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MANARA UNIVERSITY

Composite Steel-Concrete Structures
Sem. 2
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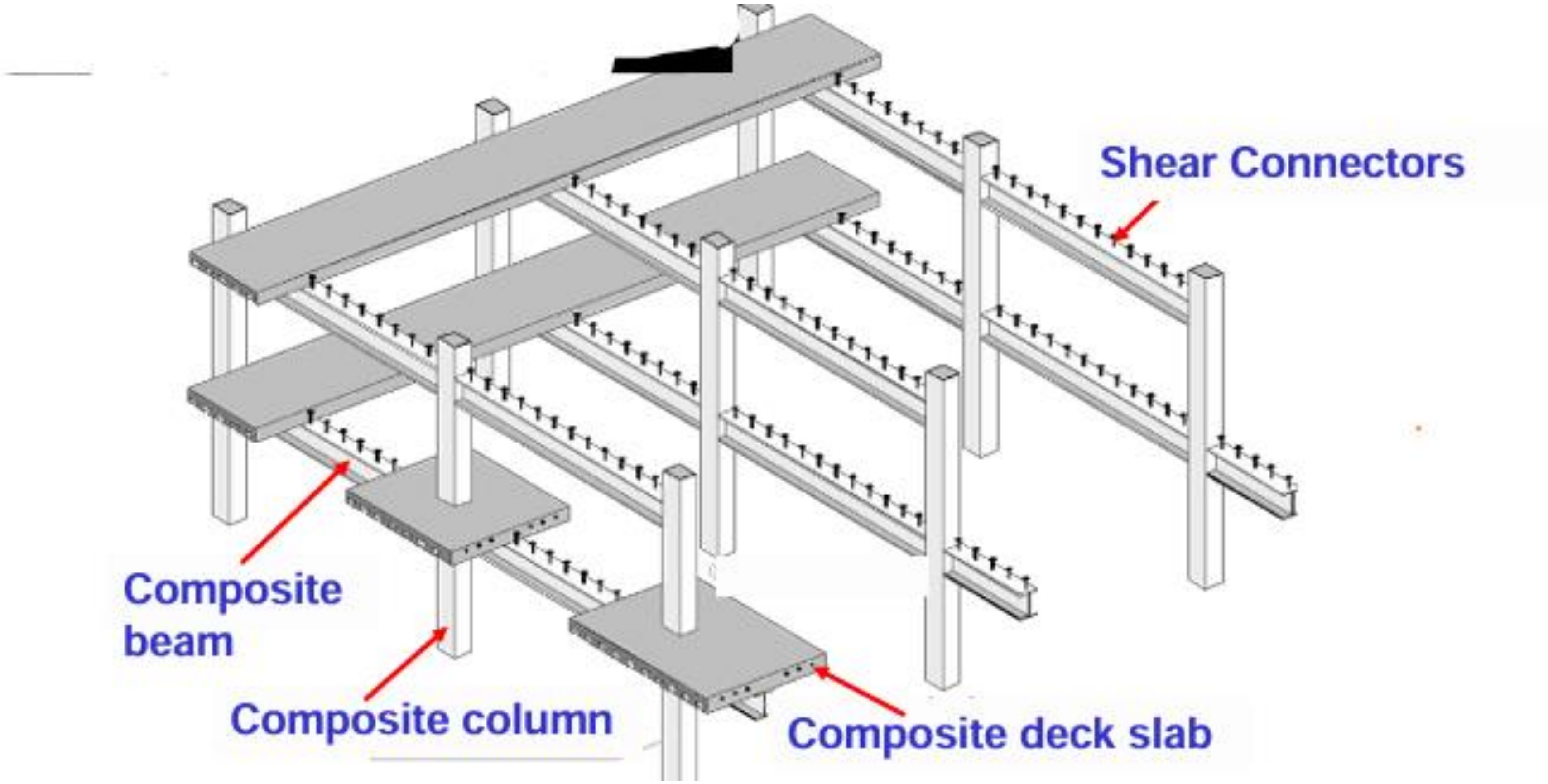
Lecture 5-6

- Properties of composite cross sections con.
- 2 Plastic moment resistance in hogging bending; longitudinal shear not critical
- 3 Elastic moment resistance and flexural stiffness in sagging bending; complete interaction
- 4 Elastic moment resistance and flexural stiffness in hogging bending; complete interaction
- 5 Resistance to vertical shear
- 6 Combined bending and vertical shear
- Partially-encased beams
- Resistance of simply supported composite be


Shear Connectors
Composite beam
Composite column
Composite deck slab

Principles of design

As a starting point for describing the design process a number of key subjects will be clarified, in particular:

1– partial factors;
2– material properties;
3– properties of shear connectors;
4– cross-section classification;
5– effective width.

Properties of composite cross sections

1 Plastic moment resistance in sagging bending;Longitudinal shear not critical

This section considers the plastic moment resistance $M_{pl,Rd}$ for sagging bending, assuming that the longitudinal shear connection is not critical (full shear connection).

The following types of steel sections are considered:

– symmetrical sections made of grade S355 steel or lower;

– asymmetric sections;

– sections of grade S420 or S460 steel.

Asymmetric steel sections

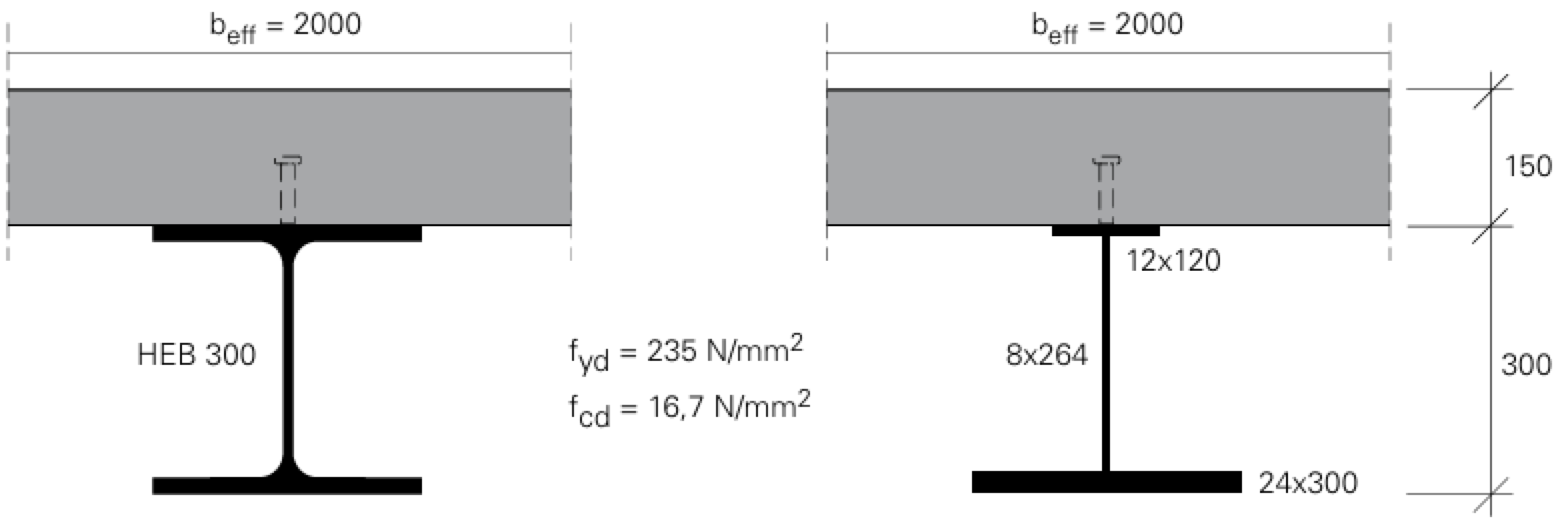
Composite structures 1, section 1.1.1 and figure 1.6, suggests that it may be **efficient to use** an asymmetric steel section because, after **curing** of the concrete, the concrete slab acts as the **top flange of a composite** beam

Figure demonstrates the calculation of $M_{pl,Rd}$ for one cross section with a symmetric hot rolled steel section, and one with a welded asymmetric section

For a very similar moment resistance, the cross section area A – and consequently the weight – of the asymmetric section (10752 mm2) is approximately 30% lower than that of the symmetric section (14900 mm2)

It is worth noting that this does not automatically mean the asymmetric solution is cheaper, because whilst material costs will be lower there will be higher fabrication costs.

Asymmetric steel sections



$$A = 14900 \text{ mm}^2$$

$$N_{pl} = 14900 \cdot 235 \cdot 10^{-3} = 3502 \text{ kN}$$

$$x = \frac{3502}{2{,}0 \cdot 0{,}85 \cdot 16{,}7} = 123 \text{ mm}$$

$$z = 150 + 150 - \frac{1}{2} 123 = 238 \text{ mm}$$

$$M_{pl,Rd} = 3502 \cdot 238 \cdot 10^{-3} = 833 \text{ kNm}$$

$$A = 12 \cdot 120 + 8 \cdot 264 + 24 \cdot 300 = 1440 + 2112 + 7200 = 10752 \text{ mm}^2$$

$$N_{pl} = 10752 \cdot 235 \cdot 10^{-3} = 2527 \text{ kN}$$

$$x = \frac{2527}{2{,}0 \cdot 0{,}85 \cdot 16{,}7} = 89 \text{ mm}$$

n.a.

$$\begin{aligned} M_{pl,Rd} = \; & 7200 \cdot 235 \cdot (450 - 12 - 44{,}5) \cdot 10^{-6} = && 666 \text{ kNm} \\ & + 2112 \cdot 235 \cdot (450 - 24 - \tfrac{1}{2} 264 - 44{,}5) \cdot 10^{-6} = && 124 \text{ kNm} \\ & + 1440 \cdot 235 \cdot (150 + 6 - 44{,}5) \cdot 10^{-6} = && 38 \text{ kNm} \\ & && \overline{828 \text{ kNm}} \end{aligned}$$

Determination of $M_{pl,Rd}$ for a symmetric and an asymmetric steel section.

Sections with grade S420 or S460 steel

When using a steel grade that is higher than S355, the calibration factor of 0,85 that reduces the axial resistance of the concrete component is not always sufficient to justify the use of an ideal plastic concrete stress block (see fig.)

