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NORME EUROPÉENNE
EUROPÄISCHE NORM

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English version

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Design of composite steel and concrete structures

Part 2
Rules for bridges

CEN

European Committee for Standardization
Comité Européen de Normalisation
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Clean version, only bridge clauses

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Foreword

This European Standard EN 1994-1-1, Eurocode 4: Design of composite steel and concrete structures: General rules and rules for buildings, has been prepared on behalf of Technical Committee CEN/TC250 « Structural Eurocodes », the Secretariat of which is held by BSI. CEN/TC250 is responsible for all Structural Eurocodes.

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The text of the draft standard was submitted to the formal vote and was approved by CEN as EN 1994-1-1 on **YYYY-MM-DD**.

No existing European Standard is superseded.

Background of the Eurocode programme

In 1975, the Commission of the European Community decided on an action programme in the field of construction, based on article 95 of the Treaty. The objective of the programme was the elimination of technical obstacles to trade and the harmonisation of technical specifications.

Within this action programme, the Commission took the initiative to establish a set of harmonised technical rules for the design of construction works which, in a first stage, would serve as an alternative to the national rules in force in the Member States and, ultimately, would replace them.

For fifteen years, the Commission, with the help of a Steering Committee with Representatives of Member States, conducted the development of the Eurocodes programme, which led to the first generation of European codes in the 1980s.

In 1989, the Commission and the Member States of the EU and EFTA decided, on the basis of an agreement[1] between the Commission and CEN, to transfer the preparation and the publication of the Eurocodes to CEN through a series of Mandates, in order to provide them with a future status of European Standard (EN). This links *de facto* the Eurocodes with the provisions of all the Council's Directives and/or Commission's Decisions dealing with European standards (*e.g.* the Council Directive 89/106/EEC on construction products - CPD - and Council Directives 93/37/EEC, 92/50/EEC and 89/440/EEC on public works and services and equivalent EFTA Directives initiated in pursuit of setting up the internal market).

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

[1] Agreement between the Commission of the European Communities and the European Committee for Standardisation (CEN) concerning the work on EUROCODES for the design of building and civil engineering works (BC/CEN/03/89).

EN 1990	Eurocode :	Basis of Structural Design
EN 1991	Eurocode 1:	Actions on structures
EN 1992	Eurocode 2:	Design of concrete structures
EN 1993	Eurocode 3:	Design of steel structures
EN 1994	Eurocode 4:	Design of composite steel and concrete structures
EN 1995	Eurocode 5:	Design of timber structures
EN 1996	Eurocode 6:	Design of masonry structures
EN 1997	Eurocode 7:	Geotechnical design
EN 1998	Eurocode 8:	Design of structures for earthquake resistance
EN 1999	Eurocode 9:	Design of aluminium structures

Eurocode standards recognise the responsibility of regulatory authorities in each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level where these continue to vary from State to State.

Status and field of application of Eurocodes

The Member States of the EU and EFTA recognise that Eurocodes serve as reference documents for the following purposes:

– as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 – Mechanical resistance and stability – and Essential Requirement N°2 – Safety in case of fire ;

– as a basis for specifying contracts for construction works and related engineering services ;

– as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs)

The Eurocodes, as far as they concern the construction works themselves, have a direct relationship with the Interpretative Documents[2] referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standards[3]. Therefore, technical aspects arising from the Eurocodes work need to be adequately considered by CEN Technical Committees and/or EOTA Working Groups working on product standards with a view to achieving full compatibility of these technical specifications with the Eurocodes.

The Eurocode standards provide common structural design rules for everyday use for the design of whole structures and component products of both a traditional and an innovative nature. Unusual forms of construction or design conditions are not specifically covered and additional expert consideration will be required by the designer in such cases.

[2] According to Art. 3.3 of the CPD, the essential requirements (ERs) shall be given concrete form in interpretative documents for the creation of the necessary links between the essential requirements and the mandates for harmonised ENs and ETAGs/ETAs.

[3] According to Art. 12 of the CPD the interpretative documents shall :

a) give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;

b) indicate methods of correlating these classes or levels of requirement with the technical specifications, *e.g.* methods of calculation and of proof, technical rules for project design, etc. ;

c) serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.

The Eurocodes, *de facto*, play a similar role in the field of the ER 1 and a part of ER 2.

National Standards implementing Eurocodes

The National Standards implementing Eurocodes will comprise the full text of the Eurocode (including any annexes), as published by CEN, which may be preceded by a National title page and National foreword, and may be followed by a National annex.

The National annex may only contain information on those parameters which are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned, *i.e.*:

- values and/or classes where alternatives are given in the Eurocode,
- values to be used where a symbol only is given in the Eurocode,
- country specific data (geographical, climatic, etc.), e.g. snow map,
- the procedure to be used where alternative procedures are given in the Eurocode.

It may also contain:

- decisions on the use of informative annexes, and
- references to non-contradictory complementary information to assist the user to apply the Eurocode.

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

There is a need for consistency between the harmonised technical specifications for construction products and the technical rules for works[4.] Furthermore, all the information accompanying the CE Marking of the construction products which refer to Eurocodes shall clearly mention which Nationally Determined Parameters have been taken into account.

Additional information specific to EN 1994-2

EN 1994-2 gives Principles and application rules, additional to the general rules given in EN 1994-1-1, for the design of composite steel and concrete bridges or composite members of bridges.

EN 1994-2 is intended for use by clients, designers, contractors and public authorities.
EN 1994-2 is intended to be used with EN 1990, the relevant parts of EN 1991, EN 1993 for the design of steel structures and EN 1992 for the design of concrete structures.

National annex for EN 1994-2

This standard gives alternative procedures, values and recommendations for classes with notes indicating where national choices may have to be made. Therefore, the National Standard implementing EN 1994-2 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.

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

National choice is allowed in EN 1994-2 through clauses:

1.1.3 (3)
5.4.2.5 (3)
6.2.3 (1)
6.3.1 (1)
6.6.1.1 (13)
6.6.3.1 (4)
6.8.2 (2)
6.9 (3)
7.2.2 (2)
7.2.2 (4)
7.4.1 (6)
8.4.3 (4)

Section 1 General

1.1 Scope

1.1.3 Scope of Part 2 of Eurocode 4

(1) Part 2 of Eurocode 4 gives design rules for steel-concrete composite bridges or members of bridges, additional to the general rules in EN 1994-1-1. Cable stayed bridges are not fully covered by this part.

(2) The following subjects are dealt with in Part 2:

Section 1: General

Section 2: Basis of design

Section 3: Materials

Section 4: Durability

Section 5: Structural analysis

Section 6: Ultimate limit states

Section 7: Serviceability limit states

Section 8: Decks with precast concrete slabs

Section 9: Composite plates in bridges

(3) Provisions for shear connectors are given only for welded headed studs.

Note: Reference to guidance for other types as shear connectors may be given in the National Annex.

1.2 Normative references

1.2.3 Additional general and other reference standards for composite bridges

EN 1990:Annex 2	Basis of structural design : Application for bridges
EN 1991-2:200x	Actions on structures : Traffic loads on bridges
EN 1992-2:200x	Design of concrete structures. Part 2 – Bridges
EN 1993-2:200x	Design of steel structures. Part 2 – Bridges
EN 1994-1-1:200x	Design of steel and concrete composite structures. General rules and rules for buildings

[Drafting note: This list will require updating at the time of publication]

1.3 Assumptions

(2) In addition to the general assumptions of EN 1990, the following assumptions apply for bridges :

– those given in clauses 1.3 of EN1992-2 and EN1993-2.

1.5 Definitions

1.5.2 Additional terms and definitions used in this Standard

1.5.2.13
filler beam deck
a deck consisting of a reinforced concrete slab and concrete-encased steel beams, having their bottom flange on the level of the slab bottom.

1.5.2.14
composite plate
composite member subjected mainly to bending, consisting of a flat plate connected to a concrete slab, in which both the length and width are much larger than the thickness.

1.7 Additional symbols used in Part 2

Latin upper case letters

A_p	Area of prestressing steel
$(EA)_{eff}$	Effective longitudinal stiffness of cracked concrete
F_d	Component in the direction of the steel beam of the design force of a bonded or unbonded tendon applied after the shear connection has become effective
I_{eff}	Effective second moment of area of filler beams
L_{A-B}	Length of inelastic region, between points A and B, corresponding to $M_{el,Rd}$ and $M_{Ed,max}$, respectively
L_v	Length of shear connection
$M_{f,Rd}$	Design resistance moment to 5.2.6.1 of EN1993-1-5
N_{cd}	Design compressive force in concrete slab corresponding to $M_{Ed,max}$
$N_{Ed,serv}$	Normal force of concrete tension member for SLS
$N_{Ed,ult}$	Normal force of concrete tension member for ULS
$N_{s,el}$	Tensile force in cracked concrete slab corresponding to $M_{el,Rd}$ taking into account the effects of tension stiffening
P_{Ed}	Longitudinal force on a connector at distance x from the nearest web
V_L	Longitudinal shear force, acting along the steel-concrete flange interface
$V_{L,Ed}$	Longitudinal shear force acting on length L_{A-B} of the inelastic region

