the bearing or bearings.

The nodes at the bearings must be checked using the criteria given in 6.5 and 6.7 of EN 1992-1-1.

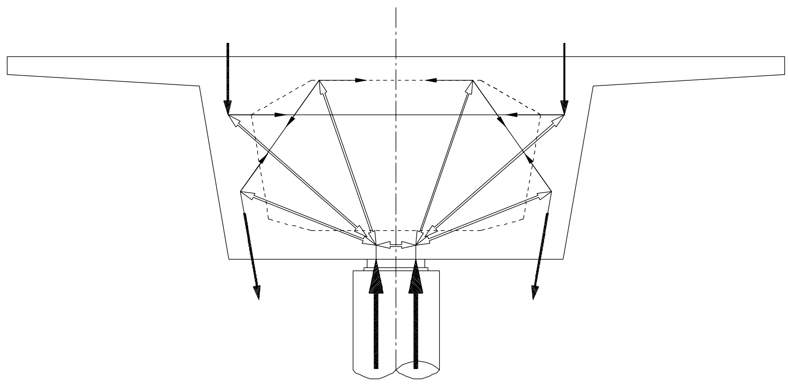
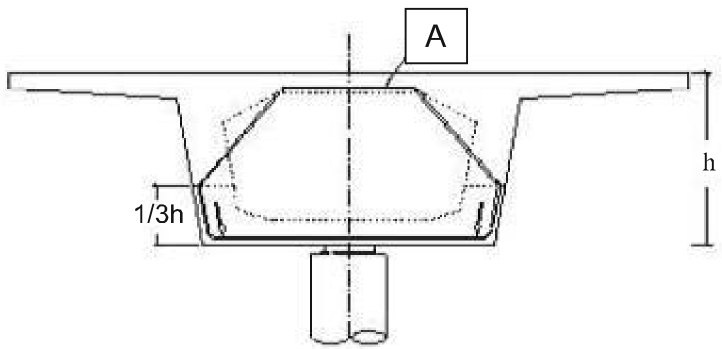


Figure OO.5 — Diaphragms with indirect support. Strut and tie model

(102) Reinforcement should be designed for the tie forces obtained from the resistance mechanisms adopted, taking account of limitations on tension in the reinforcement indicated in 6.5 of EN 1992-1-1. In general, due to the way in which vertical shear is transmitted, it will be necessary to provide suspension reinforcement. If inclined bars are used for this, special attention should be paid to the anchorage conditions (Figure OO.6).



A Reinforcement

Figure OO.6 — Diaphragms with indirect support. Anchorage of the suspension reinforcement

(103) If the suspension reinforcement is provided in the form of closed stirrups, these must enclose the reinforcement in the upper face of the box girder (Figure OO.7).

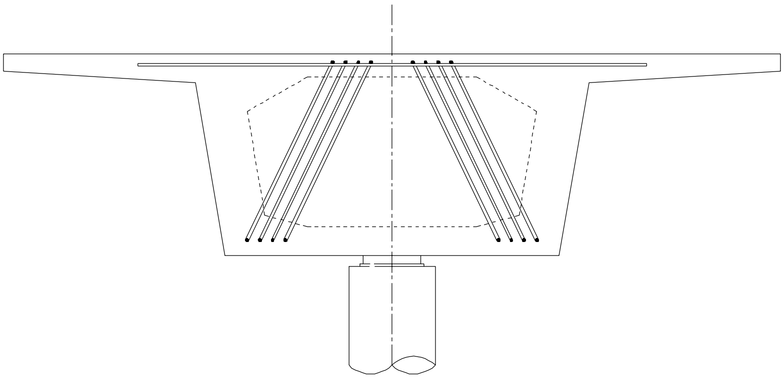


Figure OO.7 — Diaphragms with indirect support. Links as suspension reinforcement

(104) In cases where prestressing is used, such as post-tensioned tendons, the design will clearly define the order in which these have to be tensioned (diaphragm prestressing should generally be carried out before longitudinal prestressing). Special attention should be paid to the losses in the prestressing, given the short length of the tendons.

