BS EN 1994-1-1:2004

Eurocode 4: Design of composite steel and concrete structures —

Part 1-1: General rules and rules for buildings

The European Standard EN 1994-1-1:2004 has the status of a British Standard

ICS 91.010.30; 91.080.10; 91.080.40

British Standards

EN 10025-6: 2002	Hot-rolled products of structural steels: Technical delivery conditions for flat products of high yield strength structural steels in the quenched and tempered condition
EN 10147: 2000	Continuously hot-dip zinc coated structural steels strip and sheet: Technical delivery conditions
EN 10149-2: 1995	Hot-rolled flat products made of high yield strength steels for cold-forming: Delivery conditions for thermomechanically rolled steels
EN 10149-3: 1995	Hot-rolled flat products made of high yield strength steels for cold-forming: Delivery conditions for normalised or normalised rolled steels

1.3 Assumptions

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

– those given in clauses 1.3 of EN1992-1-1 and EN1993-1-1.

1.4 Distinction between principles and application rules

(1) The rules in EN 1990, 1.4 apply.

1.5 Definitions

1.5.1 General

(1) The terms and definitions given in EN 1990, 1.5, EN 1992-1-1, 1.5 and EN 1993-1-1, 1.5 apply.

1.5.2 Additional terms and definitions used in this Standard

1.5.2.1 Composite member
a structural member with components of concrete and of structural or cold-formed steel, interconnected by shear connection so as to limit the longitudinal slip between concrete and steel and the separation of one component from the other

1.5.2.2 Shear connection
an interconnection between the concrete and steel components of a composite member that has sufficient strength and stiffness to enable the two components to be designed as parts of a single structural member

1.5.2.3 Composite behaviour
behaviour which occurs after the shear connection has become effective due to hardening of concrete

1.5.2.4 Composite beam
a composite member subjected mainly to bending

1.5.2.5 Composite column
a composite member subjected mainly to compression or to compression and bending

1.5.2.6 Composite slab
a slab in which profiled steel sheets are used initially as permanent shuttering and subsequently combine structurally with the hardened concrete and act as tensile reinforcement in the finished floor

1.5.2.7 Composite frame
a framed structure in which some or all of the elements are composite members and most of the remainder are structural steel members

1.5.2.8 Composite joint
a joint between a composite member and another composite, steel or reinforced concrete member, in which reinforcement is taken into account in design for the resistance and the stiffness of the joint

1.5.2.9 Propped structure or member
a structure or member where the weight of concrete elements is applied to the steel elements which are supported in the span, or is carried independently until the concrete elements are able to resist stresses

1.5.2.10 Un-propped structure or member
a structure or member in which the weight of concrete elements is applied to steel elements which are unsupported in the span

1.5.2.11 Un-cracked flexural stiffness
the stiffness E_aI_1 of a cross-section of a composite member where I_1 is the second moment of area of the effective equivalent steel section calculated assuming that concrete in tension is un-cracked

1.5.2.12 Cracked flexural stiffness
the stiffness E_aI_2 of a cross-section of a composite member where I_2 is the second moment of area of the effective equivalent steel section calculated neglecting concrete in tension but including reinforcement

1.5.2.13 Prestress
the process of applying compressive stresses to the concrete part of a composite member, achieved by tendons or by controlled imposed deformations

1.6 Symbols

For the purpose of this Standard the following symbols apply.

Latin upper case letters

A	Cross-sectional area of the effective composite section neglecting concrete in tension
A_a	Cross-sectional area of the structural steel section
A_b	Cross-sectional area of bottom transverse reinforcement
A_{bh}	Cross-sectional area of bottom transverse reinforcement in a haunch
A_c	Cross-sectional area of concrete
A_{ct}	Cross-sectional area of the tensile zone of the concrete
A_{fc}	Cross-sectional area of the compression flange
A_p	Cross-sectional area of profiled steel sheeting

2.2 Principles of limit states design

(1)P For composite structures, relevant stages in the sequence of construction shall be considered.

2.3 Basic variables

2.3.1 Actions and environmental influences

(1) Actions to be used in design may be obtained from the relevant parts of EN 1991.

(2)P In verification for steel sheeting as shuttering, account shall be taken of the ponding effect (increased depth of concrete due to the deflection of the sheeting).

2.3.2 Material and product properties

(1) Unless otherwise given by Eurocode 4, actions caused by time-dependent behaviour of concrete should be obtained from EN 1992-1-1.

2.3.3 Classification of actions

(1)P The effects of shrinkage and creep of concrete and non-uniform changes of temperature result in internal forces in cross sections, and curvatures and longitudinal strains in members; the effects that occur in statically determinate structures, and in statically indeterminate structures when compatibility of the deformations is not considered, shall be classified as primary effects.

(2)P In statically indeterminate structures the primary effects of shrinkage, creep and temperature are associated with additional action effects, such that the total effects are compatible; these shall be classified as secondary effects and shall be considered as indirect actions.

2.4 Verification by the partial factor method

2.4.1 Design values

2.4.1.1 Design values of actions

(1) For pre-stress by controlled imposed deformations, e.g. by jacking at supports, the partial safety factor γ_P should be specified for ultimate limit states, taking into account favourable and unfavourable effects.

Note: Values for γ_P may be given in the National Annex. The recommended value for both favourable and unfavourable effects is 1,0.

2.4.1.2 Design values of material or product properties

(1)P Unless an upper estimate of strength is required, partial factors shall be applied to lower characteristic or nominal strengths.

(2)P For concrete, a partial factor γ_C shall be applied. The design compressive strength shall be given by:

$$f_{cd} = f_{ck} / \gamma_C \qquad (2.1)$$

where the characteristic value f_{ck} shall be obtained by reference to EN 1992-1-1, 3.1 for normal concrete and to EN 1992-1-1, 11.3 for lightweight concrete.

Note: The value for γ_C is that used in EN 1992-1-1.

(3)P For steel reinforcement, a partial factor γ_S shall be applied.

Note: The value for γ_S is that used in EN 1992-1-1.

(4)P For structural steel, steel sheeting and steel connecting devices, partial factors γ_M shall be applied. Unless otherwise stated, the partial factor for structural steel shall be taken as γ_{M0}.

Note: Values for γ_M are those given in EN 1993.

(5)P For shear connection, a partial factor γ_V shall be applied.

Note: The value for γ_V may be given in the National Annex. The recommended value for γ_V is 1,25.

(6)P For longitudinal shear in composite slabs for buildings, a partial factor γ_{VS} shall be applied.

Note: The value for γ_{VS} may be given in the National Annex. The recommended value for γ_{VS} is 1,25.

(7)P For fatigue verification of headed studs in buildings, partial factors γ_{Mf} and $\gamma_{Mf,s}$ shall be applied.

Note: The value for γ_{Mf} is that used the relevant Parts of EN 1993. The value for $\gamma_{Mf,s}$. may be given in the National Annex. The recommended value for $\gamma_{Mf,s}$ is 1,0.

2.4.1.3 Design values of geometrical data

(1) Geometrical data for cross-sections and systems may be taken from product standards hEN or drawings for the execution and treated as nominal values.

2.4.1.4 Design resistances

(1)P For composite structures, design resistances shall be determined in accordance with EN 1990, expression (6.6a) or expression (6.6c).

2.4.2 Combination of actions

(1) The general formats for combinations of actions are given in EN 1990, Section 6.

Note: For buildings, the combination rules may be given in the National Annex to Annex A of EN 1990.

2.4.3 Verification of static equilibrium (EQU)

(1) The reliability format for the verification of static equilibrium for buildings, as described in EN 1990, Table A1.2(A), also applies to design situations equivalent to (EQU), e.g. for the design of hold down anchors or the verification of uplift of bearings of continuous beams.

Section 3 Materials

3.1 Concrete

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

(2) This Part of EN 1994 does not cover the design of composite structures with concrete strength classes lower than C20/25 and LC20/22 and higher than C60/75 and LC60/66.

(3) Shrinkage of concrete should be determined taking account of the ambient humidity, the dimensions of the element and the composition of the concrete.

(4) Where composite action is taken into account in buildings, the effects of autogenous shrinkage may be neglected in the determination of stresses and deflections.

Note: Experience shows that the values of shrinkage strain given in EN 1992-1-1 can give overestimates of the effects of shrinkage in composite structures. Values for shrinkage of concrete may be given in the National Annex. Recommended values for composite structures for buildings are given in Annex C.

3.2 Reinforcing steel

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

(2) For composite structures, the design value of the modulus of elasticity E_s may be taken as equal to the value for structural steel given in EN 1993-1-1, 3.2.6.

3.3 Structural steel

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

(2) The rules in this Part of EN 1994 apply to structural steel of nominal yield strength not more than 460 N/mm^2.

3.4 Connecting devices

3.4.1 General

(1) Reference should be made to EN 1993-1-8 for requirements for fasteners and welding consumables.

3.4.2 Headed stud shear connectors

(1) Reference should be made to EN 13918.

3.5 Profiled steel sheeting for composite slabs in buildings

(1) Properties should be obtained by reference to EN 1993-1-3, 3.1 and 3.2.

(2) The rules in this Part of EN 1994 apply to the design of composite slabs with profiled steel sheets manufactured from steel in accordance with EN 10025, cold formed steel sheet in accordance with EN 10149-2 or EN 10149-3 or galvanised steel sheet in accordance with EN 10147.

Note: The minimum value for the nominal thickness t of steel sheets may be given in the National Annex. The recommended value is 0,70 mm.

Section 4 Durability

4.1 General

(1) The relevant provisions given in EN 1990, EN 1992 and EN 1993 should be followed.

(2) Detailing of the shear connection should be in accordance with 6.6.5.

5.3.2.2 Global imperfections

(1) The effects of imperfections should be allowed for in accordance with EN 1993-1-1, 5.3.2.

5.3.2.3 Member imperfections

(1) Design values of equivalent initial bow imperfection for composite columns and composite compression members should be taken from Table 6.5.

(2) For laterally unrestrained composite beams the effects of imperfections are incorporated within the formulae given for buckling resistance moment, see 6.4.

(3) For steel members the effects of imperfections are incorporated within the formulae given for buckling resistance, see EN 1993-1-1, 6.3.

5.4 Calculation of action effects

5.4.1 Methods of global analysis

5.4.1.1 General

(1) Action effects may be calculated by elastic global analysis, even where the resistance of a cross-section is based on its plastic or non-linear resistance.

(2) Elastic global analysis should be used for serviceability limit states, with appropriate corrections for non-linear effects such as cracking of concrete.

(3) Elastic global analysis should be used for verifications of the limit state of fatigue.

(4)P The effects of shear lag and of local buckling shall be taken into account if these significantly influence the global analysis.

(5) The effects of local buckling of steel elements on the choice of method of analysis may be taken into account by classifying cross-sections, see 5.5.

(6) The effects of local buckling of steel elements on stiffness may be ignored in normal composite sections. For cross-sections of Class 4, see EN 1993-1-5, 2.2.

(7) The effects on the global analysis of slip in bolt holes and similar deformations of connecting devices should be considered.

(8) Unless non-linear analysis is used, the effects of slip and separation on calculation of internal forces and moments may be neglected at interfaces between steel and concrete where shear connection is provided in accordance with 6.6.

5.4.1.2 Effective width of flanges for shear lag

(1)P Allowance shall be made for the flexibility of steel or concrete flanges affected by shear in their plane (shear lag) either by means of rigorous analysis, or by using an effective width of flange.

(2) The effects of shear lag in steel plate elements should be considered in accordance with EN 1993-1-1, 5.2.1(5).

(3) The effective width of concrete flanges should be determined in accordance with the following provisions.

(4) When elastic global analysis is used, a constant effective width may be assumed over the whole of each span. This value may be taken as the value $b_{eff,1}$ at mid-span for a span supported at both ends, or the value $b_{eff,2}$ at the support for a cantilever.

(5) At mid-span or an internal support, the total effective width b_{eff} , see Figure 5.1, may be determined as:

$$b_{eff} = b_0 + \sum b_{ei} \tag{5.3}$$

where:

b_0 is the distance between the centres of the outstand shear connectors;

b_{ei} is the value of the effective width of the concrete flange on each side of the web and taken as $L_e / 8$ but not greater than the geometric width b_i . The value b_i should be taken as the distance from the outstand shear connector to a point mid-way between adjacent webs, measured at mid-depth of the concrete flange, except that at a free edge b_i is the distance to the free edge. The length L_e should be taken as the approximate distance between points of zero bending moment. For typical continuous composite beams, where a moment envelope from various load arrangements governs the design, and for cantilevers, L_e may be assumed to be as shown in Figure 5.1.

(6) The effective width at an end support may be determined as:

$$b_{eff} = b_0 + \sum \beta_i \, b_{ei} \tag{5.4}$$

with:

$$\beta_i = (0{,}55 + 0{,}025 \, L_e / b_{ei}) \leq 1{,}0 \tag{5.5}$$

where:

b_{ei} is the effective width, see (5), of the end span at mid-span and L_e is the equivalent span of the end span according to Figure 5.1.

(7) The distribution of the effective width between supports and midspan regions may be assumed to be as shown in Figure 5.1.

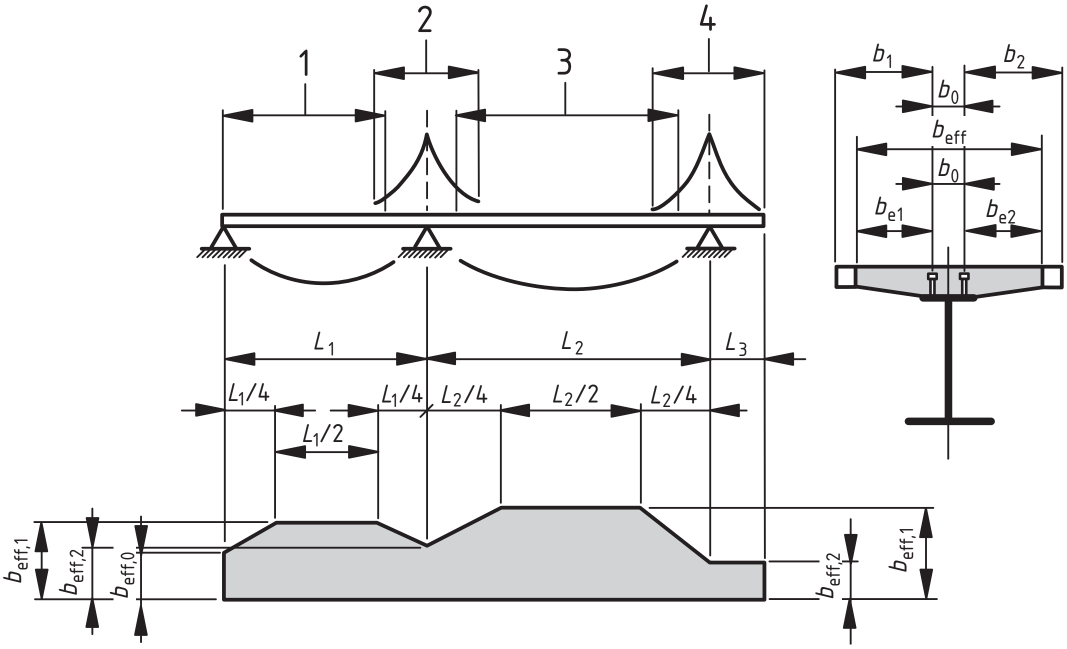
(8) Where in buildings the bending moment distribution is influenced by the resistance or the rotational stiffness of a joint, this should be considered in the determination of the length L_e.