Latin lower case letters

a_w	Steel flange projection outside the web of the beam
b	Half the distance between adjacent webs, or the distance between the web and the free edge of the flange
b_{ei}	Effective width of composite bottom flange of a box section
c_{st}	Concrete cover above the steel beams of filler beam decks
e_d	Either of $2e_h$ or $2e_v$
e_h	Lateral distance from the point of application of force F_d to the relevant steel web, if F_d is applied to the concrete slab
e_v	Vertical distance from the point of application of force F_d to the plane of shear connection concerned, if F_d is applied to the steel element
f_{pd}	Limiting stress of prestressing tendons according to 3.3.3 of EN1992-1:200x
f_{pk}	characteristic value of yield strength of prestressing tendons
n_{tot}	See 9.4
n_{0G}	Modular ratio (shear moduli) for short term loading
n_{LG}	Modular ratio (shear moduli) for long term loading

n_w See 9.4
s_f Clear distance between the upper flanges of the steel beams of filler beam decks
s_w Spacing of webs of steel beams of filler beam decks
t_f Thickness of the steel flange of the steel beams of filler beam decks
$v_{max,Ed}$ Maximum shear force per unit length of shear connection
v_{Ed} Design longitudinal shear per unit length at an interface between steel and concrete in a composite member
x Distance of a shear connector from the nearest web

Greek lower case letters

α Factor see 6.4.2 (6)
β Half of the angle of spread of longitudinal shear force V_ℓ into the concrete slab
$\lambda_{v,1}$ Factor to be used for the determination of the damage equivalent factor λ_v for headed studs in shear

Section 2 Basis of design

2.4 Verification by the partial factor method

2.4.2 Combination of actions

(2) For bridges the combinations of actions are given in Annex A2 of EN 1990.

2.4.3 Verification of static equilibrium (EQU)

(2) For bridges, the reliability format for the verification of static equilibrium, as described in EN 1990, Table A2.4(A), should also apply to design situations equivalent to (EQU), e.g. for the design of hold down anchors or the verification of uplift of bearings of continuous bridges.

Section 3 Materials

3.1 Concrete

(1) Unless otherwise given by Eurocode 4, properties should be obtained by reference to EN 1992-2, 3.1 for normal concrete and to EN 1992-2, 11.3 for lightweight concrete.

(4) Where composite action is taken into account in bridges, the effects of autogenous shrinkage may be neglected in the determination of stresses and deflections and at ultimate limit states but should be considered as stated in 7.4.1(7).

3.2 Reinforcing steel

(1) Properties should be obtained by reference to EN 1992-2, 3.2.

3.3 Structural steel

(1) Properties should be obtained by reference to EN 1993-2, 3.1 and 3.2.

(3) For simplification in design calculations for composite structures, the value of the coefficient of linear thermal expansion for structural steel may be taken as 10×10^{-6} per ^{o}C. The coefficient of thermal expansion should be taken as $12x10^{-6}$ for calculation of change in length of the bridge.

3.5 Prestressing steel and devices

(1) Reference should be made to clauses 3.3 and 3.4 of EN1992-2.

3.6 Cables

(1) Reference should be made to EN 1993-1-11.

Section 4 Durability

4.2 Corrosion protection at the steel-concrete interface in bridges

(1) The corrosion protection should extend into the steel-concrete interface at least 50 mm. For additional rules for bridges with pre-cast deck slabs, see Section 8.

Section 5 Structural analysis

5.1 Structural modelling for analysis

5.1.2 Joint modelling

(3) In bridge structures semi-continuous composite joints should not be used. For other types of steel joints EN 1993-2 applies.

5.1.3 Ground-structure interaction

(2) Where settlements have to be taken into account and where no design values have been specified, appropriate estimated values of predicted settlement should be used.

(3) Effects due to settlements may normally be neglected in ultimate limit states other than fatigue for composite members where all cross sections are in class 1 or 2 and bending resistance is not reduced by lateral torsional buckling.

5.2 Structural stability

5.2.2 Methods of analysis for bridges

(1) For bridge structures EN 1993-2, 5.2 applies.

5.3 Imperfections

5.3.2 Imperfections for bridges

(1) Suitable equivalent geometric imperfections should be used with values that reflect the possible effects of system imperfections and member imperfections (e.g in bowstring arches, trusses, transverse frames) unless these effects are included in the resistance formulae.

(2) The imperfections and design transverse forces for stabilising transverse frames should be calculated in accordance with EN 1993-2, 5.3 and 6.3.4.2.

(3) For composite columns and composite compression members, member imperfections should always be considered when verifying stability within a member's length in accordance with 6.7.3.6 or 6.7.3.7. Design values of equivalent initial bow imperfection should be taken from Table 6.5.

(4) Imperfections within steel compression members should be considered in accordance with EN 1993-2, 5.3.

5.4 Calculation of action effects

5.4.1 Methods of global analysis

5.4.1.1 General

(9) For erection stages uncracked global analysis and the distribution of effective width according to 5.4.1.2(4) may be used.

5.4.1.2 Effective width of flanges for shear lag

(8) The transverse distribution of stresses due to shear lag may be taken in accordance with EN 1993-1-5, 4.3 for both concrete and steel flanges.

(9) For cross-sections with bending moments resulting from the main-girder system and from a local system (for example in composite trusses with direct actions on the chord between nodes) the relevant effective widths for the main girder system and the local system should be used for the relevant bending moments.

5.4.2 Linear elastic analysis

5.4.2.1 General

(2) For serviceability limit states, to ensure the performance required, the bridge or parts of the bridge should be classified into design categories for serviceability limit states according to EN 1992-2, 7.1.2 for both the construction phases and for persistent situations. For Categories A, B and C for serviceability limit states and for the ultimate limite state of fatigue uncracked linear elastic global analysis without redistribution should be used.

(3) For the ultimate limit states, other than fatigue, of bridge structures in Categories A, B and C according to EN 1992-2, 7.1.2 effects of cracking may be taken into account according to 5.4.2.3 or 5.4.4.

(4) For Categories D and E for ultimate and serviceability limit states the effects of cracking may be taken into account according to 5.4.2.3 or 5.4.4.

5.4.2.2 Creep and shrinkage

(11) The torsional stiffness of box girders should be calculated for a transformed cross section in which the slab thickness is reduced by the modular ratio $n_{0G}=G_a/G_c$ where G_a and G_c are the elastic shear moduli of structural steel and concrete respectively. The effects of creep may be taken into account in accordance with (2) with the modular ratio $n_{L.G}= n_{0,G}\,(1+\psi_L\varphi_t)$.

5.4.2.3 Effects of cracking of concrete

(5) Unless a more precise method is used, in multiple beam decks where transverse composite members are not subjected to tensile forces, it may be assumed that the transverse members are uncracked throughout.

(6) The torsional stiffness of box girders should be calculated for a transformed cross section. In areas where the concrete slab is assumed to be cracked due to bending and where membrane shear stresses are so large that shear reinforcement is required, the calculation should be performed considering a slab thickness reduced to one half, unless the effect of cracking is considered in a more precise way.

(7) For ultimate limit states the effects of cracking on the longitudinal shear forces at the interface between the steel and concrete section should be taken into account according to 6.6.2.

(8) For serviceability limit states the longitudinal shear forces at the interface between the steel und concrete section should normally calculated by uncracked analysis. The effects of cracking may be taken into account under a proper consideration of tension stiffening and overstrength of concrete in tension.

5.4.2.5 Temperature effects

(3) If during concreting and hardening of concrete the temperature in the steel top flange due to extreme climatic conditions is very low additional differential temperature should be considered.

Note: Further provisions may be given in an National Annex

5.4.2.7 Prestressing by tendons

(1) Internal forces and moments due to prestressing by bonded tendons should be determined in accordance with EN 1992-2, 5.10.2 taking into account effects of creep and shrinkage of concrete and cracking of concrete where relevant.

(2) In global analysis, forces in unbonded tendons should be treated as external forces. For the determination of forces in permanently unbonded tendons, deformations of the whole structure should be taken into account.

5.4.2.8 Tension members in composite bridges

(1) In paragraphs (1) to (5) of this clause, “tension member” means a reinforced concrete tension member acting together with a tension member of structural steel or the reinforced concrete part of a composite tension member. This clause is applicable to structures in which shear connection causes global tensile forces in reinforced concrete or composite members. Typical examples are bowstring arches and trusses where the concrete or composite members act as a tension member in the main system.

(2)P For the determination of the forces of a tension member, the non linear behaviour due to cracking of concrete and the effects of tension stiffening of concrete shall be considered for the global analyses for ultimate and serviceability limit states and for the limit state of fatigue. Account shall be taken effects resulting from overstrength of concrete in tension.

(3) For the calculation of the internal forces of a cracked tension member the effects of shrinkage of concrete between cracks should be taken into account. The effects of autogenous shrinkage may be neglected. For simplification and where (6) and (7) are used, the free shrinkage strain of the uncracked member should be used for the determination of secondary effects due to shrinkage.

(4) Unless more accurate method according to (2) and (3) is used, the simplified method given in (5) or (6) and (7) below may be used.

(5) For a tension member the effects of tension stiffening of concrete may be neglected, if in the global analysis the internal forces of the tension member are determined by uncracked analysis and the sectional and internal forces of structural steel members are determined by cracked analysis, neglecting concrete in tension and effects of tension stiffening .