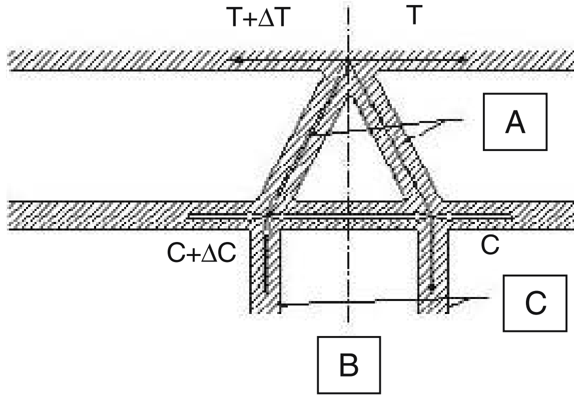
(105) AC1 In addition to the reinforcement obtained on the basis of the above resistant mechanism, splitting reinforcement should be provided, if necessary, with regard to concentrated support forces. AC1

OO.3 Diaphragms in monolithic deck-pier joints

(101) In cases where the deck and pier are monolithic, the difference in deck moments in adjacent spans on either side of the pier must be transmitted to the pier. This moment transmission will generate additional forces to those identified in the previous clauses.

(102) In the case of triangular diaphragms (Figure OO.8), transmission of the vertical load and the force caused by the difference in moments is direct, as long as the continuity of the compression struts and overlapping (or anchorage) of the tension reinforcement is provided.

(103) In the case of a double vertical diaphragm, the flow of forces from the deck to the piers is more complex. In this case, it is necessary to carefully check the continuity of the compression flow.



A Diaphragm

B Longitudinal section

C Pier

Figure OO.8 — Diaphragm in monolith joint with double diaphragm: Equivalent system of struts and ties.

OO.4 Diaphragms in decks with double T sections and bearings under the webs

(101) In this case, the diaphragms will be subject to forces generated by the transmission of shear in the horizontal axis (Figure OO.9), or forces due to the transformation of the torsional moment in the deck into a pair of forces in the case where two supports are present(Figure OO.10).

(102) In general, from Figures OO.9 and OO.10, it can be seen that the flow of forces from the webs is channelled directly at the supports without any forces being induced in the central part of the diaphragm. The forces from the upper flange result in forces being applied to the diaphragm and these have to be considered in the design.