(9) For analysis of building structures, b_0 may be taken as zero and b_i measured from the centre of the web.

5.4.2 Linear elastic analysis

5.4.2.1 General

(1) Allowance should be made for the effects of cracking of concrete, creep and shrinkage of concrete, sequence of construction and pre-stressing.



Key

1 $L_e = 0{,}85L_1$ for $b_{eff,1}$

2 $L_e = 0{,}25(L_1 + L_2)$ for $b_{eff,2}$

3 $L_e = 0{,}70L_2$ for $b_{eff,1}$

4 $L_e = 2L_3$ for $b_{eff,2}$

Figure 5.1 : Equivalent spans, for effective width of concrete flange

5.4.2.2 Creep and shrinkage

(1)P Appropriate allowance shall be made for the effects of creep and shrinkage of concrete.

(2) Except for members with both flanges composite, the effects of creep may be taken into account by using modular ratios n_L for the concrete. The modular ratios depending on the type of loading (subscript L) are given by:

$$n_L = n_0 \left(1 + \psi_L \varphi_t\right) \quad (5.6)$$

where:

n_0 is the modular ratio E_a / E_{cm} for short-term loading;

E_{cm} is the secant modulus of elasticity of the concrete for short-term loading according to EN 1992-1-1, Table 3.1 or Table 11.3.1;

φ_t is the creep coefficient $\varphi(t,t_0)$ according to EN 1992-1-1, 3.1.4 or 11.3.3, depending on the age (t) of concrete at the moment considered and the age (t_0) at loading,

ψ_L is the creep multiplier depending on the type of loading, which be taken as 1,1 for permanent loads, 0,55 for primary and secondary effects of shrinkage and 1,5 for pre-stressing by imposed deformations.

a) the grade of structural steel does not exceed S355,
b) the contribution of any reinforced concrete encasement in compression is neglected when calculating the design resistance moment,
c) all effective cross-sections at plastic hinge locations are in Class 1; and all other effective cross-sections are in Class 1 or Class 2,
d) each beam-to-column joint has been shown to have sufficient design rotation capacity, or to have a design resistance moment at least 1,2 times the design plastic resistance moment of the connected beam,
e) adjacent spans do not differ in length by more than 50% of the shorter span,
f) end spans do not exceed 115% of the length of the adjacent span,
g) in any span in which more than half of the total design load for that span is concentrated within a length of one-fifth of the span, then at any hinge location where the concrete slab is in compression, not more than 15% of the overall depth of the member should be in compression; this does not apply where it can be shown that the hinge will be the last to form in that span and
h) the steel compression flange at a plastic hinge location is laterally restrained.

(5) Unless verified otherwise, it should be assumed that composite columns do not have rotation capacity.

(6) Where the cross-section of a steel member varies along its length, EN 1993-1-1, 5.6(3) is applicable.

(7) Where restraint is required by (3)(c) or 4(h), it should be located within a distance along the member from the calculated hinge location that does not exceed half the depth of the steel section.

5.5 Classification of cross-sections

5.5.1 General

(1)P The classification system defined in EN 1993-1-1, 5.5.2 applies to cross-sections of composite beams.

(2) A composite section should be classified according to the least favourable class of its steel elements in compression. The class of a composite section normally depends on the direction of the bending moment at that section.

(3) A steel compression element restrained by attaching it to a reinforced concrete element may be placed in a more favourable class, provided that the resulting improvement in performance has been established.

(4) For classification, the plastic stress distribution should be used except at the boundary between Classes 3 and 4, where the elastic stress distribution should be used taking into account sequence of construction and the effects of creep and shrinkage. For classification, design values of strengths of materials should be used. Concrete in tension should be neglected. The distribution of the stresses should be determined for the gross cross-section of the steel web and the effective flanges.

(5) For cross-sections in Class 1 and 2 with bars in tension, reinforcement used within the effective width should have a ductility Class B or C, see EN 1992-1-1, Table C.1. Additionally for a section

whose resistance moment is determined by 6.2.1.2, 6.2.1.3 or 6.2.1.4, a minimum area of reinforcement A_s within the effective width of the concrete flange should be provided to satisfy the following condition:

$$A_s \geq \rho_s A_c \tag{5.7}$$

with

$$\rho_s = \delta \frac{f_y}{235} \frac{f_{ctm}}{f_{sk}} \sqrt{k_c} \tag{5.8}$$

where:

A_c is the effective area of the concrete flange;

f_y is the nominal value of the yield strength of the structural steel in N/mm^2;

f_{sk} is the characteristic yield strength of the reinforcement;

f_{ctm} is the mean tensile strength of the concrete, see EN1992-1-1, Table 3.1 or Table 11.3.1;

k_c is a coefficient given in 7.4.2;

δ is equal to 1,0 for Class 2 cross-sections, and equal to 1,1 for Class 1 cross-sections at which plastic hinge rotation is required.

(6) Welded mesh should not be included in the effective section unless it has been shown to have sufficient ductility, when built into a concrete slab, to ensure that it will not fracture.

(7) In global analysis for stages in construction, account should be taken of the class of the steel section at the stage considered.

5.5.2 Classification of composite sections without concrete encasement

(1) A steel compression flange that is restrained from buckling by effective attachment to a concrete flange by shear connectors may be assumed to be in Class 1 if the spacing of connectors is in accordance with 6.6.5.5.

(2) The classification of other steel flanges and webs in compression in composite beams without concrete encasement should be in accordance with EN 1993-1-1, Table 5.2. An element that fails to satisfy the limits for Class 3 should be taken as Class 4.

(3) Cross-sections with webs in Class 3 and flanges in Classes 1 or 2 may be treated as an effective cross-section in Class 2 with an effective web in accordance with EN1993-1-1, 6.2.2.4.

5.5.3 Classification of composite sections for buildings with concrete encasement

(1) A steel outstand flange of a composite section with concrete encasement in accordance with (2) below may be classified in accordance with Table 5.2.

(2) For a web of a concrete encased section, the concrete that encases it should be reinforced, mechanically connected to the steel section, and capable of preventing buckling of the web and of any part of the compression flange towards the web. It may be assumed that the above requirements are satisfied if:

a) the concrete that encases a web is reinforced by longitudinal bars and stirrups, and/or welded mesh,

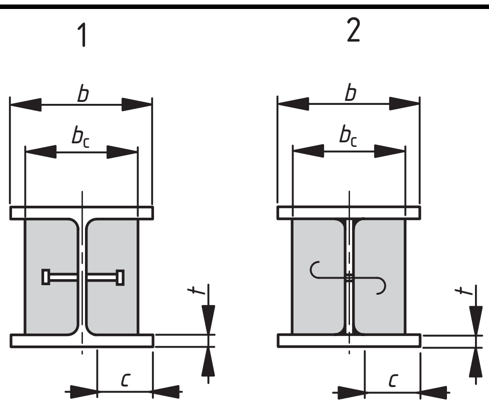
b) the requirements for the ratio b_c / b given in Table 5.2 are fulfilled,

c) the concrete between the flanges is fixed to the web in accordance with Figure 6.10 by welding the stirrups to the web or by means of bars of at least 6 mm diameter through holes and/or studs with a diameter greater than 10 mm welded to the web and

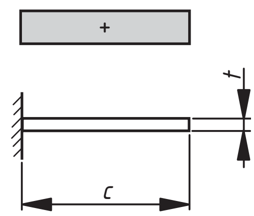
d) the longitudinal spacing of the studs on each side of the web or of the bars through holes is not greater than 400 mm. The distance between the inner face of each flange and the nearest row of fixings to the web is not greater than 200 mm. For steel sections with a maximum depth of not less than 400 mm and two or more rows of fixings, a staggered arrangement of the studs and/or bars through holes may be used.

(3) A steel web in Class 3 encased in concrete in accordance with (2) above may be represented by an effective web of the same cross-section in Class 2.

Table 5.2 : Classification of steel flanges in compression for partially-encased sections



$$0{,}8 \le \frac{b_c}{b} \le 1{,}0$$



Stress distribution (compression positive)

Class	Type	Limit
1	(1) rolled or (2) welded	$c/t \le 9\varepsilon$
2		$c/t \le 14\varepsilon$
3		$c/t \le 20\varepsilon$

Section 6 Ultimate limit states

6.1 Beams

6.1.1 Beams for buildings

(1)P Composite beams are defined in 1.5.2. Typical types of cross-section are shown in Figure 6.1 with either a solid slab or a composite slab. Partially–encased beams are those in which the web of the steel section is encased by reinforced concrete and shear connection is provided between the concrete and the steel components.

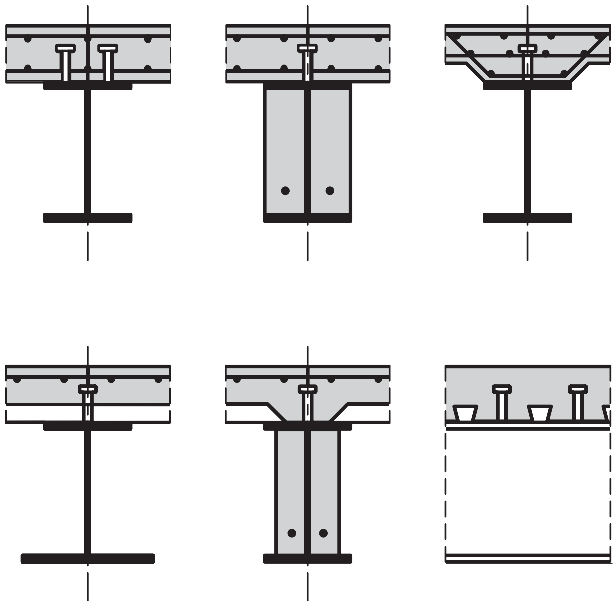


Figure 6.1 : Typical cross-sections of composite beams

(2) Design resistances of composite cross-sections in bending or/and vertical shear should be determined in accordance with 6.2 for composite beams with steel sections and 6.3 for partially-encased composite beams.

(3)P Composite beams shall be checked for:

- resistance of critical cross-sections (6.2 and 6.3);
- resistance to lateral-torsional buckling (6.4);
- resistance to shear buckling (6.2.2.3) and transverse forces on webs (6.5);
- resistance to longitudinal shear (6.6).

(4)P Critical cross-sections include:

- sections of maximum bending moment;
- supports;
- sections subjected to concentrated loads or reactions;
- places where a sudden change of cross-section occurs, other than a change due to cracking of concrete.

(5) A cross-section with a sudden change should be considered as a critical cross-section when the ratio of the greater to the lesser resistance moment is greater than 1,2.

(6) For checking resistance to longitudinal shear, a critical length consists of a length of the interface between two critical cross-sections. For this purpose critical cross-sections also include:

- free ends of cantilevers;
- in tapering members, sections so chosen that the ratio of the greater to the lesser plastic resistance moments (under flexural bending of the same direction) for any pair of adjacent cross-sections does not exceed 1,5.

(7)P The concepts "full shear connection" and "partial shear connection" are applicable only to beams in which plastic theory is used for calculating bending resistances of critical cross-sections.

A span of a beam, or a cantilever, has full shear connection when increase in the number of shear connectors would not increase the design bending resistance of the member. Otherwise, the shear connection is partial.

Note: Limits to the use of partial shear connection are given in 6.6.1.2.

6.1.2 Effective width for verification of cross-sections

(1) The effective width of the concrete flange for verification of cross-sections should be determined in accordance with 5.4.1.2 taking into account the distribution of effective width between supports and mid-span regions.

(2) As a simplification for buildings, a constant effective width may be assumed over the whole region in sagging bending of each span. This value may be taken as the value $b_{eff,1}$ at mid-span. The same assumption applies over the whole region in hogging bending on both sides of an intermediate support. This value may be taken as the value $b_{eff,2}$ at the relevant support.

6.2 Resistances of cross-sections of beams

6.2.1 Bending resistance

6.2.1.1 General

(1)P The design bending resistance shall be determined by rigid-plastic theory only where the effective composite cross-section is in Class 1 or Class 2 and where pre-stressing by tendons is not used.

(2) Elastic analysis and non-linear theory for bending resistance may be applied to cross-sections of any class.

(3) For elastic analysis and non-linear theory it may be assumed that the composite cross-section remains plane if the shear connection and the transverse reinforcement are designed in accordance with 6.6, considering appropriate distributions of design longitudinal shear force.

(4)P The tensile strength of concrete shall be neglected.

(5) Where the steel section of a composite member is curved in plan, the effects of curvature should be taken into account.

6.2.1.2 Plastic resistance moment $M_{pl,Rd}$ of a composite cross-section

(1) The following assumptions should be made in the calculation of $M_{pl,Rd}$:

a) there is full interaction between structural steel, reinforcement, and concrete;

b) the effective area of the structural steel member is stressed to its design yield strength f_{yd} in tension or compression;

c) the effective areas of longitudinal reinforcement in tension and in compression are stressed to their design yield strength f_{sd} in tension or compression. Alternatively, reinforcement in compression in a concrete slab may be neglected;

d) the effective area of concrete in compression resists a stress of $0{,}85 f_{cd}$, constant over the whole depth between the plastic neutral axis and the most compressed fibre of the concrete, where f_{cd} is the design cylinder compressive strength of concrete.

Typical plastic stress distributions are shown in Figure 6.2.

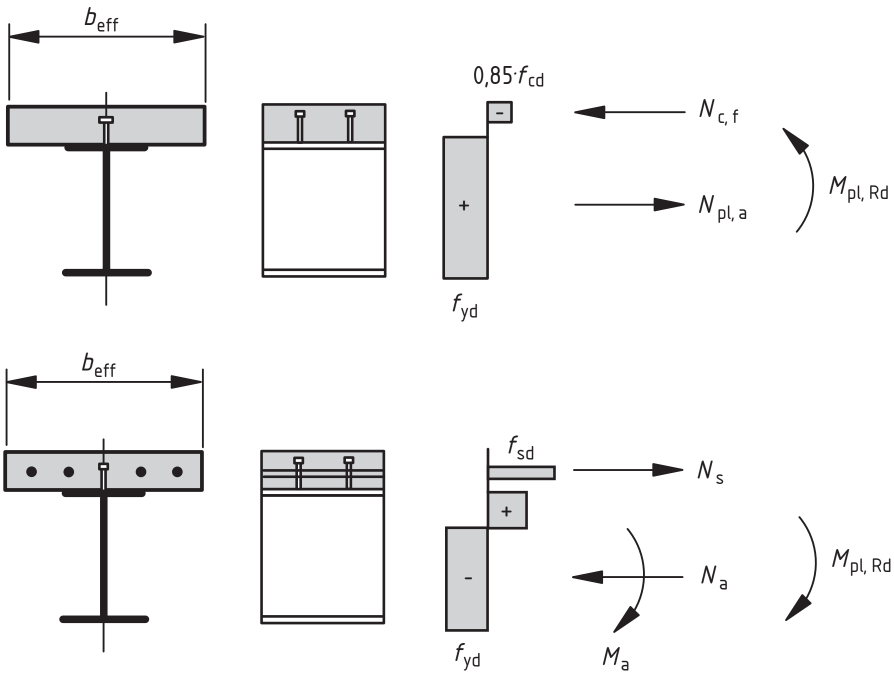


Figure 6.2 : Examples of plastic stress distributions for a composite beam with a solid slab and full shear connection in sagging and hogging bending

(2) For composite cross-sections with structural steel grade S420 or S460, where the distance x_{pl} between the plastic neutral axis and the extreme fibre of the concrete slab in compression exceeds 15% of the overall depth h of the member, the design resistance moment M_{Rd} should be taken as $\beta M_{pl,Rd}$ where β is the reduction factor given in Figure 6.3. For values of x_{pl}/h greater than 0,4 the resistance to bending should be determined from 6.2.1.4 or 6.2.1.5.

