



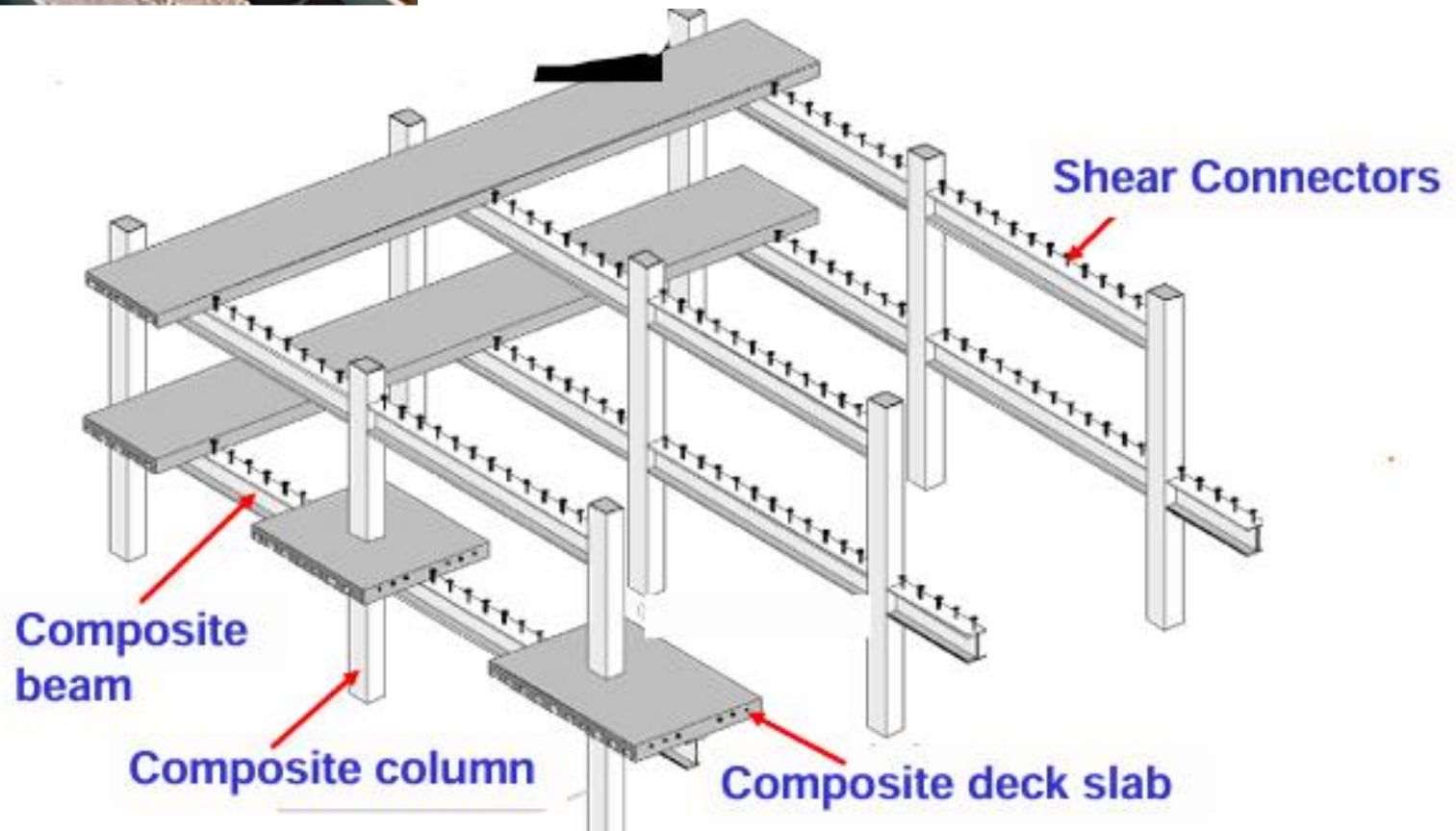
Lecture 3

Composite columns

Applications of composite structures

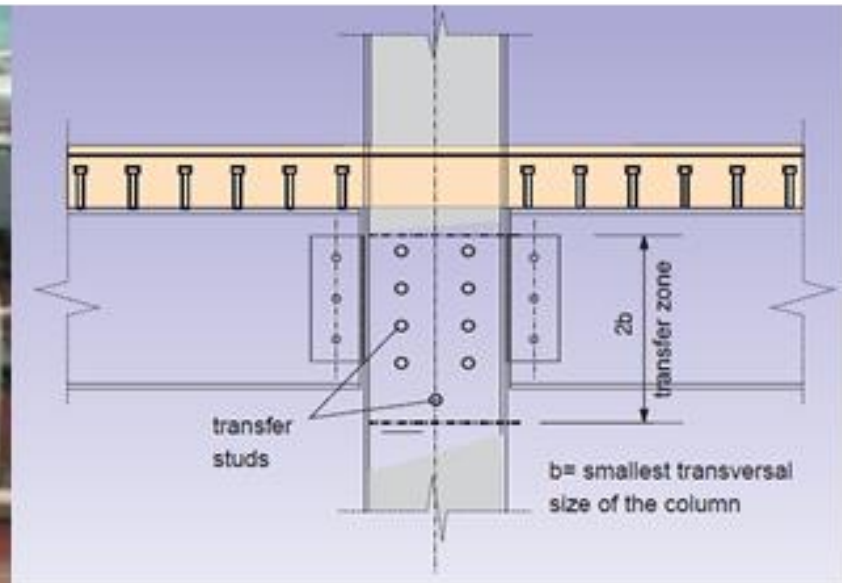
EN 1994 (Eurocode 4)





Composite columns

To exhibit efficient structural response, composite columns should employ **shear connectors** along the height.



Composite columns

Increasing the fire resistance of columns is often a reason to apply a composite structure.

Depending on the execution

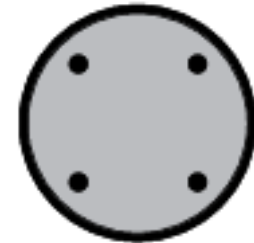
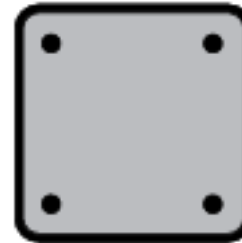
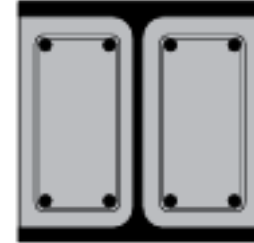
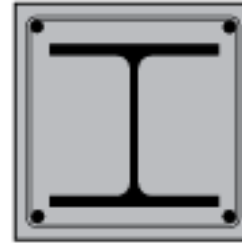
- **complete or partial** filling of the steel
- and the possibility of installing **additional** reinforcement, any fire resistance is achievable.

Structural system

- A composite column consists of a **steel section** that has been **partially or fully** encased in concrete, or a **hollow section** filled with concrete (fig.).
- Both options have a high fire resistance.



H section columns with a concrete encasement between the flanges



Frequently used cross-sections of composite columns.

Structural system

- It became practice to assume the column behaves as a composite section also at normal temperature.

- The **disadvantage** of a full encasement is that formwork for an entire column is required.

Moreover, an important advantage of building in steel is **lost**, namely **the short erection time**.

- In **prefabricated** composite column It might be feasible to only fill the space between the flanges of the section with concrete (fig.).

- By **adding reinforcement**, in theory any desired fire resistance can be achieved.

Design

- EN 1994-1-1 contains **assessment** rules for composite columns.
 - The calculation method is in line with the method described in EN 1993-1-1 for steel columns.
 - In the preliminary design, composite columns are usually dimensioned using design tables.
- For a **hollow section** filled with concrete – both with and without reinforcement – with a required fire resistance of 60 minutes or more, the **fire load** is almost always governing.
- It is recommended to choose a hollow section with a minimum wall thickness in a low steel grade, like S235

Design

In a fire situation, because the **contribution** of the steel section with regards to the total resistance is limited in any case.

- A structural engineer has the choice for, for example,
- a **filled** hollow section with a low reinforcement percentage (1%),
 - a smaller **hollow** section with a higher reinforcement percentage (4%)
 - or a larger hollow section without reinforcement.
- The **final decision** is made based on aesthetic, practical and financial reasons

Design

With the guidelines provided in EN 1994-1-1, cl. 6.7.4 it can be determined whether:

shear connectors are necessary to ensure sufficient interaction between the steel and concrete.

For this purpose, **headed studs**, **strips** or **angles** can be used.

With **relatively stocky**, circular columns:

the contribution of concrete with regards to the total resistance increases due to its encasement, which results on so-called concrete confinement.

Confinement increases the effective strength of the concrete.

Fire resistance

Unprotected circular hollow sections often have a fire resistance of no more than **20** minutes.

The behaviour of a circular hollow column during a fire improves by filling the column with concrete, resulting in a **fire resistance of at least 30** minutes.

A fire resistance of 60 minutes is achievable in some cases, depending on the **geometry** (especially the diameter and the slenderness), **the load and the way** in which the column has been dimensioned at normal temperature.

The addition of reinforcing steel ensures a further increase in fire resistance

Fire resistance

- By **applying** a full concrete encasement, the temperature of steel **increases less quickly** than without any concrete encasing.

The fire resistance depends on the **dimensions** of the concrete and the **cover** of the steel section and the reinforcement. -

- EN 1994-1-2 contains simple design tables to determine the fire resistance.

Filling the space between the flanges of a H section with concrete also **improves** the fire resistance, but less effective than with a full encasement.

Fire resistance

- The concrete ensures that the temperature of the directly heated steel surface **rises less quickly** than a steel column without any concrete.

In case of a **partially encased** section, the temperature of the **flanges increases fairly quickly** in a fire situation.

However, the temperature of the web **rises significantly slower**, because the web is completely encased by concrete

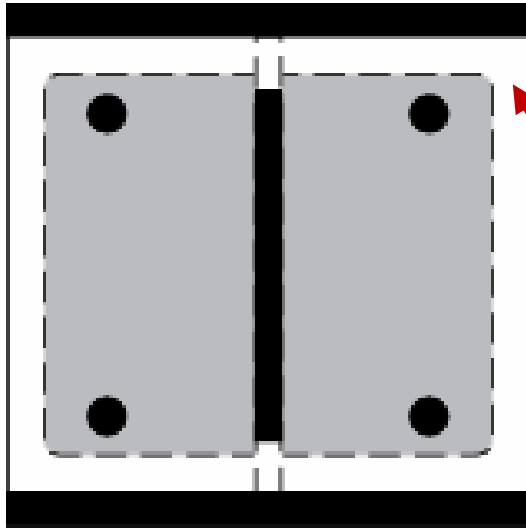
The web maintains its full resistance

Fire resistance

Due to the **decrease in strength** and stiffness of the flanges of the column, a redistribution of forces occurs towards the inner parts of the composite cross-section.

-In combination with the strength reduced flanges and additional reinforcement, the **concrete** and the **web** of the steel section together provide the **required** resistance in a fire situation. Figure : shows the model used in EN 1994-1-2.

