
# Advanced Eurocode Training

## EN 1994-1-1: Composite Structures

# Table of contents

# Overview

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

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

EN1994-1-1 describes the Principles and requirements for safety, serviceability and durability of composite steel and concrete structures, together with specific provisions for buildings. It is based on the limit state concept used in conjunction with a partial factor method.

National choice is allowed in EN 1994-1-1 through the following clauses:
- 2.4.1.1(1)
- 2.4.1.2(5)
- 2.4.1.2(6)
- 2.4.1.2(7)
- 3.1(4)
- 3.5(2)
- 6.4.3(1)(h)
- 6.6.3.1(1)
- 6.6.3.1(3)
- 6.6.4.1(3)
- 6.8.2(1)
- 6.8.2(2)
- 9.1.1(2)
- 9.6(2)
- 9.7.3(4)
- 9.7.3(8)
- 9.7.3(9)
- B.2.5(1)
- B.3.6(5)

# EN 1994-1-1 Design of composite steel and concrete structures: General rules and rules for buildings

## Section 1: General

The following subjects are dealt with in EN 1993-1-1:

| | |
|---|---|
| Section 1: | General |
| Section 2: | Basis of design |
| Section 3: | Materials |
| Section 4: | Durability |
| Section 5: | Structural analysis |
| Section 6: | Ultimate limit states |
| Section 7: | Serviceability limit states |
| Section 8: | Composite joints in frames for building |
| Section 9: | Composite slabs with profiles steel sheeting for buildings |

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

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

## Section 2: Basis of design

Actions for the design of steel structures should be taken from EN 1991. For the combination for actions and partial factors of actions see Annex A to EN 1990.

Partial safety factors:

- Concrete: $\gamma_C$
- Steel reinforcement: $\gamma_S$
- Structural steel: $\gamma_M$

  steel sheeting: $\gamma_M$

  steel connecting devices: $\gamma_M$
- Shear connection: $\gamma_V$
- Longitudinal shear connection in composite slabs: $\gamma_{VS}$

  Fatigue verification of headed studs: $\gamma_{Mf}$ and $\gamma_{Mf,s}$

# Section 3: Materials

Concrete

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

Reinforcing steel

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

Structural steel

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

When looking at the Composite check in Scia Engineer, the reference article is EN 1994-1-1, 3.3:

| Cross-section Classification | Value | Unit | Reference |
|---|---|---|---|
| Design strength of steel | 235,0 | MPa | 3.3 |
| Effective width | 500 | mm | 5.4.1.2 |
| Critical section | 0,000 | m | |
| Flange class | Class 1 | | |
| Web class | Class 1 | | |
| Cross-section class | Class 1 | | 5.5 |

Which will refer to EN 1993-1-1,3.1 and 3.2

Connecting devices

Reference should be made to EN 1993-1-8 for requirements for fasteners and welding consumables. And for the headed stud shear connectors, reference should be made to EN 13918.

Profiled sheeting for composite slabs in buildings

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

# Section 4: Durability

The relevant provision given in EN 1990, EN 1992 and EN 1993 should be followed

# Section 5: Structural analysis

## Second order calculation and imperfections

Following EN 1994-1-1, §5.1.1(2)
Where the structural behaviour is essentially that of a reinforced or pre-stressed concrete structure, with only a few composite members, global analysis should be generally in accordance with EN 1992-1-1.

EN 1994-1-1 **article 5.2**

The action effects may generally be determined using either:

- First order analysis, using the initial geometry of the structure or
- Second-order analysis, taking into account the influence of the deformation of the structure.

First order analysis may be used if the increase of relevant internal forces or moments caused by the deformations given by first-)rder analysis is less than 10%. This condition may be assumed to be fulfilled, if the following criterion is satisfied:

$$\alpha_{cr} \geq 10 \qquad (5.1)$$

Where $\alpha_{cr:}$ The factor by which the design loading has to be increased to cause elastic instability.