(3) Where plastic theory is used and reinforcement is in tension, that reinforcement should be in accordance with 5.5.1(5).

(4)P For buildings, profiled steel sheeting in compression shall be neglected.

(5) For buildings, any profiled steel sheeting in tension included within the effective section should be assumed to be stressed to its design yield strength $f_{yp,d}$.

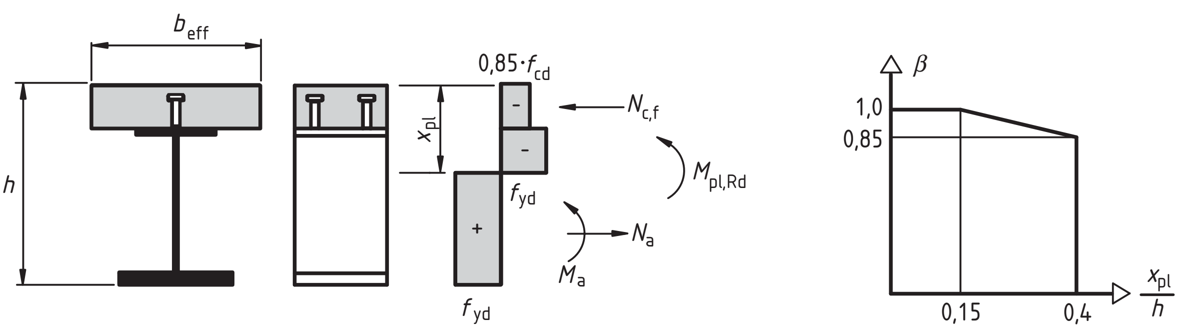


Figure 6.3 : Reduction factor β for $M_{pl,Rd}$

6.2.1.3 Plastic resistance moment of sections with partial shear connection in buildings

(1) In regions of sagging bending, partial shear connection in accordance with 6.6.1 and 6.6.2.2 may be used in composite beams for buildings.

(2) Unless otherwise verified, the plastic resistance moment in hogging bending should be determined in accordance with 6.2.1.2 and appropriate shear connection should be provided to ensure yielding of reinforcement in tension.

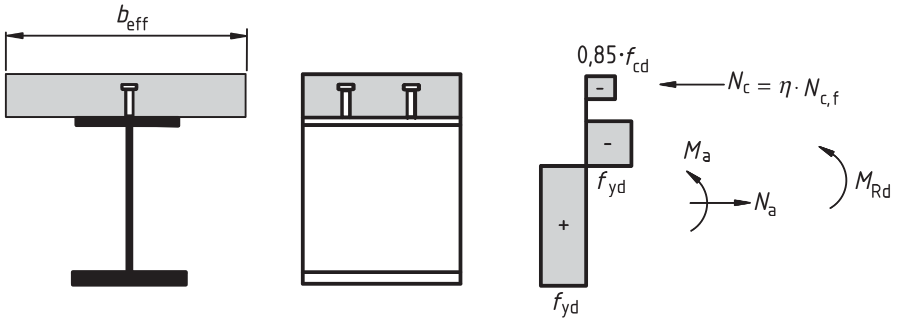
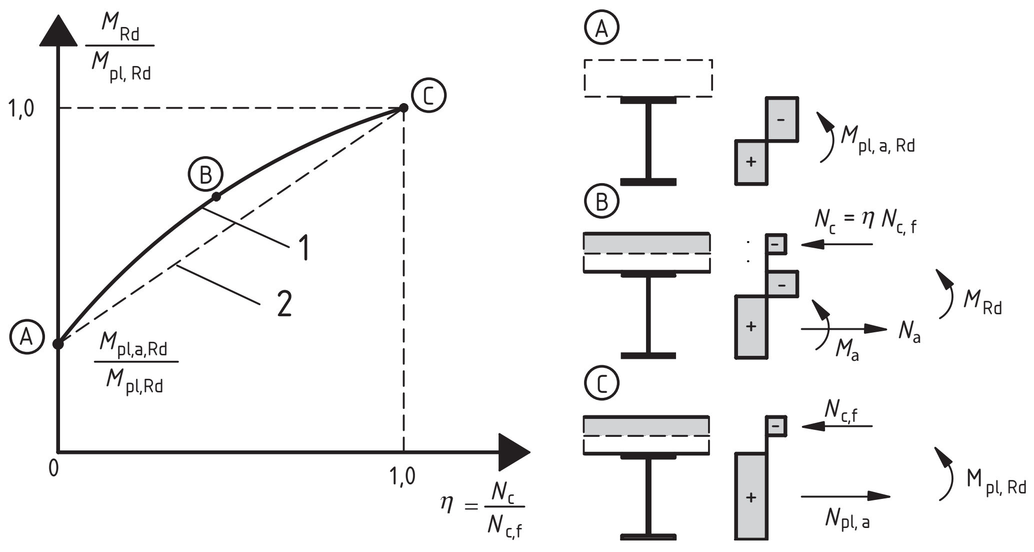


Figure 6.4 : Plastic stress distribution under sagging bending for partial shear connection

(3) Where ductile shear connectors are used, the resistance moment of the critical cross-section of the beam M_{Rd} may be calculated by means of rigid plastic theory in accordance with 6.2.1.2, except that a reduced value of the compressive force in the concrete flange N_c should be used in place of the force N_{cf} given by 6.2.1.2(1)(d). The ratio $\eta = N_c/N_{c,f}$ is the degree of shear connection. The location of the plastic neutral axis in the slab should be determined by the new force N_c , see Figure 6.4. There is a second plastic neutral axis within the steel section, which should be used for the classification of the web.



Key

1 plastic theory

2 simplified method

Figure 6.5 : Relation between M_{Rd} and N_c (for ductile shear connectors)

(4) The relation between M_{Rd} and N_c in (3) is qualitatively given by the convex curve ABC in Figure 6.5 where $M_{pl,a,Rd}$ and $M_{pl,Rd}$ are the design plastic resistances to sagging bending of the structural steel section alone, and of the composite section with full shear connection, respectively.

(5) For the method given in (3), a conservative value of M_{Rd} may be determined by the straight line AC in Figure 6.5:

$$M_{Rd} = M_{pl,a,Rd} + \left(M_{pl,Rd} - M_{pl,a,Rd}\right)\frac{N_c}{N_{cf}} \quad (6.1)$$

6.2.1.4 Non-linear resistance to bending

(1)P Where the bending resistance of a composite cross-section is determined by non-linear theory, the stress-strain relationships of the materials shall be taken into account.

(2) It should be assumed that the composite cross-section remains plane and that the strain in bonded reinforcement, whether in tension or compression, is the same as the mean strain in the surrounding concrete.

(3) The stresses in the concrete in compression should be derived from the stress-strain curves given in EN 1992-1-1, 3.1.7.

(4) The stresses in the reinforcement should be derived from the bi-linear diagrams given in EN 1992-1-1, 3.2.7.

(5) The stresses in structural steel in compression or tension should be derived from the bi-linear diagram given in EN 1993-1-1, 5.4.3(4) and should take account of the effects of the method of construction (e.g. propped or un-propped).

(6) For Class 1 and Class 2 composite cross-sections with the concrete flange in compression, the non-linear resistance to bending M_{Rd} may be determined as a function of the compressive force in the concrete N_c using the simplified expressions (6.2) and (6.3), as shown in Figure 6.6:

$$M_{Rd} = M_{a,Ed} + (M_{el,Rd} - M_{a,Ed})\frac{N_c}{N_{c,el}} \qquad \text{for } N_c \le N_{c,el} \quad (6.2)$$

$$M_{Rd} = M_{el,Rd} + (M_{pl,Rd} - M_{el,Rd})\frac{N_c - N_{c,el}}{N_{c,f} - N_{c,el}} \qquad \text{for } N_{c,el} \le N_c \le N_{c,f} \quad (6.3)$$

with:

$$M_{el,Rd} = M_{a,Ed} + k\, M_{c,Ed} \quad (6.4)$$

where:

$M_{a,Ed}$ is the design bending moment applied to the structural steel section before composite behaviour;

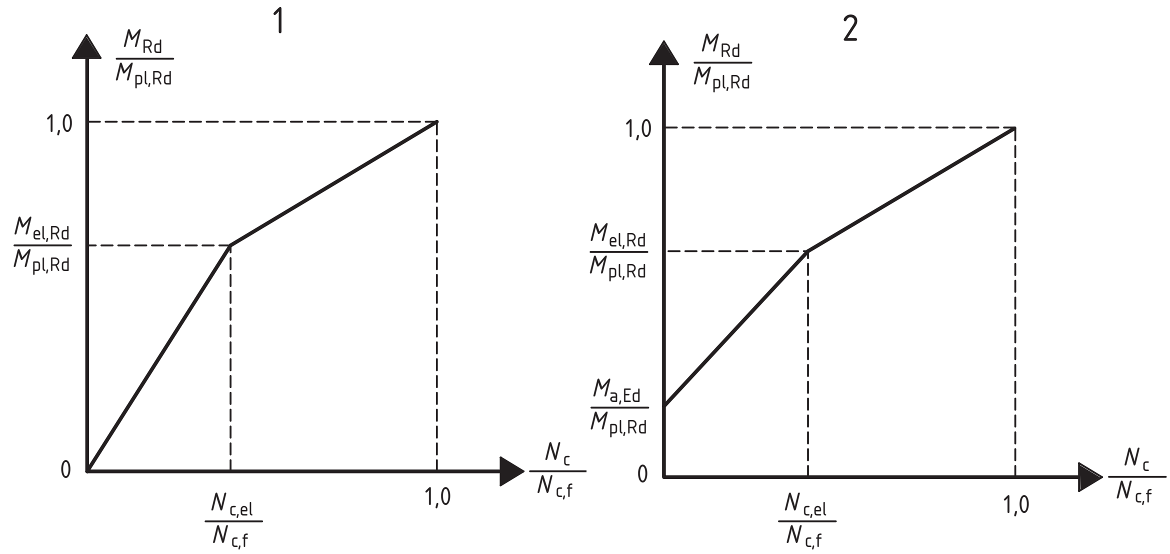
$M_{c,Ed}$ is the part of the design bending moment applied to the composite section;

k is the lowest factor such that a stress limit in 6.2.1.5(2) is reached; where un-propped construction is used, the sequence of construction should be taken into account;

$N_{c,el}$ is the compressive force in the concrete flange corresponding to moment $M_{el,Rd}$.

For cross sections where 6.2.1.2 (2) applies, in expression (6.3) and in Figure 6.6 instead of $M_{pl,Rd}$ the reduced value $\beta M_{pl,Rd}$ should be used.

(7) For buildings, the determination of $M_{el.Rd}$ may be simplified using 5.4.2.2(11).



Key

1 propped construction

2 unpropped construction

Figure 6.6 : Simplified relationship between M_{Rd} and N_c for sections with the concrete slab in compression

6.2.1.5 Elastic resistance to bending

(1) Stresses should be calculated by elastic theory, using an effective width of the concrete flange in accordance with 6.1.2. For cross-sections in Class 4, the effective structural steel section should be determined in accordance with EN 1993-1-5, 4.3.

(2) In the calculation of the elastic resistance to bending based on the effective cross-section, the limiting stresses should be taken as:

- f_{cd} in concrete in compression;
- f_{yd} in structural steel in tension or compression;
- f_{sd} in reinforcement in tension or compression. Alternatively, reinforcement in compression in a concrete slab may be neglected.

(3)P Stresses due to actions on the structural steelwork alone shall be added to stresses due to actions on the composite member.

(4) Unless a more precise method is used, the effect of creep should be taken into account by use of a modular ratio according to 5.4.2.2.

(5) In cross-sections with concrete in tension and assumed to be cracked, the stresses due to primary (isostatic) effects of shrinkage may be neglected.

6.2.2 Resistance to vertical shear

6.2.2.1 Scope

(1) Clause 6.2.2 applies to composite beams with a rolled or welded structural steel section with a solid web, which may be stiffened.

6.2.2.2 Plastic resistance to vertical shear

(1) The resistance to vertical shear $V_{pl,Rd}$ should be taken as the resistance of the structural steel section $V_{pl,a,Rd}$ unless the value for a contribution from the reinforced concrete part of the beam has been established.

(2) The design plastic shear resistance $V_{pl,a,Rd}$ of the structural steel section should be determined in accordance with EN 1993-1-1, 6.2.6.

6.2.2.3 Shear buckling resistance

(1) The shear buckling resistance $V_{b,Rd}$ of an uncased steel web should be determined in accordance with EN 1993-1-5, 5.

(2) No account should be taken of a contribution from the concrete slab, unless a more precise method than the one of EN 1993-1-5, 5 is used and unless the shear connection is designed for the relevant vertical force.

6.2.2.4 Bending and vertical shear

(1) Where the vertical shear force V_{Ed} exceeds half the shear resistance V_{Rd} given by $V_{pl,Rd}$ in 6.2.2.2 or $V_{b,Rd}$ in 6.2.2.3, whichever is the smaller, allowance should be made for its effect on the resistance moment.

(2) For cross-sections in Class 1 or 2, the influence of the vertical shear on the resistance to bending may be taken into account by a reduced design steel strength $(1 - \rho) f_{yd}$ in the shear area as shown in Figure 6.7 where:

$$\rho = (2V_{Ed} / V_{Rd} - 1)^2 \qquad (6.5)$$

and V_{Rd} is the appropriate resistance to vertical shear, determined in accordance with 6.2.2.2 or 6.2.2.3.

(3) For cross-sections in Class 3 and 4, EN 1993-1-5, 7.1 is applicable using the calculated stresses of the composite section.

6.5 Transverse forces on webs

6.5.1 General

(1) The rules given in EN 1993-1-5, 6 to determine the design resistance of an unstiffened or stiffened web to transverse forces applied through a flange are applicable to the non-composite steel flange of a composite beam, and to the adjacent part of the web.

(2) If the transverse force acts in combination with bending and axial force, the resistance should be verified according to EN 1993-1-5, 7.2.

(3) For buildings, at an internal support of a beam designed using an effective web in Class 2 in accordance with 5.5.2(3), transverse stiffening should be provided unless it has been verified that the un-stiffened web has sufficient resistance to crippling and buckling.

6.5.2 Flange-induced buckling of webs

(1) EN 1993-1-5, 8 is applicable provided that area A_{fc} is taken equal to the area of the non-composite steel flange or the transformed area of the composite steel flange taking into account the modular ratio for short-term loading, whichever is the smaller.

6.6 Shear connection

6.6.1 General

6.6.1.1 Basis of design

(1) Clause 6.6 is applicable to composite beams and, as appropriate, to other types of composite member.

(2)P Shear connection and transverse reinforcement shall be provided to transmit the longitudinal shear force between the concrete and the structural steel element, ignoring the effect of natural bond between the two.

(3)P Shear connectors shall have sufficient deformation capacity to justify any inelastic redistribution of shear assumed in design.

