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.

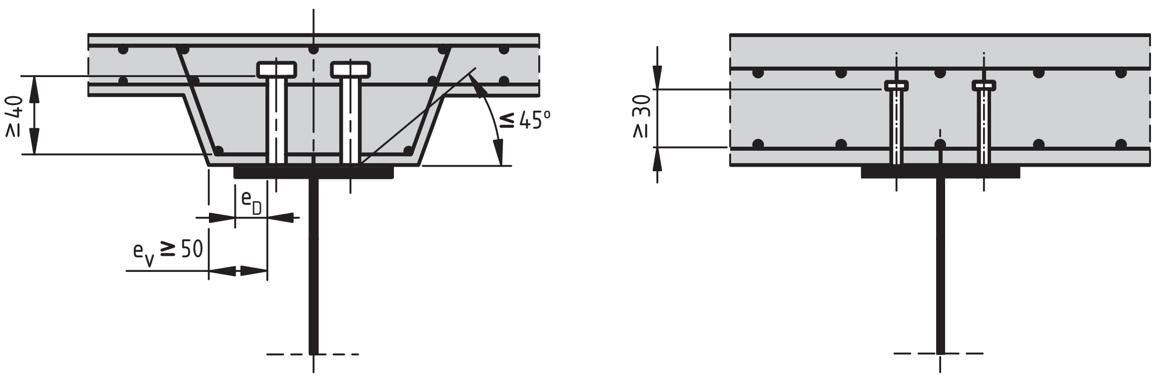


Figure 6.14 : Detailing

(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 AC› Class 3 or Class 4 ‹AC 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.

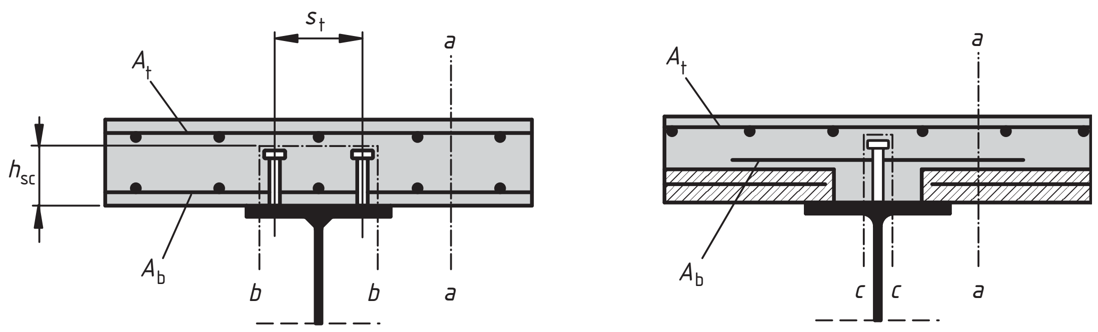
6.6.6.2 Design resistance to longitudinal shear

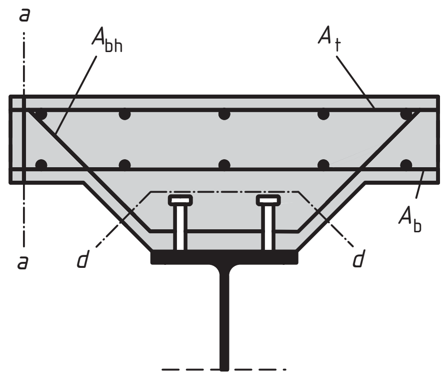
(1) The design shear strength of the concrete flange (shear planes a-a illustrated in Figure 6.15) should be determined in accordance with EN 1992-1-1, 6.2.4.

(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	$\mathbf{A_{sf} / s_f}$
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.