EN 1994-1-2 provides both **design tables and simple assessment rules** for different types of composite

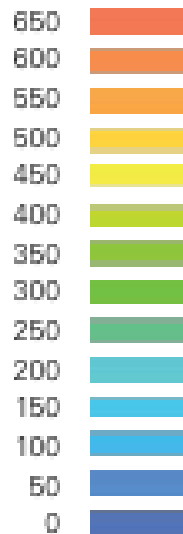
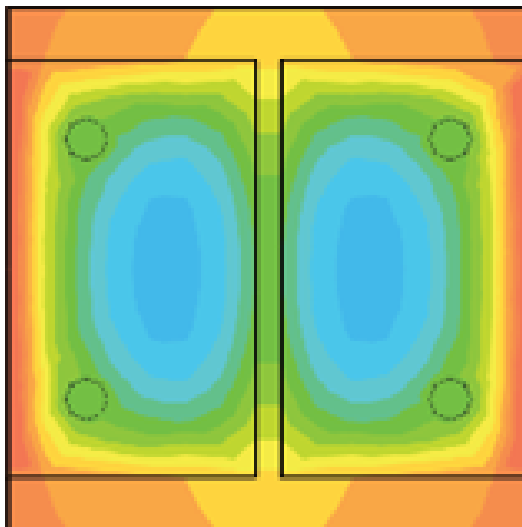


Design model according to EN 1994-1-2 for determining the resistance under fire situation for a partially encased section.

As shown in the upper figure in white, parts of the web and parts of the concrete cross-section are neglected.

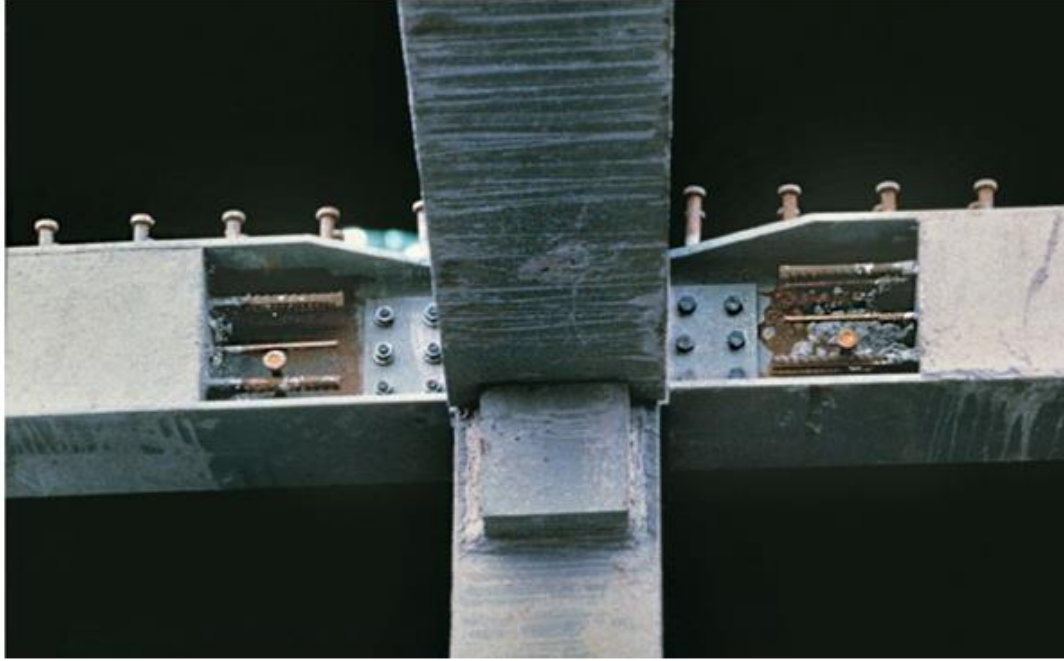
The lower figure is an illustration of a calculated temperature distribution over the cross-section.

temperature (°C)



Execution

- **No separate formwork** is required for columns with partially encased cross-sections (fig.).
 - Because the steel section is cast horizontally, formwork is only required at the **two column ends**. The assembly can be done with regular steel joints (fig.).
- When composite columns are **prefabricated**, construction time can be significantly reduced compared to in situ filled columns.



Partially encased H and I sections can be erected using regular joints.



H section columns with concrete encasement between the flanges being cast horizontally.

Execution

For **tubular columns** filled with concrete, at least **two holes** with a diameter of approximately 12 mm or more must be present in the hollow section wall:

one hole at **the base** and one hole at **the top** of the column.

The maximum distance between the holes is **thirty times** the transverse width of the column (fig.).

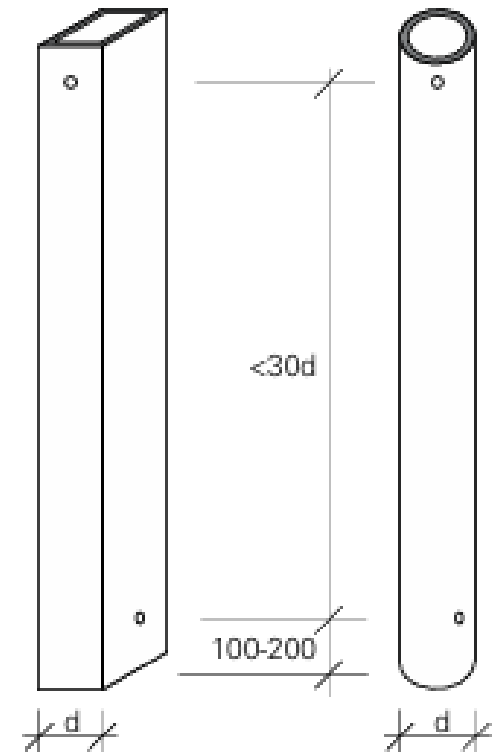
These holes are required to ensure that the **moisture** present in the concrete can escape from the column in case of a fire.

If water vapour cannot escape, pressure may build up until the column **'explodes'**.

Filling hollow sections with concrete can either be done at the workshop or at the construction site.



Hollow sections up to four storeys in height can be filled with concrete from ground level in one go.



Holes in the column wall prevent the build-up of pressure due to water vapour.

Applications

The **choice for composite** structures in buildings (slabs, beams and columns) depends on various factors, which may vary per project. Mostly, it involves

- 1- weight reduction,
- 2- architectural freedom
- 3- and cost-saving.

1- Weight reduction

An important factor – especially for composite **slabs** – is often the **low self-weight**, which saves costs and reduces material.

- The steel sheets are usually **delivered in packages**, which reduces the amount of crane use.

- The concrete is casted in a **separate** process.

- The steel sheets can easily be adjusted (cut to fit).

Without additional measures, a composite floor can form a **diaphragm** as part of the stability system.

- Due to the profiled cross-section, a composite floor has a large surface area at the bottom.

Freedom of design

The design of a structure should not only consider (**resistance, stiffness, deformation capacity and costs**); **architectural** creative freedom is just as important.

The implementation of composite structures provides for the architectural design:

- **light structures** for large column
- free spaces** and large cantilevers;
- **slender beams** (compared to regular steel beams) and columns for maximum transparency;
- **high fire resistance**, allowing the steel structure to still be visible;
- **smaller** floor heights;
- **design freedom** for the floor plan in combination with composite floors.

2- Freedom of design

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3- Cost savings

Cost-optimization can be achieved using composite structures.

In high-rise buildings composite slabs offer fast **erection and the low crane use.**

The short cycle time is reached when the floor does not have to be **propped** and if the steel sheeting can be lifted onto a floor in a **bundle.**

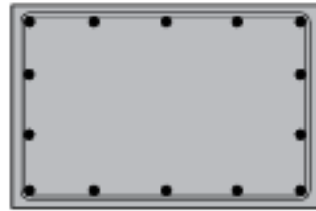
When considering the overall costs of the floor structure – including erection time, weight reduction, fire-resistant coating and material costs –

a composite floor is still a competitive alternative.

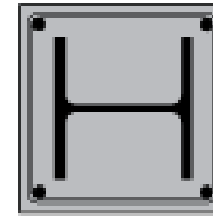
3- Cost savings

in combination with the ability to **integrate services** between (or even through) the floor beams, reduces the **total depth** of the floor package.

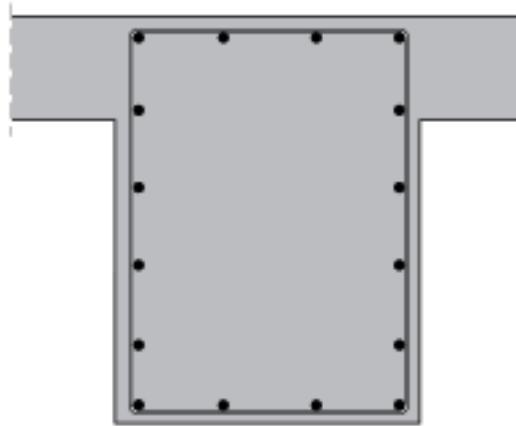
This may allow a reduction in the: **total building height**, which in turn results in a reduction of the **façade costs**, or – without changing the building height – may allow an **extra floor level** to be included.



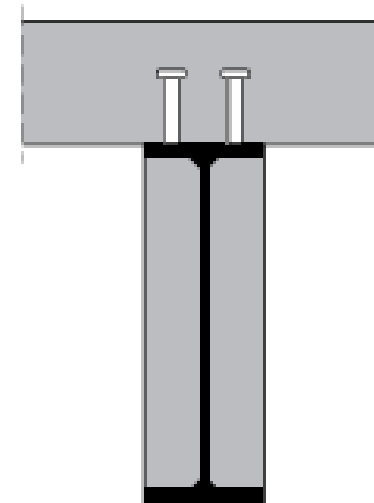
column
reinforced concrete: 800x1200 mm



column
composite: 700x700 mm



beam
reinforced concrete: 1600x1200 mm



beam
composite: 1600x400 mm

Comparable heavily loaded composite and reinforced concrete members, showing the former can be significantly more slender but still carry the same loads.

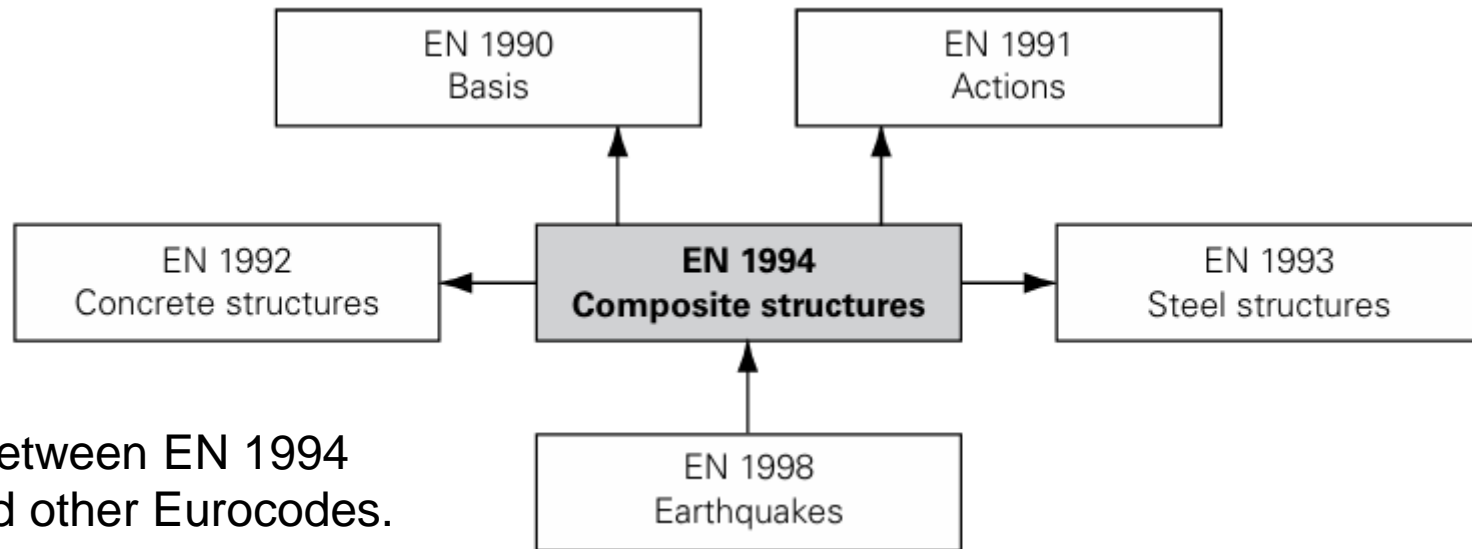
EN 1994 (Eurocode 4)