(4)P Ductile connectors are those with sufficient deformation capacity to justify the assumption of ideal plastic behaviour of the shear connection in the structure considered.

(5) A connector may be taken as ductile if the characteristic slip capacity δ_{uk} is at least 6mm.

Note: An evaluation of δ_{uk} is given in Annex B.

(6)P Where two or more different types of shear connection are used within the same span of a beam, account shall be taken of any significant difference in their load-slip properties.

(7)P Shear connectors shall be capable of preventing separation of the concrete element from the steel element, except where separation is prevented by other means.

(8) To prevent separation of the slab, shear connectors should be designed to resist a nominal ultimate tensile force, perpendicular to the plane of the steel flange, of at least 0,1 times the design ultimate shear resistance of the connectors. If necessary they should be supplemented by anchoring devices.

(9) Headed stud shear connectors in accordance with 6.6.5.7 may be assumed to provide sufficient resistance to uplift, unless the shear connection is subjected to direct tension.

(10)P Longitudinal shear failure and splitting of the concrete slab due to concentrated forces applied by the connectors shall be prevented.

(11) If the detailing of the shear connection is in accordance with the appropriate provisions of 6.6.5 and the transverse reinforcement is in accordance with 6.6.6, compliance with 6.6.1.1(10) may be assumed.

(12) Where a method of interconnection, other than the shear connectors included in 6.6, is used to transfer shear between a steel element and a concrete element, the behaviour assumed in design should be based on tests and supported by a conceptual model. The design of the composite member should conform to the design of a similar member employing shear connectors included in 6.6, in so far as practicable.

(13) For buildings, the number of connectors should be at least equal to the total design shear force for the ultimate limit state, determined according to 6.6.2, divided by the design resistance of a single connector P_{Rd}. For stud connectors the design resistance should be determined according to 6.6.3 or 6.6.4, as appropriate.

(14)P If all cross-sections are in Class 1 or Class 2, in buildings partial shear connection may be used for beams. The number of connectors shall then be determined by a partial connection theory taking into account the deformation capacity of the shear connectors.

6.6.1.2 Limitation on the use of partial shear connection in beams for buildings

(1) Headed studs with an overall length after welding not less than 4 times the diameter, and with a shank of nominal diameter not less than 16 mm and not greater than 25 mm, may be considered as ductile within the following limits for the degree of shear connection, which is defined by the ratio $\eta = n / n_f$:

For steel sections with equal flanges:

$$L_e \leq 25: \qquad \eta \geq 1 - \left(\frac{355}{f_y}\right)(0{,}75 - 0{,}03\, L_e), \qquad \eta \geq 0{,}4 \tag{6.12}$$

$$L_e > 25: \qquad \eta \geq 1 \tag{6.13}$$

For steel sections having a bottom flange with an area equal to three times the area of the top flange:

$$L_e \leq 20: \qquad \eta \geq 1 - \left(\frac{355}{f_y}\right)(0{,}30 - 0{,}015\, L_e), \qquad \eta \geq 0{,}4 \tag{6.14}$$

$$L_e > 20: \qquad \eta \geq 1 \tag{6.15}$$

where:

L_e is the distance in sagging bending between points of zero bending moment in metres; for typical continuous beams, L_e may be assumed to be as shown in Figure 5.1;

n_f is the number of connectors for full shear connection determined for that length of beam in accordance with 6.6.1.1(13) and 6.6.2.2(2);

n is the number of shear connectors provided within that same length.

(2) For steel sections having a bottom flange with an area exceeding the area of the top flange but less than three times that area, the limit for η may be determined from expressions (6.12) – (6.15) by linear interpolation.

(3) Headed stud connectors may be considered as ductile over a wider range of spans than given in (1) above where:

(a) the studs have an overall length after welding not less than 76 mm, and a shank of nominal diameter of 19 mm,

(b) the steel section is a rolled or welded I or H with equal flanges,

(c) the concrete slab is composite with profiled steel sheeting that spans perpendicular to the beam and the concrete ribs are continuous across it,

(d) there is one stud per rib of sheeting, placed either centrally within the rib or alternately on the left side and on the right side of the trough throughout the length of the span,

(e) for the sheeting $b_0 / h_p \geq 2$ and $h_p \leq 60$ mm, where the notation is as in Figure 6.13 and

(f) the force N_c is calculated in accordance with the simplified method given in Figure 6.5.

Where these conditions are satisfied, the ratio η should satisfy:

$$L_e \leq 25: \qquad \eta \geq 1 - \left(\frac{355}{f_y}\right)(1{,}0 - 0{,}04\,L_e), \qquad \eta \geq 0{,}4 \tag{6.16}$$

$$L_e > 25: \qquad \eta \geq 1 \tag{6.17}$$

Note: The requirements in 6.6.1.2 are derived for uniform spacing of shear connectors.

6.6.1.3 Spacing of shear connectors in beams for buildings

(1)P The shear connectors shall be spaced along the beam so as to transmit longitudinal shear and to prevent separation between the concrete and the steel beam, considering an appropriate distribution of design longitudinal shear force.

(2) In cantilevers and hogging moment regions of continuous beams, tension reinforcement should be curtailed to suit the spacing of the shear connectors and should be adequately anchored.

(3) Ductile connectors may be spaced uniformly over a length between adjacent critical cross-sections as defined in 6.1.1 provided that:

– all critical sections in the span considered are in Class 1 or Class 2,

– η satisfies the limit given by 6.6.1.2 and

– the plastic resistance moment of the composite section does not exceed 2,5 times the plastic resistance moment of the steel member alone.

(4) If the plastic resistance moment exceeds 2,5 times the plastic resistance moment of the steel member alone, additional checks on the adequacy of the shear connection should be made at intermediate points approximately mid-way between adjacent critical cross-sections.

(5) The required number of shear connectors may be distributed between a point of maximum sagging bending moment and an adjacent support or point of maximum hogging moment, in accordance with the longitudinal shear calculated by elastic theory for the loading considered. Where this is done, no additional checks on the adequacy of the shear connection are required.

6.6.2 Longitudinal shear force in beams for buildings

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

(1) If non-linear or elastic theory is applied to cross-sections, the longitudinal shear force should be determined in a manner consistent with 6.2.1.4 or 6.2.1.5 respectively.

6.6.2.2 Beams in which plastic theory is used for resistance of cross sections

(1)P The total design longitudinal shear shall be determined in a manner consistent with the design bending resistance, taking account of the difference in the normal force in concrete or structural steel over a critical length.

(2) For full shear connection, reference should be made to 6.2.1.2, or 6.3.2, as appropriate.

(3) For partial shear connection, reference should be made to 6.2.1.3 or 6.3.2, as appropriate.

6.6.3 Headed stud connectors in solid slabs and concrete encasement

6.6.3.1 Design resistance

(1) The design shear resistance of a headed stud automatically welded in accordance with EN 14555 should be determined from:

$$P_{\mathrm{Rd}} = \frac{0{,}8\, f_{\mathrm{u}}\, \pi\, d^2 / 4}{\gamma_{\mathrm{V}}} \tag{6.18}$$

or:

$$P_{\mathrm{Rd}} = \frac{0{,}29\, \alpha d^2 \sqrt{f_{\mathrm{ck}} E_{\mathrm{cm}}}}{\gamma_{\mathrm{V}}} \tag{6.19}$$

whichever is smaller, with:

$$\alpha = 0{,}2\left(\frac{h_{\mathrm{sc}}}{d} + 1\right) \quad \text{for } 3 \le h_{\mathrm{sc}} / d \le 4 \tag{6.20}$$

$$\alpha = 1 \quad \text{for } h_{\mathrm{sc}} / d > 4 \tag{6.21}$$

where:

γ_{V} is the partial factor;

d is the diameter of the shank of the stud, 16 mm $\le d \le$ 25 mm;

f_{u} is the specified ultimate tensile strength of the material of the stud but not greater than 500 N/mm^2;

f_{ck} is the characteristic cylinder compressive strength of the concrete at the age considered, of density not less than 1750 kg/m^3;

h_{sc} is the overall nominal height of the stud.

Note: The value for γ_V may be given in the National Annex. The recommended value for γ_V is 1,25.

(2) The weld collars should comply with the requirements of EN 13918.

(3) Where studs are arranged in a way such that splitting forces occur in the direction of the slab thickness, (1) is not applicable.

Note: For buildings, further information may be given in the National Annex.

6.6.3.2 Influence of tension on shear resistance

(1) Where headed stud connectors are subjected to direct tensile force in addition to shear, the design tensile force per stud F_{ten} should be calculated.

(2) If $F_{ten} \leq 0{,}1P_{Rd}$, where P_{Rd} is the design shear resistance defined in 6.6.3.1, the tensile force may be neglected.

(3) If $F_{ten} > 0{,}1P_{Rd}$, the connection is not within the scope of EN 1994.

6.6.4 Design resistance of headed studs used with profiled steel sheeting in buildings

6.6.4.1 Sheeting with ribs parallel to the supporting beams

(1) The studs are located within a region of concrete that has the shape of a haunch, see Figure 6.12. Where the sheeting is continuous across the beam, the width of the haunch b_0 is equal to the width of the trough as given in Figure 9.2. Where the sheeting is not continuous, b_0 is defined in a similar way as given in Figure 6.12. The depth of the haunch should be taken as h_p, the overall depth of the sheeting excluding embossments.

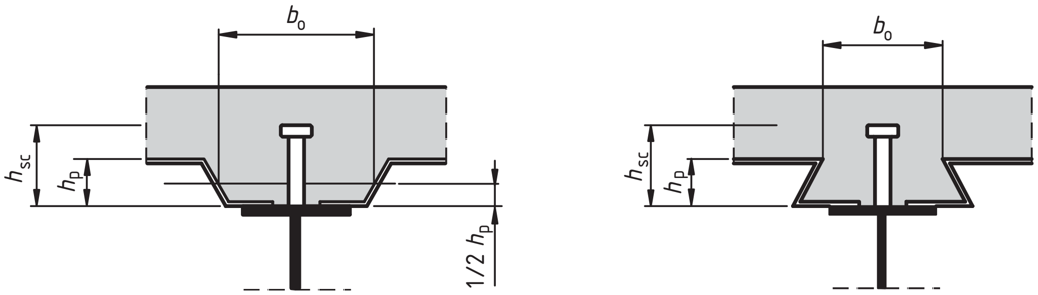


Figure 6.12 : Beam with profiled steel sheeting parallel to the beam

(2) The design shear resistance should be taken as the resistance in a solid slab, see 6.6.3.1, multiplied by the reduction factor k_ℓ given by the following expression:

$$k_\ell = 0{,}6\,\frac{b_0}{h_p}\left(\frac{h_{sc}}{h_p} - 1\right) \leq 1{,}0 \qquad (6.22)$$

where:

h_{sc} is the overall height of the stud, but not greater than h_p + 75 mm.

(3) Where the sheeting is not continuous across the beam, and is not appropriately anchored to the beam, that side of the haunch and its reinforcement should satisfy 6.6.5.4.

Note: Means to achieve appropriate anchorage may be given in the National Annex.

6.6.4.2 Sheeting with ribs transverse to the supporting beams

(1) Provided that the conditions given in (2) and (3) are satisfied, the design shear resistance should be taken as the resistance in a solid slab, calculated as given by 6.6.3.1 (except that f_u should not be taken as greater than 450 N/mm²) multiplied by the reduction factor k_t given by:

$$k_t = \frac{0{,}7}{\sqrt{n_r}} \frac{b_0}{h_p} \left(\frac{h_{sc}}{h_p} - 1 \right) \tag{6.23}$$

where:

n_r is the number of stud connectors in one rib at a beam intersection, not to exceed 2 in computations, and other symbols are as defined in Figure 6.13.

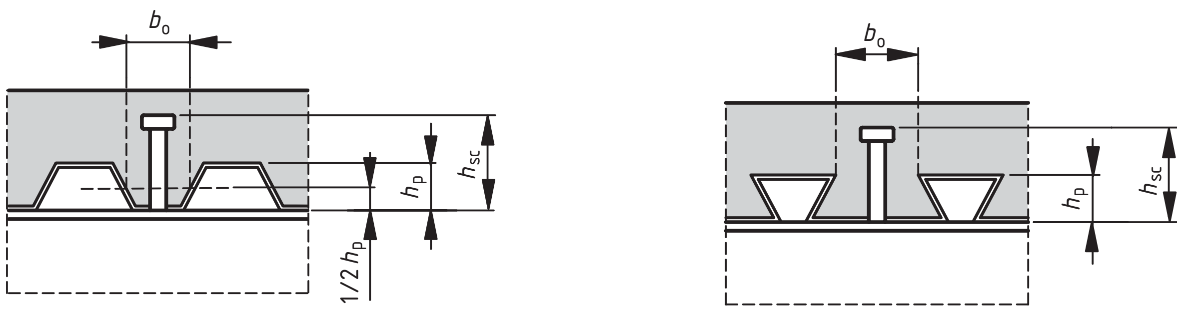


Figure 6.13 : Beam with profiled steel sheeting transverse to the beam

(2) The factor k_t should not be taken greater than the appropriate value $k_{t,max}$ given in Table 6.2.

(3) The values for k_t given by (1) and (2) are applicable provided that:

- the studs are placed in ribs with a height h_p not greater than 85 mm and a width b_0 not less than h_p and
- for through deck welding, the diameter of the studs is not greater than 20mm, or
- for holes provided in the sheeting, the diameter of the studs is not greater than 22mm.

Table 6.2 : Upper limits $k_{t,max}$ for the reduction factor k_t

Number of stud connectors per rib	Thickness t of sheet (mm)	Studs not exceeding 20 mm in diameter and welded through profiled steel sheeting	Profiled sheeting with holes and studs 19 mm or 22mm in diameter
$n_r = 1$	$\leq 1,0$	0,85	0,75
	$> 1,0$	1,0	0,75
$n_r = 2$	$\leq 1,0$	0,70	0,60
	$> 1,0$	0,8	0,60

6.6.4.3 Biaxial loading of shear connectors

(1) Where the shear connectors are provided to produce composite action both for the beam and for the composite slab, the combination of forces acting on the stud should satisfy the following:

$$\frac{F_\ell^2}{P_{\ell,\mathrm{Rd}}^2} + \frac{F_\mathrm{t}^2}{P_{\mathrm{t,Rd}}^2} \leq 1 \qquad (6.24)$$

where:

F_ℓ is the design longitudinal force caused by composite action in the beam;

F_t is the design transverse force caused by composite action in the slab, see Section 9;

$P_{\ell,\mathrm{Rd}}$ and $P_{\mathrm{t,Rd}}$ are the corresponding design shear resistances of the stud.

6.6.5 Detailing of the shear connection and influence of execution

6.6.5.1 Resistance to separation

(1) The surface of a connector that resists separation forces (for example, the underside of the head of a stud) should extend not less than 30 mm clear above the bottom reinforcement, see Figure 6.14.

6.6.5.2 Cover and concreting for buildings

(1)P The detailing of shear connectors shall be such that concrete can be adequately compacted around the base of the connector.

(2) If cover over the connector is required, the nominal cover should be:

a) not less than 20 mm, or

b) as recommended by EN 1992-1-1, Table 4.4 for reinforcing steel, less 5 mm,

whichever is the greater.

(3) If cover is not required the top of the connector may be flush with the upper surface of the concrete slab.