If the depth of the compression zone is:

$0,15h < x_{pl} < 0,4h$,

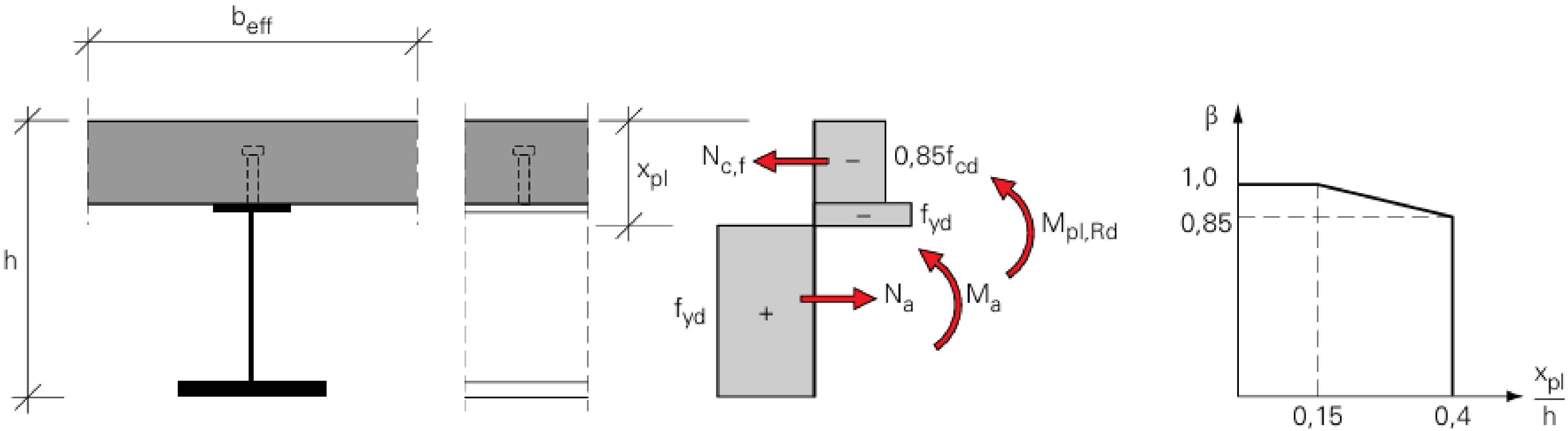
then $M_{pl,Rd}$ must be reduced by a factor β, as shown in figure.

For values of:

xpl > 0,4h,

the use of plastic theory to determine section resistance is not allowed.

Sections with grade S420 or S460 steel



Reduction factor β for Mpl,Rd

2 Plastic moment resistance in hogging bending; longitudinal shear not critical

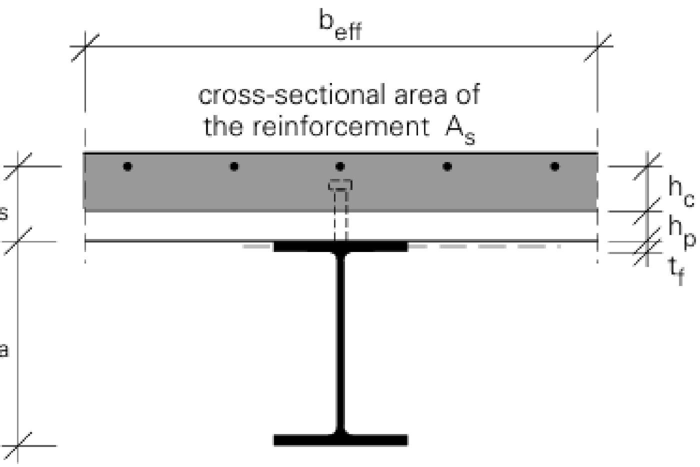
A different calculation model is used to determine the plastic resistance moment in hogging bending $M_{pl,s,Rd}$, because in this case the concrete is in tension and therefore not effective.

Tension stresses in the slab are only resisted by the reinforcement.

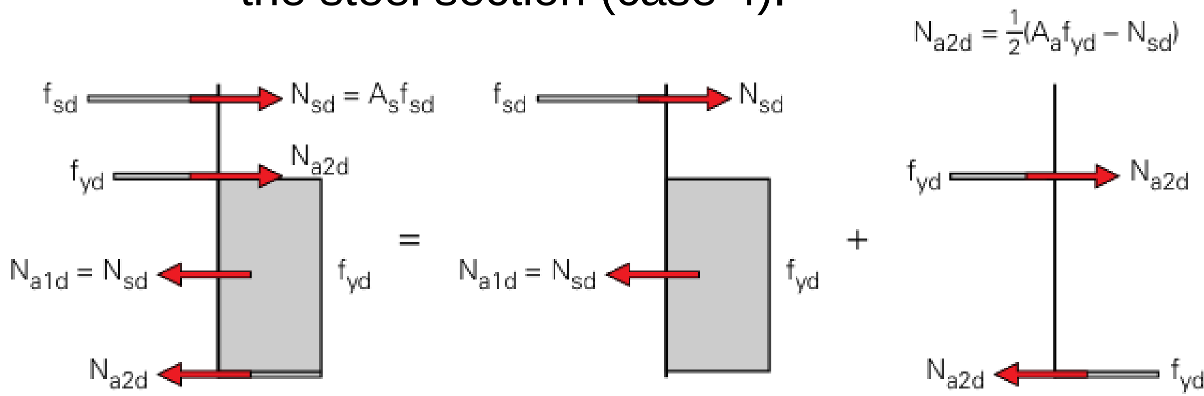
It is assumed that the axial force needed to yield the reinforcement $A_s f_{sd}$ is less than that needed to yield the steel section $A_a f_{yd}$.

Depending on the position of the neutral axis, the resistance of a cross section above a support (i.e. in hogging) is determined as follows.

2 Plastic moment resistance in hogging bending; longitudinal shear not critical



Stress distribution for hogging moment with the plastic neutral axis in the top flange of the steel section (case 4).



Case 4. Neutral axis in the upper flange of the steel section.

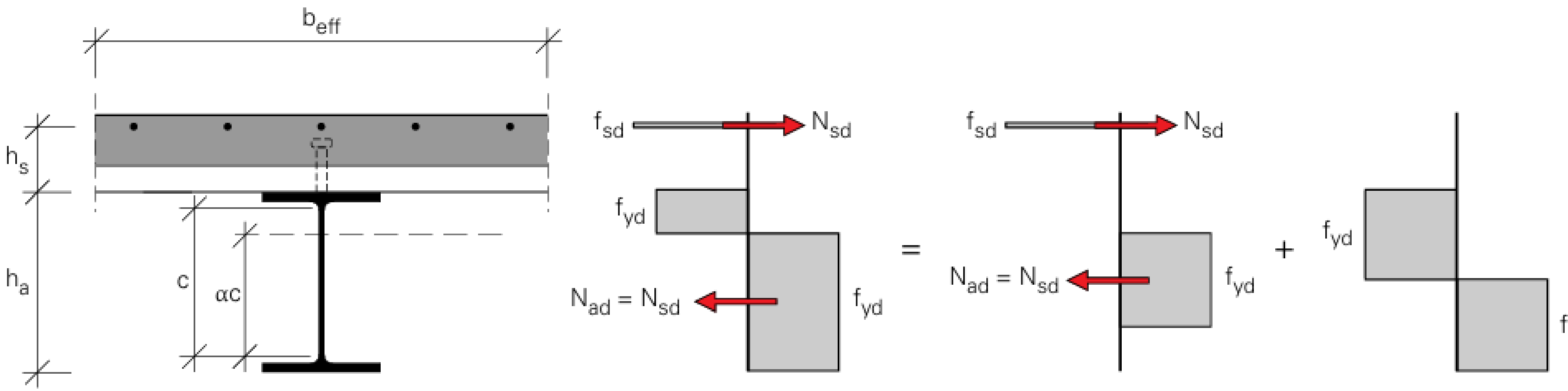
This situation occurs when

$$A_a f_{yd} > A_s f_{sd} > (h_a - 2t_f)t_w f_{yd}.$$

The moment resistance at the intermediate support $M_{pl,s,Rd}$ comprises two components, in which $N_{sd} = A_s f_{sd}$ (fig. 2.21):

$$M_{pl,s,Rd} = N_{sd}\left(\frac{1}{2}h_a + h_s\right) + \frac{1}{2}\left(A_a f_{yd} - N_{sd}\right)\left(h_a - \frac{A_a f_{yd} - N_{sd}}{2b_a f_{yd}}\right) \qquad (2.12)$$

2 Plastic moment resistance in hogging bending; longitudinal shear not critical



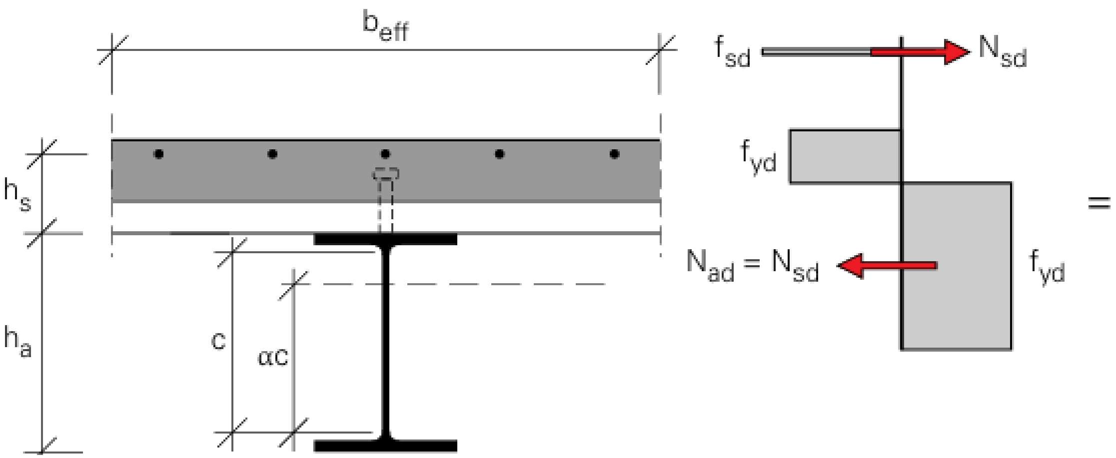
Case 5. Neutral axis in the web of the steel section. This situation occurs when

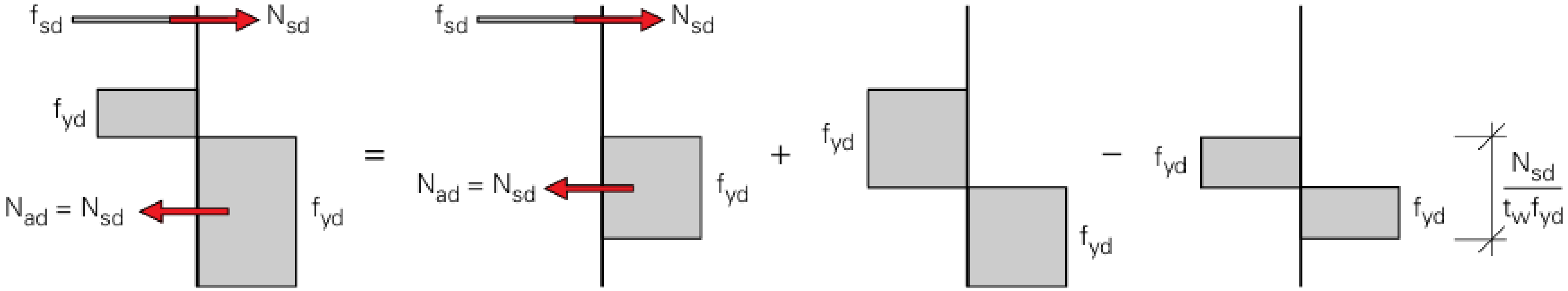
$$A_s f_{sd} < (h_a - 2t_f)t_w f_{yd}.$$