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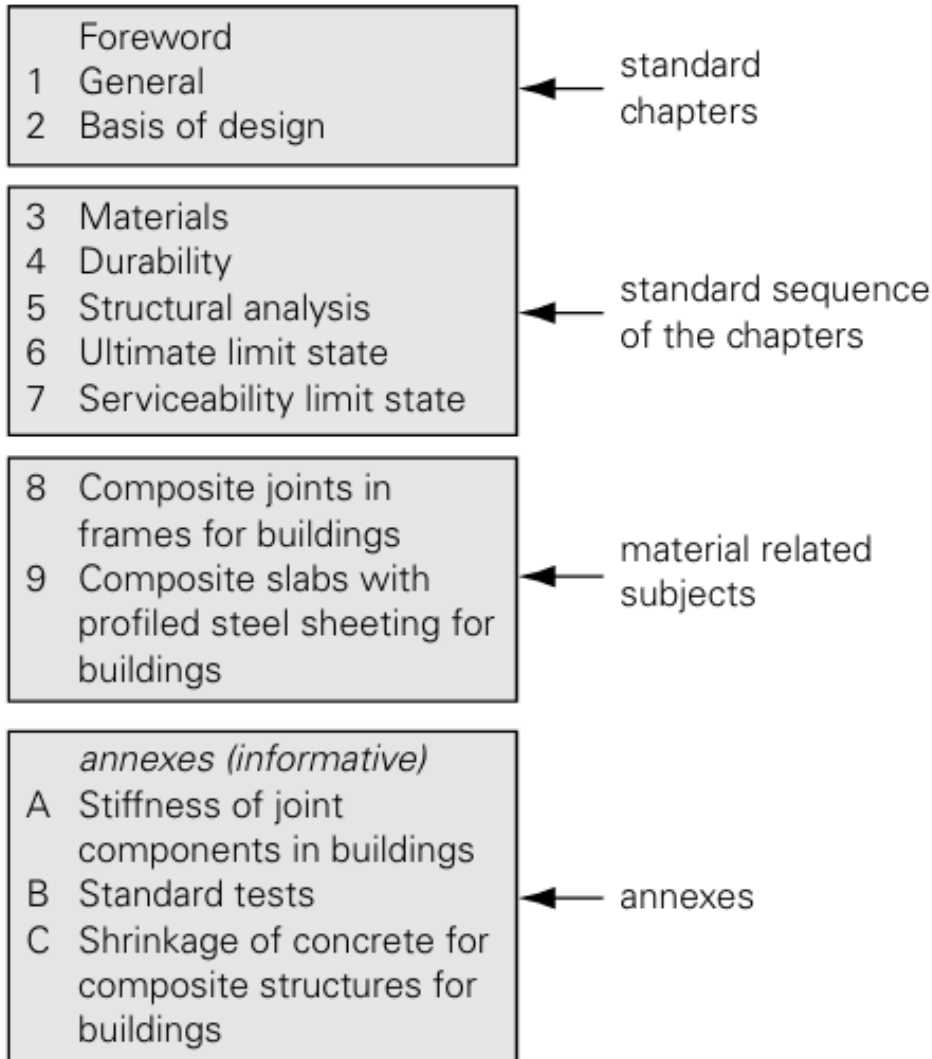
Design of composite steel and concrete structures

there are a lot of aspects of composite design which are **covered in other Eurocodes**, so EN 1994 explicitly refers to these codes.

The referral system is schematically shown in figure



Relationship between EN 1994 (Eurocode 4) and other Eurocodes.



Structure and layout

EN 1994 is divided into three parts:

- part 1-1 General rules and rules for buildings;
- part 1-2 General rules. Structural fire design;
- part 2 General rules and rules for bridges.

Chapters and annexes in EN 1994-1-1.

clause	parameter or item	recommendation or remark
2.4.1.1(1)	partial factor γ_P	$\gamma_P = 1,0$ for favourable and unfavourable effects
2.4.1.2(5)	partial factor γ_V	$\gamma_V = 1,25$
2.4.1.2(6)	partial factor γ_{VS}	$\gamma_{VS} = 1,25$
2.4.1.2(7)	partial factors γ_{Mf} and $\gamma_{Mf,s}$	γ_{Mf} as given in the relevant parts of EN 1993 $\gamma_{Mf,s} = 1,0$
3.1(4)	shrinkage of concrete	values of shrinkage strain in EN 1992-1-1 can give overestimates of effects of shrinkage in composite structures; recommended values for buildings are given in annex C
3.5(2)	minimum thickness t of the profiled sheeting	$t_{min} = 0,70$ mm
6.4.3(1)h	provisions of lateral instability	for other types of steel sections than given in table 6 provisions may be given in the National Annex; no recommendation
6.6.3.1(1)	partial factor γ_V	$\gamma_V = 1,25$
6.6.3.1(3)	studs causing splitting forces	for buildings, further information may be given in the National Annex

Overview of National Determined Parameters in EN 1994-1-1.

6.6.4.1(3)	methods for anchoring steel sheeting	means to achieve appropriate anchorage may be given in the National Annex; no recommendation
6.8.2(1)	partial factor $\gamma_{Mf,s}$	$\gamma_{Mf,s} = 1,0$
6.8.2(2)	partial factor γ_{Ff}	values may be given in the National Annex; no recommendation
9.1.1(2)	upper limit b_r/b_s	$b_r/b_s = 0,6$
9.6(2)	maximum deflection $\delta_{s,max}$	$\delta_{s,max} = L/180$
9.7.3(4)	partial factor γ_{VS}	$\gamma_{VS} = 1,25$
9.7.3(8)	partial factor γ_{VS}	$\gamma_{VS} = 1,25$
9.7.3(9)	nominal factor μ	$\mu = 0,5$
B.2.5(1)	partial factor γ_V	$\gamma_V = 1,25$
B.3.6(5)	partial factor γ_{VS}	$\gamma_{VS} = 1,25$

Overview of National Determined Parameters in EN 1994-1-1.

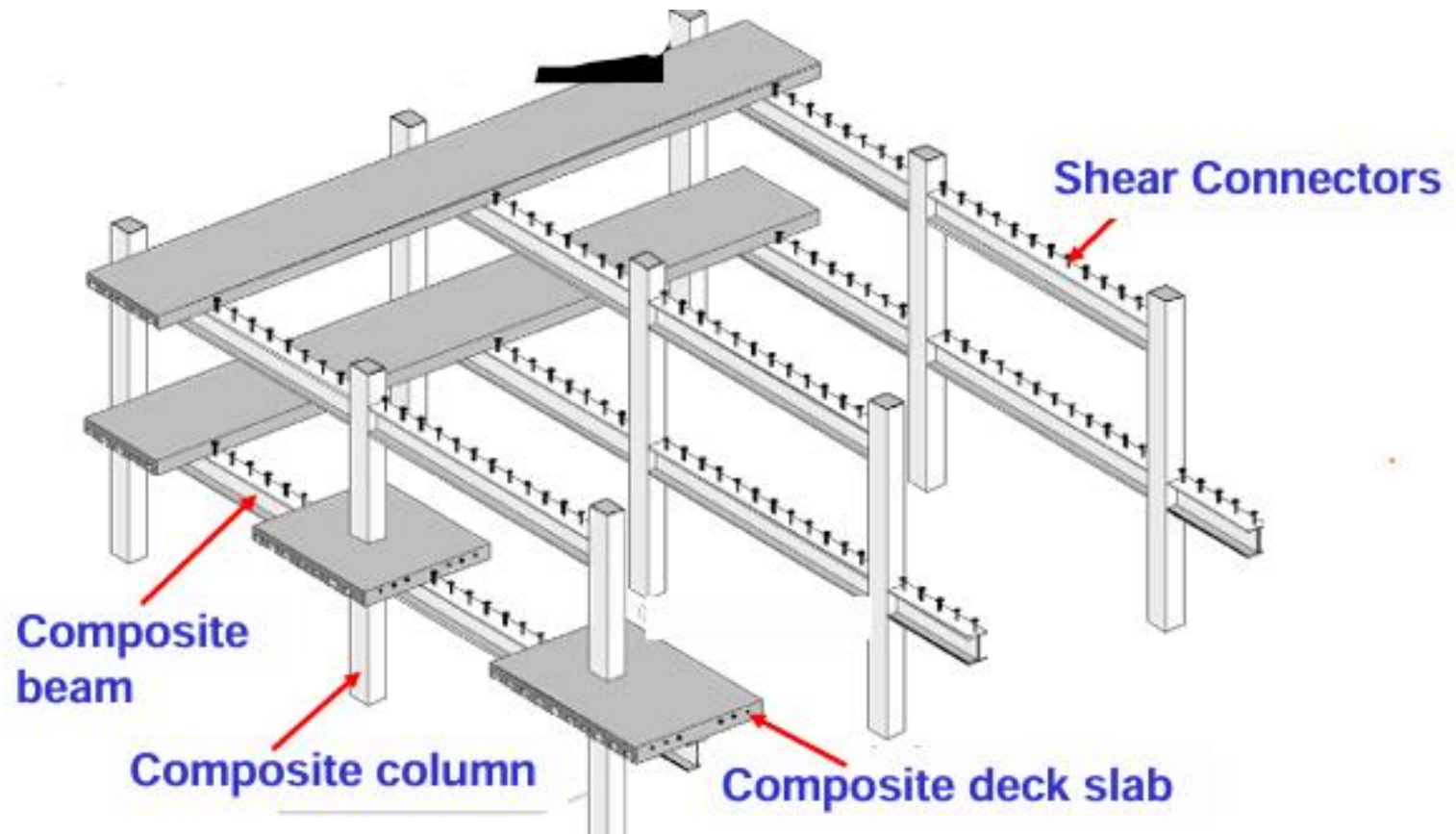
clause	parameter or item	recommendation or remark
1.1(16)	concrete strength > C50/60	use of these concrete strength may be specified in the National Annex
2.1.3(2)	max. temperature increase $\Delta\theta_1$ $\Delta\theta_2$	$\Delta\theta_1 = 200$ K $\Delta\theta_2 = 240$ K
2.3(1)P	partial factor $\gamma_{M,fi,a}$ $\gamma_{M,fi,s}$ $\gamma_{M,fi,c}$ $\gamma_{M,fi,v}$	$\gamma_{M,fi,a} = 1,0$ as in EN 1993-1-2 $\gamma_{M,fi,s} = 1,0$ as in EN 1992-1-2 $\gamma_{M,fi,c} = 1,0$ as in EN 1992-1-2 $\gamma_{M,fi,v} = 1,0$
2.3(2)P	partial factor $\gamma_{M,fi}$	$\gamma_{M,fi} = 1,0$
2.4.2(3)	reduction factor η_{fi}	two alternative options in note 1 and note 2
3.3.2(9)	thermal conductivity λ_c	use upper limit according to equation 3.6a
4.1(1)P	advanced calculation models	permission for application to be given in the National Annex
4.3.5.1(10)	buckling length L_{ei} L_{et}	$L_{ei} = 0,5L$ $L_{et} = 0,7L$

Overview of National Determined Parameters in EN 1994-1-2 (continued).