(4) In execution, the rate and sequence of concreting should be required to be such that partly matured concrete is not damaged as a result of limited composite action occurring from deformation of the steel beams under subsequent concreting operations. Wherever possible, deformation should not be imposed on a shear connection until the concrete has reached a cylinder strength of at least 20 N/mm^2.

6.6.5.3 Local reinforcement in the slab

(1) Where the shear connection is adjacent to a longitudinal edge of a concrete slab, transverse reinforcement provided in accordance with 6.6.6 should be fully anchored in the concrete between the edge of the slab and the adjacent row of connectors.

(2) To prevent longitudinal splitting of the concrete flange caused by the shear connectors, the following additional recommendations should be applied where the distance from the edge of the concrete flange to the centreline of the nearest row of shear connectors is less than 300 mm:

a) transverse reinforcement should be supplied by U-bars passing around the shear connectors,

b) where headed studs are used as shear connectors, the distance from the edge of the concrete flange to the centre of the nearest stud should not be less than 6*d*, where *d* is the nominal diameter of the stud, and the U-bars should be not less than 0,5*d* in diameter and

c) the U-bars should be placed as low as possible while still providing sufficient bottom cover.

(3)P At the end of a composite cantilever, sufficient local reinforcement shall be provided to transfer forces from the shear connectors to the longitudinal reinforcement.

6.6.5.4 Haunches other than formed by profiled steel sheeting

(1) Where a concrete haunch is used between the steel section and the soffit of the concrete slab, the sides of the haunch should lie outside a line drawn at 45º from the outside edge of the connector, see Figure 6.14.

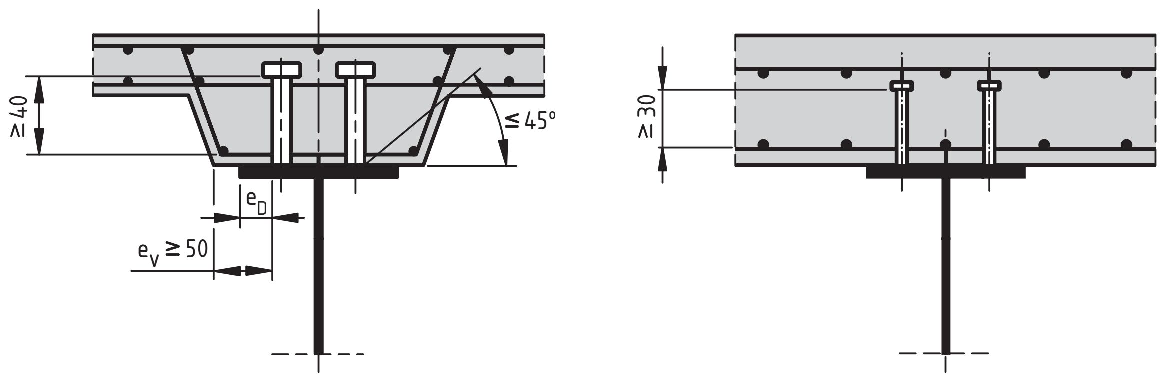


Figure 6.14 : Detailing

(2) The nominal concrete cover from the side of the haunch to the connector should be not less than 50 mm.

(3) Transverse reinforcing bars sufficient to satisfy the requirements of 6.6.6 should be provided in the haunch at not less than 40 mm clear below the surface of the connector that resists uplift.

6.6.5.5 Spacing of connectors

(1)P Where it is assumed in design that the stability of either the steel or the concrete member is ensured by the connection between the two, the spacing of the shear connectors shall be sufficiently close for this assumption to be valid.

(2) Where a steel compression flange that would otherwise be in a lower class is assumed to be in Class 1 or Class 2 because of restraint from shear connectors, the centre-to-centre spacing of the shear connectors in the direction of compression should be not greater than the following limits:

– where the slab is in contact over the full length (e.g. solid slab): $22\ t_f\sqrt{235/f_y}$

– where the slab is not in contact over the full length (e.g. slab with ribs transverse to the beam): $15\ t_f\sqrt{235/f_y}$

where:

t_f is the thickness of the flange;

f_y is the nominal yield strength of the flange in N/mm^2.

In addition, the clear distance from the edge of a compression flange to the nearest line of shear connectors should be not greater than $9\ t_f\sqrt{235/f_y}$.

(3) In buildings, the maximum longitudinal centre-to-centre spacing of shear connectors should be not greater than 6 times the total slab thickness nor 800 mm.

6.6.5.6 Dimensions of the steel flange

(1)P The thickness of the steel plate or flange to which a connector is welded shall be sufficient to allow proper welding and proper transfer of load from the connector to the plate without local failure or excessive deformation.

(2) In buildings, 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 be not less than 20 mm.

6.6.5.7 Headed stud connectors

(1) The overall height of a stud should be not less than 3*d*, where *d* is the diameter of the shank.

(2) The head should have a diameter of not less than 1,5*d* and a depth of not less than 0,4*d*.

(3) For elements in tension and subjected to fatigue loading, the diameter of a welded stud should not exceed 1,5 times the thickness of the flange to which it is welded, unless test information is provided to establish the fatigue resistance of the stud as a shear connector. This applies also to studs directly over a web.

(4) The spacing of studs in the direction of the shear force should be not less than 5*d*; the spacing in the direction transverse to the shear force should be not less than 2,5*d* in solid slabs and 4*d* in other cases.

(5) Except when the studs are located directly over the web, the diameter of a welded stud should be not greater than 2,5 times the thickness of that part to which it is welded, unless test information is provided to establish the resistance of the stud as a shear connector.

6.6.5.8 Headed studs used with profiled steel sheeting in buildings

(1) The nominal height of a connector should extend not less than $2d$ above the top of the steel deck, where d is the diameter of the shank.

(2) The minimum width of the troughs that are to be filled with concrete should be not less than 50 mm.

(3) Where the sheeting is such that studs cannot be placed centrally within a trough, they should be placed alternately on the two sides of the trough, throughout the length of the span.

6.6.6 Longitudinal shear in concrete slabs

6.6.6.1 General

(1)P Transverse reinforcement in the slab shall be designed for the ultimate limit state so that premature longitudinal shear failure or longitudinal splitting shall be prevented.

(2)P The design longitudinal shear stress for any potential surface of longitudinal shear failure within the slab v_{Ed} shall not exceed the design longitudinal shear strength of the shear surface considered.

(3) The length of the shear surface b-b shown in Figure 6.15 should be taken as equal to $2h_{sc}$ plus the head diameter for a single row of stud shear connectors or staggered stud connectors, or as equal to $(2h_{sc} + s_t)$ plus the head diameter for stud shear connectors arranged in pairs, where h_{sc} is the height of the studs and s_t is the transverse spacing centre-to-centre of the studs.

(4) The design longitudinal shear per unit length of beam on a shear surface should be determined in accordance with 6.6.2 and be consistent with the design and spacing of the shear connectors. Account may be taken of the variation of longitudinal shear across the width of the concrete flange.

(5) For each type of shear surface considered, the design longitudinal shear stress v_{Ed} should be determined from the design longitudinal shear per unit length of beam, taking account of the number of shear planes and the length of the shear surface.

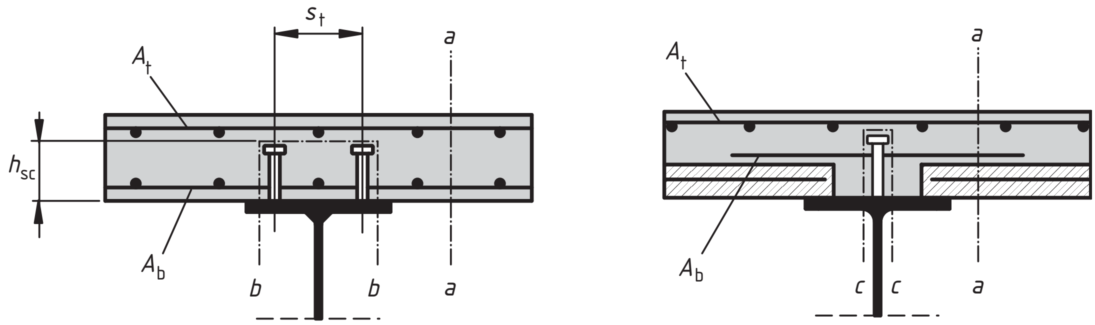
6.6.6.2 Design resistance to longitudinal shear

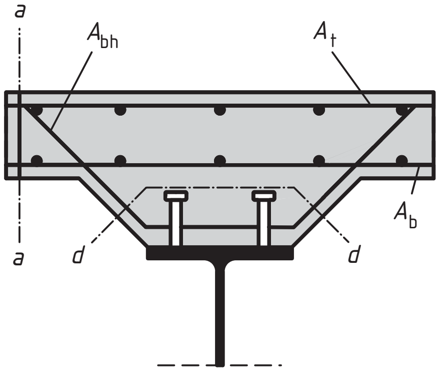
(1) The design shear strength of the concrete flange (shear planes a-a illustrated in Figure 6.15) should be determined in accordance with EN 1992-1-1, 6.2.4.

(2) In the absence of a more accurate calculation the design shear strength of any surface of potential shear failure in the flange or a haunch may be determined from EN 1992-1-1, 6.2.4(4). For a shear surface passing around the shear connectors (e.g. shear surface b-b in Figure 6.15), the dimension h_f should be taken as the length of the shear surface.

(3) The effective transverse reinforcement per unit length, A_{sf} / s_f in EN 1992-1-1, should be as shown in Figure 6.15, in which A_b, A_t and A_{bh} are areas of reinforcement per unit length of beam anchored in accordance with EN 1992-1-1, 8.4 for longitudinal reinforcement.

(4) Where a combination of pre-cast elements and in-situ concrete is used, the resistance to longitudinal shear should be determined in accordance with EN 1992-1-1, 6.2.5.





type	A_{sf} / s_f
a-a	$A_b + A_t$
b-b	$2A_b$
c-c	$2A_b$
d-d	$2A_{bh}$

Figure 6.15 : Typical potential surfaces of shear failure

6.6.6.3 Minimum transverse reinforcement

(1) The minimum area of reinforcement should be determined in accordance with EN 1992-1-1, 9.2.2(5) using definitions appropriate to transverse reinforcement.

6.6.6.4 Longitudinal shear and transverse reinforcement in beams for buildings

(1) Where profiled steel sheeting is used and the shear surface passes through the depth of the slab (e.g. shear surface a-a in Figure 6.16), the dimension h_f should be taken as the thickness of the concrete above the sheeting.

(2) Where profiled steel sheeting is used transverse to the beam and the design resistances of the studs are determined using the appropriate reduction factor k_t as given in 6.6.4.2, it is not necessary to consider shear surfaces of type b-b in Figure 6.16.

(3) Unless verified by tests, for surfaces of type c-c in Figure 6.16 the depth of the sheeting should not be included in h_f.

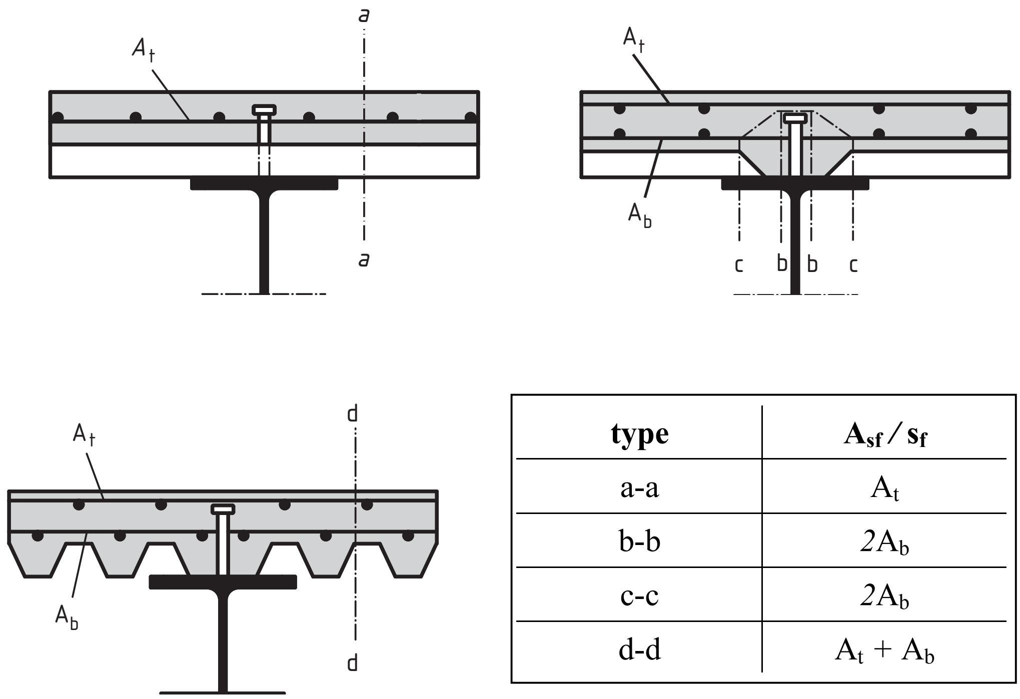
(4) Where profiled steel sheeting with mechanical or frictional interlock and with ribs transverse to the beam is continuous across the top flange of the steel beam, its contribution to the transverse reinforcement for a shear surface of type a-a may be allowed for by replacing expression (6.21) in EN 1992-1-1, 6.2.4(4) by:

$$(A_{sf} f_{yd} / s_f) + A_{pe} f_{yp,d} > v_{Ed} h_f / \cot\theta \tag{6.25}$$

where:

A_{pe} is the effective cross-sectional area of the profiled steel sheeting per unit length of the beam, see 9.7.2(3); for sheeting with holes, the net area should be used;

$f_{yp,d}$ is its design yield strength.



type	A_{sf} / s_f
a-a	A_t
b-b	$2A_b$
c-c	$2A_b$
d-d	$A_t + A_b$

Figure 6.16 : Typical potential surfaces of shear failure where profiled steel sheeting is used

(5) Where the profiled steel sheeting with ribs transverse to the beam is discontinuous across the top flange of the steel beam, and stud shear connectors are welded to the steel beam directly through the profiled steel sheets, the term $A_p f_{yp,d}$ in expression (6.25) should be replaced by:

$$P_{pb,Rd} / s \quad \text{but} \quad \leq A_p f_{yp,d} \tag{6.26}$$

where:

$P_{pb,Rd}$ is the design bearing resistance of a headed stud welded through the sheet according to 9.7.4;

s is the longitudinal spacing centre-to-centre of the studs effective in anchoring the sheeting.

(6) With profiled steel sheeting, the requirement for minimum reinforcement relates to the area of concrete above the sheeting.

6.7 Composite columns and composite compression members

6.7.1 General

(1)P Clause 6.7 applies for the design of composite columns and composite compression members with concrete encased sections, partially encased sections and concrete filled rectangular and circular tubes, see Figure 6.17.