The moment resistance comprises three components (fig.):

$$M_{pl,s,Rd} = N_{sd}\left(\frac{1}{2}h_a + h_s\right) + W_{pl,a} f_{yd} - \frac{N_{sd}^2}{4t_w f_{yd}} \qquad (2.13)$$

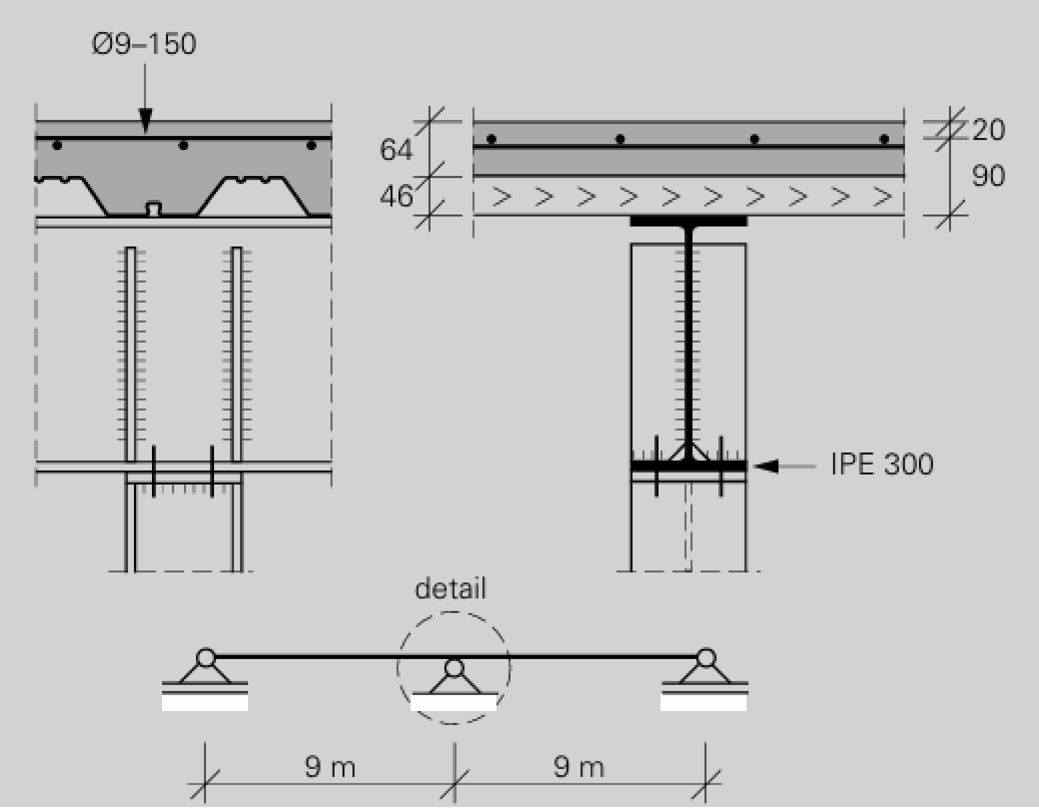
2 Plastic moment resistance in hogging bending; longitudinal shear not critical





Stress distribution for hogging moment with the plastic neutral axis in the web of the steel section (case 5). (2-22)

Example 2.3



Continuous composite beam with two spans.

- *Given*. A two span continuous composite beam, each span being 9 m (fig. 2.23). The concrete is of class C25/30 with f_{cd} = 16,7 N/mm^2. Key dimensions of the composite floor are h_c = 64 mm and h_p = 46 mm. The reinforcement in the concrete slab consists of Ø9 bars at 150 mm centre, in steel B500 grade with f_{sd} = 435 N/mm^2, and with a concrete cover c_s = 15 mm. The steel beam is an IPE 300 in grade S235 steel with f_{yd} = 235 N/mm^2.
- *Question*. Determine the plastic moment resistance $M_{pl,s,Rd}$ above the intermediate support (i.e. in hogging).

Answer: moment resistance $M_{pl,s,Rd}$ at the intermediate support

The effective width of the concrete slab above the intermediate support is given by:

$$b_{eff} = 2 \cdot \frac{L_e}{8} = 2 \cdot \frac{0,25 \cdot (9,0 + 9,0)}{8} = 1,125 \text{ m}$$

This assumes a single row of shear studs, i.e. $b_0 = 0$. The cross sectional area of the reinforcement within the effective width is as follows:

$$A_s = \frac{1}{4}\pi d^2 \cdot \frac{b_{eff}}{s} = \frac{1}{4}\pi \cdot 9^2 \cdot \frac{1125}{150} = 477 \text{ mm}^2$$

The percentage of reinforcement therefore equals:

$$\rho = 100 \cdot \frac{A_s}{A_c} = 100 \cdot \frac{477}{1125 \cdot 64} = 0,66\%$$

The resistance of the reinforcement is:

$$N_{sd} = A_s f_{sd} = 477 \cdot 435 \cdot 10^{-3} = 207 \text{ kN}$$

Example 2.3

To achieve longitudinal equilibrium the depth of the web in compression must be equal to (see fig. 2.22):

$$\frac{N_{sd}}{t_w f_{yd}} = \frac{207 \cdot 10^3}{7{,}1 \cdot 235} = 124 \text{ mm} < h_a - 2t_f$$

This means that the neutral axis is in the web of the steel section (case 5).
The concrete cover to the reinforcement is c_s = 15 mm, so the effective depth of the concrete slab is h_s = 110 – 15 – 5 = 90 mm.
The moment resistance $M_{pl,s,Rd}$ at the intermediate support is given by equation (2.13):

$$M_{pl,s,Rd} = N_{sd}\left(\frac{1}{2}h_a + h_s\right) + W_{pl,a} f_{yd} - \frac{N_{sd}^2}{4 t_w f_{yd}}$$

$$= 207 \cdot \left(\frac{1}{2} \cdot 300 + 90\right) \cdot 10^{-3} + 628 \cdot 235 \cdot 10^{-3} - \frac{207^2}{4 \cdot 7{,}1 \cdot 235}$$

$$= 50 + 148 - 6 = 192 \text{ kNm}$$

Example 2.3

Finally, the cross section should be classified to confirm that the use of plastic theory to determine the moment resistance is justified.

To use plastic theory, the section must be in class 1 or 2.

The bottom flange of the steel section, which is in compression, must satisfy (see EN 199311, table 5.2, page 2):

$$\frac{c}{t} = \frac{b - t_w - 2r}{2t_f} = \frac{150 - 7,1 - 2\cdot 15}{2\cdot 10,7} = 5,3 < 9\varepsilon = 9\cdot 1 = 9$$

The bottom flange is therefore class 1. Classification of the part of the web in compression is as follows (see fig. 2.22):

$$c = h_a - 2\left(t_f + r\right) = 300 - 2\cdot\left(10,7 + 15\right) = 249 \text{ mm}$$

$$\alpha c = 0,5\left(c + \frac{N_{sd}}{t_w f_{yd}}\right) = 0,5\cdot\left(249 + 124\right) = 187 \quad \Rightarrow \quad \alpha = \frac{187}{249} = 0,75$$

The web must satisfy (from EN 199311, table 5.2, page 1):

The web therefore also falls into class 1, which means that the section is class 1, and the use of plastic theory is justified.

3 Elastic moment resistance and flexural stiffness in sagging bending; complete interaction

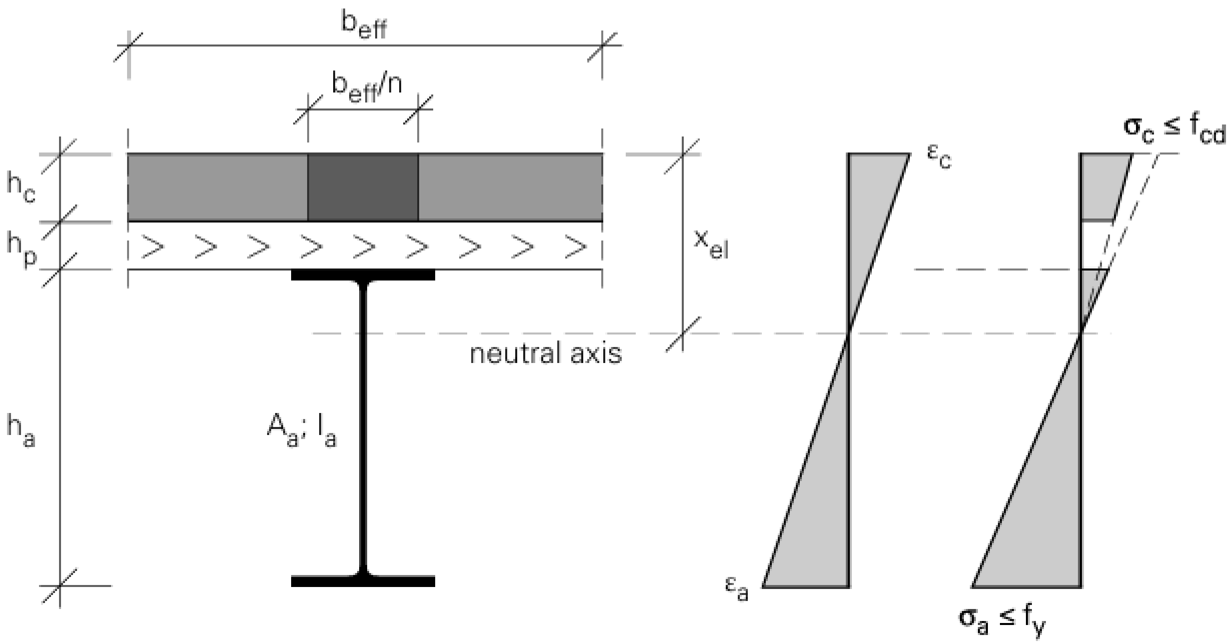
To calculate the elastic moment resistance $M_{el,Rd}$, the steel concrete cross section is replaced by an **equivalent all steel section** with an upper flange width of b_e/n.

The variable n is the modular ratio; that is the ratio between the modulus of elasticity of structural steel E_a and the modulus of elasticity of concrete E_{cm} (see section 2.1.2). The stresses in the concrete slab may be calculated by dividing the stresses in the equivalent steel section by n. $n = E_a / E_{cm}$

Strain and stress distributions are depicted in figure. The position of the neutral axis depends on the dimensions of the composite section.