(6) The internal forces in bowstring arches with tension members consisting of a structural steel member and a reinforced concrete member may be determined as follows:

- determination of the internal forces of the steel structure with an effective longitudinal stiffness $(EA_s)_{eff}$ of the cracked concrete tension member according to equation (5.6-1).

$$(E\,A_s)_{\mathrm{eff}} = \frac{E_{\mathrm{s}}\,A_{\mathrm{s}}}{1-0{,}35/(1+n_0\,\rho_{\mathrm{s}})} \qquad (5.6\text{-}1)$$

  where n_o is the modular ratio for short term loading according to 5.4.2.2(2), A_s is the longitudinal reinforcement of the tension member within the effective width and ρ_s is the reinforcement ratio $\rho_s = A_s/A_c$ determined with the effective concrete cross-section area A_c,

- the normal forces of the reinforced concrete tension member $N_{Ed,serv}$ for the serviceability limit state and $N_{Ed,ult}$ for the ultimate limit state are given by

$$N_{\mathrm{Ed,serv.}} = 1{,}15\;\; A_{\mathrm{c}}\;\; f_{\mathrm{ct,eff}}\,(1+n_0\,\rho_{\mathrm{s}}) \qquad (5.6\text{-}2)$$

$$N_{\mathrm{Ed,ult.}} = 1{,}45\;\; A_{\mathrm{c}}\;\; f_{\mathrm{ct,eff}}\,(1+n_0\,\rho_{\mathrm{s}}) \qquad (5.6\text{-}3)$$

  where the symbols are defined above and $f_{ct,eff}$ is the effective tensile strength of concrete. Unless verified by more accurate methods, the effective tensile strength may be assumed as $f_{ct,eff}$ = 0,7 f_{ctm} where the tension member is simultaneously acting as a deck and is subjected to combined global and local effects.

(7) For composite tension members subjected to normal forces and bending moments the cross section properties of the cracked section and the cross-sectional forces of the composite section should be determined with the longitudinal stiffness of the concrete member according to equation (5.6-1). If the sectional normal forces of the reinforced concrete part of the member do not exceed the values given by the equations (5.6-2) and (5.6-3), these values should be used for design.

5.4.2.9 Filler beam decks for bridges

(1) Where the detailing is in accordance with 6.3, in longitudinal bending the effects of slip between the concrete and the steel beams and effects of shear lag may be neglected. The contribution of formwork supported from the steel beams, which becomes part of the permanent construction, should be neglected.

(2) Where the distribution of loads applied after hardening of concrete is not uniform in the direction transverse to the span of the filler beams, the analysis should take account of the transverse distribution of forces due to the difference between the deformation of adjacent filler beams, unless it is verified that sufficient accuracy is obtained by a simplified analysis assuming rigid behaviour in the transverse direction.

(3) Account may be taken of these deformations by using one of the following methods of analysis:

- modelling by an orthotropic continuum by smearing of the steel beams,
- considering the concrete as discontinuous so as to have a plane grid with members having flexural and torsional stiffness where the torsional stiffness of the steel section may be neglected. For the determination of internal forces in the transverse direction, the flexural and torsional stiffness of the transverse members may be assumed to be 50 % of the uncracked stiffness,
- general methods according to 5.4.3.

The nominal value of Poisson's ratio, if needed for calculation, may be assumed to be in all directions zero for ultimate limit states and 0.2 for serviceability limit states.

(4) Internal forces and moments should be determined by elastic analysis, neglecting redistribution of moments and internal forces due to cracking of concrete.

(5) Hogging bending moments of continuous filler beams with Class1 cross-sections at internal supports may be redistributed for ultimate limit states other than fatigue by amounts not exceeding 15% to take into account inelastic behaviour of materials. For each load case the internal forces and moments after redistribution should be in equilibrium with the loads.

(6) Effects of creep on deformations may be taken into account according to 5.4.2.3. The effects of shrinkage of concrete may be neglected.

(7) For the determination of deflections and precamber for the serviceability limit state as well as for dynamic analysis the effective flexural stiffness of filler beams decks may be taken as

$$E_a I_{eff} = 0{,}5\ (E_a I_1 + E_a I_2) \tag{5.6-4}$$

where I_1 and I_2 are the uncracked and the cracked values of second moment of area of the composite cross-section subjected to sagging bending as defined in 1.5.2.11 and 1.5.2.12. The second moment of area I_2 should be determined with the effective cross-section of structural steel, reinforcement and concrete in compression. The area of concrete in compression may be determined from the plastic stress distribution.

(8) The influences of differences and gradients of temperature may be ignored, except for the determination of deflections of railway bridges without ballast bed or railway bridges with non ballasted slab track.

5.4.4 Linear elastic analysis with limited redistribution for allowing cracking of concrete in bridges

(1) For continuous beams in categorie E or D , including longitudinal beams in multiple–beam decks with the concrete slab above the steel beam, the method according to (2) for allowing cracking of concrete may be used, except where the sensitivity of the results of global analysis to the extent of cracking of concrete is very high.

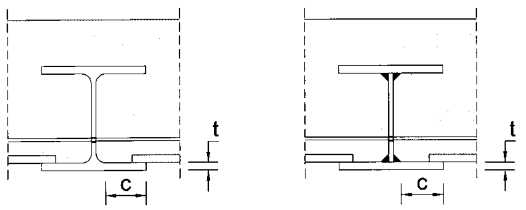
(2) Where for composite members according to (1) the bending moments are calculated by uncracked analysis, at internal supports the bending moments acting on the composite section should be reduced by 10%. For each load case the internal forces and moments after redistribution should be in equilibrium with the loads.

5.5 Classification of cross-sections

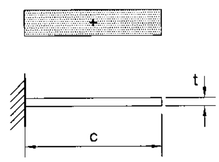
5.5.3 Classification of sections of filler beam decks for bridges

(1) A steel outstand flange of a composite section should be classified in accordance with table 5.2 .

Table 5.2: Classification of steel flanges of filler beams



$$\varepsilon = \sqrt{\frac{235}{f_y}} \text{ with } f_y \text{ in N/mm}^2$$



Stress distribution
(compression positive)

Class	Type	Limit
1	Rolled or welded	c/t $\leq 9\varepsilon$
2		c/t $\leq 14\varepsilon$
3		c/t $\leq 20\varepsilon$

(2) A web in Class3 that is encased in concrete may be represented by an effective web of the same cross-section in Class 2.

Section 6 Ultimate limit states

6.1 Beams

6.1.1 Beams for bridges

(1) Composite beams should be checked for:
- resistance of cross-sections (see 6.2 and 6.3)
- resistance to lateral-torsional buckling (see 6.4)
- resistance to shear buckling and in-plane forces applied to webs (see 6.2.2 and 6.5)
- resistance to longitudinal shear (see 6.6)
- resistance to fatigue (see 6.8).

6.2 Resistances of cross-sections of beams

6.2.1 Bending resistance

6.2.1.3 Additional rules for beams in bridges

(1) Where a composite beam is subjected to biaxial bending, combined bending and torsion, or combined global and local effects, account should be taken of 6.1 and 6.2 of EN 1993-1-1:20xx when determining the contribution of the steel element of a composite flange to the resistance.

(2) Where elastic global analysis is used for a continuous beam, M_{Ed} should not exceed 0.9 $M_{pl,Rd}$ at any cross-section in Class 1 or 2 in sagging bending with the concrete slab in compression where both:
- a cross-section in hogging bending at or near an adjacent support is in Class 3 or 4, and
- the ratio of lengths of the spans adjacent to that support (shorter/longer) is less than 0.6.

Alternatively, a more accurate global analysis that takes account of inelastic behaviour should be used.

(3) For the determination of forces in permanently unbonded tendons, the deformations of the whole member should normally be taken into account.

6.2.1.4 Non-linear resistance to bending

(7) For bridges, paragraph (6) is applicable to sections where the concrete flange is in compression, whether the bending is sagging or hogging; and $N_{c,f}$ is the compressive force corresponding to the resistance $M_{pl,Rd}$, determined according to 6.2.1.2.

[Drafting note: (7) will be deleted if 'in sagging bending' in line 1 of (6) is changed to 'with the concrete flange in compression']

(8) Where the bending resistance of a composite cross-section is determined by non-linear theory, the stresses in prestressing steel should be derived from the design curves in 3.3.6 of EN 1992-1-1:200X. The design initial pre-strain in prestressing tendons should be taken into account when assessing the stresses in the tendons.

6.2.1.5 Elastic resistance to bending

(6) In compression flanges susceptible to lateral torsional buckling, the compressive stress in the steel flange should not exceed that given by 6.4.

(7) In the calculation of the elastic resistance to bending based on the effective cross-section, the limiting stress in prestressing tendons should be taken as f_{pd} according to 3.3.6 of EN 1992-1-1:200X. The stress due to initial prestrain in prestressing tendons should be taken into account in accordance with 5.10.8 of EN 1992-2:200X.

(8) For composite bridges with cross-sections in Class 4, the sum of stresses from different stages of construction and use, calculated on gross sections, may be used for calculating the effective steel cross-section to EN1993-1-5. This single effective cross-section should be used should be used in calculations for design stresses.

(9) As an alternative to (7) and (8), Section 10 of EN 1993-1-5 may be used.

6.2.2 Resistance to vertical shear

6.2.2.5 Additional rules for beams in bridges

(1) When applying 5.4(1) of EN 1993-1-5 for a beam with one flange composite, the dimension of the non-composite flange may be used even if that is the larger steel flange. The axial normal force N_{Ed} in 5.4(2) of EN 1993-1-5 should be taken as the axial force acting on the composite section.