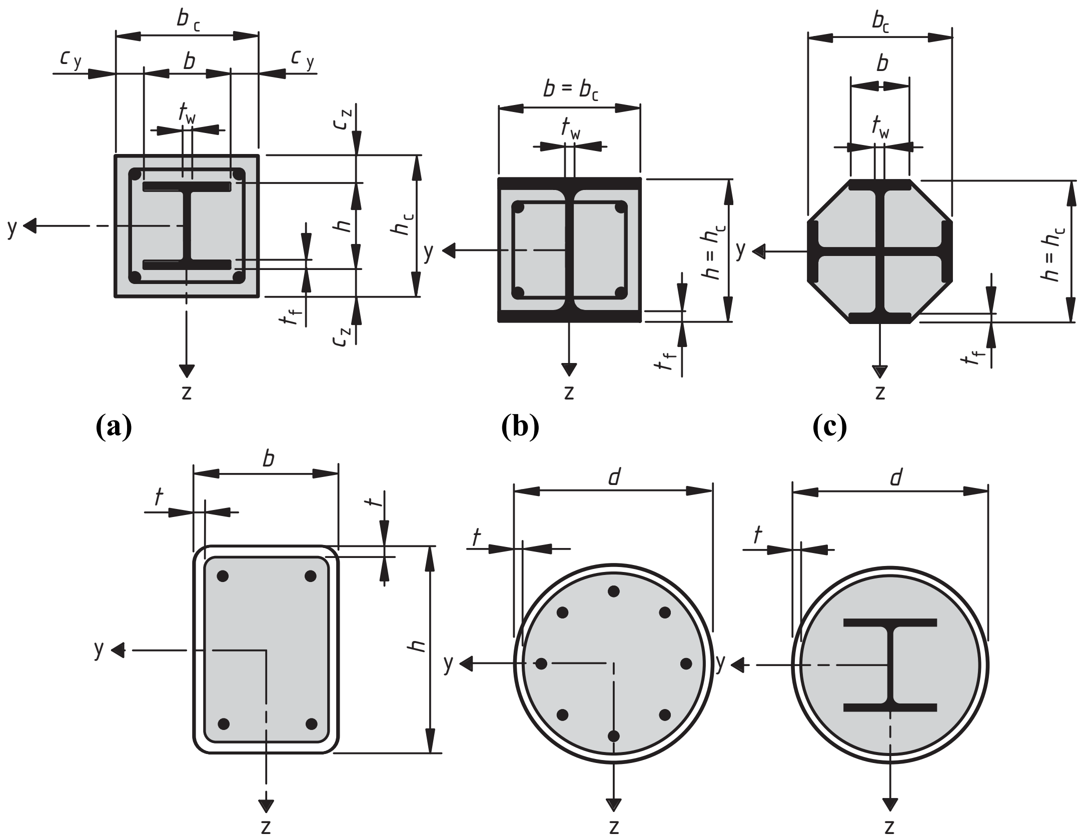


Figure 6.17 : Typical cross-sections of composite columns and notation

(2)P This clause applies to columns and compression members with steel grades S235 to S460 and normal weight concrete of strength classes C20/25 to C50/60.

(3) This clause applies to isolated columns and columns and composite compression members in framed structures where the other structural members are either composite or steel members.

(4) The steel contribution ratio δ should fulfil the following condition:

$$0{,}2 \leq \delta \leq 0{,}9 \qquad (6.27)$$

where:

δ is defined in 6.7.3.3(1).

(5) Composite columns or compression members of any cross-section should be checked for:

– resistance of the member in accordance with 6.7.2 or 6.7.3,

– resistance to local buckling in accordance with (8) and (9) below,

– introduction of loads in accordance with 6.7.4.2 and

– resistance to shear between steel and concrete elements in accordance with 6.7.4.3.

(6) Two methods of design are given:

– a general method in 6.7.2 whose scope includes members with non-symmetrical or non-uniform cross-sections over the column length and

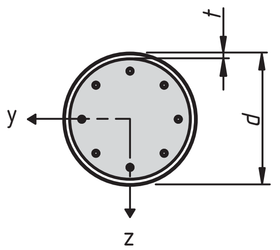
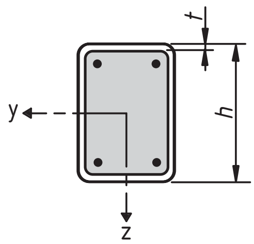
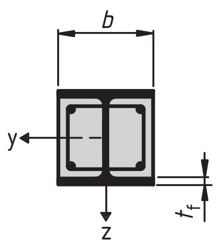
– a simplified method in 6.7.3 for members of doubly symmetrical and uniform cross section over the member length.

(7) For composite compression members subjected to bending moments and normal forces resulting from independent actions, the partial factor γ_F for those internal forces that lead to an increase of resistance should be reduced by 20%.

(8)P The influence of local buckling of the steel section on the resistance shall be considered in design.

(9) The effects of local buckling may be neglected for a steel section fully encased in accordance with 6.7.5.1(2), and for other types of cross-section provided the maximum values of Table 6.3 are not exceeded.

Table 6.3 : Maximum values (*d/t*), (*h/t*) and (*b/t*$_f$) with f_y in N/mm^2

Cross-section		Max (*d/t*), max (*h/t*) and max (*b/t*)
Circular hollow steel sections		$\max\ (d/t) = 90\dfrac{235}{f_y}$
Rectangular hollow steel sections		$\max\ (h/t) = 52\sqrt{\dfrac{235}{f_y}}$
Partially encased I-sections		$\max\ (b/t_f) = 44\sqrt{\dfrac{235}{f_y}}$

6.7.2 General method of design

(1)P Design for structural stability shall take account of second-order effects including residual stresses, geometrical imperfections, local instability, cracking of concrete, creep and shrinkage of concrete and yielding of structural steel and of reinforcement. The design shall ensure that instability does not occur for the most unfavourable combination of actions at the ultimate limit state and that the resistance of individual cross-sections subjected to bending, longitudinal force and shear is not exceeded.

(2)P Second-order effects shall be considered in any direction in which failure might occur, if they affect the structural stability significantly.

(3)P Internal forces shall be determined by elasto-plastic analysis.

(4) Plane sections may be assumed to remain plane. Full composite action up to failure may be assumed between the steel and concrete components of the member.

(5)P The tensile strength of concrete shall be neglected. The influence of tension stiffening of concrete between cracks on the flexural stiffness may be taken into account.

(6)P Shrinkage and creep effects shall be considered if they are likely to reduce the structural stability significantly.

(7) For simplification, creep and shrinkage effects may be ignored if the increase in the first-order bending moments due to creep deformations and longitudinal force resulting from permanent loads is not greater than 10%.

(8) The following stress-strain relationships should be used in the non-linear analysis :

– for concrete in compression as given in EN 1992-1-1, 3.1.5;
– for reinforcing steel as given in EN 1992-1-1, 3.2.7;
– for structural steel as given in EN 1993-1-1, 5.4.3(4).

(9) For simplification, instead of the effect of residual stresses and geometrical imperfections, equivalent initial bow imperfections (member imperfections) may be used in accordance with Table 6.5.

6.7.3 Simplified method of design

6.7.3.1 General and scope

(1) The scope of this simplified method is limited to members of doubly symmetrical and uniform cross-section over the member length with rolled, cold-formed or welded steel sections. The simplified method is not applicable if the structural steel component consists of two or more unconnected sections. The relative slenderness $\bar{\lambda}$ defined in 6.7.3.3 should fulfill the following condition:

$$\bar{\lambda} \leq 2{,}0 \tag{6.28}$$

(2) For a fully encased steel section, see Figure 6.17a, limits to the maximum thickness of concrete cover that may be used in calculation are:

$$\max c_z = 0{,}3h \qquad \max\ c_y = 0{,}4\,b \tag{6.29}$$

(3) The longitudinal reinforcement that may be used in calculation should not exceed 6% of the concrete area.

(4) The ratio of the depth to the width of the composite cross-section should be within the limits 0,2 and 5,0.

6.7.3.2 Resistance of cross sections

(1) The plastic resistance to compression $N_{pl,Rd}$ of a composite cross-section should be calculated by adding the plastic resistances of its components:

$$N_{pl,Rd} = A_a f_{yd} + 0{,}85 A_c f_{cd} + A_s f_{sd} \tag{6.30}$$

Expression (6.30) applies for concrete encased and partially concrete encased steel sections. For concrete filled sections the coefficient 0,85 may be replaced by 1,0.

(2) The resistance of a cross-section to combined compression and bending and the corresponding interaction curve may be calculated assuming rectangular stress blocks as shown in Figure 6.18, taking account of the design shear force V_{Ed} in accordance with (3). The tensile strength of the concrete should be neglected.

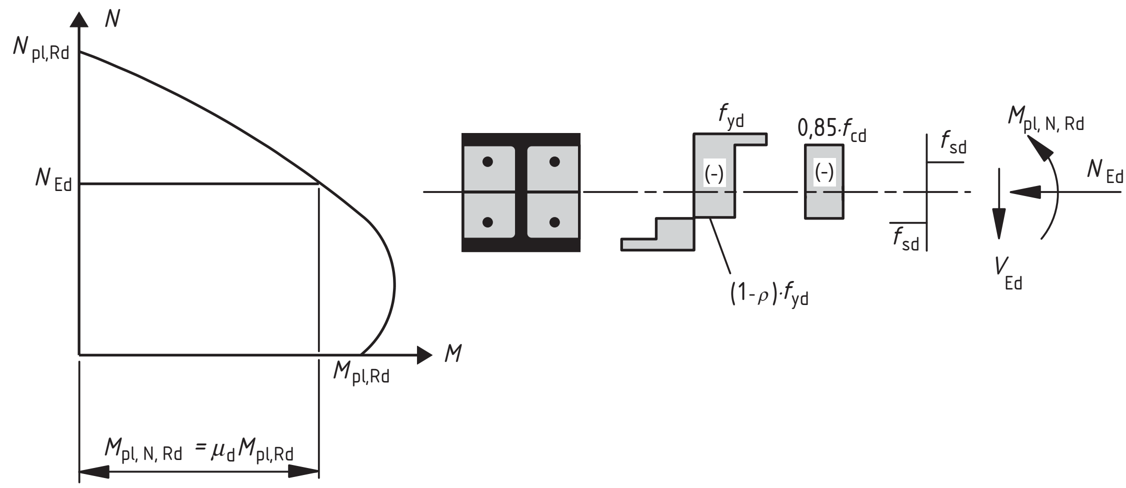


Figure 6.18 : Interaction curve for combined compression and uniaxial bending

(3) The influence of transverse shear forces on the resistance to bending and normal force should be considered when determining the interaction curve, if the shear force $V_{a,Ed}$ on the steel section exceeds 50% of the design shear resistance $V_{pl,a,Rd}$ of the steel section, see 6.2.2.2.

Where $V_{a,Ed} > 0{,}5 V_{pl,a,Rd}$, the influence of the transverse shear on the resistance in combined bending and compression should be taken into account by a reduced design steel strength $(1 - \rho) f_{yd}$ in the shear area A_v in accordance with 6.2.2.4(2) and Figure 6.18.
The shear force $V_{a,Ed}$ should not exceed the resistance to shear of the steel section determined according to 6.2.2. The resistance to shear $V_{c,Ed}$ of the reinforced concrete part should be verified in accordance with EN 1992-1-1, 6.2.

(4) Unless a more accurate analysis is used, V_{Ed} may be distributed into $V_{a,Ed}$ acting on the structural steel and $V_{c,Ed}$ acting on the reinforced concrete section by :

$$V_{a,Ed} = V_{Ed} \frac{M_{pl,a,Rd}}{M_{pl,Rd}} \tag{6.31}$$

$$V_{c,Ed} = V_{Ed} - V_{a,Ed} \tag{6.32}$$

where:

$M_{pl,a,Rd}$ is the plastic resistance moment of the steel section and

$M_{pl,Rd}$ is the plastic resistance moment of the composite section.

For simplification V_{Ed} may be assumed to act on the structural steel section alone.

(5) As a simplification, the interaction curve may be replaced by a polygonal diagram (the dashed line in Figure 6.19). Figure 6.19 shows as an example the plastic stress distribution of a fully encased cross section for the points A to D. $N_{pm,Rd}$ should be taken as 0,85 f_{cd} A_c for concrete encased and partially concrete encased sections, see Figures 6.17(a) – (c), and as f_{cd} A_c for concrete filled sections, see Figures 6.17(d) – (f).

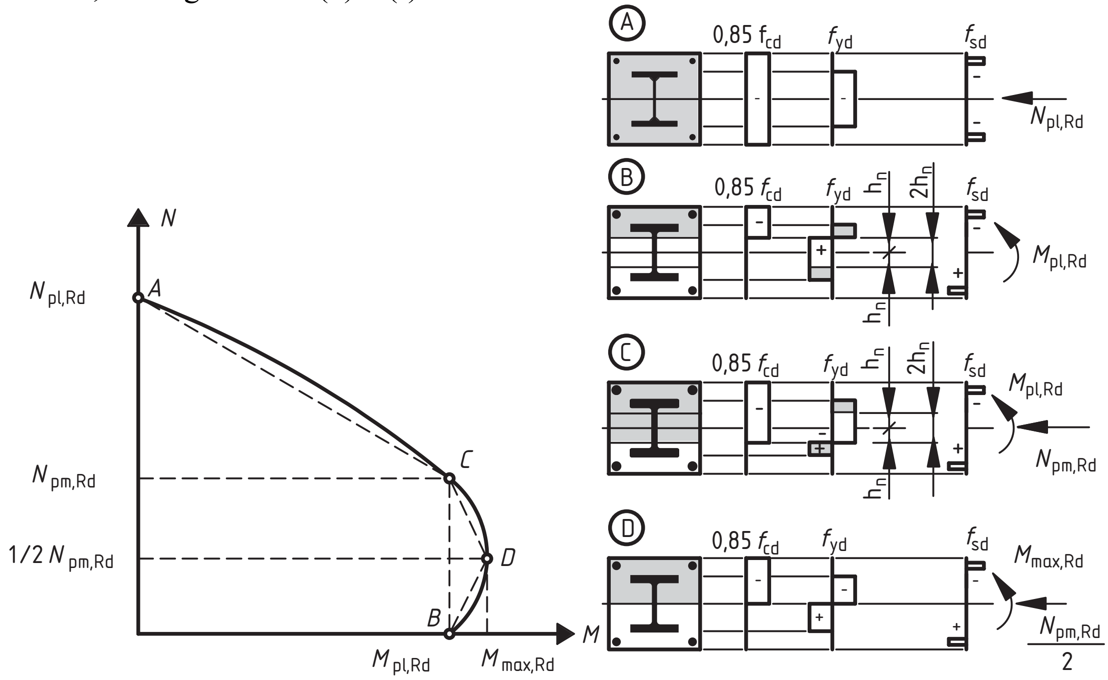


Figure 6.19 : Simplified interaction curve and corresponding stress distributions

(6) For concrete filled tubes of circular cross-section, account may be taken of increase in strength of concrete caused by confinement provided that the relative slenderness $\bar{\lambda}$ defined in 6.7.3.3 does not exceed 0,5 and $e/d < 0{,}1$, where e is the eccentricity of loading given by M_{Ed} / N_{Ed} and d is the external diameter of the column. The plastic resistance to compression may then be calculated from the following expression:

$$N_{pl,Rd} = \eta_a A_a f_{yd} + A_c f_{cd} \left(1 + \eta_c \frac{t}{d} \frac{f_y}{f_{ck}} \right) + A_s f_{sd} \tag{6.33}$$

where:

t is the wall thickness of the steel tube.