Lecture 4

- Composite Beams
- Applications
- EN 1994 (Eurocode 4)





Composite Beams

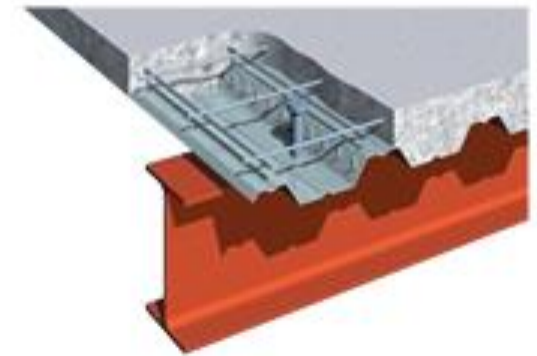
A composite beam generally consists of three parts:

- **concrete slab** (acting as a top flange);
- **steel beam**;
- **connection devices** (for example headed studs).

Longitudinal shear forces are transferred between the concrete and the steel by **shear connectors**.

In this way composite interaction is achieved between the concrete slab and the steel beam, as they act together as one structural element

explanation of behaviour of both statically determinate and statically indeterminate composite beams



explanation of the design methods given in EN 199411 for the:
loadbearing capacity (resistance) and
the deflection (stiffness) of a composite beam at normal temperature

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.

1– partial factors

Partial factors In order to determine the design resistance R_d of the individual components of a composite beam, the partial factors γ , as provided by the appropriate National Annex, are taken into account.

The recommended values are:

structural steel $\gamma_M = 1,00 (= \gamma_{M0})$

reinforcing steel $\gamma_S = 1,15$

concrete $\gamma_C = 1,50$

shear connection $\gamma_V = 1,25$

2– material properties

The design values for the strength of **structural steel** (f_{yd}), **reinforcing steel** (f_{sd}), and **concrete** (f_{cd}), are determined by dividing the characteristic values by the corresponding partial factor. The characteristic values are generally equal to nominal values of strength.

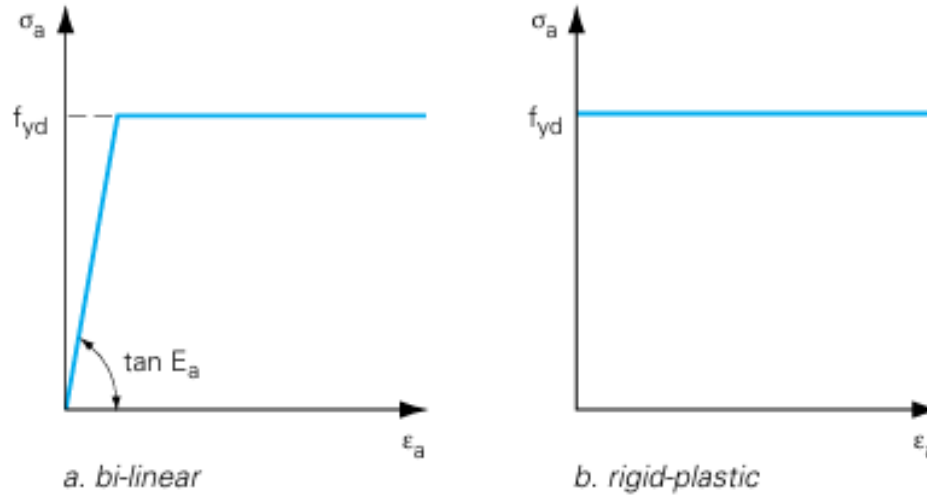
• **Structural steel.**

In the Eurocodes the **stress/strain** relationship for structural steel is schematised, as usual, as a bi linear relationship as shown in figure a. The modulus of elasticity of steel is taken as $E_a = 2,1 \cdot 10^5 \text{ N/mm}^2$. When calculating the cross section resistance according to **plastic theory**, this relationship is further simplified to **rigid plastic** behaviour as shown in figure b.

• **Reinforcing steel.**

The stress/strain relationship for reinforcing steel is schematised in the same way as for structural steel (fig), using the appropriate value of peak stress (f_{sd} rather than f_{yd}).

2– material properties



Stress/strain relationship for structural steel.

- **Concrete.**

The stress/strain relationship for concrete is shown in figure a. The dotted line represents the actual behaviour and the solid line the schematised behaviour according to EN 199211, cl. 3.1.7(2).

The coefficient α_{cc} (a factor which takes into account long-term effects on the compression strength of concrete) according to EN 199211, cl. 3.1.6 should not be taken into account for composite structures.

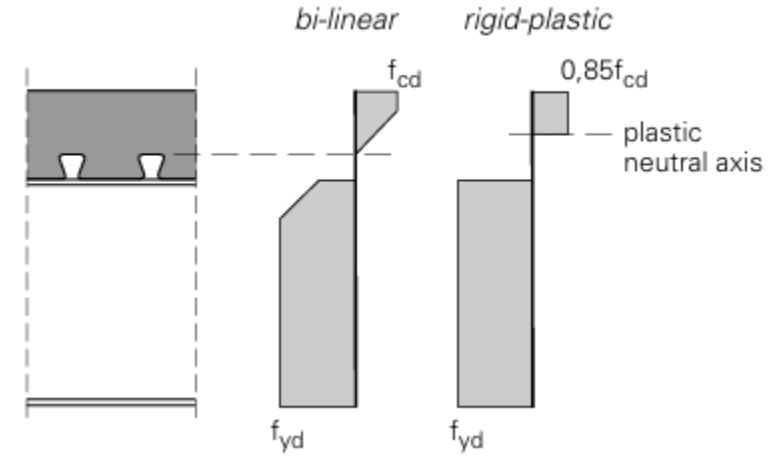
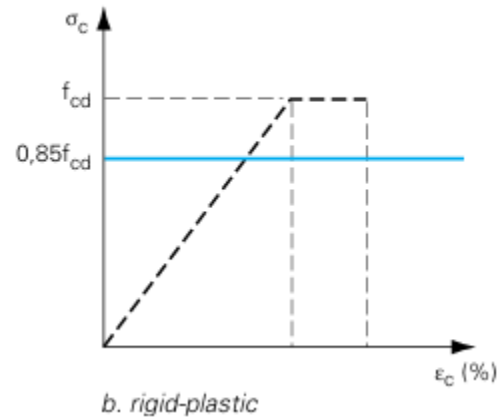
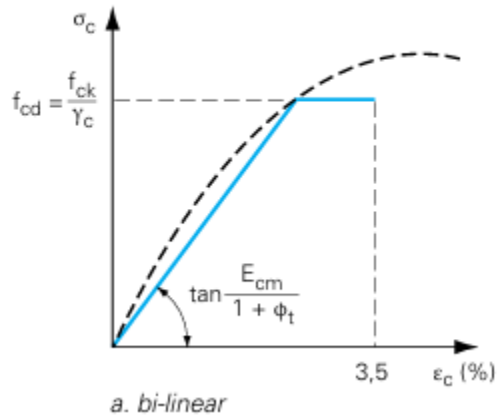
When a composite beam is designed using the principles of **plastic theory**, the bi linear σ/ϵ relationship may be further simplified into **rigid plastic** material behaviour (fig. b).

This simplification makes it much easier to determine the moment resistance using rectangular stress distributions (fig. c).

As part of this simplification the compression strength of the concrete should be reduced using a **calibration coefficient** $k = 0,85$.

This value is based on comparative designs of composite beams,

2– material properties



Stress/strain relationship for concrete.

C: Stress distribution in a beam cross section at the ultimate limit state.

The slope of the initial part of the curve shown in figure a is a **function** of the modulus of elasticity of concrete E_{cm} and the creep coefficient φ_t .

The influence of the **creep of concrete** under long term loading is thus **taken into account** by a reduction of the modulus of elasticity.

2– material properties

Mean values of the modulus of elasticity of concrete under short term loading.

class	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60
E_{cm} (N/mm ²)	30.000	31.000	33.000	34.000	35.000	36.000	37.000

The value of E_{cm} depends on the **class of the concrete** (table).

The presentation of class C comprises a first number referring to cylinder strength, and a second number to cube strength.

According to EN 199411, cl. 5.4.2.2 the effects of creep may be taken into account by using a modular ratio n_L , which is the **ratio** between the elastic modulus of structural steel (E_a) and that of concrete (E_{cm}):

$$n_L = n_0(1 + \psi_L \phi_t)$$

Where:

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

ψ_L creep multiplier depending on the type of loading;

ϕ_t creep coefficient.

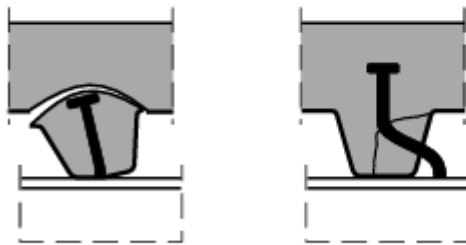
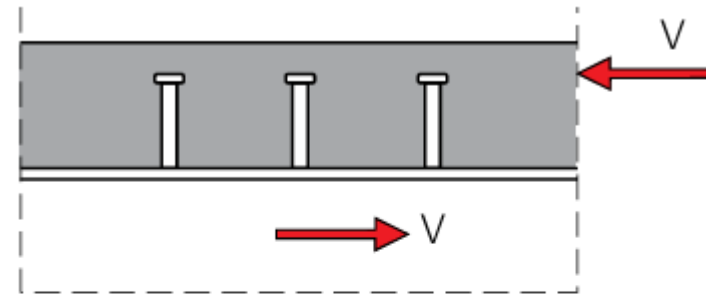
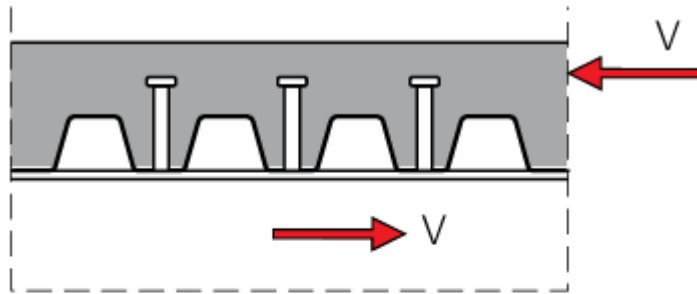
Approximate values for E_c and thus for n may be used in the design of buildings

3 Properties of shear connectors

There are various **types of shear connector**. EN 1994-1-1 only provides principles and application rules for **welded headed studs** (hereafter also referred to as stud).

Some National Annexes provide additional rules for other types of shear connectors.

When considering the behaviour of headed studs, a **distinction must be made** between headed studs in a solid concrete slab and headed studs in the ribs of a composite slab.



a. composite slab



b. solid concrete slab

Transfer of shear forces by studs with corresponding specific failure modes.

2- material properties

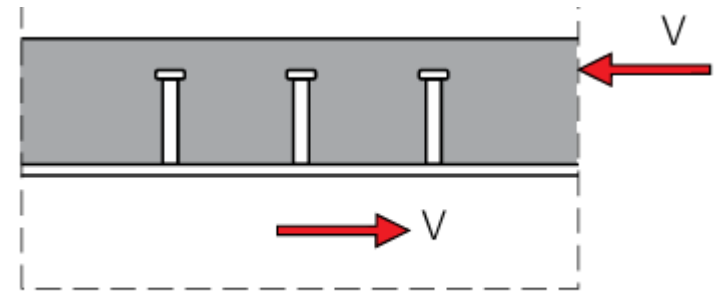
Headed studs in a solid concrete

slab Stud resistance is based on a highly simplified model that is similar to that of a bolt subjected to bearing and shearing.