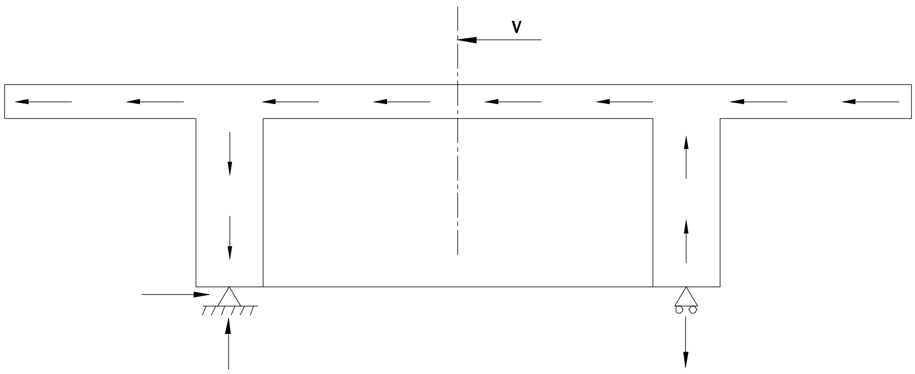


Figure OO.9 — Horizontal shear and reactions in supports

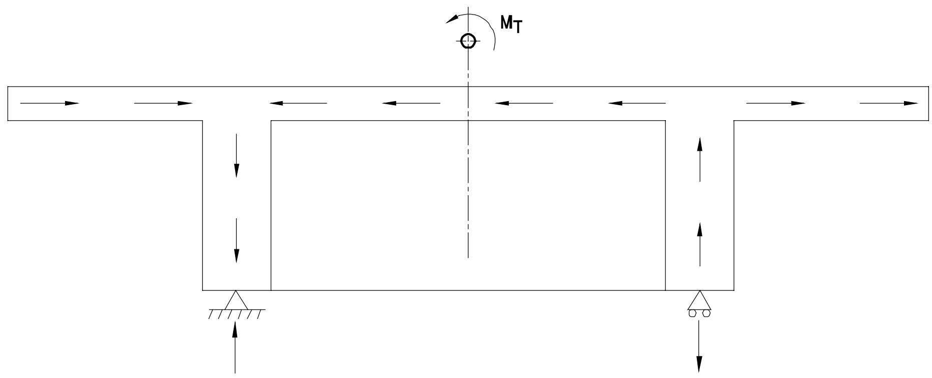


Figure OO.10 — Torsion in the deck slab and reactions in the supports

Figure OO.11 shows a possible resistance mechanism that enables the required reinforcement to be determined.

In general, if the thickness of the diaphragm is equal to or greater than the dimension of the bearing area in the longitudinal direction of the bridge, it will only be necessary to check the support nodes in accordance with 6.5 of EN 1992-1-1.

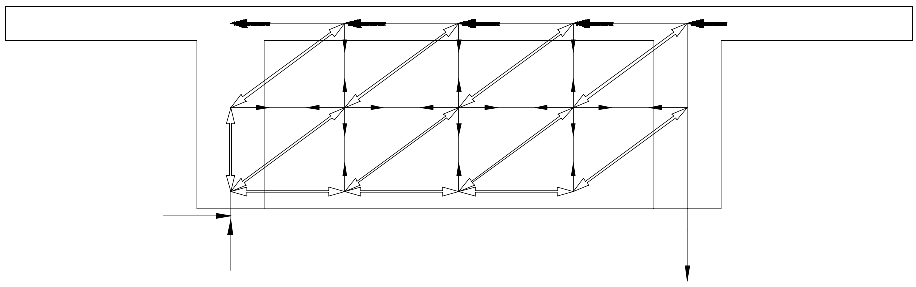


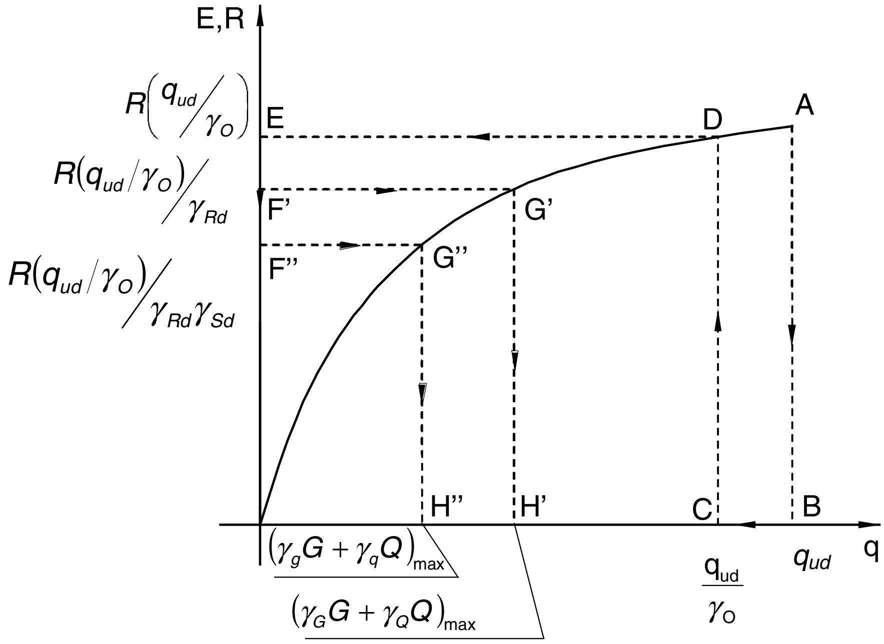
Figure OO.11 — Model of struts and ties for a typical diaphragm of a slab

Annex PP
(informative)