(2) For the calculation of $M_{f,Rd}$ in 5.4(1) of EN 1993-1-5, the resistance to axial force of each flange of the composite section should be determined in accordance with 6.2.1.2(1) and (2). The resistance moment $M_{f,Rd}$ should be taken as the product of the smaller force and the distance between the centroids of the flanges. Where 6.2.1.2(2) applies, the same value of β should be used for $M_{f,Rd}$ as for $M_{pl,Rd}$.

6.2.3 Vertical shear in concrete flanges of composite beams

(1) Resistance to vertical shear due to local action effects should be verified in accordance with 6.2 of EN 1992-2.

Note: For the interaction of vertical shear and normal forces in concrete slabs without shear reinforcement, the factor k_1 should be given in the National Annex. For flanges in tension, the recommended value of k_1 is zero.

6.3 Filler beam decks

6.3.1 Scope

(1) Clauses 6.3.1 to 6.3.5 are applicable to decks consisting of a concrete slab reinforced by longitudinal steel filler beams and by reinforcing steel. A typical cross-section of a filler beam deck with non-participating permanent formwork is shown in Figure 6.8. No application rules are given for fully encased beams.

Note: a National Annex may give a reference to rules for transverse filler beams

(2) Steel beams may be rolled sections, or welded sections with a constant cross-section. For welded sections, both the width of the flanges and the depth of the web should be within the ranges that are available for rolled H- or I- sections.

(3) Spans may be simply supported or continuous. Supports may be skew or not.

(4) To be within the scope of 6.3, filler-beam decks should comply with all of the following conditions :

- the steel beams are not curved in plan;
- the skew θ of all the lines of support complies with : $0 \le \theta \le 30°$ (the value $\theta = 0$ corresponding to a non-skew deck) ;
- the nominal depth h of the steel beams complies with : $210 \text{ mm} \le h \le 1100$ mm ;

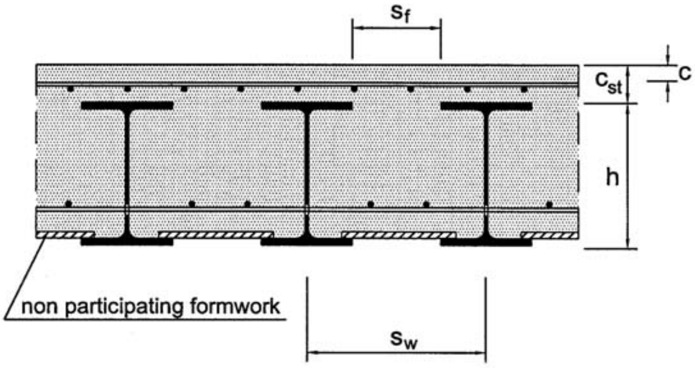


Figure 6.8 : Typical cross-section of a filler beam deck

- the spacing s_w of webs of the steel beams does not exceed the lesser of $h/3 + 600$ mm and 750 mm, where h is the nominal depth of the steel beams in mm ;
- the concrete cover c_{st} above the steel beams satisfies the conditions:

$$c_{st} \geq 70 \text{ mm}, \quad c_{st} \leq 150 \text{ mm}, \quad c_{st} \leq h/3, \quad c_{st} \leq x_{pl} - t_f$$

where x_{pl} is the distance between the plastic neutral axis for sagging bending and the extreme fibre of the concrete in compression, and t_f is the thickness of the steel flange;
- the clear distance s_f between the upper flanges of the steel beams is not less than 150 mm, so as to allow pouring and compaction of concrete;
- the soffit of the lower flange of the steel beams is not encased ;
- a bottom layer of transverse reinforcement passes through the webs of the steel beams, and is anchored beyond the end steel beams, and at each end of each bar, so as to develop its yield strength in accordance with 8.4 of EN 1992-1-1:20xx; ribbed bars in accordance with 3.2.2 and Annex C of EN 1992-1-1:20xx are used; their diameter is not less than 16 mm and their spacing is not more than 300 mm ;
- normal-density concrete is used;
- the surface of the steel beams should be descaled. The soffit, the upper surfaces and the edges of the lower flange of the steel beams should be protected against corrosion;
- for road and railway bridges the holes in the webs of the steel section should be drilled.

6.3.2 General

(1)P Filler beam decks shall be designed for the serviceability and ultimate limit states.

(2) Steel beams with bolted connections and/or welding should be checked against fatigue.

(3) Composite cross-sections should be classified according to 5.5.3.

(4) Mechanical shear connection need not be provided.

6.3.3 Bending moments

(1) The design resistance of composite cross-sections to bending moments should be determined according to 6.2.1.

(2) The design resistance of reinforced concrete sections to transverse bending moments should be determined according to 6.1 of EN 1992-2:200X.

6.3.4 Vertical shear

(1) The resistance of composite cross-sections to vertical shear should be determined according to 6.2.2, unless the value of a contribution from the reinforced concrete part has been established and verified according to 6.2 of EN 1992-2:200X.

(2) Unless a more accurate analysis is used, the distribution of the total vertical shear V_{Ed} into the parts $V_{a,Ed}$ and $V_{c,Ed}$, acting on the steel section and the reinforced concrete section, may be assumed to be in the same ratio as the contributions of the steel section and the reinforced concrete section to the bending resistance.

(3) The design resistance to vertical shear of reinforced concrete sections between filler beams should be verified according to 6.2 of EN 1992-2: 200X.

6.3.5 Resistance and stability of steel beams during execution

(1) Steel beams before the hardening of concrete should be verified according to EN 1993-1-1:200X and EN 1993-2:200X.

6.4 Lateral-torsional buckling of composite beams

6.4.2 Beams in bridges with uniform cross-sections in Class 1, 2 or 3

(1) For beams with a uniform steel cross-section in Class 1, 2, or 3, restrained in accordance with 6.4.2(5), the design buckling resistance moment should be taken as:

$$M_{b,Rd} = \chi_{LT}\ M_{Rd} \tag{6.6}$$

where :

χ_{LT} is the reduction factor for lateral-torsional buckling depending on the relative slenderness $\bar{\lambda}_{LT}$, and

M_{Rd} is the design resistance moment at the relevant cross-section.

Values of the reduction factor χ_{LT} may be obtained from 6.3.2 of EN 1993-1-1:200X.

(2) For cross-sections in Class 1 or 2, M_{Rd} should be determined according to 6.2.1.

(3) For cross-sections in Class 3, M_{Rd} should be determined using expression (6.4), but as the design bending moment that causes either a tensile stress f_{sd} in the

reinforcement or a compressive stress f_{yd} in the extreme fibre of the steel section, whichever is the smaller.

(4) The relative slenderness $\bar{\lambda}_{LT}$ may be calculated from :

$$\bar{\lambda}_{LT} = \sqrt{\frac{M_{Rk}}{M_{cr}}} \tag{6.7}$$

where :

M_{Rk} is the resistance moment of the composite section using the characteristic material properties and the method specified for M_{Rd};

M_{cr} is the elastic critical moment for lateral-torsional buckling determined at the relevant cross-section.

(5) Where the same slab is also attached to one or more supporting steel members approximately parallel to the composite beam considered and the conditions (a) and (b) below are satisfied, the calculation of the elastic critical moment, M_{cr}, may be based on the "continuous U-frame" model. As shown in Figure 6.10, this model takes into account the lateral displacement of the bottom flange causing bending of the steel web, and the rotation of the top flange that is resisted by bending of the slab.

a) One of the flanges of the steel member is attached to a reinforced concrete slab by shear connectors in accordance with 6.6.

b) At each support of the steel member, its bottom flange is laterally restrained and its web is stiffened. Elsewhere, the web should be un-stiffened.

(6) At the level of the top steel flange, a rotational stiffness k_s per unit length of steel beam may be adopted to represent the U-frame model by a beam alone:

$$k_s = \frac{k_1 k_2}{k_1 + k_2} \tag{6.8}$$

where:

k_1 is the flexural stiffness of the cracked concrete slab in the direction transverse to the steel beam, which may be taken as:

$$k_1 = \alpha\, E_a I_2 / a \tag{6.9}$$

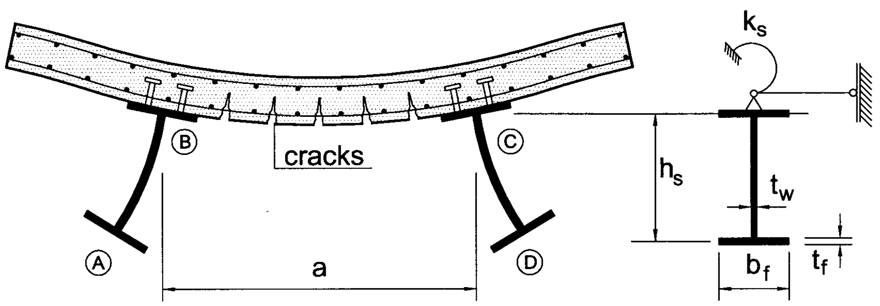


Figure 6.10 U-frame model

with $\alpha = 4$ for a slab continuous across the steel beam, and $\alpha = 2$ for an end slab with or without a cantilever;

a is the spacing between the parallel beams;

$E_a I_2$ is the "cracked" flexural stiffness per unit width of the concrete or composite slab, as defined in 1.5.2.12, where I_2 should be taken as the lower of the value at midspan, for sagging bending, and the value at the supporting steel member, for hogging bending;

k_2 is the flexural stiffness of the steel web, to be taken as:

$$k_2 = \frac{E_a t_w^3}{4(1-\nu_a^2)h_s} \tag{6.10}$$

where ν_a is Poisson's ratio for steel and h_s and t_w are defined in Figure 6.10.