EN 1994-1-1 **article 5.2.2**

- For beam-and-column type plane frames, $\alpha_{cr}$ may be calculated using the expression given in EN 1993-1-1, 5.2.1(4):

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right)$$

- If second-order effects in individual members and relevant member imperfections are fully accounted for in the global analysis of the structure, individual stability checks for the members are un-necessary.

**Global imperfections**

Should be taken into account in accordance with EN 1993-1-1, 5.3.2:

$$\varphi = \varphi_0\, \alpha_h\, \alpha_m$$

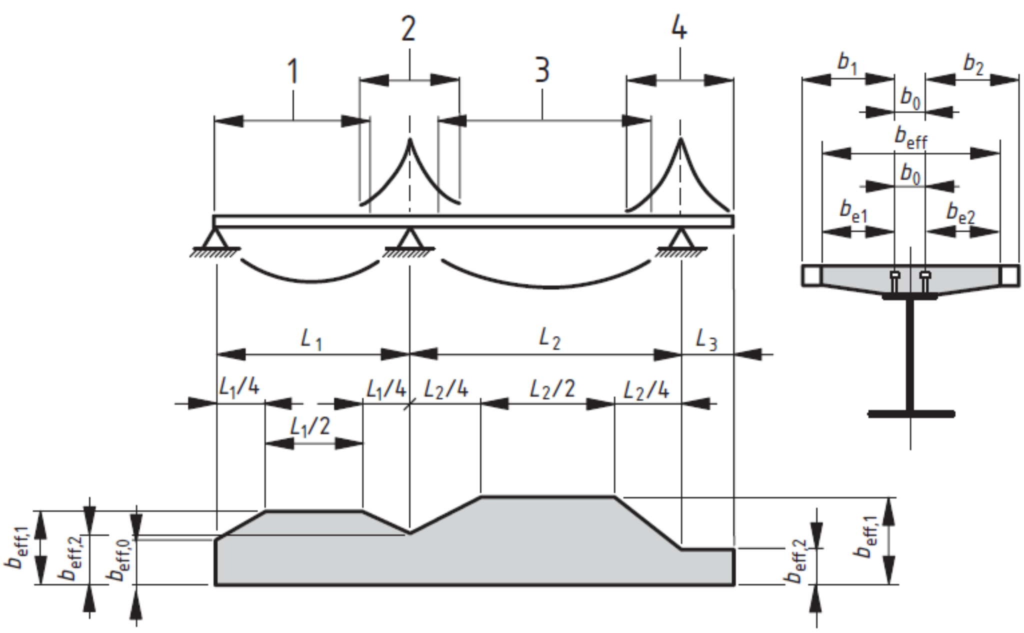
**Member imperfections**

The member imperfections are described in chapter6. Except for steel members the effects of imperfections are incorporated within the formulae given for buckling resistance, see EN 1993-1-1, 6.3.

## Effective width for flanges for shear lag

EN 1994-1-1 **article 5.4.1.2.**

- The effects of shear lag in steel plate elements should be considered in accordance with EN 1993-1-1, 5.2.1 (5).
- When elastic global analysis is used, a constant effective width may be assumed over the whole of each span. This value may be taken as the value $b_{eff,1}$ at mid-span for a span supported at both ends, or the value $b_{eff,2}$ at the support for a cantilever.




**Key**

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

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

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

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

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

- At the mid-span: $b_{eff}$:

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

Where: $b_0$ is the distance between the centers of the outstand shear connectors

$b_{ei}$ is the value of the effective width of the concrete flange on each side of the web and taken as $L_e$/ 8 but not greater than the geometric width $b_i$

$b$i should be taken as the distance from the outstand shear connector to a point mid-way between adjacent webs, measured at mid-depth