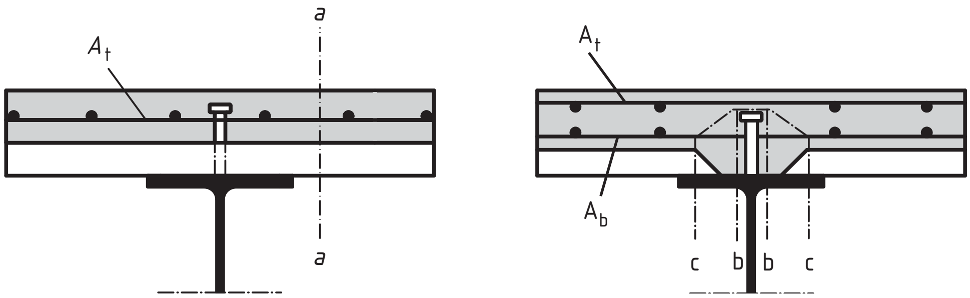
(4) Where profiled steel sheeting with mechanical or frictional interlock and with ribs transverse to the beam is continuous across the top flange of the steel beam, its contribution to the transverse reinforcement for a shear surface of type a-a may be allowed for by replacing expression (6.21) in EN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

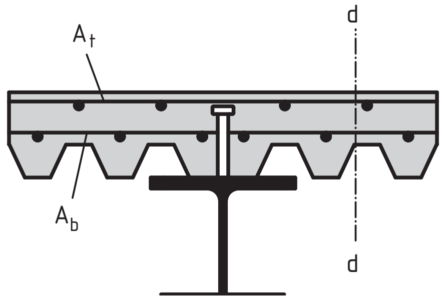
$$(A_{sf} f_{yd} / s_f) + A_{pe} f_{yp,d} > v_{Ed} h_f / \cot\theta \qquad (6.25)$$

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 AC⟩ $A_{pe} f_{yp,d}$ ⟨AC in expression (6.25) should be replaced by:

$$P_{pb,Rd} / s \quad \text{but} \quad \leq \text{AC⟩} \; A_{pe} f_{yp,d} \; \text{⟨AC} \tag{6.26}$$

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.