(8) In the U-frame model, the favourable effect of the St. Venant torsional stiffness, $G_a I_{at}$, of the steel section may be taken into account for the calculation of M_{cr}.

6.4.3 General methods for buckling of members and frames

6.4.3.1 General method

(1) For composite members outside the scope of 6.7 and for composite frames, clause 6.3.4 of EN 1993-1-1 is applicable. For the determination of α_{ult} and α_{crit}, appropriate resistances and stiffnesses of concrete and composite members should be used, in accordance with EN 1992 and EN 1994.

6.4.3.2 Simplified method

(1) Clause 6.3.4.2 and Annex D2.4 of EN 1993-2 are applicable to structural steel flanges of composite beams and chords of composite trusses. Where restraint is provided by concrete or composite members, appropriate elastic stiffnesses should be used, in accordance with EN 1992 and EN 1994.

6.6 Shear connection

6.6.1 General

6.6.1.1 Basis of design

(13) Adjacent to cross frames and vertical web stiffeners, and for composite box girders, the effects should be considered of restraint by the shear connection of rotation of the slab about an axis parallel to the axis of the steel beam.

Note: reference to further guidance may be given in a National Annex.

6.6.1.2 Ultimate limit states other than fatigue

(1) For verifications for ultimate limit states, the size and spacing of shear connectors may be kept constant over any length where the design longitudinal shear per unit length does not exceed the design shear resistance P_{Rd} by more than 10% Over every such length, the total design longitudinal shear force should not exceed the total design shear resistance.

6.6.2 Longitudinal shear force in beams for bridges

6.6.2.1 Beams in which elastic or non-linear theory is used for resistances of cross-sections

(1) For any combination and arrangement of design actions, the longitudinal shear per unit length at an interface between steel and concrete in a composite member, v_{Ed}, should be found from the rate of change of the longitudinal force in either the steel element or the concrete element of the composite section. Where elastic theory is used for resistances of sections, the envelope of transverse shear force in the relevant direction may be used.

(2) In uncracked members, and in cracked members where effects of tension stiffening are neglected in global analysis, the elastic properties of the uncracked section should be used for the determination of the longitudinal shear force. Where account is taken in global analysis of the effects of tension stiffening and possible over-strength of concrete, these effects should be allowed for in the determination of longitudinal shear.

(3) Where concentrated longitudinal shear forces occur, account should be taken of the local effects of longitudinal slip; for example, as provided in 6.6.2.3 to 6.6.2.5. Otherwise, the effects of longitudinal slip may be neglected.

(4) Where a sudden change of cross-section leads to an excessive local value for v_{Ed}, and in the absence of a more precise analysis, the distribution along the interface of the longitudinal shear force $V_{L,Ed}$ caused by the change of cross-section may be assumed to be as given in 6.6.2.3, with e_d taken as zero.

(5) For composite box girders, the longitudinal shear force on the connectors should include the effects of bending and torsion, and also of distortion according to 6.2.7 of EN 1993-2:200X, if this is not negligible. For box girders with a flange designed as a composite plate, see 9.4.

6.6.2.2 Beams in bridges with some cross-sections in Class 1 or 2 and inelastic behaviour

(1)P Where in members with some cross-sections in Class 1 or 2 the design bending moment M_{Ed} exceeds the elastic bending resistance $M_{el,Rd}$, within the inelastic lengths of the member account shall be taken of the non-linear relationship between transverse shear and longitudinal shear.

(2) This paragraph applies to regions where the concrete slab is in compression, as shown in Figure 6.11. Unless otherwise verified, the number of shear connectors provided within the inelastic length $L_{A\text{-}B}$ should be sufficient to resist the longitudinal shear force $V_{L,Ed}$, resulting from the difference between the normal forces N_{cd} and $N_{c,el}$ in the concrete slab at the points B and A. The bending resistance $M_{el,Rd}$ is defined in 6.2.1.4. If the maximum bending moment $M_{Ed,max}$ at point B is smaller than the plastic bending resistance $M_{pl,Rd}$, the normal force N_{cd} at point B may be determined according 6.2.1.4 (6) and Figure 6.6, or alternatively with the simplified linear relationship according to Figure 6.11.

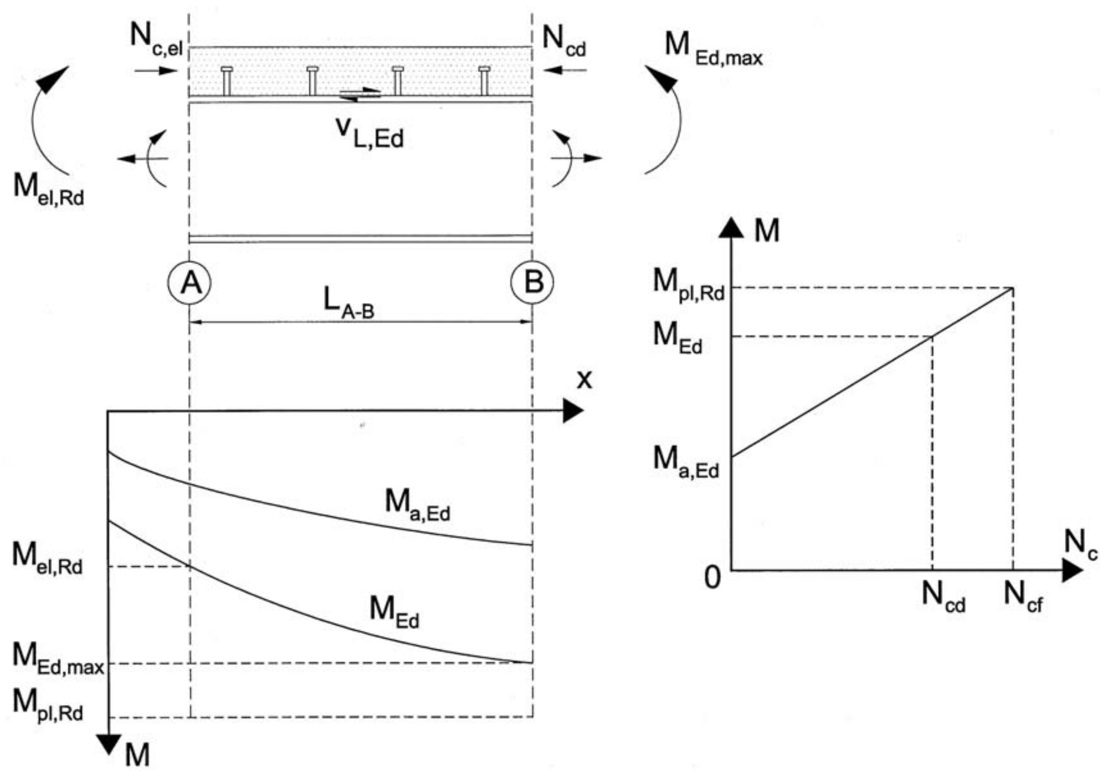


Figure 6.11: Longitudinal shear in beams with cross sections in Class 1 or 2, concrete flanges in compression and inelastic behaviour of cross sections

(3) Paragraphs (3) to (5) apply to regions where the concrete slab is in tension. For the determination of the elastic region, the bending moment $M_{el,Rd}$ and factor k should be found according to 6.2.1.4(6) and 6.2.1.5(6).

(4) In regions where M_{Ed} does not exceed the elastic bending resistance $M_{el,Rd}$, longitudinal shear forces should be determined by elastic analysis with the cross-section properties of the uncracked section taking into account effects of the sequence of construction.

(5) In inelastic regions, unless a more precise method is used, longitudinal shear forces should be determined as in paragraph (4).

6.6.2.3 Local effects of concentrated longitudinal shear force

(1) Clause 6.6.2.3 is applicable to the determination of the distribution along an interface between steel and concrete of the design longitudinal shear force $V_{L,Ed}$ caused by the statically determinate effects of the application of a force F_{Ed} to the concrete or steel element by a bonded or unbonded tendon. The force F_{Ed} is the component of the relevant force in a direction parallel to the longitudinal axis of the composite beam, applied after the shear connection has become effective.

(2) The distribution of $V_{L,Ed}$ caused by several forces F_{Ed} may be obtained by summation.

(3) In the absence of a more precise determination, the force $F_{Ed} - V_{L,Ed}$ may be assumed to disperse into the concrete or steel element at an angle of spread 2β, where $\beta =$ arc tan 2/3.

(4) The force $V_{L,Ed}$ may be assumed to be distributed along a length L_v of shear connection as shown (for example) in Figure 6.12, with a maximum shear force per unit length given by

$$v_{L,Ed,max} = V_{L,Ed} / (e_d + b_{eff}/2), \qquad (6.12)$$

where

b_{eff} is the effective width for global analysis, given by 5.4.1.2,

e_d is either $2e_h$ or $2e_v$,

e_h is the lateral distance from the point of application of force F_{Ed} to the relevant steel web, if it is applied to the slab,

e_v is the vertical distance from the point of application of force F_{Ed} to the plane of the shear connection concerned, if it is applied to the steel element.

(5) Where the force F_{Ed} is applied over a length that is not negligible, that length may be added to e_d.