If the neutral axis is in the concrete slab, the part of the concrete section in tension (the part below the neutral axis) should be ignored.

This simplification has a negligible impact on the bending resistance and stiffness. It means that the concrete section is assumed to be cracked for calculation purposes.



Elastic strain and stress distributions.

The position of the neutral axis is given by:

$$x_{el} = h - \frac{\frac{1}{2}A_a h_a + \frac{1}{n} b_{eff} h_c \left(h_a + h_p + \frac{1}{2} h_c \right)}{A_a + \frac{1}{n} b_{eff} h_c} \qquad (2.14)$$

The second moment of area is calculated by:

$$I_c = \frac{A_a \left(h_c + 2h_p + h_a \right)^2}{4\left(1 + nr\right)} + \frac{b_{eff} h_c^3}{12n} + I_a \quad \text{with} \quad r = \frac{A_a}{b_{eff} h_c} \qquad (2.15)$$

4 Elastic moment resistance and flexural stiffness in hogging bending; complete interaction

The concrete section, which is all in tension under hogging moment, is considered to be cracked and therefore ignored. Only the steel section and reinforcement remain, and they have the same modulus of elasticity.

Figure shows the distributions of stress and strain.

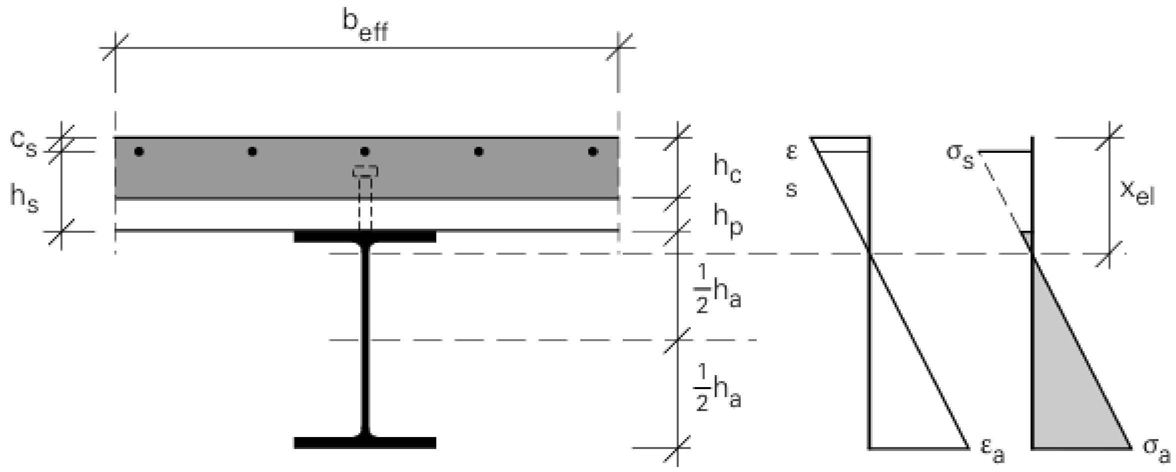
The position of the neutral axis is given by:

$$x_{el} = \frac{A_a\left(\frac{1}{2}h_a + h_p + h_c\right) + A_s c_s}{A_a + A_s} \qquad (2.16)$$

The second moment of area is determined by:

$$I_2 = I_a + A_a\left(\frac{1}{2}h_a + h_p + h_c - x_{el}\right)^2 + A_s\left(x_{el} - c_s\right)^2 \qquad (2.17)$$

4 Elastic moment resistance and flexural stiffness in hogging bending; complete interaction



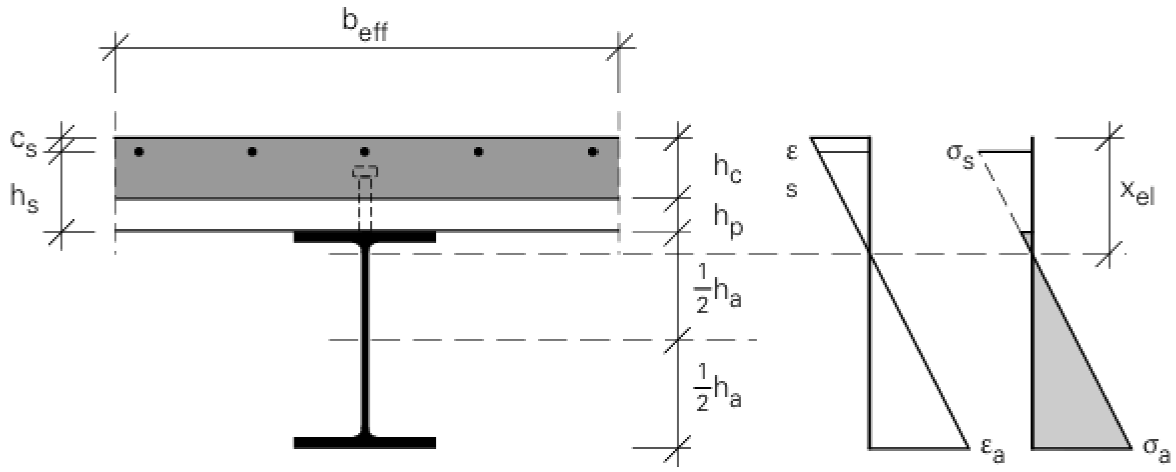
The position of the neutral axis is given by:

$$x_{el} = \frac{A_a\left(\frac{1}{2}h_a + h_p + h_c\right) + A_s c_s}{A_a + A_s} \qquad (2.16)$$

The second moment of area is determined by:

$$I_2 = I_a + A_a\left(\frac{1}{2}h_a + h_p + h_c - x_{el}\right)^2 + A_s\left(x_{el} - c_s\right)^2 \qquad (2.17)$$

4 Elastic moment resistance and flexural stiffness in hogging bending; complete interaction



The position of the neutral axis is given by:

$$x_{el} = \frac{A_a\left(\frac{1}{2}h_a + h_p + h_c\right) + A_s c_s}{A_a + A_s} \qquad (2.16)$$

The second moment of area is determined by:

$$I_2 = I_a + A_a\left(\frac{1}{2}h_a + h_p + h_c - x_{el}\right)^2 + A_s\left(x_{el} - c_s\right)^2 \qquad (2.17)$$

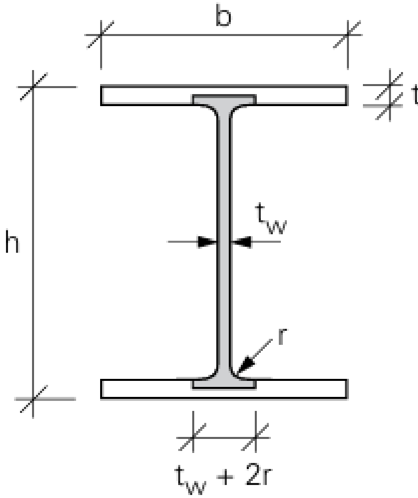
5 Resistance to vertical shear

Any contribution from the concrete slab to the vertical shear resistance of the composite section is normally neglected,

so the shear force is generally assumed to be carried by the steel web alone. EN 199411, cl. 6.2.2.2 does allow inclusion of the contribution of the concrete in the resistance if this contribution has specifically been established.

In the absence of a bending moment the design plastic shear resistance $V_{pl,Rd}$ according to EN 199311, cl. 6.2.6(2) is given by:

$$V_{pl,Rd} = \frac{A_v f_{yd}}{\sqrt{3}} \qquad (2.18)$$



Where A_v is the shear area, which may be assumed equal to the area of the web, or for rolled sections a slightly greater area may be taken $A_v = A - 2bt_f + (t_w + 2r)t_f$

An approximation is $A_v = 1{,}04ht_w$.

For slender uncased webs with **$h_w/t_w > 72$** (EN 199311, cl. 6.2.6) the shear buckling resistance $V_{b,Rd}$ should be assessed according to EN 1993-1-5. Maximum available cross sectional area of a rolled section to resist vertical shear force.

Maximum available cross sectional area of a rolled section to resist vertical shear force.

6 Combined bending and vertical shear

At an internal support of a continuous composite beam, the cross section is always subjected to combined (hogging) bending and a relatively large vertical shear force.

Combined bending and shear could also be critical for a simply supported beam subject to concentrated load(s).

The critical combination of vertical shear and bending may be determined in three steps.

Step 1. Determine the maximum plastic resistance to vertical shear $V_{pl,Rd}$ and, if necessary, shear buckling $V_{b,Rd}$ in the absence of any bending moment (see section.5).

Step 2. Determine the applied vertical shear force V_{Ed}. If $\mathbf{V_{Ed} \leq 0,5V_{pl,Rd}}$, or where relevant $\mathbf{V_{Ed} \leq 0,5V_{b,Rd}}$, it is not necessary to take into account any interaction between shear and bending.

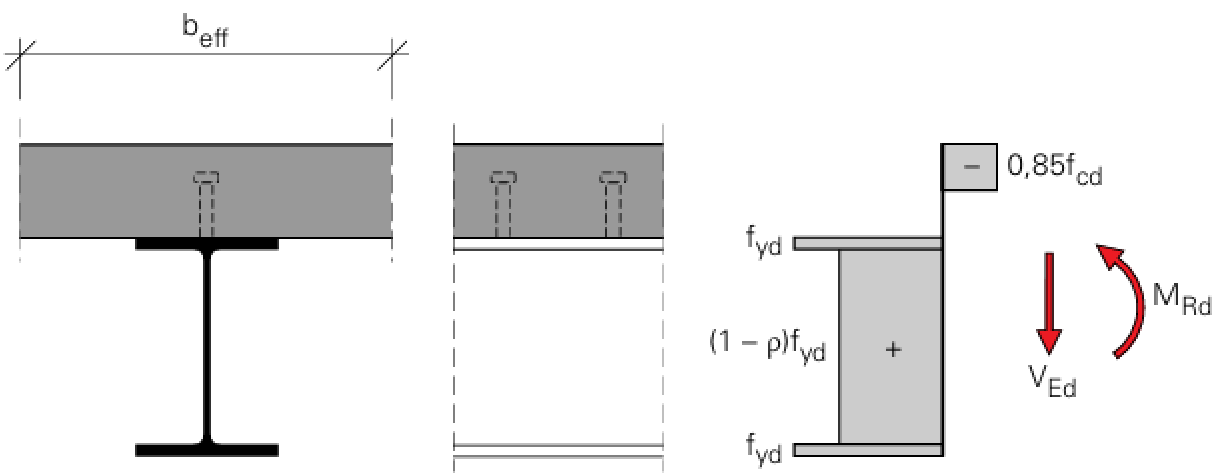
6 Combined bending and vertical shear

Step 3. If the step 2 check(s) concludes that interaction between shear and bending should be taken into account, EN 1994-1-1, cl. 6.2.2.4(2) should be applied.