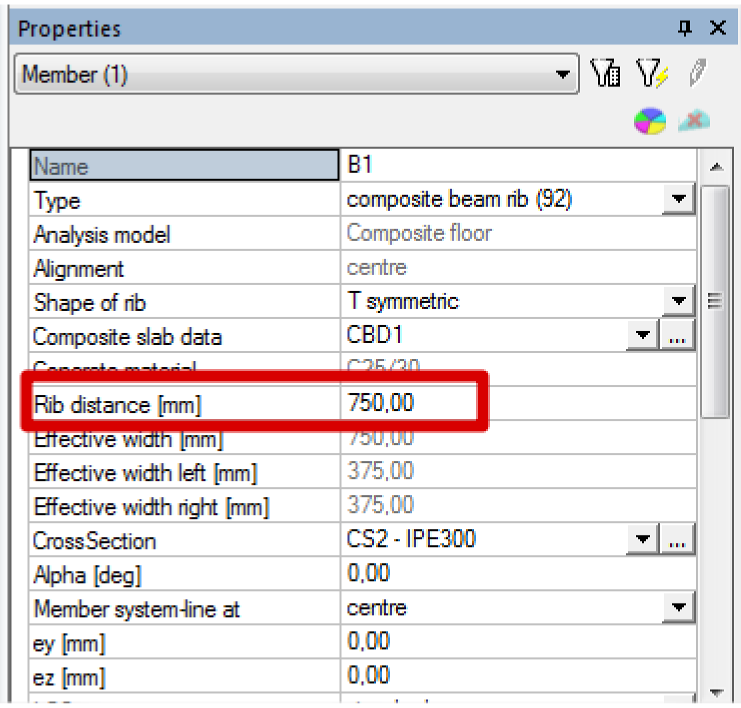
| | |
|---|---|
| | of the concrete flange, except that at a free edge $b_i$ is the distance to the free edge. |
| $L_e$ | should be taken as the approximate distance between points of zero bending moment. For typical continuous composite beams, where a moment envelope from various load arrangements governs the design, and for cantilevers, *Le* may be assumed to be as shown in Figure 5.1. |

- $b_{eff}$ at an end support::

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

| | |
|---|---|
| Where: $b_0$ | is the effective width, see previous formula, of the end span at mid-span |
| $L_e$ | is the equivalent span of the end span according to Figure 5.1. |
| $\beta_i$ | $= (0{,}55 + \frac{0{,}025 L_e}{b_{ei}}) \leq 1{,}0$ |

In Scia Engineer the user has to input the effective width himself in the properties of the beam:




And this inputted value will also be shown in the Composite check afterwards:

| Cross-section Classification | Value | Unit | Reference |
|---|---|---|---|
| Design strength of steel | 275.0 | MPa | 3.3 |
| Effective width | 750,00 | mm | 5.4.1.2 |
| Critical section | 0,000 | m | |
| Flange class | Class 1 | | |
| Web class | Class 1 | | |
| Cross-section class | Class 1 | | 5.5 |

## Creep and shrinkage

EN 1994-1-1 **article 5.4.2.2.**

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

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

Where:

| | |
|---|---|
| $n_0$ | is the modular ratio $E_a / E_{cm}$ for short-term loading |
| $E_{cm}$ | is the secant modulus of elasticity of the concrete for short-term loading according to EN 1992-1-1, Table 3.1 or Table 11.3.1 |
| $\varphi_t$ | is the creep coefficient $\varphi_{(t,t0)}$ according to EN 1992-1-1, 3.1.4 or 11.3.3, Depending on the age ($t$) of concrete at the moment considered and the age ($t_0$) at loading |
| $\psi_L$ | is the creep multiplier depending on the type of loading, which be taken as 1,1 for permanent loads, 0,55 for primary and secondary effects of shrinkage and 1,5 for prestressing by imposed deformations. |

## Linear elastic analysis with limited redistribution for buildings

EN 1994-1-1 **article 5.4.4.**
The bending moments in composite beams determined by linear elastic global analysis may be modified by reducing the maximum hogging moments by amounts not exceeding the percentages given in Table 5.1.
(also some other methods are given for Class 1 and Class2 profiles in this article).