(6) Where stud shear connectors are used, a rectangular distribution of shear force per unit length may be assumed, so that within a length of concrete flange,

$$v_{L,Ed,max} = V_{L,Ed} / (e_d + b_{eff}) \qquad (6.13)$$

and at an end of a flange,

$$v_{L,Ed,max} = 2\, V_{L,Ed} / (e_d + b_{eff}). \qquad (6.14)$$

(7) Where shear connection is not present along the whole of the length L_v, (e.g., at a free end of a slab), the distribution of $v_{L,Ed}$ should be modified as appropriate.

(8) If the force F_{Ed} is applied at a free end of the concrete or steel element, the distribution of the force $V_{L,Ed}$ may be assumed to be half that given by paragraph (4); i.e., extending along a length of shear connection $L_v/2$, as in Fig. 6.12, with a maximum shear force per unit length given by

$$v_{L,Ed,max} = 2\, V_{L,Ed} / (e_d + b_{eff}/2). \qquad (6.15)$$

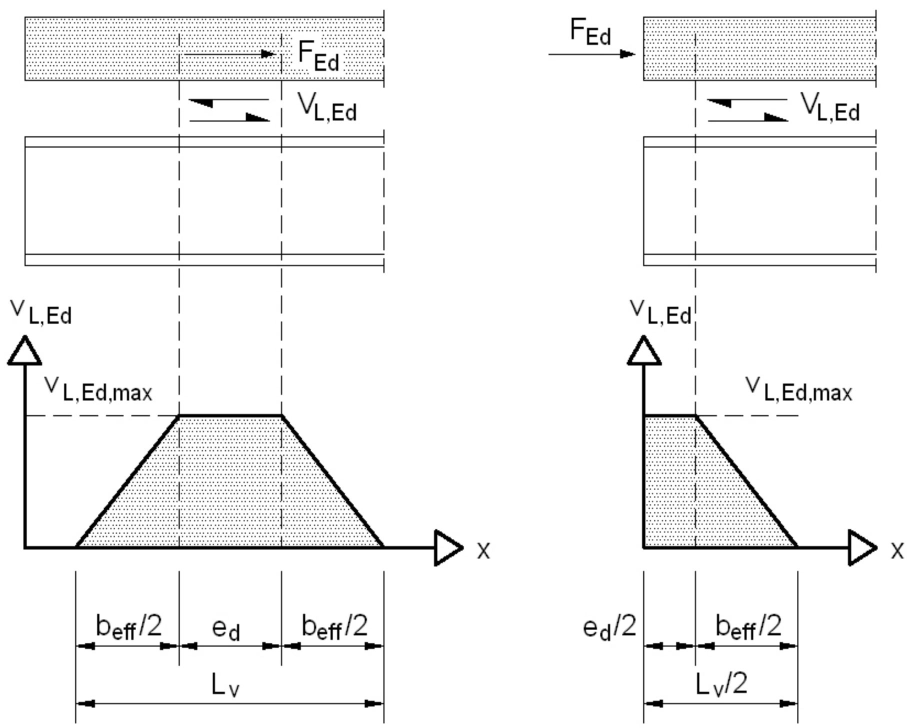


Figure 6.12: Distribution of longitudinal shear force along the interface

6.6.2.4 Temperature effects

(1)P Longitudinal shear due to thermal actions in accordance with EN 1991-1-5 shall be considered, and taken into account where appropriate.

(2) Calculations for primary effects give a design longitudinal shear force, $V_{L,Ed}$ to be transferred across the interface between steel and concrete at each free end of the member considered. The distribution of this force may be assumed to be triangular, with a maximum shear force per unit length

$$v_{L,Ed,max} = 2\ V_{L,Ed} / b_{eff} \qquad (6.16)$$

at the free end of the slab, where b_{eff} is the effective width for global analysis, given by 5.4.1.2. Where stud shear connectors are used, the distribution may alternatively be assumed to be uniform along a length b_{eff} adjacent to the free end of the slab.

(3) The forces transferred by shear connectors may be assumed to disperse into the concrete slab at an angle of spread 2β, where β = arc tan 2/3.

6.6.2.5 Shrinkage effects

(1)P Where the effects of shrinkage of concrete adversely affect the maximum resultant forces on the shear connectors or the maximum resultant stresses in the concrete element, they shall be taken into account where appropriate.

(2) Where it is necessary to estimate the effects of shrinkage, they may be determined by a method similar to that given for temperature effects in 6.6.2.4, using a modular ratio that takes account of the reduction of shrinkage effects by creep of concrete in accordance with 5.4.2.2.

(3) Where the primary effects of shrinkage are determined at intermediate stages of the construction of a concrete slab, the breadth b_{eff} in 6.6.2.4 should be determined for an equivalent span equal to the continuous length of concrete slab where the shear connection is effective, within the span considered.

6.6.3 Headed stud connectors in solid slabs and concrete encasement

(2) The formulae in 6.6.3.1(1) should be verified by tests, see B.2 of EN 1994-1-1:200X, before being used for studs of diameter greater than 25 mm, or studs with weld collars which do not comply with the requirements of EN ISO 13918:1998.

(3) Where, in bridges, headed stud connectors are arranged in such a way that splitting forces can occur in the direction of the slab thickness, paragraph (1) for the design shear resistance may be assumed to apply, provided that the local reinforcement is in accordance with 6.6.5.3(4).

(4) Where the condition of paragraph (3) on local reinforcement is not satisfied, the shear resistance of the studs should be based on evidence from tests.

Note: the National Annex may refer to relevant information.

6.6.5 Detailing of the shear connection and influence of execution

6.6.5.2 Cover and concreting

(5) For bridges, cover over shear connectors should be provided, according to (2).

6.6.5.3 Local reinforcement in the slab

(4) Where, in bridges, headed stud connectors of diameter d are arranged in such a way that splitting forces can occur in the direction of the slab thickness, transverse reinforcement should be provided, as shown in Figure 6.13, such that $e_v \geq 6\ d$. For connectors with shear resistance P_{Rd} at longitudinal spacing s, the cross-sectional area of reinforcement per unit length should satisfy $A_{sv} \geq P_{Rd} / (3 f_{sd}\ s)$.

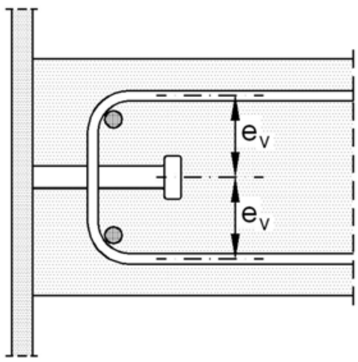


Figure 6.13: Local reinforcement for splitting forces

6.6.5.5 Spacing of connectors

(3) Connectors may be placed in groups, with the spacing of groups greater than that specified for individual connectors, provided that consideration is given in design to:

- the non-uniform flow of longitudinal shear,

- the greater possibility of slip and vertical separation between the slab and the steel member,
- buckling of the steel flange, and
- the local resistance of the slab to the concentrated force from the connectors.

(4) The maximum longitudinal centre-to-centre spacing of shear connectors should not exceed the lesser of four times the slab thickness and 800 mm.

6.6.5.6 Dimensions of the steel flange

(2) The distance e_D between the edge of a connector and the edge of the flange of the beam to which it is welded, see Figure 6.14, should not be less than 25 mm.

6.8 Fatigue

6.8.1 General

(4) For structural steel, no fatigue assessment is required where 9.1.1(2) of EN 1993-2 applies.

(5) For concrete and reinforcement, no fatigue assessment is required where 6.8.1(3) of EN 1992-2 applies.

6.8.2 Partial safety factors for fatigue assessment

(2) Partial safety factors for fatigue loading γ_{Ff} should be applied.

Note: Partial safety factors γ_{Ff} are given in Notes in EN 1992-2 and EN 1993-2.

6.8.4 Internal forces and fatigue loadings

(2) Fatigue loading should be obtained from EN 1991-2. Where no fatigue loading is specified, Annex A.1 of EN 1993-1-9 may be used.

(3) For road bridges simplified methods according to EN 1992-2 and EN 1993-2, based on Fatigue Load Model 3 of clause 4.6 of EN 1991-2 may be used for verifications of fatigue resistance.

(4) For road bridges prestressed by tendons and/or imposed deformations, the factored load model according to clause N2.1 of EN 1992-2 should be used for the verification of reinforcement and tendons.

(5) For railway bridges the characteristic values for load model 71 according EN 1991-2 should be used.

6.8.5 Stresses

6.8.5.5 Stresses in reinforcement and prestressing steel in members prestressed by bonded tendons

(1)P For members with bonded tendons the different bond behaviour of reinforcement and tendons shall be taken into account for the determination of stresses in reinforcement and tendons .

(2) Stresses should be determined in accordance with 6.8.5.4 but calculating the stress $\sigma_{s,max,f}$ according 7.4.3 (4).

6.8.6 Stress ranges in structural steel, reinforcement, tendons and shear connectors

6.8.6.1 Structural steel, reinforcement and tendons

(3) The damage equivalent factor λ depends on the loading spectrum and the slope of the fatigue strength curve. The factor λ for structural steel elements is given in EN 1993-2, 9.5.2 for road bridges and in EN1993-2, 9.5.3 for railway bridges.

(4) The factors $\lambda=\lambda_e$ for reinforcement and prestressing steel are given in EN 1992-2, Annex N; in N.2 for road bridges and in N.3 for railway bridges.

(5) For railway bridges the damage equivalent impact factor ϕ is defined in EN 1991-2, 6.4.5.