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

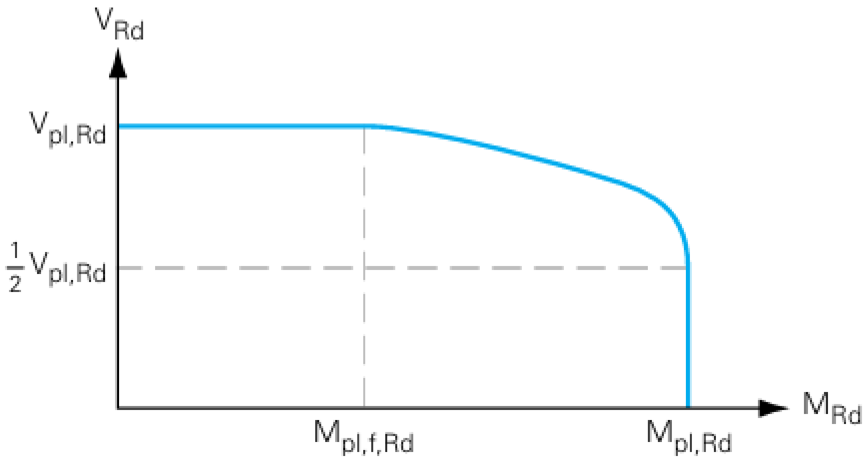
$$\rho = \left(\frac{2V_{Ed}}{V_{Rd}} - 1\right)^2 \qquad (2.19)$$

Adjustment of plastic stress distribution in bending due to the influence of vertical shear force.



Where V is the appropriate resistance to vertical shear, either $V_{pl,Rd}$ or $V_{b,Rd}$. The interaction between vertical shear V_{Rd} and bending moment M_{Rd} is shown graphically in figure. Here $M_{pl,f,Rd}$ is the maximum plastic moment resistance of the section neglecting any contribution from the web.

6 Combined bending and vertical shear



Interaction diagram between bending moment and vertical shear.

Class 3 and 4 sections should be assessed using a typical stress check according to elastic theory. Calculation rules from EN 1993 apply.

7 Partially-encased beams

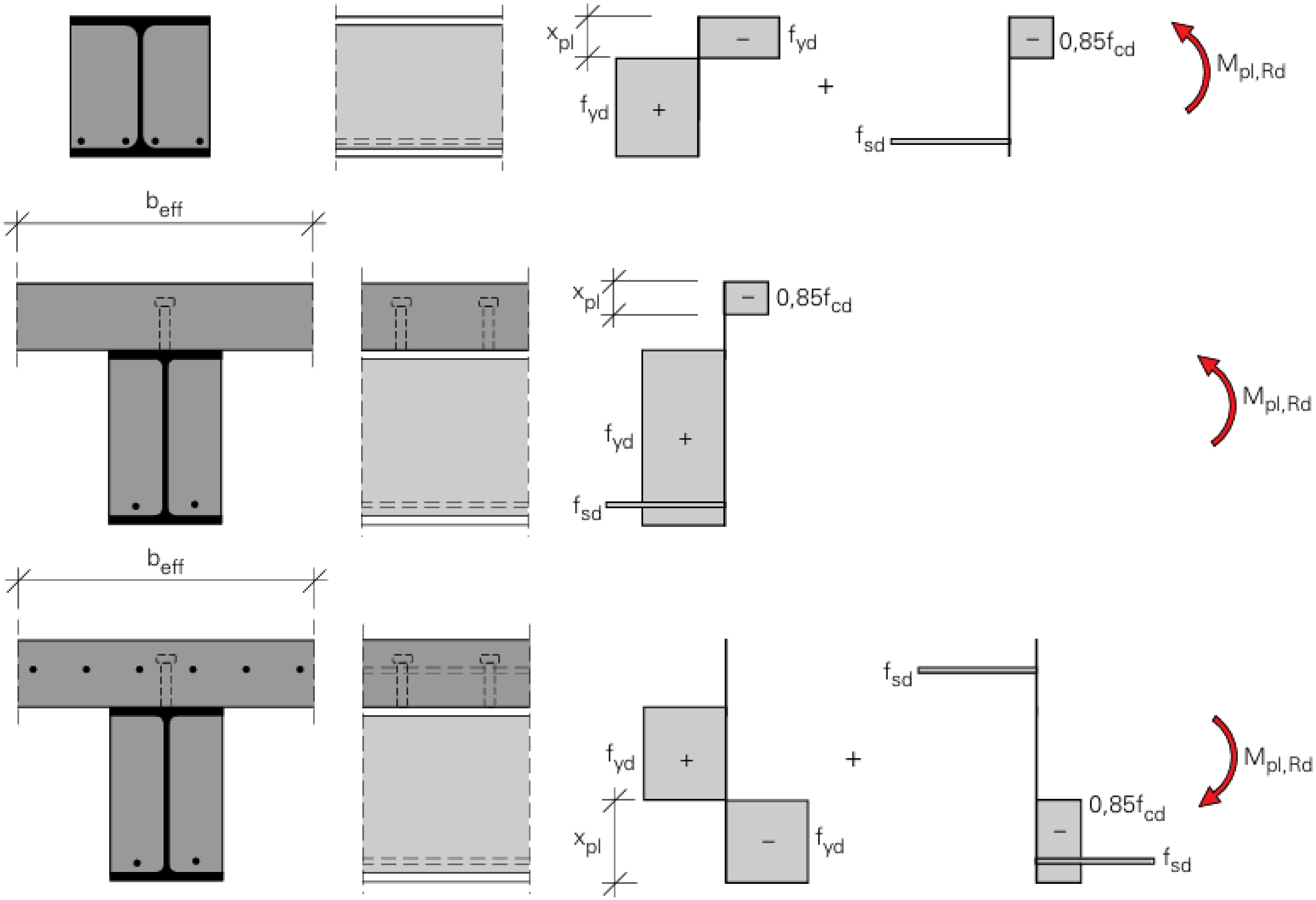
The same principles adopted for unencased beams may be used to calculate the plastic moment resistance of partially encased beams, see section1.

Figure shows some examples of plastic stress distributions for partially encased beams. From the plastic stress distribution, $M_{pl,Rd}$ may be determined by considering equilibrium.

The shear resistance of a partially encased steel section is determined as explained in section.5.

If sufficient stirrups are used, the contribution of the web encasement to shear may be taken into account according to EN 199411, cl. 6.3.3(2).

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



Examples of the plastic stress distribution for the effective cross section of partially encased beams.

Resistance of simply supported composite beams

This section covers the assessment of simply supported composite beams.

The following subjects are dealt with:

1– criteria for the assessment;

2– assessment of moment resistances;

3– assessment of longitudinal shear; full shear connection;

4– assessment of longitudinal shear; partial shear connection.

Two distinct cases of partial shear connection (case 6 and 7) can be identified. Calculation of the cross section resistance is explained for both cases.

1 Criteria for the assessment

The resistance of a simply supported composite beam depends on three different criteria, corresponding to three critical failure zones (fig.).

I: sagging moment resistance.

The beam resistance is determined by the optimal stress distri bution in the critical cross section, as described in section 2.2 for the calculation of the moment resistance.

This optimum stress distribution can only be achieved when the shear studs at the interface between steel and concrete can transmit sufficient longitudinal shear force.

This is known as a beam with full shear connection.

1 Criteria for the assessment

II: longitudinal shear resistance.

A minimum number of shear connectors (in table 2.31 indicated as 100% connectors) is required to ensure that the optimal stress distribution can be developed in the critical cross section. The moment resistance does not increase when more connectors are applied (because either the concrete would crush or the steel section would yield).

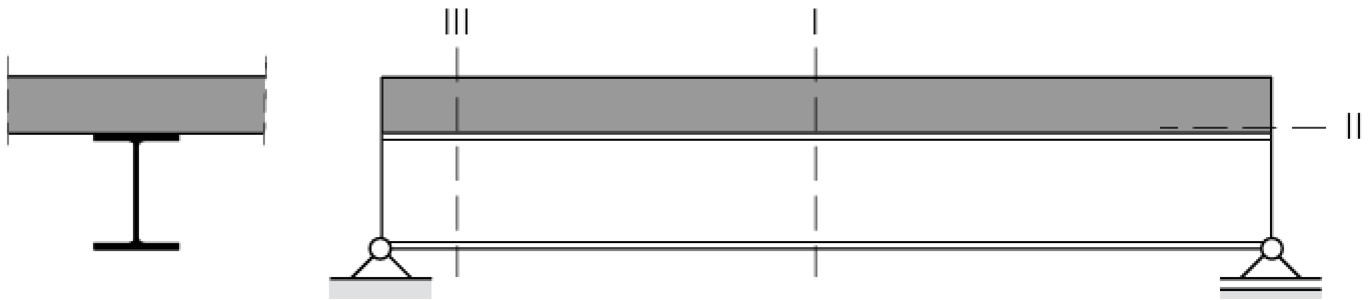
However, if fewer connectors are used the moment resistance decreases, and so the beam resistance decreases, depending on the number of shear connectors.

This is known as a beam with partial shear connection (with >100% connectors).

1 Criteria for the assessment

III: vertical shear resistance.

This criterion is discussed in section 5.



Failure zones of a simply supported (statically determinate) composite beam.

property of connectors	classification	characteristic
strength	full shear connection partial shear connection	100% connectors <100% connectors
stiffness	complete interaction incomplete interaction	no slip slip

Classification system for longitudinal shear connection.

1 Criteria for the assessment

The terms full and partial shear connection refer to the total resi stance of the connectors in shear plane II (see fig.)

However, the behaviour of a composite beam also depend on the stiffness of the connectors.