Safety format for non linear analysis

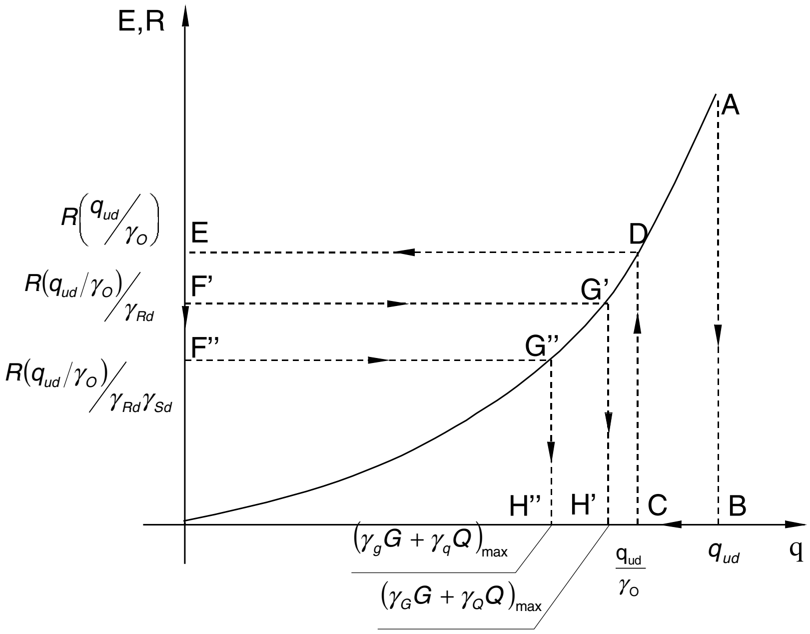
PP.1 Practical application

(101) For the case of scalar combination of internal actions, AC1 reverse application of inequalities (5.102 aN) and (5.102 bN) is shown diagrammatically in Figures AC1 PP.1 and PP.2, for underproportional and overproportional structural behaviour respectively.



A Final point of N.L. Analysis

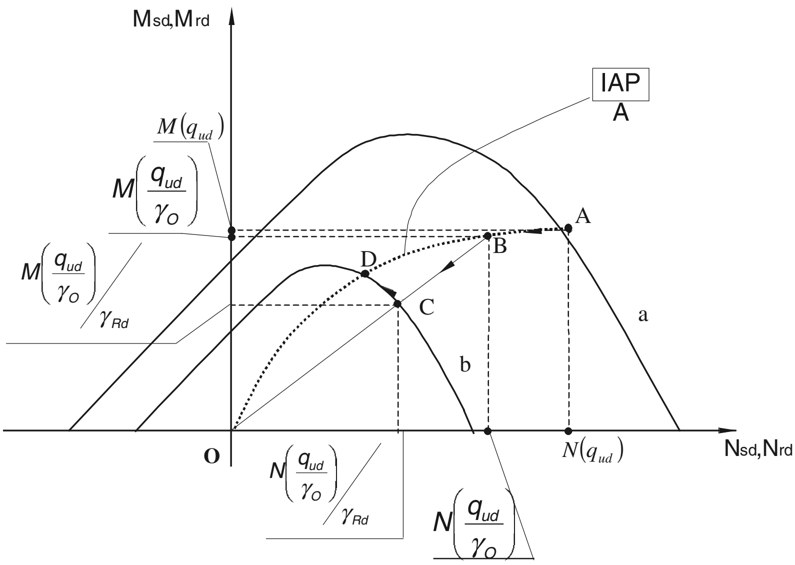
Figure PP.1 — Safety format application for scalar underproportional behaviour