(6) For road bridges the damage equivalent impact factor may be taken as equal to 1.0, because it is included in fatigue load model 3.

6.8.6.2 Shear connection

(5) For bridges the damage equivalent factor λ_v for headed studs in shear should be determined from

$$\lambda_v = \lambda_{v,1}\ \lambda_{v,2}\ \lambda_{v,3}\ \lambda_{v,4} \tag{6.55}$$

where the factors $\lambda_{v,1}$ to $\lambda_{v,4}$ are defined in (6) and (7).

(6) For road bridges of span up to 100 m the factor $\lambda_{v,1}$=1,55 should be used. The factors $\lambda_{v,2}$ to $\lambda_{v,4}$ should be determined in accordance with 9.5.2 (4) to (7) of EN 1993-2 but using exponents 8 and 1/8 in place of those given, to allow for the relevant slope m = 8 of the fatigue strength curve for headed studs, given in 6.8.3.

(7) For railway bridges the factor $\lambda_{v,1}$ should be taken from Figure 6.27. The factors $\lambda_{v,2}$ to $\lambda_{v,4}$ should be determined in accordance with N3.1.4 to N3.1.6 of Annex N of EN 1992-2 but using instead of the exponent k_2 the exponent m = 8 for headed studs.

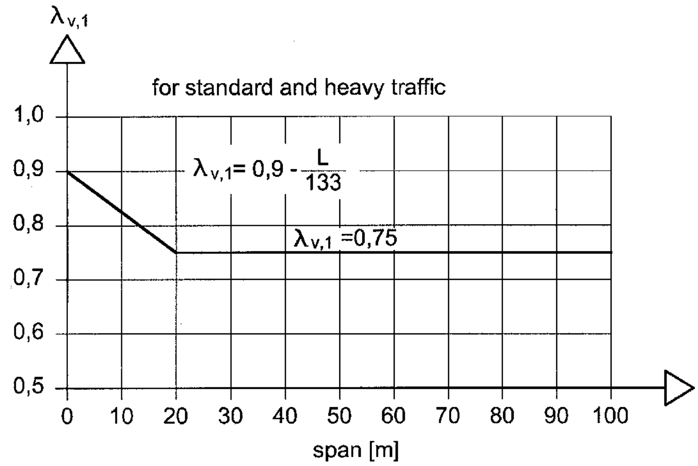


Figure 6.27: Values $\lambda_{v,1}$ for load model 71 according to EN 1991-2:200X

6.8.7 Fatigue assessment based on nominal stress ranges

6.8.7.1 Structural steel, reinforcement, pre-stressing steel and concrete

(3) For bridges the fatigue assessment for structural steel should follow section 9 of EN 1993-2.

6.9 Tension members in composite bridges

(1) A concrete tension member in a composite system should be designed in accordance with Sections 6 and 9 of EN 1992-1. For prestressing by tendons the effect of different bond behaviour of prestressing and reinforcing steel should be taken into account according to 6.8.2 of EN 1992-1:200X.

Note: ‘Composite system’ is defined in 5.4.2.8.

(2) For tension members in half-through bridges or bowstring arch bridges where the tension member is simultaneously acting as a deck and is subjected to combined global and local effects, the design shear resistance for local vertical shear and for punching shear due to permanent loads and traffic loads should be verified. Unless a more precise method is used, the verification should be according to 6.2 and 6.4 of EN 1992-1:200X by taking into account the normal force of the reinforced concrete element according to 5.4.2.8(3) and (6).

(3) At the ends of a concrete part of a tension member, for the introduction of the normal force, a concentrated group of shear connectors designed according to 6.6 should be provided. The shear connection should be able to transfer the design value of the normal force of the concrete tension element over a length 1.5 b, where b is the larger of the outstand of the concrete member and half the distance between adjacent steel elements. Where the shear connectors are verified for a normal force determined by 5.4.2.8(6), equation (5.6-3) should be used.

(4)P Provision shall be made for internal forces and moments from members connected to the ends of a composite tension member to be distributed between the structural steel and reinforced concrete elements.

(5) For composite tension members subject to tension and bending a shear connection should be provided according to 6.6.

(6) For composite tension members such as diagonals in trusses, the introduction length for the shear force should not be assumed to exceed twice the minimum transverse dimension of the member.

Section 7 Serviceability limit states

7.1 General

(3)P Composite bridges or specific parts of it shall be classified into environmental classes according to Table 4.1 of En 1992-2 and into categories according to Table 7.0 of EN 1992-2.

(4) In order to ensure the performance required, the bridge or parts of the bridge may be classified into design categories according to EN 1992-2, 7.1.2(2) for both the construction phases and for persistent situations.

(5) Where relevant, requirements and criteria given in A2.4 of Annex A2 of EN 1990 should be taken into account.

(6) Serviceability limit states for composite plates are covered in section 9.

7.2 Stresses

7.2.1 General

(8) Stresses in the concrete slab and its reinforcement caused by simultaneous global and local actions should be added.

7.2.2 Stress limitation for bridges

(1)P Excessive creep and microcracking shall be avoided by limiting the compressive stress in concrete.

(2) Stress limitations for concrete to the value $k_i f_{ck}$ should be in accordance with EN 1992-2, 7.2(2)and (3).

Note: The value $k_i = k_1$ and $k_i = k_2$ may be given in the National Annex. The recommended values are the values proposed in the notes of EN 1992-2, 7.2.

(3)P The stress in reinforcing steel and in prestressing tendons shall be such that inelastic strains in the steel are avoided.

(4) The tensile stresses in the reinforcing steel and tendons under the characteristic combination of actions should be limited to the values $k_i f_{sk}$ and $k_5 f_{pk}$, respectively, according to EN 1992-2, 7.2(5).

Note: The values $k_i = k_3$ and $k_i = k_4$ for reinforcing steel and k_5 for tendons may be given in the National Annex. The recommended values are the values proposed in the note of EN 1992-2, 7(5).

(5) The stresses in structural steel should be in accordance with 7.3 of EN 1993-2.

(6) For serviceability limit states, forces applied to shear connectors should be limited according to 6.8.1(3).

7.2.3 Web breathing

(1) The slenderness of unstiffened or stiffened web plates of composite girders should be limited according to 7.4 of EN 1993-2.

7.3 Deformations in bridges

7.3.1 Deflections

(1) Deflections due to loading applied to the steel member alone should be calculated in accordance with EN 1993-1-1.

(2) Deflections due to loading applied to the composite member should be calculated using elastic analysis in accordance with Section 5.

(3)P Deformations during construction shall be controlled such that the concrete is not impaired during its placing and setting by uncontrolled displacements and the required long-term geometry is achieved.

(4) For the limit state of deformation see EN 1990, A2-4 of Annex A2 and EN 1993-2, 7.5 to 7.8 and 7.12, where relevant.

7.3.2 Vibrations

(1) For the limit state of vibration see EN 1990, A2-4 of Annex A2, EN 1991-2, 5.7 and 6.4 and EN 1993-2, 7.7 to 7.10 where relevant.

7.4 Cracking of concrete

7.4.1 General

(4) For general use, application rules are given in 7.4.2 and 7.4.3 for design crack widths w_k of 0,3 mm for reinforced concrete and 0,2 mm for prestressed concrete. For general use the reinforcing bars should have bond properties in accordance with EN 1992-1, 3.2.2.

(5) Where composite action becomes effective as concrete hardens, effects of heat of hydration of cement and corresponding thermal shrinkage should be taken into account only during the construction stage for the serviceability limit state to define areas where tension is expected.

(6) Unless specific measures are taken to limit the effects of heat of hydration of cement, a temperature difference between the concrete section and the steel section (concrete cooler) should be assumed. For the determination of the cracked regions, the short term modulus should be used and the mean values of the tensile strength of the concrete should be defined in accordance with EN 1992-2, 7.3.2(2).

Note: National Annex may give specific measures and a temperature difference, the recommended value for the temperature difference is 20K.

7.4.2 Minimum reinforcement

(4) For the determination of the minimum reinforcement in concrete flanges with variable depth the local depth should be used.

(5) The minimum reinforcement according to (1) and (2) should be placed where under the characteristic combination of actions, the stresses in concrete are tensile. For members prestressed by bonded tendons, 7.3.2(4) of EN 1992-2 applies.

(6) Where bonded tendons are used, the contribution of bonded tendons to minimum reinforcement may be taken into account in accordance with EN 1992-2, 7.3.2(3).

7.4.3 Control of cracking due to direct loading

(4) Where bonded tendons are used, the stresses in reinforcement or prestressing steel may be calculated in accordance with EN 1992-2, 7.3.3(6). For the determination of stresses according to 7.7a and 7.7b in 7.3.3(6) of EN 1992-2, the stress σ_s^{II} should be determined with the equations 7.4 and 7.5 of this code, where instead of ρ_s, the geometric ratio ρ_{tot} should be used.

7.5 Filler beam decks

7.5.1 General

(1) The action effects for the serviceability limit states should be determined according to paragraphs 1 to 4 and 5 to 8 of 5.4.2.9.

7.5.2 Cracking of concrete

(1) The application rules of 7.4.1 should be considered.

(2) For the reinforcing bars in the direction of the steel beams within the whole thickness of the deck, 7.5.3 and 7.5.4 should be applied.