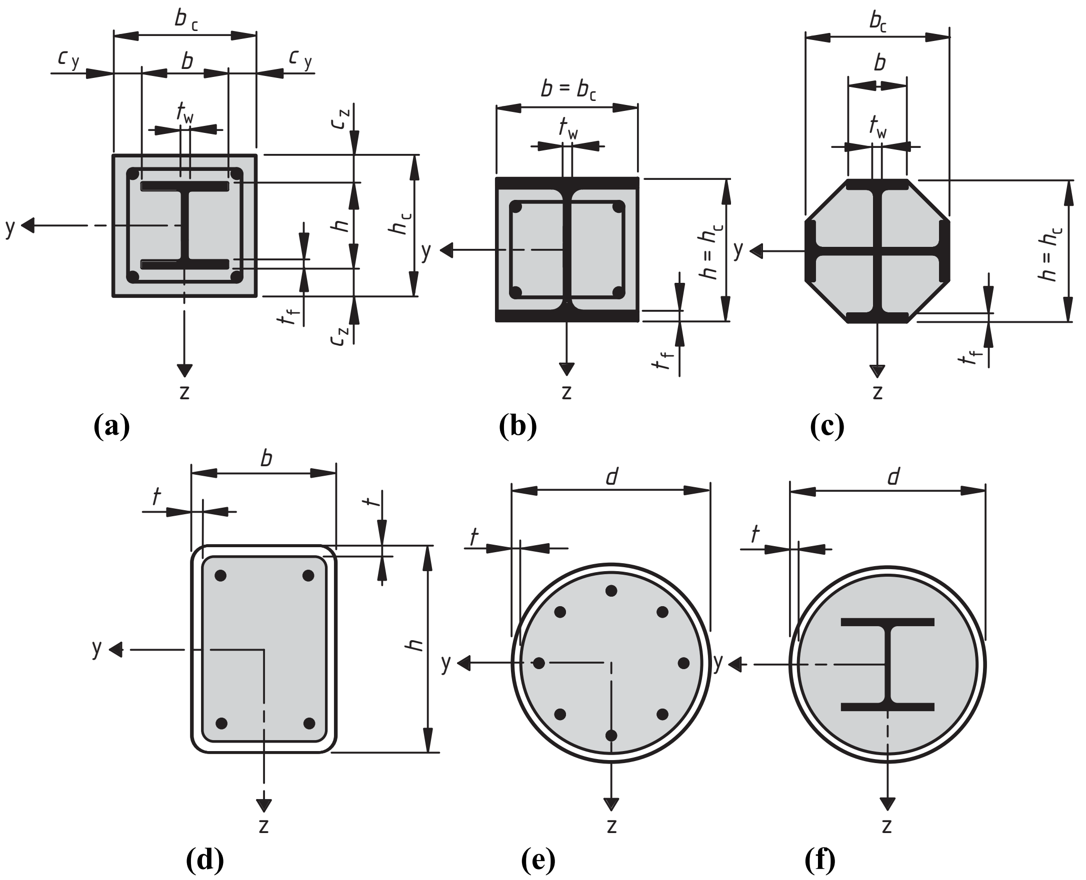


Figure 6.17 : Typical cross-sections of composite columns and notation

(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:

$$0,2 \leq \delta \leq 0,9 \tag{6.27}$$

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:

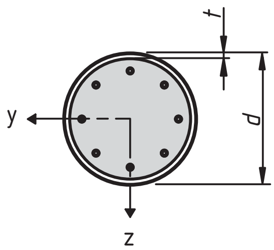
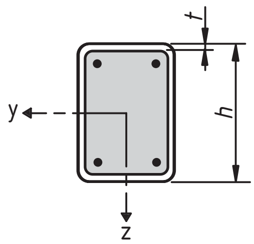
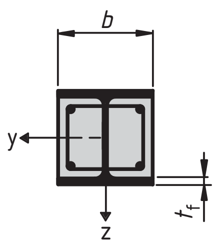
- a general method in 6.7.2 whose scope includes members with non-symmetrical or non-uniform cross-sections over the column length and
- a simplified method in 6.7.3 for members of doubly symmetrical and uniform cross section over the member length.

(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/mm²

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:

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

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.

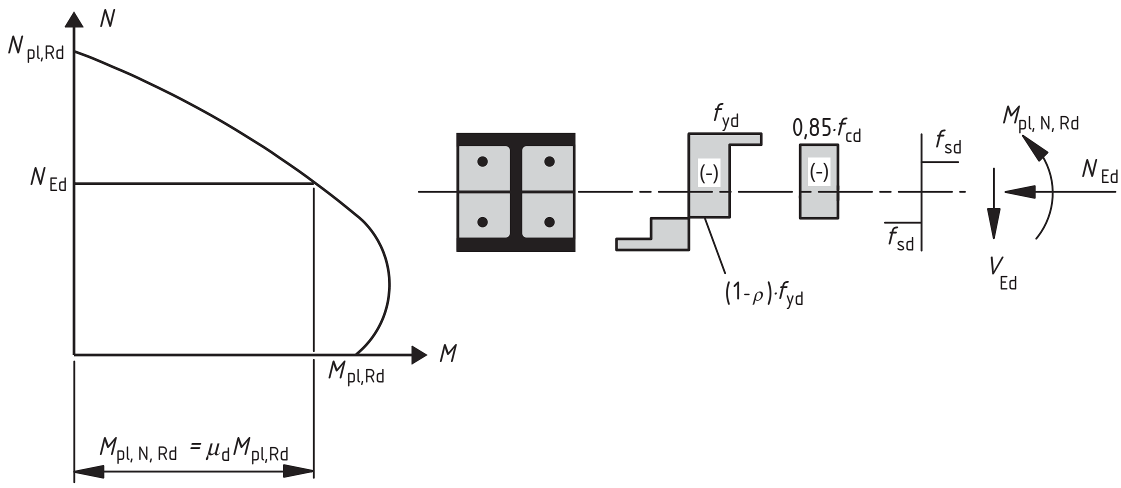


Figure 6.18 : Interaction curve for combined compression and uniaxial bending

(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 :

$$V_{a,Ed} = V_{Ed} \frac{M_{pl,a,Rd}}{M_{pl,Rd}} \qquad (6.31)$$