A Final point of N.L. Analysis

Figure PP.2 — Safety format application for scalar overproportional behaviour

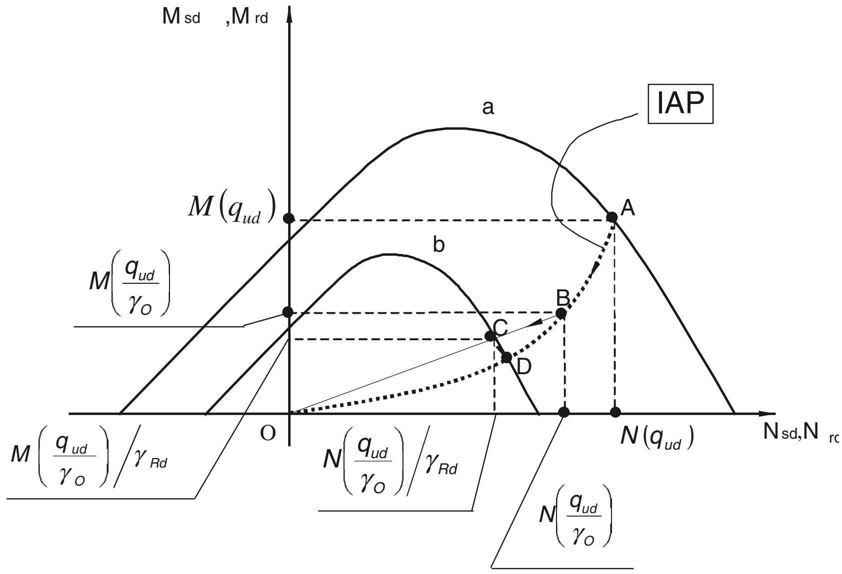
(102) For the case of vectorial combination of internal actions, AC1 the application of inequalities (5.102 aN) and (5.102 bN) is illustrated in Figures AC1 PP.3 and PP.4, for underproportional and overproportional structural behaviour respectively. Curve a represents the failure line, while curve b is obtained by scaling this line by applying safety factors γ_{Rd} and γ_o.



A Final point of N.L. Analysis

IAP Internal actions path

Figure PP.3 — Safety format application for vectorial (M,N) underproportional behaviour



[A] Final point of N.L. Analysis

[IAP] Internal actions path

Figure PP.4 — Safety format application for vectorial (M,N) overproportional behaviour

In both figures, D represents the intersection between the internal actions path and the safety domain “b”.

It should be verified that the point with coordinates

$M(\gamma_G G + \gamma_Q Q)$ and $N(\gamma_G G + \gamma_Q Q)$

i.e. the point corresponding to the internal actions (the effects of factored actions), should remain within the safety domain “b”.

An equivalent procedure applies where the partial factor for model uncertainty γ_{Sd} is introduced, but with γ_{Rd} substituted by $\gamma_{Rd}\gamma_{Sd}$ and γ_G, γ_Q substituted by γ_g, γ_q.

The same procedures applies for the combination of $N/M_x/M_y$ or $n_x/n_y/n_{xy}$.

NOTE If the procedure with $\gamma_{Rd} = \gamma_{Sd} = 1$ and $\gamma_{O'} = 1{,}27$ is applied, the safety check is satisfied if $M_{Ed} \leq M_{Rd}\ (q_{ud}/\gamma_{O'})$ and $N_{Ed} \leq N_{Rd}\ (q_{ud}/\gamma_{O'})$.

Annex QQ
(informative)

Control of shear cracks within webs

At present, the prediction of shear cracking in webs is accompanied by large model uncertainty.

Where it is considered necessary to check shear cracking, particularly for prestressed members, the reinforcement required for crack control can be determined as follows:

1. The directionally dependent concrete tensile strength f_{ctb} within the webs should be calculated from:

$$f_{ctb} = \left(1 - 0{,}8\frac{\sigma_3}{f_{ck}}\right) f_{ctk;0,05} \qquad \text{(QQ.101)}$$

where:

f_{ctb} is the concrete tensile strength prior to cracking in a biaxial state of stress

σ_3 is the larger compressive principal stress, taken as positive.

$\sigma_3 < 0{,}6\, f_{ck}$

2. The larger tensile principal stress σ_1 in the web is compared with the corresponding strength f_{ctb} obtained from expression (QQ 101).

If $\sigma_1 < f_{ctb}$, the minimum reinforcement in accordance with 7.3.2 should be provided in the longitudinal direction.

If $\sigma_1 \geq f_{ctb}$, the crack width should be controlled in accordance with 7.3.3 or alternatively calculated and verified in accordance with 7.3.4 and 7.3.1, taking into account the angle of deviation between the principal stress and reinforcement directions.

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