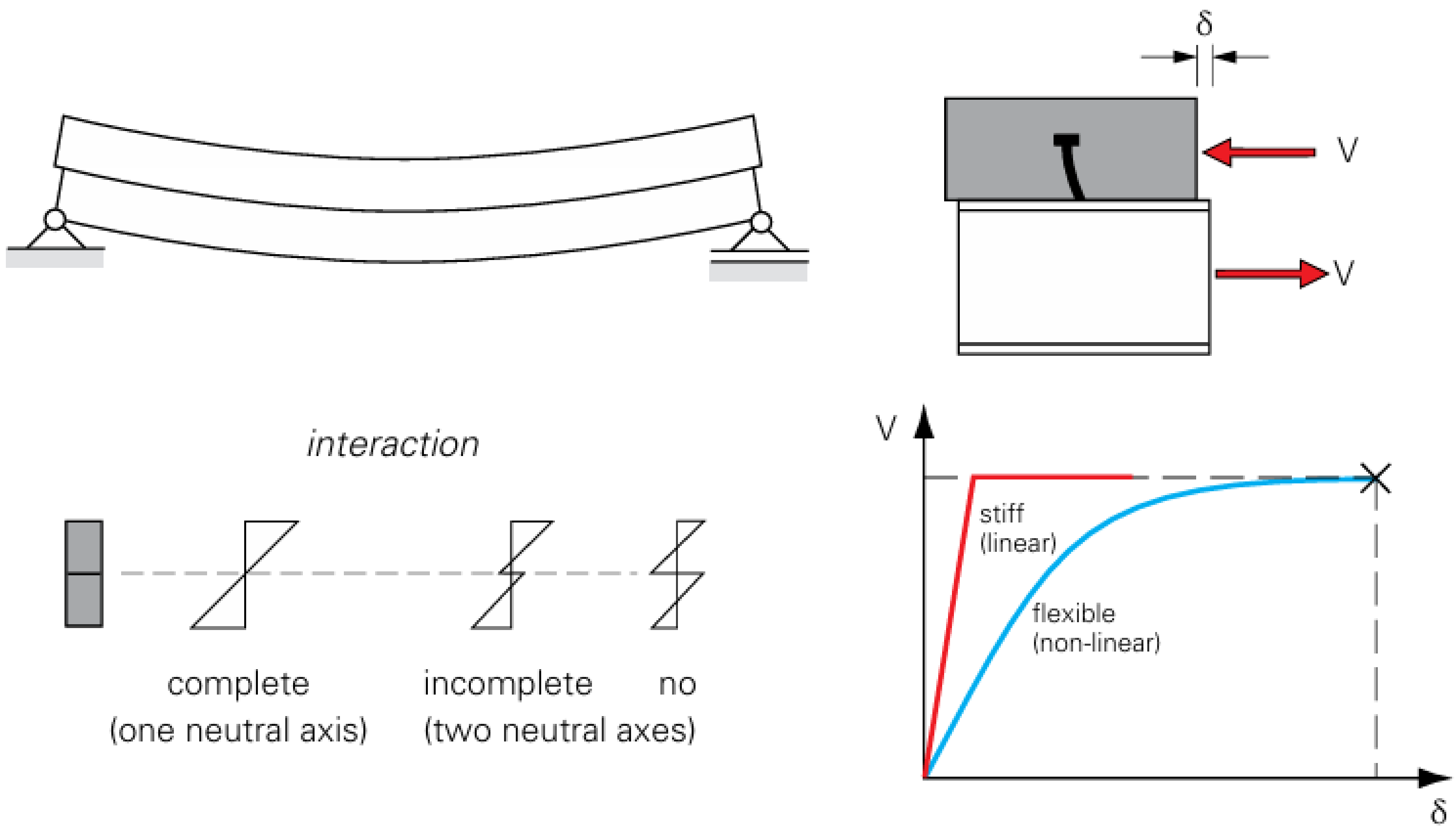
Deformation of the connectors cause slip between steel and concrete and consequently influence the elastic stress distribution (fig) and the deflection.

This is referred to as incomplete interaction.

Incomplete interaction will not influence ultimate limit state and may often be ignored for serviceability limit state

An overview of the classification system is given in table.

1 Criteria for the assessment



The stiffness of the shear connectors (headed studs) determines the level of interaction between the concrete slab and the steel section.

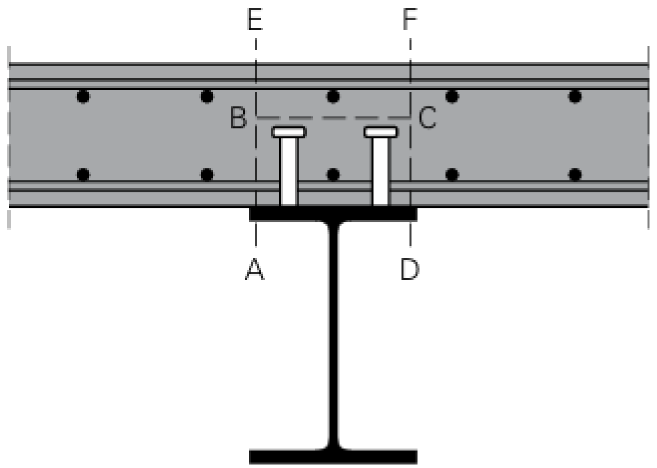
1 Criteria for the assessment

In addition to the three criteria mentioned, each of which represents a primary failure mode, other types of failure are possible, such as:

– longitudinal shear failure of the concrete slab, along the shear planes ABCD, AE and DF as shown in figure;

– splitting of the concrete slab.

The requirements in the standards for shear connector spacing and the amount of transverse reinforcement that must be present ensure that these other failure modes are not generally critical in typical composite beams.



Concrete slab may fail along transverse shear planes ABCD or planes AE and DF.

2 Assessment of the moment resistance

The assessment of the moment resistance depend on whether there is a full or partial shear connection between the cross section subject to the greatest moment and the support:
full shear ensure that either $M_{pl,Rd}$ (see section.1, and 7) or $M_{el,Rd}$ (see section3), depending on the cross section classification, exceeds the maximum applied moment;
partial shear ensure that M_{Rd} determined as given in next section 4, exceeds the maximum applied moment.

3 Assessment of the shear connection for full shear connection

If complete interaction is assumed, the shear force per unit length v_{Ed} at the shear plane is calculated (assuming elastic behaviour) as:

$$v_{Ed} = \frac{V_{Ed} S}{I_c} \quad (2.20)$$

Where:

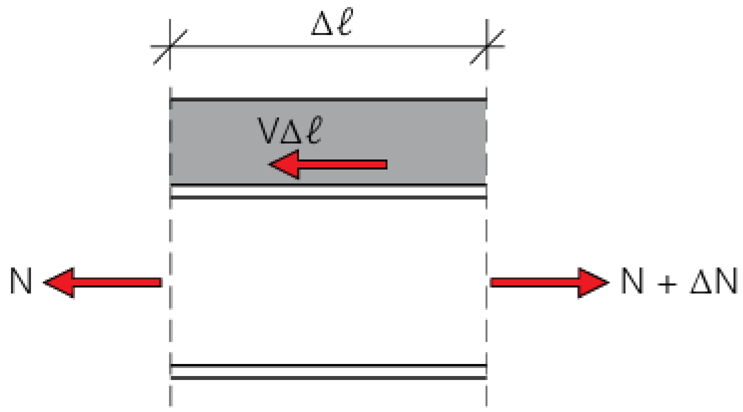
V_{Ed} design value of the vertical shear action;

S: first moment of area relative to the neutral axis of the steel section;

I_c : second moment of area of the transformed cross section.

3 Assessment of the shear connection for full shear connection

This means that with elastic behaviour the longitudinal shear distribution follows the vertical shear distribution.
Equation (2.20) can be derived by considering the horizontal equilibrium of an element of a composite beam (fig).



Determination of longitudinal shear force from the horizontal equilibrium of an element of a composite beam.

3 Assessment of the shear connection for full shear connection

Next Figure shows the distribution of the longitudinal shear force for a simply supported beam subject to a uniformly distributed load.

It follows from the need for horizontal equilibrium that the sum of the longitudinal shear forces (from individual connectors) on either side of the point of maximum moment is equal to the concrete compression force N_c, or the steel tension force N_a.

When the material starts yielding in the span because the elastic bending resistance is exceeded, the distribution of the longitudinal shear forces changes.

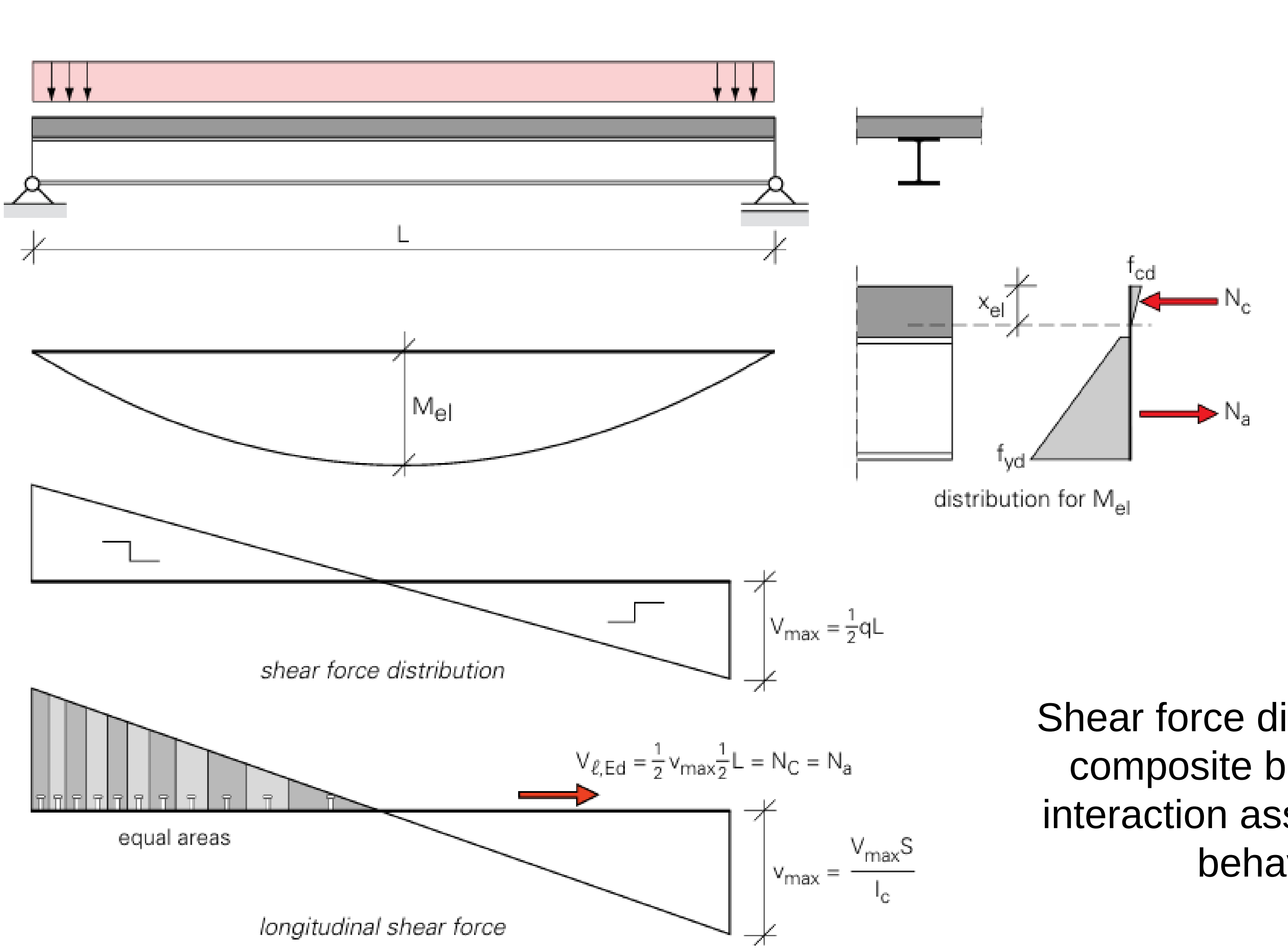
3 Assessment of the shear connection for full shear connection

At any points in the span where the applied moment remains smaller than $M_{el,Rd}$, the beam still behaves in a fully elastic manner.