The model assumes, based on test results, that at a **low concrete strength**, crushing of the concrete governs resistance, whereas at a **high concrete strength** shearing of the stud is critical.

According to this model, either of these two independent criteria may determine the strength of a headed stud.

Sheeting with ribs transverse to the supporting beam.



b. solid concrete slab

2– material properties

The **design value of the shear resistance of a welded headed stud** is therefore equal to the **smaller** value of:

$$P_{Rd} = \frac{0,8f_u \frac{1}{4} \pi d^2}{\gamma_V} \quad (\text{shear failure of the stud}) \quad (2.2)$$

and

$$P_{Rd} = \frac{0,29\alpha d^2 \sqrt{f_{ck} E_{cm}}}{\gamma_V} \quad (\text{local concrete crushing in front of the stud}) \quad (2.3)$$

Where:

f_u specified ultimate tension strength of the material of the headed stud, but not more than 500 N/mm². A common value is $f_u = 450$ N/mm²;

d diameter of the shank of the stud ($16 \text{ mm} \leq d \leq 25 \text{ mm}$);

α calibration factor reflecting the influence of the length of the stud:

$$\alpha = 1 \quad \text{if } h_{sc} > 4d$$

$$\alpha = 0,2 \left(\frac{h_{sc}}{d} + 1 \right) \quad \text{if } 3d \leq h_{sc} \leq 4d$$

2– material properties

Design value P_{Rd} (kN) for headed studs for $h_{sc} \geq 4d$.(table 2-5)

	grade or class	diameter d (mm)		
		16	19	22
equation (2.2)	S450	57,9	81,7	109,5
	S500	64,3	90,7	121,6
equation (2.3) with $\alpha = 1,0$	C25/30	52,3	73,7	98,9
	C30/37	59,1	83,3	111,7
	C35/45	64,8	91,4	122,5
	C40/50	70,3	99,1	132,9
	C45/55	75,6	106,6	142,9

Table gives a **summary of design resistances** for three diameters of headed studs in common strengths of normal weight concrete.

Headed studs in a composite slab

Figure(a) shows a section through a composite slab where the ribs run transversely to the supporting beam, and corresponding failure modes for the studs.

By way of comparison, figure b shows the same for a solid slab.

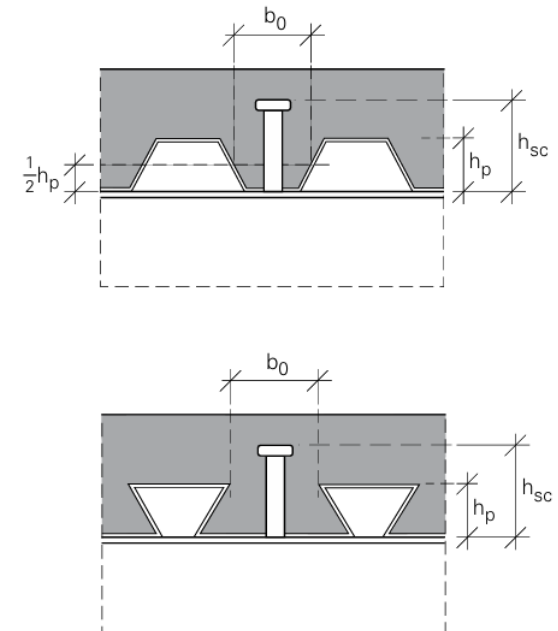
When the ribs are relatively **narrow or high**, the resistance of the studs is reduced. This potential reduction is taken into account by multiplying the **reference value** – i.e. the resistance of the stud in a solid concrete slab – by a reduction factor k_t .

A reduction must also be considered for the distinct case when the **ribs run parallel** to the beam – using a reduction factor k_r .

- **Ribs perpendicular to the steel beam.** This situation is illustrated in figure. The reduction factor is:

$$k_t = \frac{0,7}{\sqrt{n_r}} \cdot \frac{b_0}{h_p} \left(\frac{h_{sc}}{h_p} - 1 \right) \quad (2.4)$$

Where n_r is the number of studs in one rib, which should not exceed two. The value of k_t is subject to a maximum $k_{t,max}$ (table).



Sheeting with ribs transverse to the supporting beam.

Tab. (2-8) Upper limits $k_{t,max}$ for the reduction factor k_t .

n_r	thickness t of the sheet (mm)	$d \leq 20$ mm through deck welded	$d = 19$ or 22 mm in punched holes (or between sheets)
1	$\leq 1,0$	0,85	0,75
	$> 1,0$	1,00	0,75
2	$\leq 1,0$	0,70	0,60
	$> 1,0$	0,80	0,60

When determining the reference value for the resistance of a headed stud in a solid concrete slab, f_u should not be taken greater than 450 N/mm_2 . Furthermore, equation (2.4) only applies if the following three conditions are met:

- $h_p \leq 85 \text{ mm}$;
- $b_0 \geq h_p$;
- $d \leq 20 \text{ mm}$ for studs through deck welded, or $d \leq 22 \text{ mm}$ for studs in punched holes (or between sheets).

There are no assessment rules for welded studs with a diameter of 22 mm in the ribs of a composite slab, because the test results on which codified resistances are based are not sufficiently consistent.

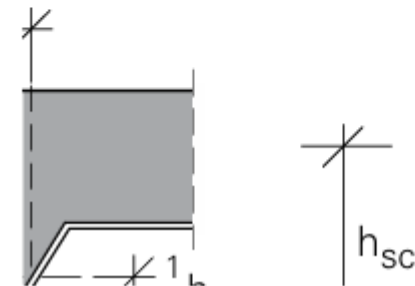
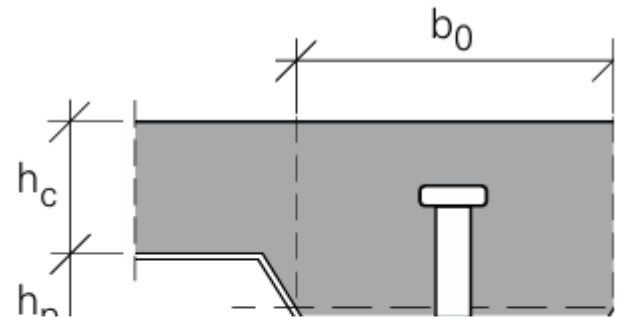
Ribs parallel to the steel beam.

This situation occurs when there is a structural arrangement in which the **primary and secondary beams are in one layer** (i.e. so that the upper surfaces of their top flanges are **at the same level**),

When the sheeting passes over the beam, the dimensions of the rib are determined by the dimensions of the sheeting.

Often the decking is not **continuous over a beam**, as shown in figure, in which case the headed studs are not welded through the deck but rather directly to the beam between adjacent sheets. The reduction factor for this situation is:

$$k_{\ell} = 0,6 \frac{b_0}{h_p} \left(\frac{h_{sc}}{h_p} - 1 \right) \leq 1,0 \quad \text{with} \quad h_{sc} \leq h_p \quad (2.5) \quad \text{mm}$$



If the sheeting above a beam is **discontinuous**, the sheeting must be properly **attached to the beam** so that the concrete in the rib is suitably confined.

If this is **not the case** the presence of the sheeting should be ignored, and reinforcement added according to EN 1994-1-1 cl. 6.6.5.4

4 Classification of cross-sections

The resistance of a cross section at **the ultimate limit state** may be based on either an **elastic or a plastic** stress distribution.

Global analysis at the ultimate limit state may be based on one of the four methods given below, according to EN 199411, cl. 5.4:

- linear elastic theory (cl. 5.4.2);
- **nonlinear global analysis (cl. 5.4.3);**
- linear elastic analysis with limited redistribution (cl. 5.4.4);
- **rigid plastic global analysis (cl. 5.4.5).**

A **rigid plastic global analysis** is preferred because of its simplicity, and the freedom of choice it gives for design, but this **method may only be used under** certain circumstances.

The type of analysis that may be used depends, amongst other things, on the **cross section classification of the beam** (EN 1994-1-1, cl. 5.5). EN 1994 follows the same classification system for composite beams as given in EN 199311, cl. 5.5.2 for steel structures. The cross section class depends on the **steel grade**, the **stress distribution**, and the **width to thickness ratio** of the cross section elements in compression..

4 Classification of cross-sections

The following should be noted

- The **flanges** of all IPE, HEB and HEM sections up to grade S355 steel belong in **class 1**.
- For some **HEA sections** the flanges do **not fall into class 1**.
- An upper steel flange that is fixed to a concrete slab by shear connectors may be taken as

class 1 provided that the stud spacing s complies with:

- $s \leq 22t_f \varepsilon$ (concrete slab is in contact with the beam over the full length); or
- $s \leq 15t_f \varepsilon$ (concrete slab with ribs transverse to the beam).

Where $\varepsilon = \sqrt{235/f_y}$ and f_y in N/mm².

- The webs of a **relatively large number of rolled sections** do not fall into class 1 when the steel cross section is subjected to a combination of bending moment and compression.

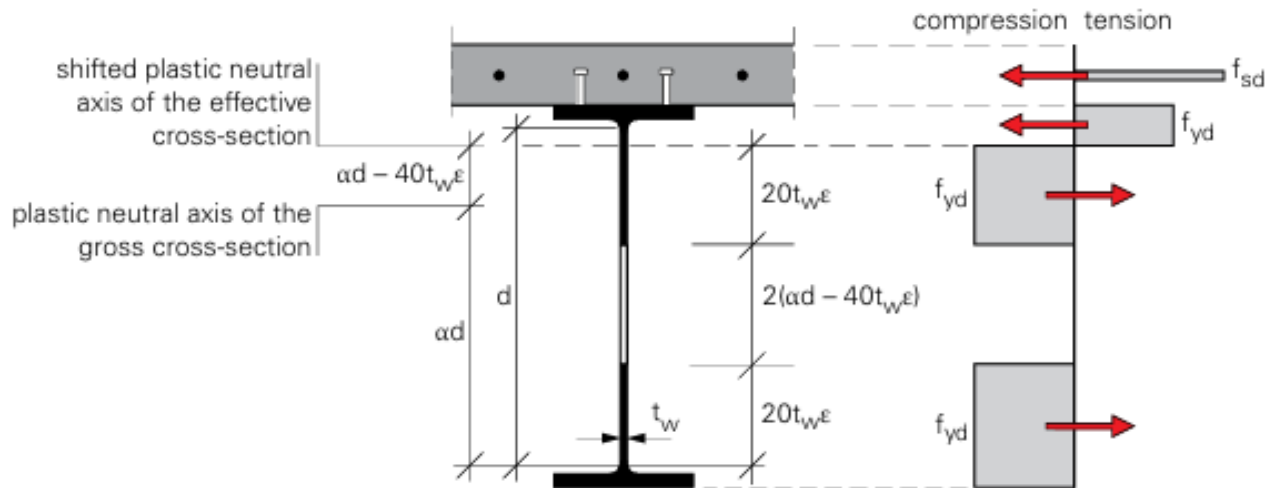
This is particularly the case for composite beams subjected to a hogging moment.

4 Classification of cross-sections

- A web that would otherwise be class 3 may be taken as class 2 with an effective area as shown in figure.

The factor α in this figure represents that part of the depth d that is in compression for the full effective cross section. The dimension d is defined as:

$$d = h_a - 2t_f - 2r \quad (2.6)$$



A web in cross section class 3 may be converted in an effective cross section in class 2.