For members with $e = 0$ the values $\eta_a = \eta_{ao}$ and $\eta_c = \eta_{co}$ are given by the following expressions:

$$\eta_{ao} = 0{,}25\,(3 + 2\bar{\lambda}) \qquad \text{(but } \leq 1{,}0\text{)} \tag{6.34}$$

$$\eta_{co} = 4{,}9 - 18{,}5\bar{\lambda} + 17\bar{\lambda}^2 \qquad \text{(but } \geq 0\text{)} \tag{6.35}$$

For members in combined compression and bending with $0 < e/d \leq 0{,}1$, the values η_a and η_c should be determined from (6.36) and (6.37), where η_{ao} and η_{co} are given by (6.34) and (6.35):

$$\eta_a = \eta_{ao} + (1 - \eta_{ao})\,(10\, e/d) \tag{6.36}$$

$$\eta_c = \eta_{co}\,(1 - 10\, e/d) \tag{6.37}$$

For $e/d > 0{,}1$, $\eta_a = 1{,}0$ and $\eta_c = 0$.

6.7.3.3 Effective flexural stiffness, steel contribution ratio and relative slenderness

(1) The steel contribution ratio δ is defined as:

$$\delta = \frac{A_a f_{yd}}{N_{pl,Rd}} \tag{6.38}$$

where:

$N_{pl,Rd}$ is the plastic resistance to compression defined in 6.7.3.2(1).

(2) The relative slenderness $\bar{\lambda}$ for the plane of bending being considered is given by:

$$\bar{\lambda} = \sqrt{\frac{N_{pl,Rk}}{N_{cr}}} \tag{6.39}$$

where:

$N_{pl,Rk}$ is the characteristic value of the plastic resistance to compression given by (6.30) if, instead of the design strengths, the characteristic values are used;

N_{cr} is the elastic critical normal force for the relevant buckling mode, calculated with the effective flexural stiffness $(EI)_{eff}$ determined in accordance with (3) and (4).

(3) For the determination of the relative slenderness $\bar{\lambda}$ and the elastic critical force N_{cr}, the characteristic value of the effective flexural stiffness $(EI)_{eff}$ of a cross section of a composite column should be calculated from:

$$(E\,I)_{eff} = E_a I_a + E_s I_s + K_e\, E_{cm} I_c \tag{6.40}$$

where:

K_e is a correction factor that should be taken as 0,6.

I_a, I_c, and I_s are the second moments of area of the structural steel section, the un-cracked concrete section and the reinforcement for the bending plane being considered.

(4) Account should be taken to the influence of long-term effects on the effective elastic flexural stiffness. The modulus of elasticity of concrete E_{cm} should be reduced to the value $E_{c,eff}$ in accordance with the following expression:

$$E_{c,eff} = E_{cm} \frac{1}{1 + (N_{G,Ed} / N_{Ed})\,\varphi_t} \tag{6.41}$$

where:

φ_t is the creep coefficient according to 5.4.2.2(2);

N_{Ed} is the total design normal force;

$N_{G,Ed}$ is the part of this normal force that is permanent.

6.7.3.4 Methods of analysis and member imperfections

(1) For member verification, analysis should be based on second-order linear elastic analysis.

(2) For the determination of the internal forces the design value of effective flexural stiffness $(EI)_{eff,II}$ should be determined from the following expression:

$$(EI)_{eff,II} = K_o\,(E_a I_a + E_s I_s + K_{e,II}\,E_{cm}\,I_c) \tag{6.42}$$

where:

$K_{e,II}$ is a correction factor which should be taken as 0,5;

K_o is a calibration factor which should be taken as 0,9.

Long-term effects should be taken into account in accordance with 6.7.3.3 (4).

(3) Second-order effects need not to be considered where 5.2.1(3) applies and the elastic critical load is determined with the flexural stiffness $(EI)_{eff,II}$ in accordance with (2).

(4) The influence of geometrical and structural imperfections may be taken into account by equivalent geometrical imperfections. Equivalent member imperfections for composite columns are given in Table 6.5, where L is the column length.

(5) Within the column length, second-order effects may be allowed for by multiplying the greatest first-order design bending moment M_{Ed} by a factor k given by:

$$k = \frac{\beta}{1 - N_{Ed} / N_{cr,eff}}\,, \qquad \geq 1{,}0 \tag{6.43}$$

where:

$N_{cr,eff}$ is the critical normal force for the relevant axis and corresponding to the effective flexural stiffness given in 6.7.3.4(2), with the effective length taken as the column length;

β is an equivalent moment factor given in Table 6.4.

6.7.3.5 Resistance of members in axial compression

(1) Members may be verified using second order analysis according to 6.7.3.6 taking into account member imperfections.

(2) For simplification for members in axial compression, the design value of the normal force N_{Ed} should satisfy:

$$\frac{N_{Ed}}{\chi N_{pl,Rd}} \leq 1{,}0 \tag{6.44}$$

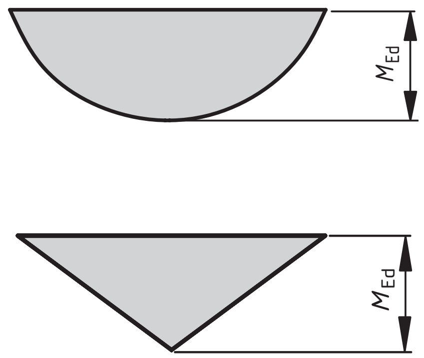
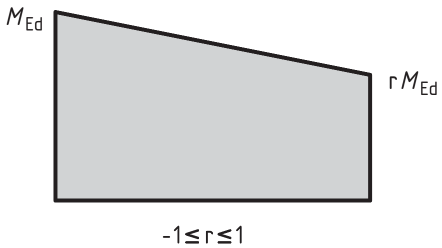
where:

$N_{pl,Rd}$ is the plastic resistance of the composite section according to 6.7.3.2(1), but with f_{yd} determined using the partial factor γ_{M1} given by EN 1993-1-1, 6.1(1);

χ is the reduction factor for the relevant buckling mode given in EN 1993-1-1, 6.3.1.2 in terms of the relevant relative slenderness $\bar{\lambda}$.

The relevant buckling curves for cross-sections of composite columns are given in Table 6.5, where ρ_s is the reinforcement ratio A_s / A_c.

Table 6.4 Factors β for the determination of moments to second order theory

Moment distribution	Moment factors β	Comment
	First-order bending moments from member imperfection or lateral load: $\beta = 1{,}0$	M_{Ed} is the maximum bending moment within the column length ignoring second-order effects
	End moments: $\beta = 0{,}66 + 0{,}44r$ but $\beta \geq 0{,}44$	M_{Ed} and $r\,M_{Ed}$ are the end moments from first-order or second-order global analysis

6.7.3.6 Resistance of members in combined compression and uniaxial bending

(1) The following expression based on the interaction curve determined according to 6.7.3.2 (2)-(5) should be satisfied:

$$\frac{M_{Ed}}{M_{pl,N,Rd}} = \frac{M_{Ed}}{\mu_d\, M_{pl,Rd}} \leq \alpha_M \qquad (6.45)$$

where:

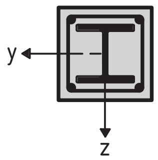
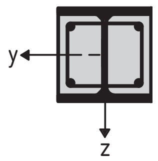
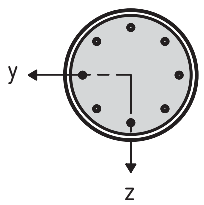
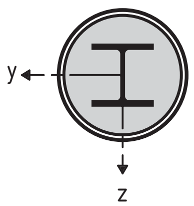
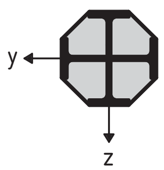
M_{Ed} is the greatest of the end moments and the maximum bending moment within the column length, calculated according to 6.7.3.4, including imperfections and second order effects if necessary;

$M_{pl,N,Rd}$ is the plastic bending resistance taking into account the normal force N_{Ed}, given by $\mu_d\, M_{pl,Rd}$, see Figure 6.18;

$M_{pl,Rd}$ is the plastic bending resistance, given by point B in Figure 6.19.

For steel grades between S235 and S355 inclusive, the coefficient α_M should be taken as 0,9 and for steel grades S420 and S460 as 0,8.

Table 6.5 : Buckling curves and member imperfections for composite columns

Cross-section	Limits	Axis of buckling	Buckling curve	Member imperfection
concrete encased section 		y-y	b	*L*/200
		z-z	c	*L*/150
partially concrete encased section 		y-y	b	*L*/200
		z-z	c	*L*/150
circular and rectangular hollow steel section 	$\rho_s \leq 3\%$	any	a	*L*/300
	$3\% < \rho_s \leq 6\%$	any	b	*L*/200
circular hollow steel sections with additional I-section 		y-y	b	*L*/200
		z-z	b	*L*/200
partially concrete encased section with crossed I-sections 		any	b	*L*/200

equivalent cross-sectional area and equivalent distances from the centre of compression and the centroid of the beam's steel section.

Annex B (Informative)

Standard tests

B.1 General

(1) In this Standard rules are given for:

a) tests on shear connectors in B.2 and

b) testing of composite floor slabs in B.3.

Note : These standard testing procedures are included in the absence of Guidelines for ETA. When such Guidelines have been developed this Annex can be withdrawn.

B.2 Tests on shear connectors

B.2.1 General

(1) Where the design rules in 6.6 are not applicable, the design should be based on tests, carried out in a way that provides information on the properties of the shear connection required for design in accordance with this Standard.

(2) The variables to be investigated include the geometry and the mechanical properties of the concrete slab, the shear connectors and the reinforcement.

(3) The resistance to loading, other than fatigue, may be determined by push tests in accordance with the requirements in this Annex.

(4) For fatigue tests the specimen should also be prepared in accordance with this Annex.

B.2.2 Testing arrangements

(1) Where the shear connectors are used in T-beams with a concrete slab of uniform thickness, or with haunches complying with 6.6.5.4, standard push tests may be used. In other cases specific push tests should be used.

(2) For standard push tests the dimensions of the test specimen, the steel section and the reinforcement should be as given in Figure B.1. The recess in the concrete slabs is optional.

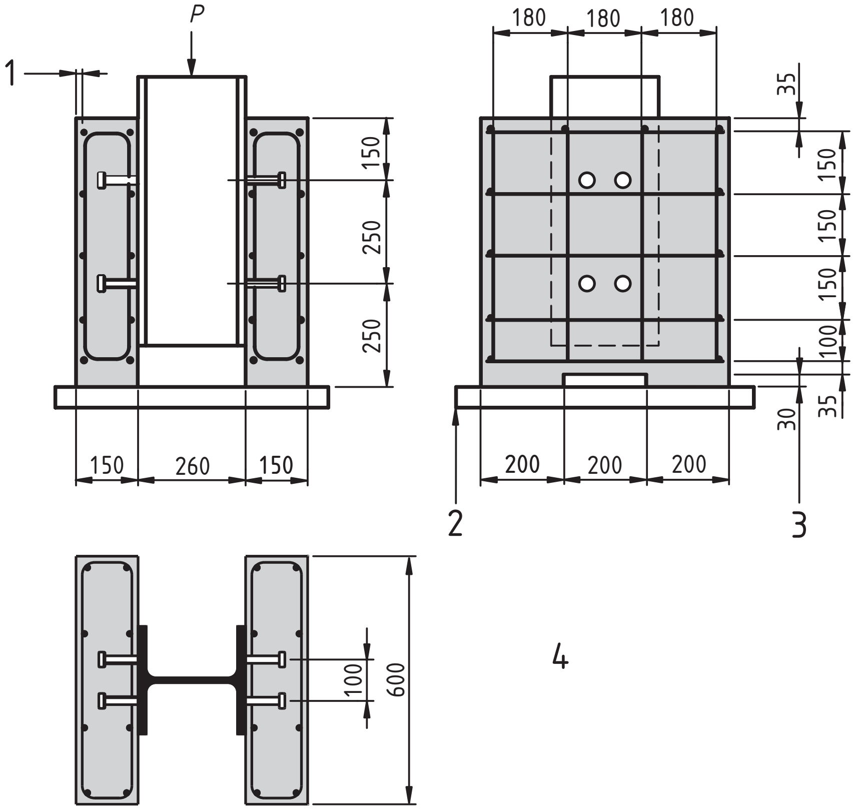
(3) Specific push tests should be carried out such that the slabs and the reinforcement are suitably dimensioned in comparison with the beams for which the test is designed. In particular:

a) the length *l* of each slab should be related to the longitudinal spacing of the connectors in the composite structure;

b) the width *b* of each slab should not exceed the effective width of the slab of the beam;

c) the thickness *h* of each slab should not exceed the minimum thickness of the slab in the beam;

d) where a haunch in the beam does not comply with 6.6.5.4, the slabs of the push specimen should have the same haunch and reinforcement as the beam.



Key

1 cover 15 mm

2 bedded in mortar or gypsum

3 recess optional

4 reinforcement: ribbed bars ϕ 10 mm resulting in a high bond with $450 \leq f_{sk} \leq 550$ N/mm^2

steel section: HE 260 B or 254 x 254 x 89 kg. UC

Figure B.1 : Test specimen for standard push test

B.2.3 Preparation of specimens

(1) Each of both concrete slabs should be cast in the horizontal position, as is done for composite beams in practice.

(2) Bond at the interface between flanges of the steel beam and the concrete should be prevented by greasing the flange or by other suitable means.

(3) The push specimens should be air-cured.

(4) For each mix a minimum of four concrete specimens (cylinders or cubes) for the determination of the cylinder strength should be prepared at the time of casting the push specimens. These concrete specimens should be cured alongside the push specimens. The concrete strength f_{cm} should be taken as the mean value.

(5) The compressive strength f_{cm} of the concrete at the time of testing should be 70% ± 10% of the specified strength of the concrete f_{ck} of the beams for which the test is designed. This requirement

can be met by using concrete of the specified grade, but testing earlier than 28 days after casting of the specimens.

(6) The yield strength, the tensile strength and the maximum elongation of a representative sample of the shear connector material should be determined.

(7) If profiled steel sheeting is used for the slabs, the tensile strength and the yield strength of the profiled steel sheet should be obtained from coupon tests on specimens cut from the sheets as used in the push tests.

B.2.4 Testing procedure

(1) The load should first be applied in increments up to 40% of the expected failure load and then cycled 25 times between 5% and 40% of the expected failure load.

(2) Subsequent load increments should then be imposed such that failure does not occur in less than 15 minutes.

(3) The longitudinal slip between each concrete slab and the steel section should be measured continuously during loading or at each load increment. The slip should be measured at least until the load has dropped to 20% below the maximum load.

(4) As close as possible to each group of connectors, the transverse separation between the steel section and each slab should be measured.

B.2.5 Test evaluation

(1) If three tests on nominally identical specimens are carried out and the deviation of any individual test result from the mean value obtained from all tests does not exceed 10%, the design resistance may be determined as follows:

- the characteristic resistance P_{Rk} should be taken as the minimum failure load (divided by the number of connectors) reduced by 10%;
- the design resistance P_{Rd} should be calculated from:

$$P_{Rd} = \frac{f_u}{f_{ut}} \frac{P_{Rk}}{\gamma_V} \leq \frac{P_{Rk}}{\gamma_V} \qquad \text{(B.1)}$$

where:

f_u is the minimum specified ultimate strength of the connector material;

f_{ut} is the actual ultimate strength of the connector material in the test specimen; and

γ_V is the partial safety factor for shear connection.

Note: The value for γ_V may be given in the National Annex. The recommended value for γ_V is 1,25.

(2) If the deviation from the mean exceeds 10%, at least three more tests of the same kind should be made. The test evaluation should then be carried out in accordance with EN 1990, Annex D.