However, at points where the moment exceeds $M_{el,Rd}$, a redistribution of stress in the cross section takes place.

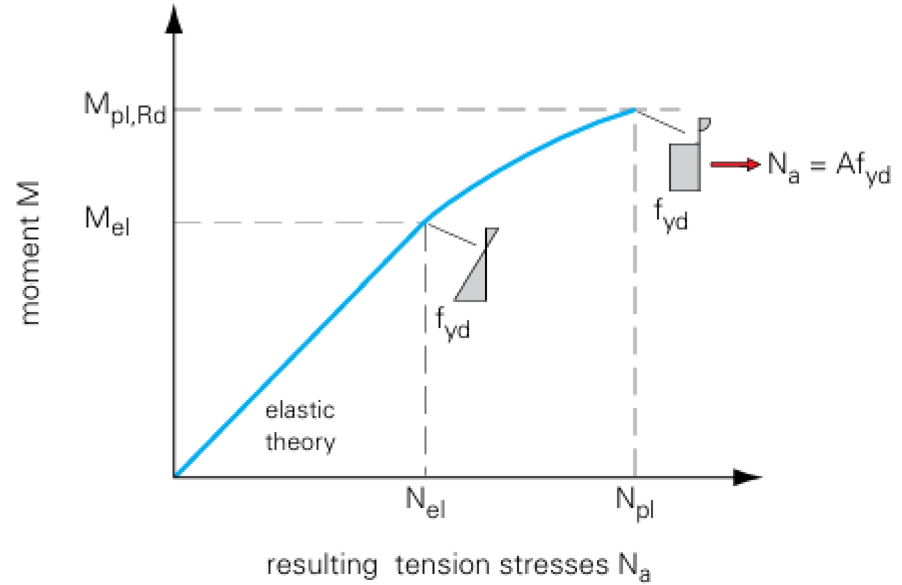
The stress in the outer fibres of the steel section remains constant (f_{yd}), while the stress in the fibres closer to the neutral axis continues to increase as the load is increased.

This means that the resultant of the steel stresses (N_a) increases more than the moment, as shown in the relationship between M and N_a in fig. Based on equilibrium the following applies: $V = dM/dx$. This means that the longitudinal shear force is no longer directly proportional to the vertical shear force once the elastic limit has been reached.



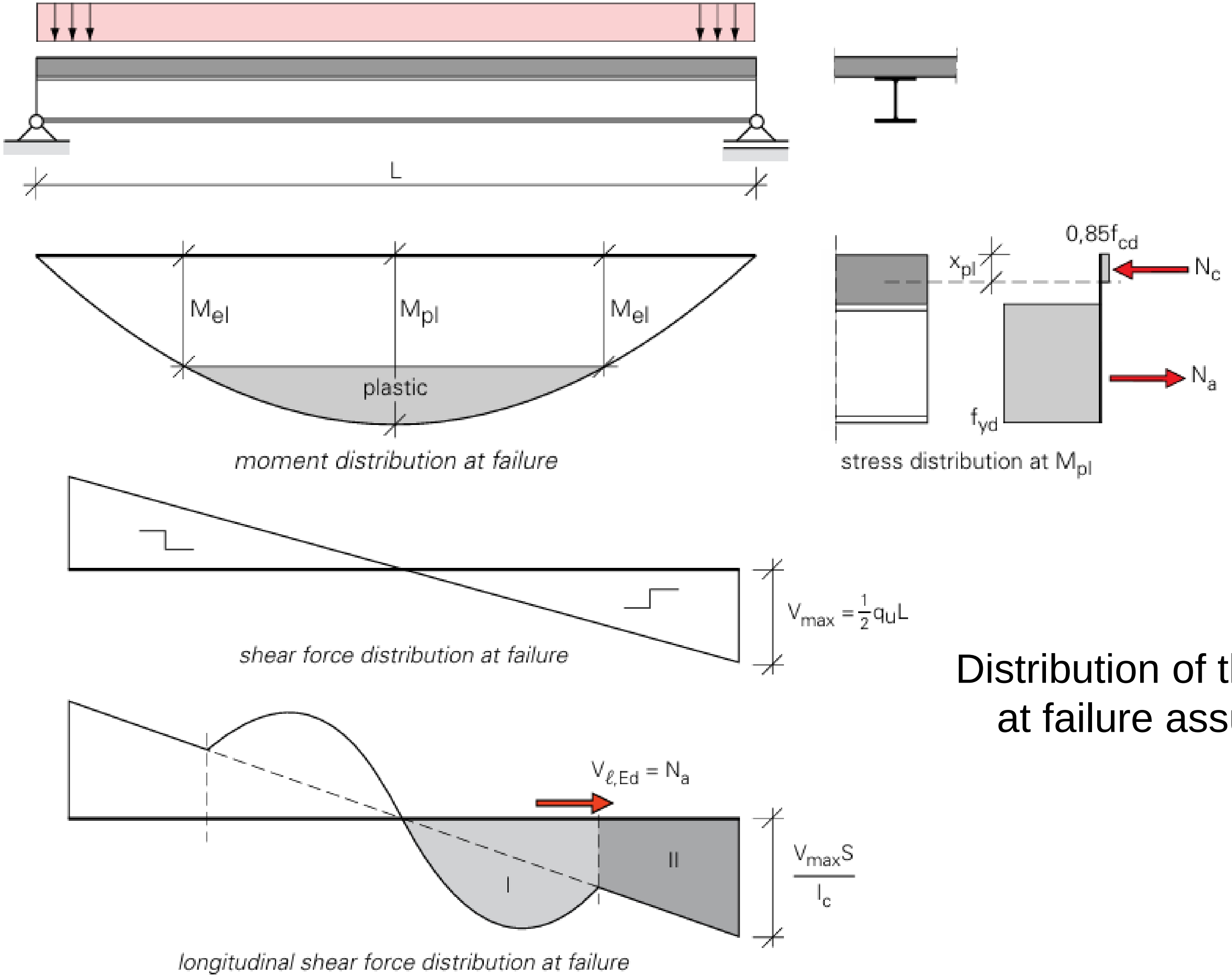
Shear force distribution for a composite beam with full interaction assuming elastic behaviour.

$$\text{number of headed studs to reach } M_{el} = \frac{\text{triangular area of the shear force distribution}}{\text{design value of a headed stud}}$$



Relationship between bending moment M
and axial tension force in the steel section Na for
full shear interaction.

Figure shows the distribution of longitudinal shear force at failure for a simply supported class 1 or 2 beam.



Distribution of the shear forces at failure assuming no slip

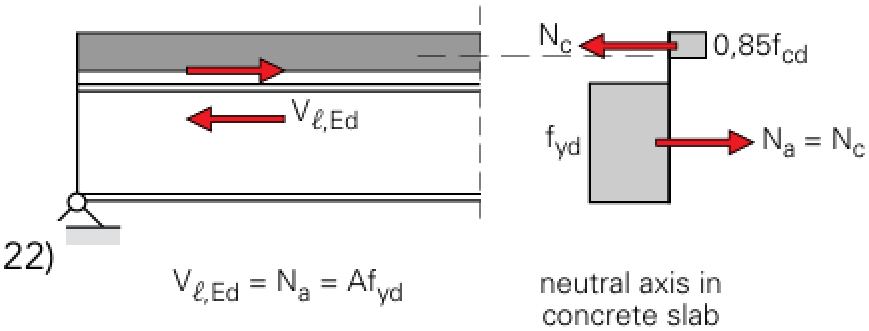
It follows from the need for horizontal equilibrium that the sum $V_{,lEd}$ of the shear forces on either side of the point of maximum moment is equal to the lesser of the concrete compression force Nc, and the steel tension force Na.

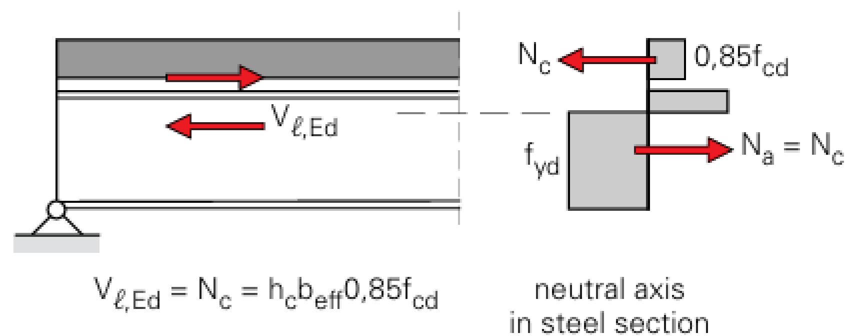
This is shown in figure. Depending on the position of the neutral axis:

$$V_{\ell,Ed} = Af_{yd} \quad \text{(neutral axis in concrete slab)} \qquad (2.21)$$

or

$$V_{\ell,Ed} = h_c b_{eff} 0{,}85 f_{cd} \quad \text{(neutral axis in steel section)} \qquad (2.22)$$





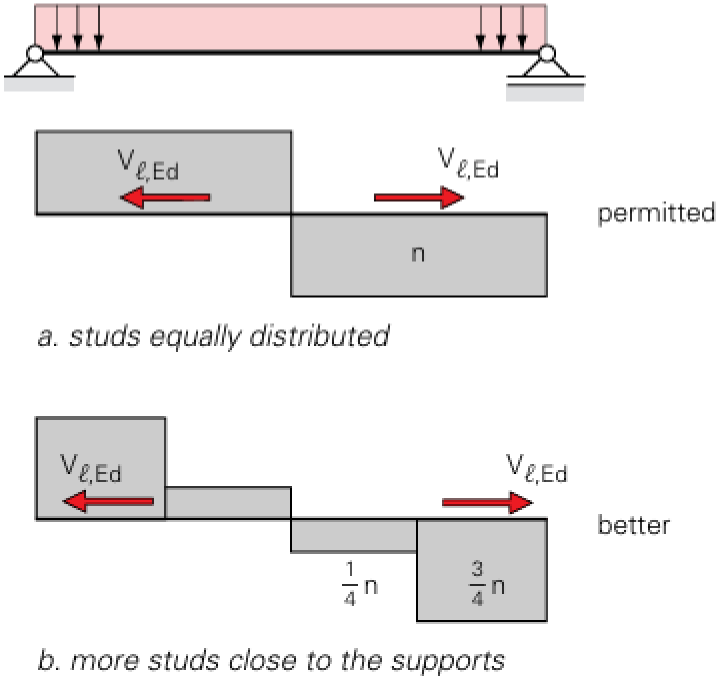
The longitudinal shear force in the shear plane depends on the location of the neutral axis.