4 Classification of cross-sections

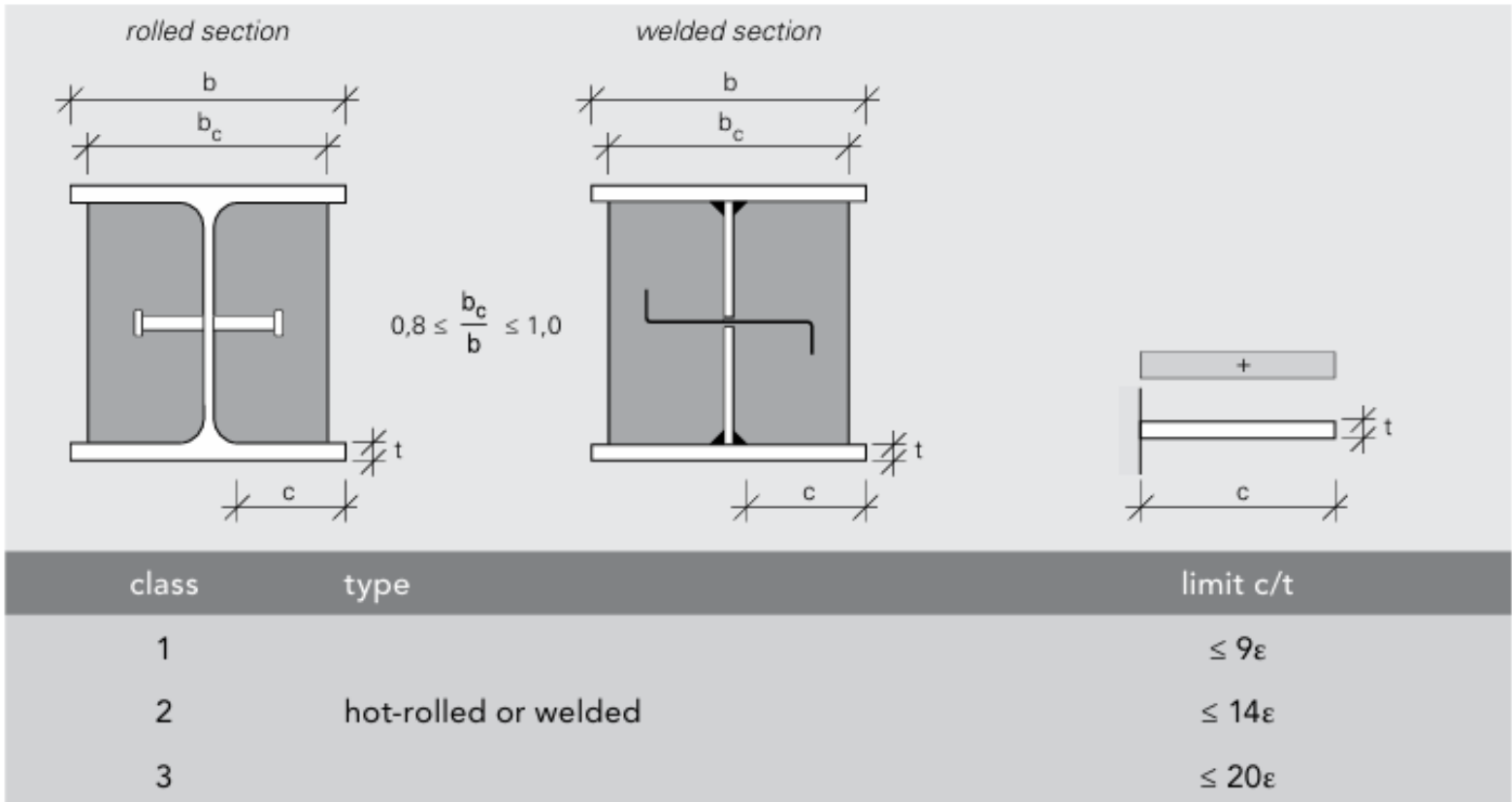
- When the web is assumed to resist only a **shear force** and no **moment or normal force**, the section classification may be based on the cross section classification of the **flanges alone**, with the exception of cross section class 1, according to EN 1993-1-1 cl. 5.5.2.(12).

The **web and flanges of partially encased steel sections** are supported against local buckling by the concrete encasement.

This kind of section may be **classified according to the values presented in table**.

A steel web that would otherwise be in class 3, when encased in concrete in accordance with EN 1994-1-1, cl. 5.5.3(2), may be 'upgraded' due to the presence of the supporting concrete to class 2.

4 Classification of cross-sections



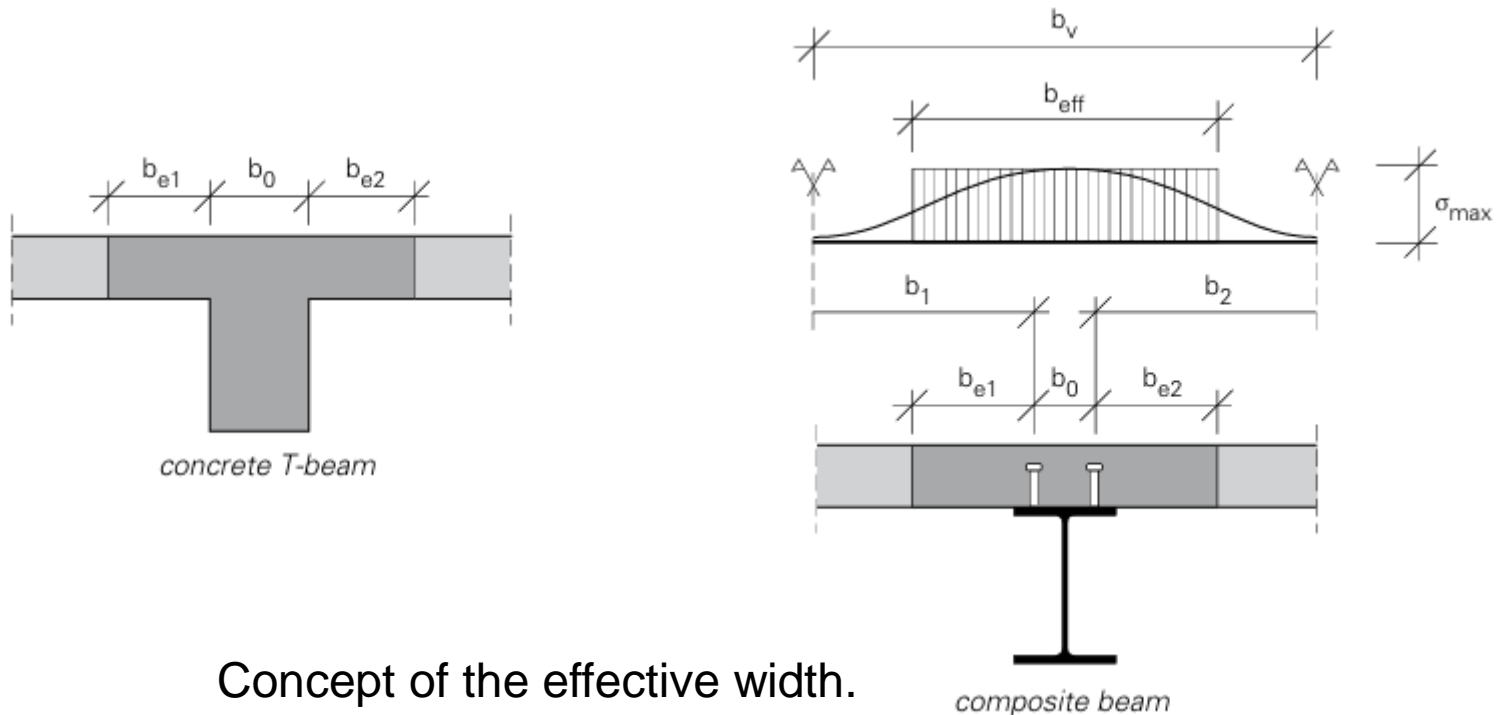
Classification of steel flanges in compression for partially encased sections.

5 Effective width

When considering the behaviour of a composite beam it is assumed – as with other types of beam – that plane sections remain plane (after bending).

However, the strains (and therefore the stresses) at the edges of the wide concrete ‘top flange’ are actually smaller than those close to the beam web due to shear lag (fig). It is common practice to take this effect into account by assuming a reduced effective width b_{eff} , rather than the full width b_w , in calculations.

This approach is also used for concrete T beams (see EN 199211, fig. 5.3).



Concept of the effective width.

The effective width of a concrete slab may be determined as:

$$b_{\text{eff}} = b_0 + \Sigma b_{\text{ei}} \quad \text{with} \quad b_{\text{ei}} = \frac{L_e}{8} \leq b_i \quad (2.8)$$

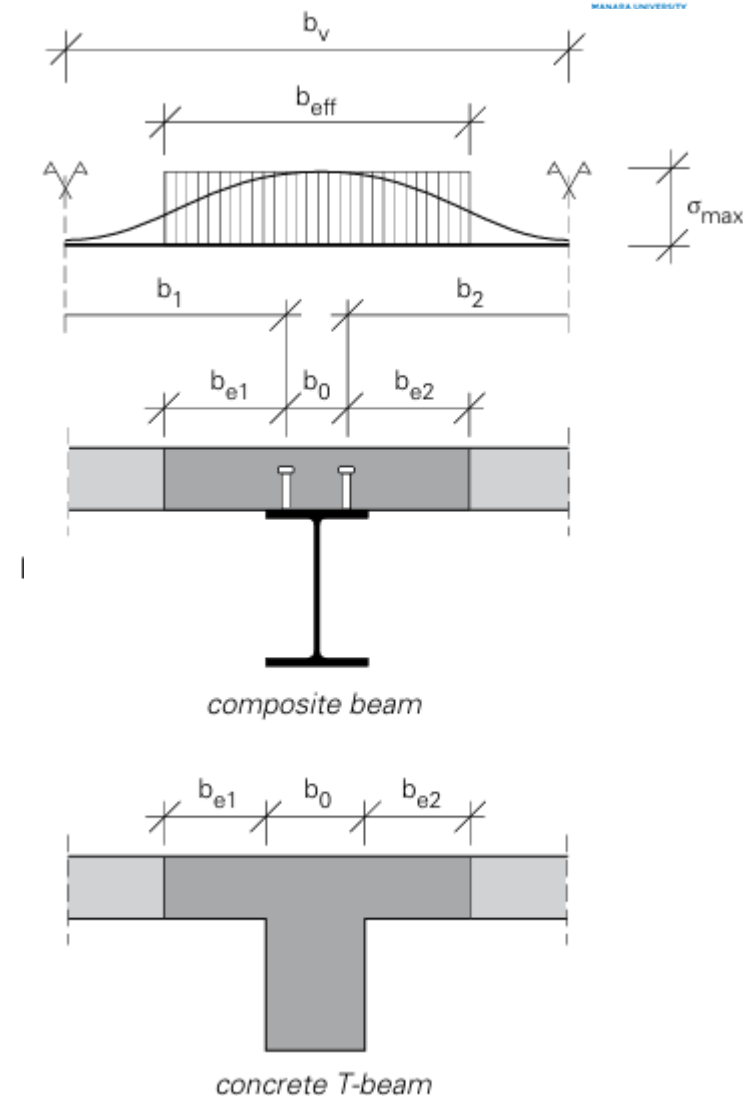
Where:

b_0 transverse distance (distance between the centres) of the outer studs if two or more (transverse) rows of studs are used;