7.5.3 Minimum reinforcement

(1) Unless verified by more accurate methods according to EN 1992-2, 7.3.2, the minimum longitudinal top reinforcement $A_{s,min}$ per filler beam should be determined from the following condition:

$$A_{s,min} \geq 0{,}01\, A_{c,eff} \qquad (7.7)$$

where

A_{ceff}	is the effective area of concrete given by $A_{c,eff} = s_w\, c_{st} \leq s_w\, d_{eff}$
d_{eff}	is the effective thickness of the concrete given by $d_{eff} = c + 7{,}5\, \varnothing_s$
$\varnothing_s$	is the diameter of the longitudinal reinforcement in [mm] within the range 10mm $\leq \varnothing s \leq$ 16mm
c, c_{st}	is the concrete cover of the longitudinal reinforcement and the structural steel section (see figure 6.8)
s_w	is defined in figure 6.8

The bar spacing s of the longitudinal reinforcement should fulfil the following condition

$$100 \text{ mm} \leq s \leq 150 \text{ mm}$$

7.5.4 Control of cracking due to direct loading

(1) 7.3.3 of EN 1992-2 should be applied. The stresses in the reinforcement should be calculated by using the cross-section properties of the cracked composite section with the second moment of area I_2 according to 1.5.2.12.

Section 8 Precast concrete slabs in composite bridges

8.1 General

(1)P This Section 8 deals with reinforced or prestressed precast concrete slabs, used either as full depth flanges of bridge decks or as partial depth slabs acting with insitu concrete.

(2) Precast bridge slabs should be designed in accordance with xxx of EN 1992-2 and also for composite action with the steel beam.

Drafting note: Awaiting progress on EN 1992-2 the references will be specified.

(3) Relevant tolerances should be in accordance with xxx of EN 1992-2 unless more strict tolerances are required for the specific project.

Drafting note: Awaiting progress on EN 1992-2 the references will be specified.

8.2 Actions

(1) EN1991-6 is applicable to precast elements acting as permanent formwork. These minimum loads are not necessarily sufficient and the requirements of the construction method should be taken into account.

8.3 Design, analysis and detailing of the bridge slab

(1) The precast slab together with any insitu concrete should be designed as continuous in both the longitudinal and the transverse directions. The joints between slabs should be designed to transmit in-plane forces as well as bending moments and shears. Compression perpendicular to the joint may be assumed to be transmitted by contact pressure if the joint is filled with mortar or glue or if it is shown by tests that the mating surfaces are in sufficiently close contact.

(2) For the use of stud connectors in groups, see 6.6.5.5(3).

(3) A stepped distribution of shear forces may be used provided that the limitations in 6.6.1.2(1) are observed.

8.4 Interface between steel beam and concrete slab

8.4.1 Bedding and tolerances

(1)P Where precast slabs are supported on steel beams without bedding the influence of the vertical tolerances of the bearing surfaces shall be considered.

8.4.2 Corrosion

(1) A steel flange under precast slabs without bedding should have the same corrosion protection as the rest of the steelwork but for a top coating provided after erection. Bedding with the purpose of protecting against corrosion may be designed to be not loadbearing.

8.4.3 Shear connection and transverse reinforcement

(1) The shear connection and transverse reinforcement should be designed in accordance with the relevant clauses of Section 6 and 7.

(2) If shear connectors welded to the steel beam project into recesses within slabs or joints between slabs, which are filled with concrete after erection, the detailing and the properties of the concrete should be such that it can be cast properly. The minimum thickness of the infill around such shear connectors should be sufficient to allow for full compaction of the filling material taking account of tolerances.

(4) If shear connectors are arranged in groups, sufficient reinforcement should be provided near each group to prevent premature local failure in either the precast or the insitu concrete.

Note: A National Annex may refer to relevant information.

Section 9 Composite plates in bridges

9.1 General

(1)P This Section 9 is valid for composite plates consisting of a nominally flat plate of structural steel connected to a site cast concrete slab by headed studs. Double skin plates or other types of connectors are not covered. The intended use of the composite plate is as a flange in a bridge deck carrying transverse loads as well as in-plane forces, or as a bottom flange in a box girder.

(2) The steel plate should be supported during casting either permanently or by temporary supports in order to limit its deflection to less than 0,05 times the slab thickness unless the additional weight of concrete is taken into account.

(3) The effective width should be determined according to 5.4.1.2, where b_0 should be taken as $2a_w$ with a_w as defined in 9.4(4).

(4) For global analysis, clauses 5.1 and 5.4 apply.

9.2 Design for local effects

(1) Local effects are bending moments and shears caused by transverse loads on the plate acting as an one- or two-way slab. For the purpose of analysis of local action effects the slab may be assumed elastic and uncracked. A top flange of an I-girder need not be designed as composite in the transverse direction.

(2) The concrete and the steel plate may be assumed to act compositely without slip. The resistance to bending and vertical shear should be verified as for a reinforced concrete slab where the steel plate is considered as reinforcement.

(3) The design resistance for shear in 6.2.2 of EN 1992-1-1 is applicable. The plate may be considered as reinforcement if the distance, in two perpendicular directions, between shear connectors does not exceed three times the slab thickness.

9.3 Design for global effects

(1)P The composite plate shall be designed to resist all forces from axial loads and global bending and torsion of all longitudinal girders or cross-girders of which it forms a part.

(2) The design resistance to in-plane compression may be taken as the sum of the design resistances of the concrete and the steel plate within the effective width. Reduction in strength due to second order effects should be considered according to 5.8 of EN 1992-1-1, if appropriate.

(3) The design resistance for tension should be taken as the sum of the design resistances of the steel plate and the reinforcement within the effective width.

(4) Interaction with local load effects should be considered for the shear connectors as stated in 9.4(1)P. Otherwise it need not be considered. Connectors that carry loads both

longitudinally and transversely may be verified for the vector sum of the simultaneous forces on the connector.

9.4 Design of shear connectors

(1)P Resistance to fatigue and requirements for serviceability limit states shall be verified for the combined local and simultaneous global effect.

(2) The design strength of stud connectors in 6.6.3 and 6.8.3 may be used provided that the concrete slab has bottom reinforcement with area not less than 0,002 times the concrete area in each of two perpendicular directions.

(3) The detailing rules of 6.6.5 are applicable.

(4) For wide girder flanges the distribution of longitudinal shear due to global effects for serviceability and fatigue limit states may be determined as follows in order to account for slip and shear lag. The longitudinal force P_{Ed} on a connector at distance x from the nearest web may be taken as

$$P_{Ed} = \frac{v_{Ed}}{n_{tot}}\left[\left(3{,}85\left(\frac{n_w}{n_{tot}}\right)^{-0{,}17} - 3\right)\left(1-\frac{x}{b}\right)^2 + 0{,}15\right] \tag{9.1}$$

where

v_{Ed} is the design longitudinal shear per unit length due to global effects,

n_{tot} is the total number of connectors of the same size per unit length of girder within the width b in Figure 9.1, provided that the number of connectors per unit area does not increase with x,

n_{w} is the number of connectors per unit length placed within a distance from the web a_{w} equal to the larger of $10t_{\mathrm{f}}$ and 200 mm, where t_{f} is the thickness of the steel plate. For these connectors x should be taken as 0,

b is equal to half the distance between adjacent webs or the distance between the web and the free edge of the flange.

In case of a flange projecting up to a_{w} outside the web, n and n_{w} may include the connectors placed on the flange. For efficiency, n_{w} should be as high as possible and the spacing of the connectors should be small enough to avoid premature local buckling of the plate, see 6.6.5.5 (4).

(5) Verification of shear connectors in composite bottom flanges of box sections according to (4) may be neglected, if the arrangement of the shear connectors is based on the following rules. Shear connectors should be concentrated in box girder corner. At least 50% of the total amount of shear connectors, which are responsible for the transfer of the longitudinal shear force in the lower flange should be placed in the area a_{w} taken as the largest of $20t_{\mathrm{f}}$, $0{,}2b_{\mathrm{ei}}$ and 400 mm. Here b_{ei} is the effective width of the lower flange according to 5.4.1.2.

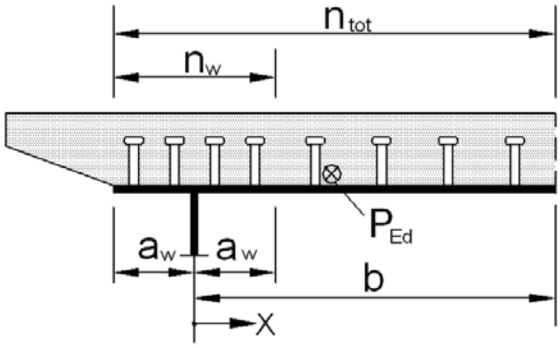


Figure 9.1: Definition of notations in equation (9.1)

(6) For ultimate limit states it may be assumed that all connectors within the effective width carry the same longitudinal force.

(7) Where restraint from shear connectors is relied upon to prevent local buckling of the steel element of a composite plate in compression, the centre-to-centre spacings of the connectors should not exceed the limits given in Table 9.1, where t is the thickness of the steel plate and $\varepsilon = \sqrt{235 / f_y}$,with f_y in N/mm^2 units.

Table 9.1: Upper limits to spacings of shear connectors in a composite plate in compression

		Class 2	Class 3
Transverse to the direction of compressive stress	outstand flange: interior flange:	14 $t\varepsilon$ 45 $t\varepsilon$	20 $t\varepsilon$ 50 $t\varepsilon$
In the direction of compressive stress	all composite plates:	22 $t\varepsilon$	25 $t\varepsilon$