The distribution of longitudinal shear forces is influenced by:
slip, temperature differences between steel and concrete, and shrinkage of the concrete.
However if the shear connectors are able to deform plastically, then these forces can be redistributed over the full set of connectors.
When all the connectors to either side of the most heavily loaded section have together sufficient resistance to transfer the required force $V_{,lEd}$, the beam achieves its maximum possible moment resistance.

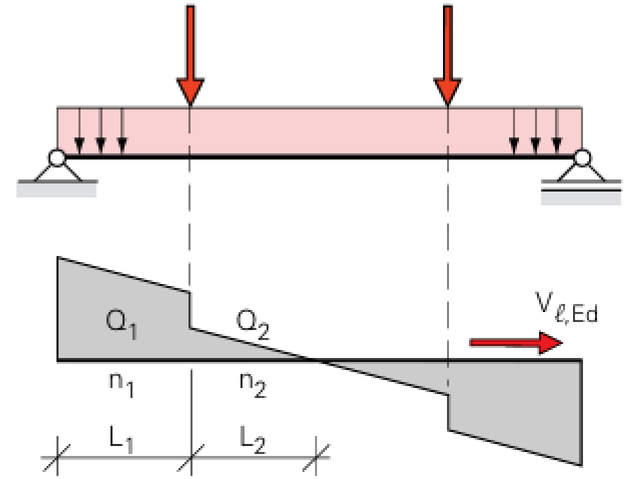
The shear force $V_{\ell,Ed}$ to be transferred is defined in equation (2.21) or (2.22).
The number of connectors required follows from:

$$n_f = \frac{V_{\ell,Ed}}{P_{Rd}} \tag{2.23}$$

Ductile connectors may be distributed evenly over the beam (fig.). However, it is recommended to concentrate them more at the beam ends to better reflect the distribution of longitudinal shear forces. With an even distribution the connectors near the centre line will not be subject to enough slip to generate their full resistance. The recommended distribution of connectors is shown in fig b.



A: Distribution of headed studs over the span of the beam for a uniformly distributed load (n = number of studs).



b: Placement of studs over the span of the beam for a uniformly distributed load in combination with concentrated loads.

In addition to uniformly distributed loads, concentrated loads can of course also act on a beam. In this case, the total number of connectors n_f should be distributed to generally reflect the area Q of the shear force diagram.

For example, for the simply supported beam shown in figure b :

$$n_1 = \frac{nQ_1}{Q_1 + Q_2} \quad \text{and} \quad n_2 = \frac{nQ_2}{Q_1 + Q_2}$$

4 Assessment of the shear connection for partial shear connection

It may be that full shear connection is not needed for supporting the applied loads, because a lesser amount of composite action (fewer connectors) offers sufficient resistance.

When partial shear connection is sufficient it saves on the number of connectors, but more importantly it provides a design method if only a limited number of shear connectors can be placed.

Partial shear connection may be considered, for example, in the following cases.

4 Assessment of the shear connection for partial shear connection

* When the floor slabs are cast without propping (unpropped), and so the execution phase determines the size of the steel section. The plastic moment resistance $M_{pl.Rd}$ with full shear connection will be unnecessarily high.

* Rolled sections are available in a limited range of standard sizes, so the chosen section may be bigger than required.

* The sizing of the composite beam is governed by serviceability requirements (stiffness, not strength). The potential resistance of the beam can be reduced by using partial shear connection.
By obtaining a better balance between stiffness and strength a more economical beam design can be achieved.

4 Assessment of the shear connection for partial shear connection

* However, partial shear connection is essential when the maximum longitudinal shear force that can be transferred is not large enough to achieve full shear connection.
For example, the maximum number of studs that can be installed on a composite beam (with transverse decking) depends on the number of ribs (with the resistance of each stud dependent on the shape of the ribs).

Example 2.4

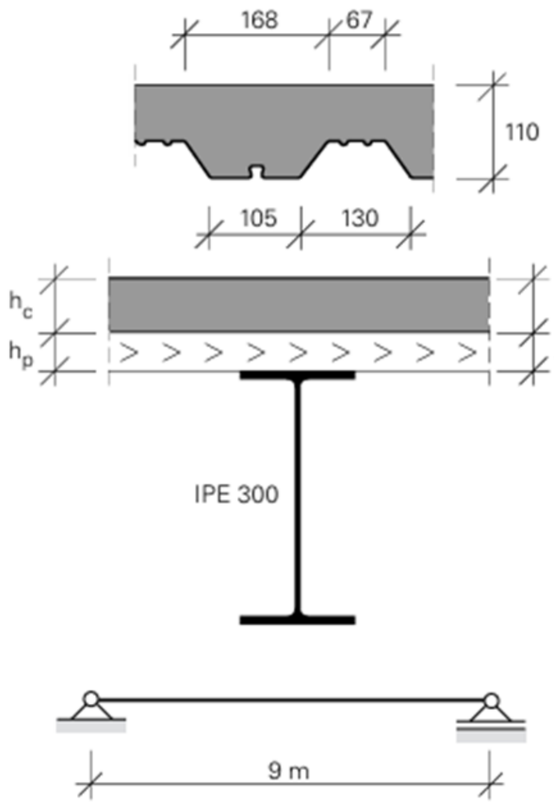
- *Given*. The composite beam in example 2.1, with studs Ø19 mm, with length h_{sc} = 85 mm and f_u = 450 N/mm^2.
- *Question*. Calculate the required number of studs to achieve full shear connection.
- *Answer*. For the given studs $h_{sc} > 4d$, so $\alpha = 1$. From table 2.5, for concrete in strength class C25/30: P_{Rd} = 73,7 kN. The reduction factor for ribs perpendicular to the supporting beam must be applied:

$$h_p = 46 \text{ mm} \leq 85 \text{ mm}; \quad b_0 = \frac{1}{2}\cdot(105 + 168) = 136 \text{ mm} \geq h_p = 46 \text{ mm}$$

$$d = 19 \text{ mm} \leq 20 \text{ mm}$$

Equation (2.4) may therefore be applied (reduction factor):

$$k_t = \frac{0,7}{\sqrt{n_r}}\cdot\frac{b_0}{h_p}\cdot\left(\frac{h_{sc}}{h_p} - 1\right) = \frac{0,7}{\sqrt{n_r}}\cdot\frac{136}{46}\cdot\left(\frac{85}{46} - 1\right) = \frac{1,75}{\sqrt{n_r}}$$



Maximum values $k_{t,max}$ for the reduction factor k_t are given in table 2.8. For a sheet thickness of 1,2 mm $k_{t,max} = 1,0$ for $n_r = 1$ and $k_{t,max} = 0,8$ for $n_r = 2$.

For $n_r = 1$ applies: $k_t = 1,75 > 1 \Rightarrow k_t = 1 \Rightarrow P_{Rd} = 73,7$ kN

For $n_r = 2$ applies: $k_t = 1,24 > 0,8 \Rightarrow k_t = 0,8 \Rightarrow P_{Rd} = 59,0$ kN

For the longitudinal shear force the smaller value given by equation (2.21) and (2.22) applies:

$$V_{\ell,Ed} = Af_{yd} = 53,8 \cdot 10^2 \cdot 235 \cdot 10^{-3} = 1264 \text{ kN (critical)}$$

$$V_{\ell,Ed} = h_c b_{eff} 0,85 f_{cd} = 64 \cdot 2250 \cdot 0,85 \cdot 16,7 \cdot 10^{-3} = 2044 \text{ kN}$$

Assume one stud per rib, so $k_t = 1$. Equation (2.23) then gives the number of studs required for full shear connection:

$$n_f = \frac{V_{\ell,Ed}}{k_t P_{Rd}} = \frac{1264}{1 \cdot 73,7} = 17,2 \Rightarrow 18 \text{ studs}$$

The number of ribs per half span (see also fig. 2.17) is 4500/235 = 19. It is concluded that one stud per rib is sufficient.

Example 2.1

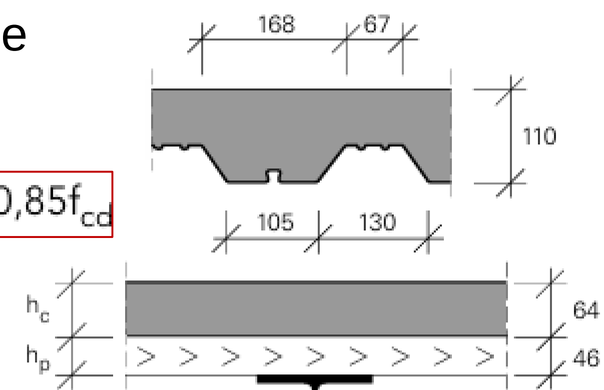
- *Given*. A statically determinate beam IPE 300 with f_{yd} = 235 N/mm^2 and a span L = 9,0 m. Key dimensions of the composite floor are h_c = 64 mm and h_p = 46 mm. The ribs of the steel sheeting are perpendicular to the beam axis (fig. 2.17). The concrete is of class C25/30 with f_{cd} = 16,7 N/mm^2.
- *Question*. Determine the plastic moment resistance $M_{pl,Rd}$.

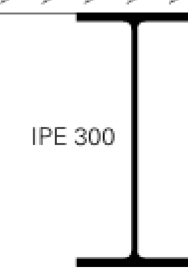
- **Answer**. One row of shear studs is assumed (b_0 = 0). The effective width is b_{eff} = 2·9000/8 =2250 mm, therefore:

$$h_c b_{eff} 0{,}85 f_{cd} = 64 \cdot 2250 \cdot 0{,}85 \cdot 16{,}7 \cdot 10^{-3} = 2044 \text{ kN}$$

$$A f_{yd} = 53{,}8 \cdot 10^2 \cdot 235 \cdot 10^{-3} = 1264 \text{ kN} < 2044 \text{ kN}$$

$$A f_{yd} < h_c b_{eff} 0{,}85 f_{cd}$$





The neutral axis is thus in the concrete slab, (case 1)

$$x_{pl} = \frac{A f_{yd}}{b_{eff} 0{,}85 f_{cd}} = \frac{53{,}8 \cdot 10^2 \cdot 235}{2250 \cdot 0{,}85 \cdot 16{,}7} = 40 \text{ mm} < h_c = 64 \text{ mm}$$

Simply supported composite beam with decking ribs running transverse to the supporting steel beam.

$$M_{pl,Rd} = A f_{yd} \cdot \left(\frac{1}{2} h_a + h_p + h_c - \frac{1}{2} x_{pl} \right) =$$

$$= 53{,}8 \cdot 10^2 \cdot 235 \cdot \left(\frac{1}{2} \cdot 300 + 46 + 64 - \frac{1}{2} \cdot 40 \right) \cdot 10^{-6}$$

$$= 303 \text{ kNm}$$