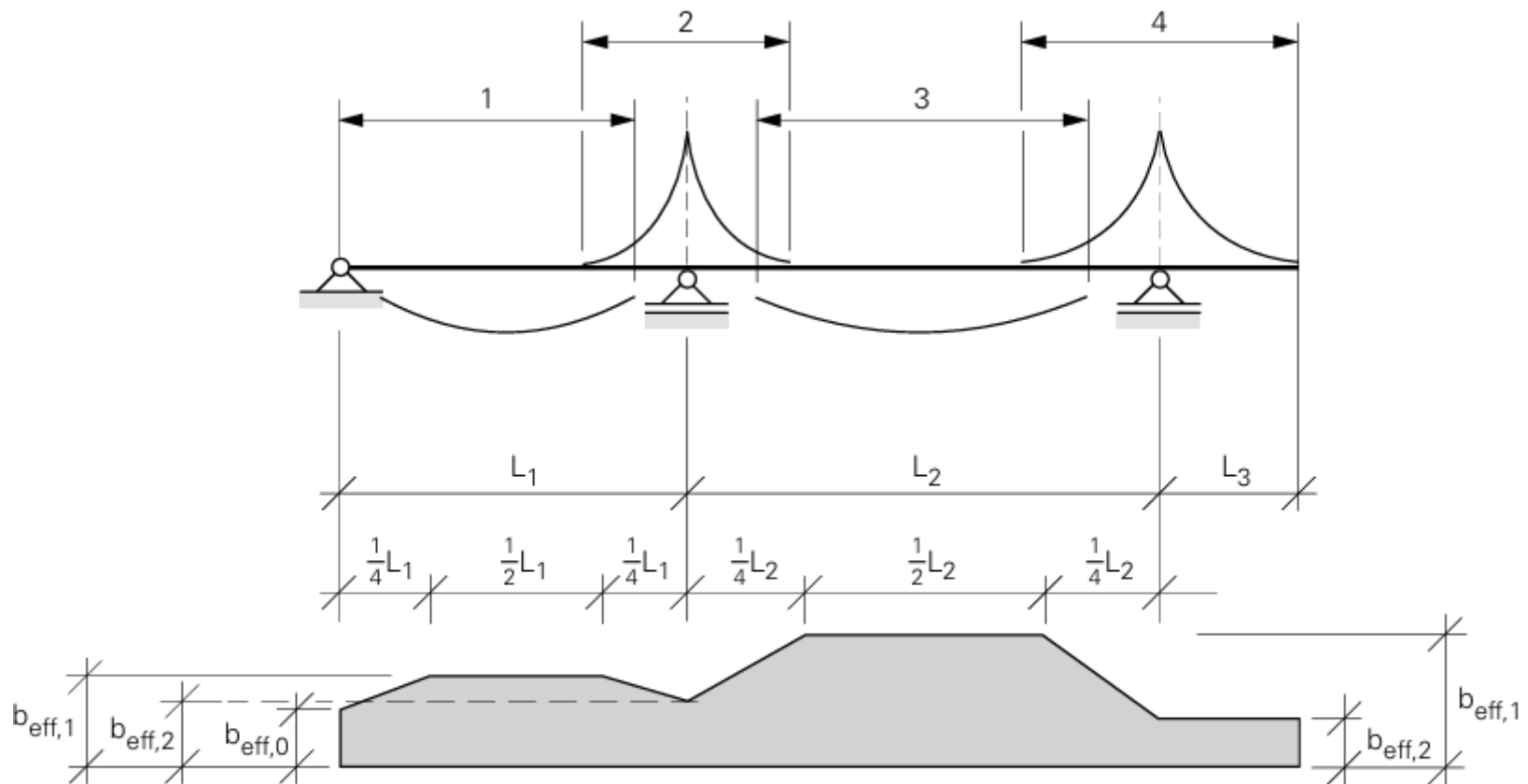
b_{ei} auxiliary term;

b_i distance from the outer row of studs (or centre line if there is a single row) to the mid point between adjacent beam webs, measured at mid-depth of the concrete flange (except that at a free edge b_i is the distance to the free edge);

L_e for a simply supported beam L_e is equal to the span and for a continuous beam L_e is the distance between points of zero bending moment, however an approximate value can normally be used as shown in figure 2.13 (see EN 1994-1-1, cl. 5.4.1.2).



Concept of the effective width.



- area 1: $L_e = 0,85L_1$ for $b_{eff,1}$
- area 2: $L_e = 0,25(L_1 + L_2)$ for $b_{eff,2}$
- area 3: $L_e = 0,70L_2$ for $b_{eff,1}$
- area 4: $L_e = 2L_3$ for $b_{eff,2}$

Determination of the length L_e for a continuous composite beam.

As a simplification (for building design) the beams may be assumed to have a **constant effective width** over the entire length in sagging of each span.

This width is the value of $b_{\text{eff},1}$, corresponding to the effective width at midspan.

Similarly,

a value of $b_{\text{eff},2}$ may be assumed on both sides of an intermediate support for members subject to a hogging moment

Properties of composite cross sections

This section considers the properties of composite beams that section in combination with a composite comprise an unencased steel or solid concrete slab.

Equations for the resistance and stiffness of the cross-section are given for different situations (cases 1 to 5), depending on the position of the neutral axis.

The following properties are discussed below:

- **plastic moment resistance** $M_{pl,Rd}$ and flexural stiffness EI in sagging and hogging bending;
- **elastic moment resistance** $M_{el,Rd}$ and flexural stiffness EI in sagging and hogging bending;
- **resistance to vertical shear** V_{Rd} ;
- **resistance to combined** bending and vertical shear.

Consideration is also given how to deal with partially encased beams.

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.

Properties of composite cross sections

Symmetrical sections made of grade S355 steel or lower

The starting point is to assume that there will be **sufficient shear connectors** to fully exploit the strength of the steel or concrete (whichever is the lower).

This means that the **longitudinal shear** connection will not be critical.

The **resistance of a cross-section** in **sagging** depends on the location of the plastic neutral axis.

The neutral axis might be positioned in:

- the concrete slab;
- the upper flange of the steel section;
- the web of the steel section.

The equations given below have been established for a **composite beam** with a composite slab.

However, they also apply to solid slabs, in which case $h_p = 0$.

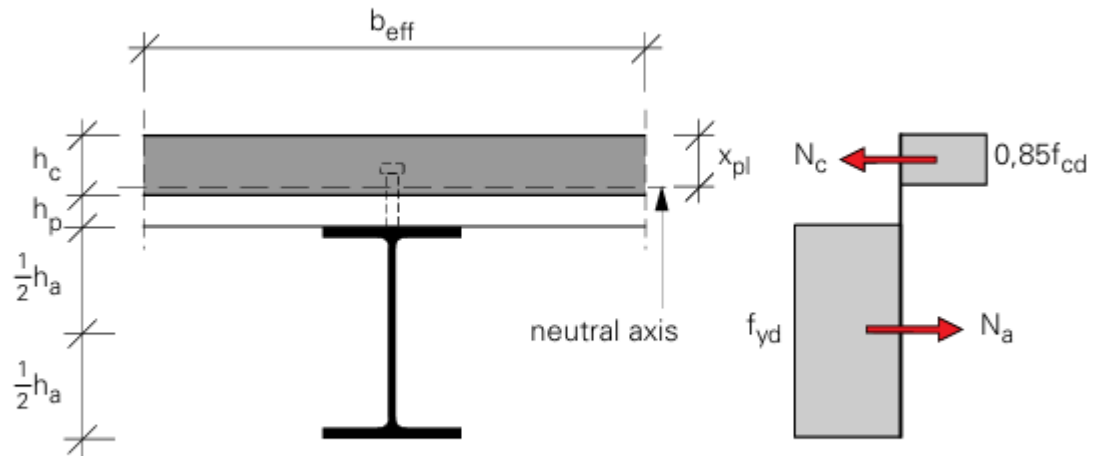
Properties of composite cross sections

- **Case 1. Neutral axis in the concrete slab**

. This situation occurs when the **axial resistance of the steel beam** is less than that of the concrete slab, or $Af_{yd} < h_c b_{eff} 0,85f_{cd}$

$$M_{pl,Rd} = Af_{yd} \left(\frac{1}{2}h_a + h_p + h_c - \frac{1}{2}x_{pl} \right) \quad (2.9)$$

$$x_{pl} = \frac{Af_{yd}}{b_{eff} 0,85f_{cd}}$$



Stress distribution with the plastic neutral axis in the concrete slab (case 1).

Properties of composite cross sections

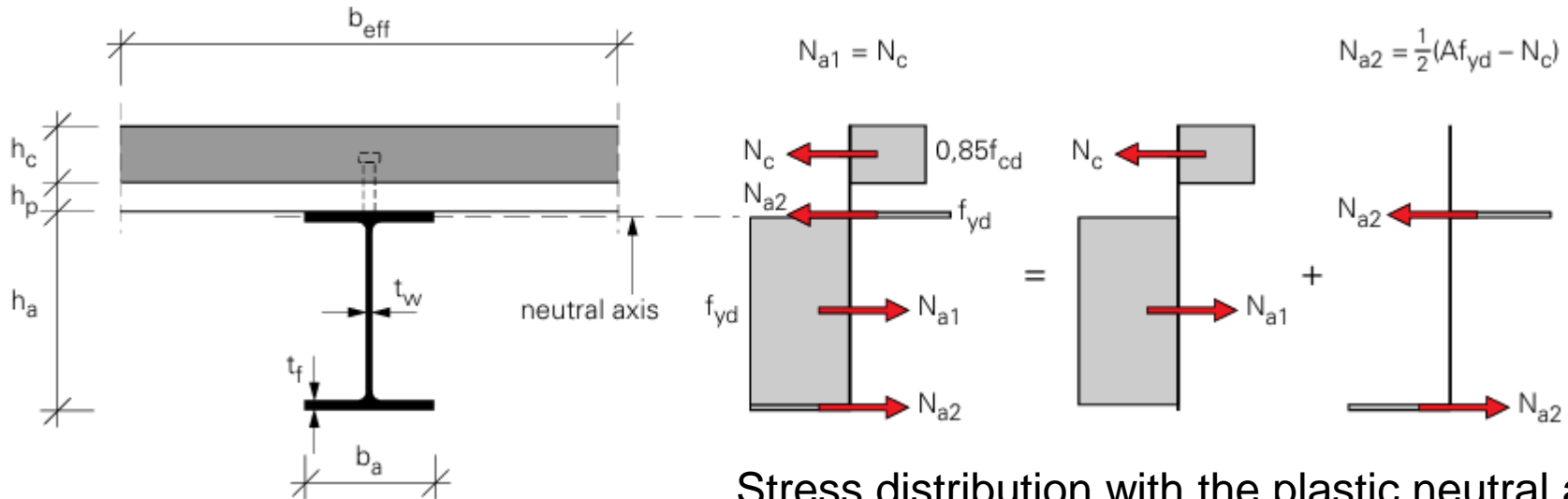
Case 2. Neutral axis in the upper steel flange.

This situation occurs when the resistance of the concrete slab is **less than that** of the steel beam, but greater than that of the steel web alone, so

$$Af_{yd} > h_c b_{eff} 0,85f_{cd} > (h_a - 2t_f)t_w f_{yd}$$

$$M_{pl,Rd} = N_c \left(\frac{1}{2}h_a + h_p + \frac{1}{2}h_c \right) + \frac{1}{2} \left(Af_{yd} - N_c \right) \left(h_a - \frac{Af_{yd} - N_c}{2b_a f_{yd}} \right) \quad (2.10)$$

$$N_c = h_c b_{eff} 0,85f_{cd}$$



Stress distribution with the plastic neutral axis in the flange of the steel section (case 2).