(3) Where the connector is composed of two separate elements, one to resist longitudinal shear and the other to resist forces tending to separate the slab from the steel beam, the ties which resist separation shall be sufficiently stiff and strong so that separation in push tests, measured when the

connectors are subjected to 80 % of their ultimate load, is less than half of the longitudinal movement of the slab relative to the beam.

(4) The slip capacity of a specimen δ_u should be taken as the maximum slip measured at the characteristic load level, as shown in Figure B.2. The characteristic slip capacity δ_{uk} should be taken as the minimum test value of δ_u reduced by 10% or determined by statistical evaluation from all the test results. In the latter case, the characteristic slip capacity should be determined in accordance with EN 1990, Annex D.

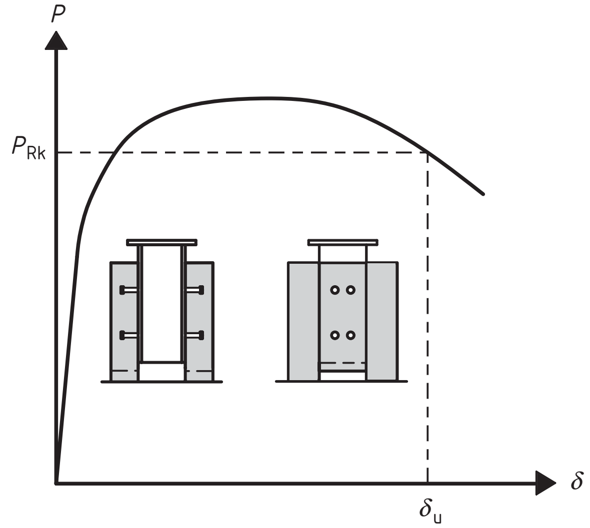


Figure B.2 : Determination of slip capacity δ_u

B.3 Testing of composite floor slabs

B.3.1 General

(1) Tests according to this section should be used for the determination of the factors *m* and *k* or the value of $\tau_{u,Rd}$ to be used for the verification of the resistance to longitudinal shear as given in Section 9.

(2) From the load-deflection curves the longitudinal shear behaviour is to be classified as brittle or ductile. The behaviour is deemed to be ductile if it is in accordance with 9.7.3(3). Otherwise the behaviour is classified as brittle.

(3) The variables to be investigated include the thickness and the type of steel sheeting, the steel grade, the coating of the steel sheet, the density and grade of concrete, the slab thickness and the shear span length L_s.

(4) To reduce the number of tests as required for a complete investigation, the results obtained from a test series may be used also for other values of variables as follows:

- for thickness of the steel sheeting t larger than tested;
- for concrete with specified strength f_{ck} not less than 0,8 f_{cm} , where f_{cm} is the mean value of the concrete strength in the tests;
- for steel sheeting having a yield strength f_{yp} not less than 0,8 f_{ypm}, where f_{ypm} is the mean value of the yield strength in the tests.

(3)B For building components under compression a minimum toughness property should be selected.

NOTE B The National Annex may give information on the selection of toughness properties for members in compression. The use of Table 2.1 of EN 1993-1-10 for $\sigma_{Ed} = 0,25\ f_y(t)$ is recommended.

(4) For selecting steels for members with hot dip galvanized coatings see EN 1461.

Table 3.1: Nominal values of yield strength f_y and ultimate tensile strength f_u for hot rolled structural steel

Standard and steel grade	Nominal thickness of the element t [mm]			
	$t \leq 40$ mm		40 mm $< t \leq 80$ mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
EN 10025-2				
S 235	235	360	215	360
S 275	275	430	255	410
S 355	355	510	335	470
S 450	440	550	410	550
EN 10025-3				
S 275 N/NL	275	390	255	370
S 355 N/NL	355	490	335	470
S 420 N/NL	420	520	390	520
S 460 N/NL	460	540	430	540
EN 10025-4				
S 275 M/ML	275	370	255	360
S 355 M/ML	355	470	335	450
S 420 M/ML	420	520	390	500
S 460 M/ML	460	540	430	530
EN 10025-5				
S 235 W	235	360	215	340
S 355 W	355	510	335	490
EN 10025-6				
S 460 Q/QL/QL1	460	570	440	550

Table 3.1 (continued): Nominal values of yield strength f_y and ultimate tensile strength f_u for structural hollow sections

Standard and steel grade	Nominal thickness of the element t [mm]			
	$t \leq 40$ mm		40 mm $< t \leq 80$ mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
EN 10210-1				
S 235 H	235	360	215	340
S 275 H	275	430	255	410
S 355 H	355	510	335	490
S 275 NH/NLH	275	390	255	370
S 355 NH/NLH	355	490	335	470
S 420 NH/NHL	420	540	390	520
S 460 NH/NLH	460	560	430	550
EN 10219-1				
S 235 H	235	360		
S 275 H	275	430		
S 355 H	355	510		
S 275 NH/NLH	275	370		
S 355 NH/NLH	355	470		
S 460 NH/NLH	460	550		
S 275 MH/MLH	275	360		
S 355 MH/MLH	355	470		
S 420 MH/MLH	420	500		
S 460 MH/MLH	460	530		

3.2.4 Through-thickness properties

(1) Where steel with improved through-thickness properties is necessary according to EN 1993-1-10, steel according to the required quality class in EN 10164 should be used.

NOTE 1 Guidance on the choice of through-thickness properties is given in EN 1993-1-10.

NOTE 2B Particular care should be given to welded beam to column connections and welded end plates with tension in the through-thickness direction.

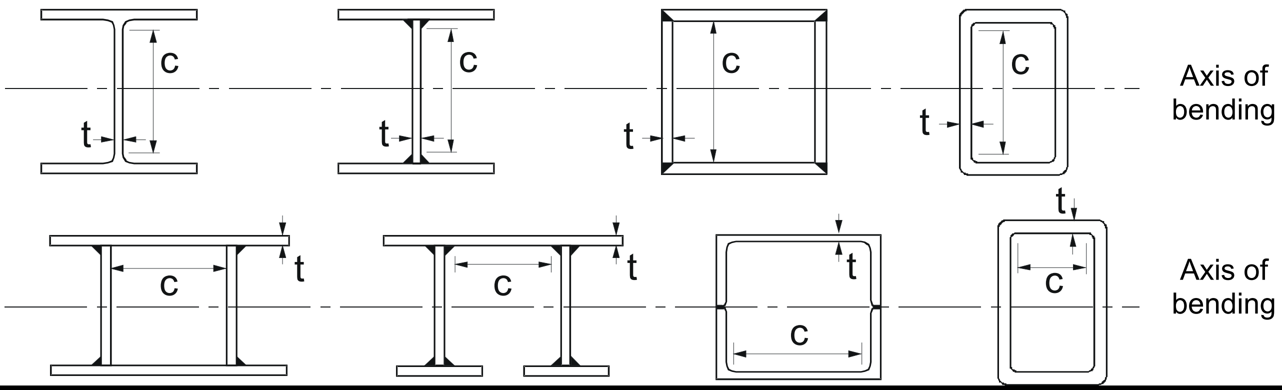
NOTE 3B The National Annex may give the relevant allocation of target values Z_{Ed} according to 3.2(2) of EN 1993-1-10 to the quality class in EN 10164. The allocation in Table 3.2 is recommended for buildings:

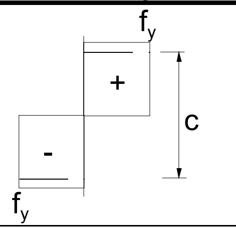
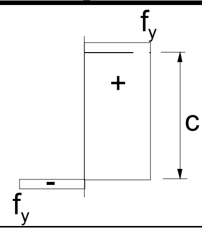
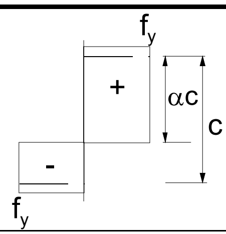
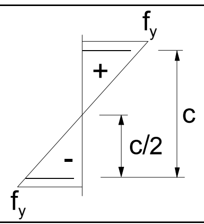
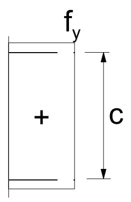
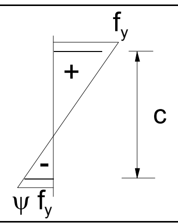
Table 3.2: Choice of quality class according to EN 10164

Target value of Z_{Ed} according to EN 1993-1-10	Required value of Z_{Rd} expressed in terms of design Z-values according to EN 10164
$Z_{Ed} \leq 10$	—
$10 < Z_{Ed} \leq 20$	Z 15
$20 < Z_{Ed} \leq 30$	Z 25
$Z_{Ed} > 30$	Z 35

Table 5.2 (sheet 1 of 3): Maximum width-to-thickness ratios for compression parts

Internal compression parts



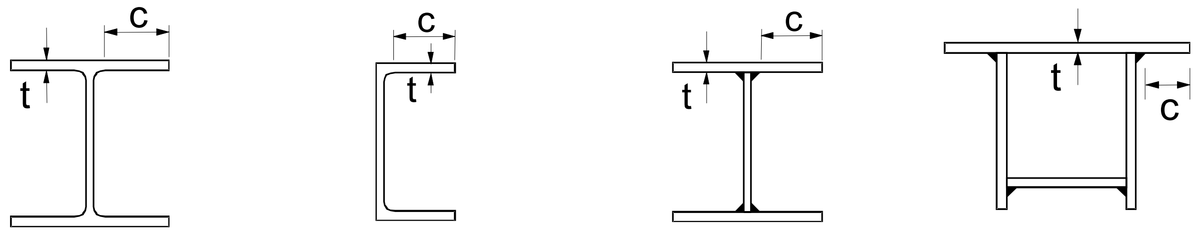
Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \leq 72\varepsilon$	$c/t \leq 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \leq \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \leq 0{,}5$: $c/t \leq \dfrac{36\varepsilon}{\alpha}$
2	$c/t \leq 83\varepsilon$	$c/t \leq 38\varepsilon$	when $\alpha > 0{,}5$: $c/t \leq \dfrac{456\varepsilon}{13\alpha - 1}$ when $\alpha \leq 0{,}5$: $c/t \leq \dfrac{41{,}5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)			
3	$c/t \leq 124\varepsilon$	$c/t \leq 42\varepsilon$	when $\psi > -1$: $c/t \leq \dfrac{42\varepsilon}{0{,}67 + 0{,}33\psi}$ when $\psi \leq -1$*): $c/t \leq 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

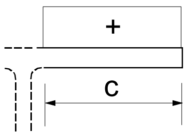
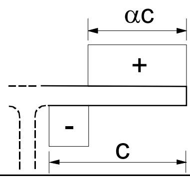
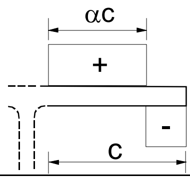
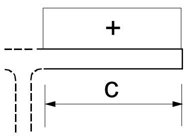
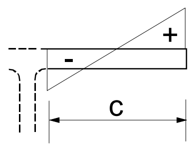
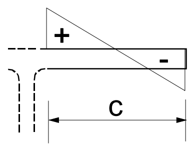
*) $\psi \leq -1$ applies where either the compression stress $\sigma \leq f_y$ or the tensile strain $\varepsilon_y > f_y/E$

Table 5.2 (sheet 2 of 3): Maximum width-to-thickness ratios for compression parts

Outstand flanges



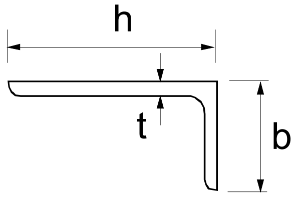
Rolled sections — Welded sections

Class	Part subject to compression	Part subject to bending and compression	
		Tip in compression	Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \leq 9\varepsilon$	$c/t \leq \dfrac{9\varepsilon}{\alpha}$	$c/t \leq \dfrac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \leq 10\varepsilon$	$c/t \leq \dfrac{10\varepsilon}{\alpha}$	$c/t \leq \dfrac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in parts (compression positive)			
3	$c/t \leq 14\varepsilon$	$c/t \leq 21\varepsilon\sqrt{k_\sigma}$ For k_σ see EN 1993-1-5	

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

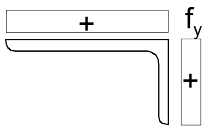
Table 5.2 (sheet 3 of 3): Maximum width-to-thickness ratios for compression parts

Angles

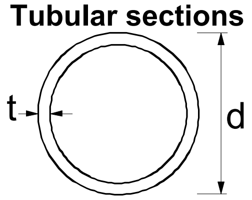


Refer also to "Outstand flanges" (see sheet 2 of 3)

Does not apply to angles in continuous contact with other components

Class	Section in compression
Stress distribution across section (compression positive)	
3	$h/t \le 15\varepsilon : \dfrac{b+h}{2t} \le 11{,}5\varepsilon$

Tubular sections



Class	Section in bending and/or compression
1	$d/t \le 50\varepsilon^2$
2	$d/t \le 70\varepsilon^2$
3	$d/t \le 90\varepsilon^2$ **NOTE** For $d/t > 90\varepsilon^2$ see EN 1993-1-6.

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71
	ε^2	1,00	0,85	0,66	0,56	0,51

Bracing systems that supply positional restraint to more than one member should be designed to resist the sum of the restraint forces from each member that they restrain, reduced by the factor k_r obtained from:

$$k_r = (0.2 + 1/N_r)^{0.5}$$

in which N_r is the number of parallel members restrained.

4.7.2 Slenderness

The slenderness λ of a compression member should generally be taken as its effective length L_E divided by its radius of gyration r about the relevant axis, except as given in **4.7.9**, **4.7.10** or **4.7.13**.

In the case of a single-angle strut with lateral restraints to its two legs alternately, the slenderness for buckling about every axis should be increased by 20 %.

4.7.3 Effective lengths

Except for angles, channels or T-sections designed in accordance with **4.7.10** the effective length L_E of a compression member should be determined from the segment length L centre-to-centre of restraints or intersections with restraining members in the relevant plane as follows.

a) Generally, in accordance with Table 22, depending on the conditions of restraint in the relevant plane, members carrying more than 90 % of their reduced plastic moment capacity M_r in the presence of axial force (see **I.2**) being taken as incapable of providing directional restraint.

b) For continuous columns in multistorey buildings of simple design, in accordance with Table 22, depending on the conditions of restraint in the relevant plane, directional restraint being based on connection stiffness as well as member stiffness.

c) For compression members in trusses, lattice girders or bracing systems, in accordance with Table 22, depending on the conditions of restraint in the relevant plane.

d) For columns in single storey buildings of simple design, see **D.1**.

e) For columns supporting internal platform floors of simple design, see **D.2**.

f) For columns forming part of a continuous structure, see Annex E.

Table 22 — Nominal effective length L_E for a compression member[a]

a) non-sway mode			
Restraint (in the plane under consideration) by other parts of the structure			L_E
Effectively held in position at both ends	Effectively restrained in direction at both ends		$0.7L$
	Partially restrained in direction at both ends		$0.85L$
	Restrained in direction at one end		$0.85L$
	Not restrained in direction at either end		$1.0L$
b) sway mode			
One end	Other end		L_E
Effectively held in position and restrained in direction	Not held in position	Effectively restrained in direction	$1.2L$
		Partially restrained in direction	$1.5L$
		Not restrained in direction	$2.0L$

[a] Excluding angle, channel or T-section struts designed in accordance with **4.7.10**.