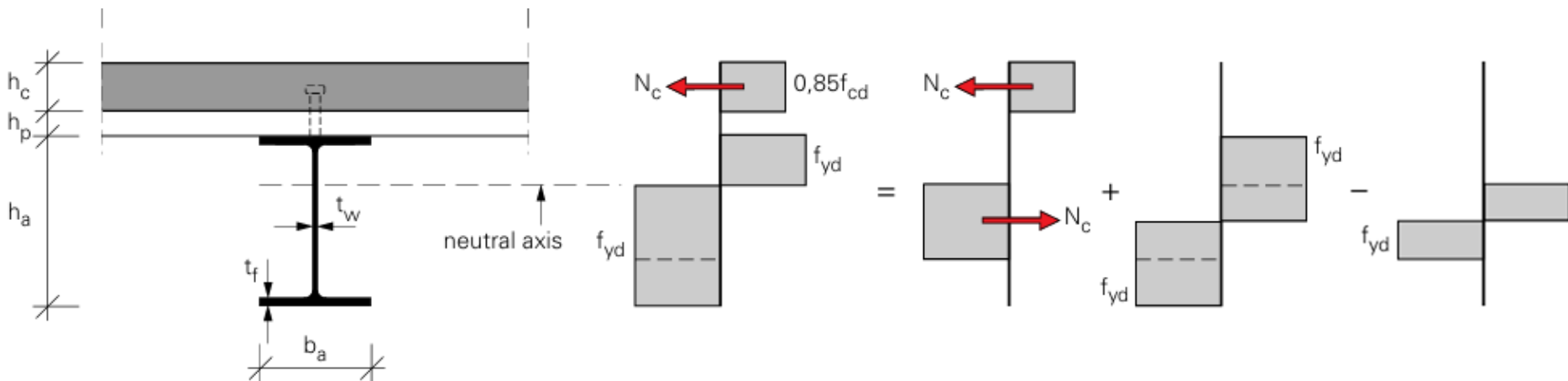
Properties of composite cross sections

• Case 3. Neutral axis in the web of the steel section.

This situation occurs when the resistance of the concrete slab is less than that of the web alone of the steel beam, $h_c b_{\text{eff}} 0,85 f_{\text{cd}} < (h_a - 2t_f) t_w f_{\text{yd}}$

$$M_{\text{pl,Rd}} = N_c \left(\frac{1}{2} h_a + h_p + \frac{1}{2} h_c \right) + W_{\text{apl}} f_{\text{yd}} - \frac{N_c^2}{4 t_w f_{\text{yd}}} \quad (2.11)$$

$$N_c = h_c b_{\text{eff}} 0,85 f_{\text{cd}}$$



Stress distribution with the plastic neutral axis in the web of the steel section (case 3).

Example 2.1

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

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

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

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

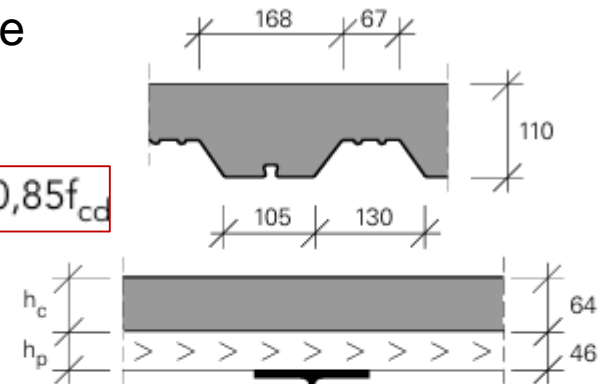
$$A f_{yd} < h_c b_{eff} 0,85 f_{cd}$$

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

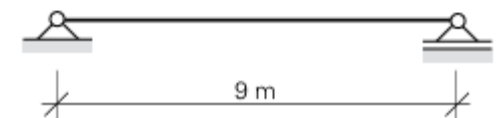
$$x_{pl} = \frac{A f_{yd}}{b_{eff} 0,85 f_{cd}} = \frac{53,8 \cdot 10^2 \cdot 235}{2250 \cdot 0,85 \cdot 16,7} = 40 \text{ mm} < h_c = 64 \text{ mm}$$

$$M_{pl,Rd} = A f_{yd} \left(\frac{1}{2} h_a + h_p + h_c - \frac{1}{2} x_{pl} \right) \cdot 10^{-6}$$

$$= 53,8 \cdot 10^2 \cdot 235 \cdot \left(\frac{1}{2} \cdot 300 + 46 + 64 - \frac{1}{2} \cdot 40 \right) \cdot 10^{-6} = 303 \text{ kNm}$$

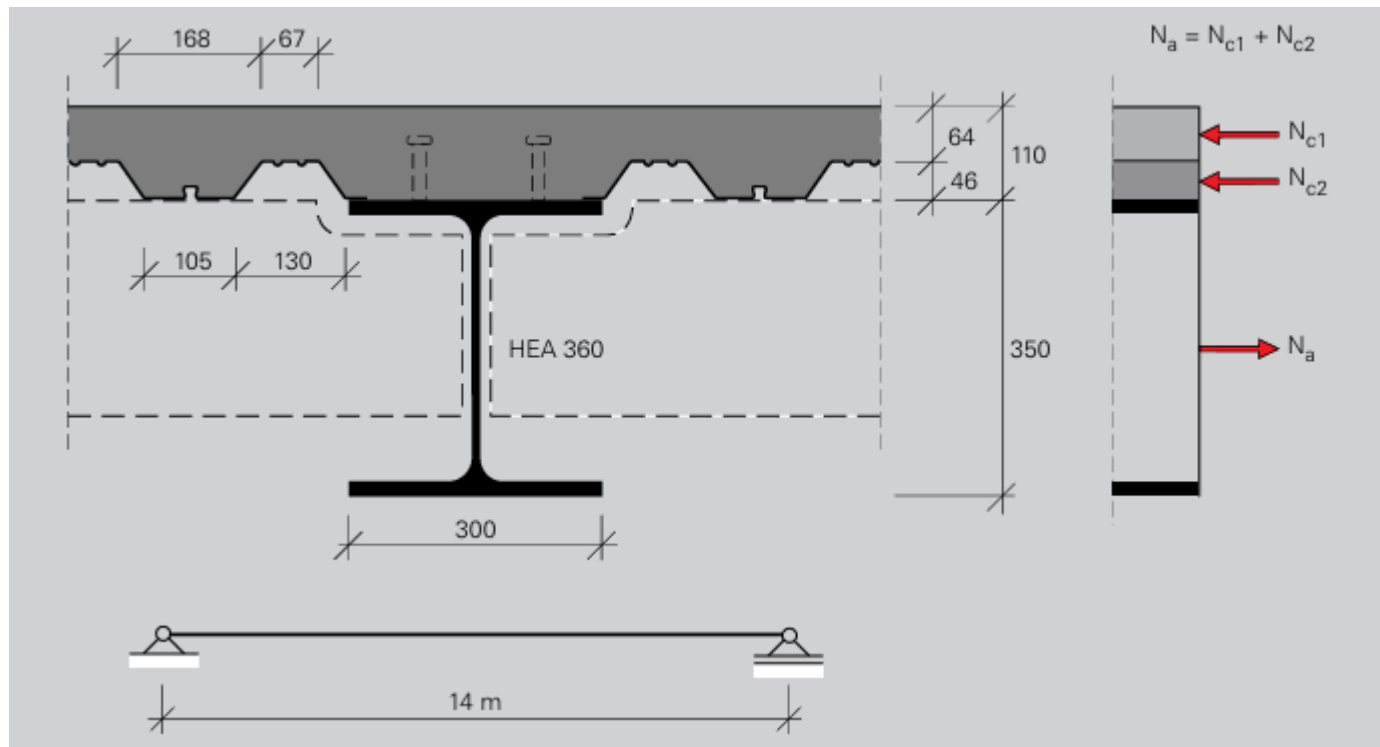


Simply supported composite beam with decking ribs running transverse to the supporting steel beam.



Example 2.2

- **Given.** A statically determinate beam HEA 360 beam with $f_{yd} = 355 \text{ N/mm}^2$ and a span $L = 14,0 \text{ m}$. Key dimensions of the composite floor are $h_c = 64 \text{ mm}$ and $h_p = 46 \text{ mm}$. The ribs of the steel sheeting run parallel to the beam axis (fig. 2.18). The concrete is of class C25/30 with $f_{cd} = 16,7 \text{ N/mm}^2$.
- **Question.** Calculate the plastic resistance moment $M_{pl,Rd}$.



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

- Answer. The effective width is $b_{\text{eff}} = 2 \cdot 14000 / 8 = 3500$ mm (any contribution of b_0 has been neglected).

Because the ribs of the sheeting run parallel to the beam, the concrete resistance consists of two components, N_{c1} and N_{c2} .

The resistance N_{c1} of the upper, solid part of the concrete slab is equal to:

$$M_{pl,Rd} = N_{c1} \left(\frac{1}{2} h_a + h_p + \frac{1}{2} h_c \right) + N_{c2} \left(\frac{1}{2} h_a + \frac{1}{2} h_p \right) + \frac{1}{2} (A f_{yd} - N_c) \left(h_a - \frac{A f_{yd} - N_c}{2 b_a f_{yd}} \right)$$

$$N_{c1} = h_c \cdot b_{\text{eff}} \cdot 0,85 \cdot f_{cd} = 64 \cdot 3500 \cdot 0,85 \cdot 16,7 \cdot 10^{-3} = 3180 \text{ kN}$$

The resistance N_{c2} of the ribbed, lower part of the concrete slab follows from the geometry of the profiled steel sheeting:

$$N_{c2} = 3500 \cdot \frac{105 + 168}{2 \cdot (130 + 105)} \cdot 46 \cdot 0,85 \cdot 16,7 \cdot 10^{-3} = 1327 \text{ kN}$$

The total resistance of the concrete slab is thus equal to:

$$N_c = N_{c1} + N_{c2} = 3180 + 1327 = 4507 \text{ kN}$$

Furthermore:

$$Af_{yd} = 142,8 \cdot 10^2 \cdot 355 \cdot 10^{-3} = 5069 \text{ kN}$$

$$(h_a - 2t_f)t_w f_{yd} = (350 - 35) \cdot 10 \cdot 355 \cdot 10^{-3} = 1118 \text{ kN}$$

and so:

$$Af_{yd} > h_c b_{\text{eff}} 0,85 f_{cd} > (h_a - 2t_f)t_w f_{yd}$$

$$Af_{yd} = 5069 > N_c = 4507 > (h_a - 2t_f)t_w f_{yd} = 1118 \text{ kN}$$

The neutral axis is thus in the upper flange of the steel section, so that case 2 applies:

$$M_{pl,Rd} = N_{c1} \left(\frac{1}{2} h_a + h_p + \frac{1}{2} h_c \right) + N_{c2} \left(\frac{1}{2} h_a + \frac{1}{2} h_p \right) + \frac{1}{2} (Af_{yd} - N_c) \left(h_a - \frac{Af_{yd} - N_c}{2b_a f_{yd}} \right)$$

$$= 3180 \cdot (175 + 46 + 32) \cdot 10^{-3} + 1327 \cdot (175 + 23) \cdot 10^{-3} + \frac{1}{2} \cdot (5069 - 4507) \cdot \left(350 - \frac{(5069 - 4507) \cdot 10^3}{2 \cdot 300 \cdot 355} \right) \cdot 10^{-3} = 805 + 263 + 98 = 1166 \text{ kNm}$$