**Table 5.1 : Limits to redistribution of hogging moments, per cent of the initial value of the bending moment to be reduced**

| Class of cross-section in hogging moment region | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| For un-cracked analysis | 40 | 30 | 20 | 10 |
| For cracked analysis | 25 | 15 | 10 | 0 |

For grades of structural steel higher than S355, redistribution should only be applied to beams with all cross-sections in Class 1 and Class 2. Redistribution by reduction of maximum hogging moments should not exceed 30% for an un-cracked analysis and 15% for a cracked analysis, unless it is demonstrated that the rotation capacity permits a higher value.

## Classification of cross-sections

### General

- The classification system defined in EN 1993-1-1, 5.5.2 applies to cross-sections of composite beams.
- For Cross-sections in Class 1 and Class 2 with bars in tension, reinforcement used within the effective width should have a ductility Class B or Class C (see EN 1992-1-1 Table C.1).
- A minimum area of reinforcement $A_s$ within the effective width of the concrete flange should be provided to satisfy the following condition:

$$A_s \geq \rho_s A_c$$

Where: $\rho_s = \delta \frac{f_y}{235} \frac{f_{ctm}}{f_{sk}} \sqrt{k_c}$

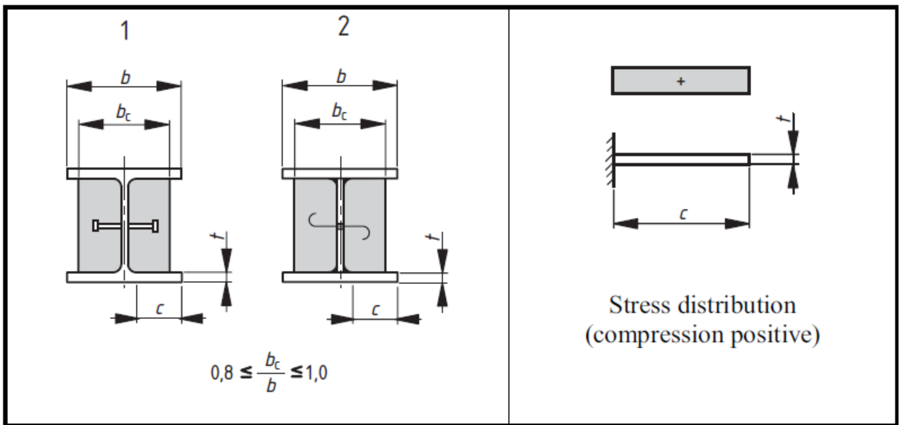
$A_c$ is the effective area of the concrete flange
$f_y$ is the nominal value of the yield strength of the structural steel in N/mm2
$f_{sk}$ is the characteristic yield strength of the reinforcement
$f_{ctm}$ is the mean tensile strength of the concrete, see EN1992-1-1, Table 3.1 or Table 11.3.1
$k_c$ is a coefficient given in 7.4.2
$\delta$ is equal to 1,0 for Class 2 cross-sections, and equal to 1,1 for Class 1 cross-sections at which plastic hinge rotation is required

## Classification of composite sections with concrete encasement

EN 1994-1-1 **article 5.4.4.**

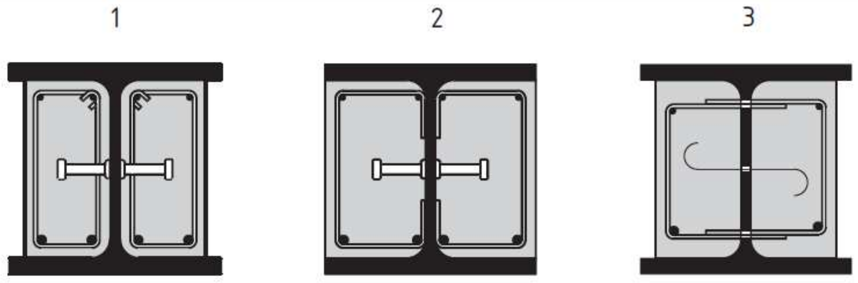
Next criteria has to be fulfilled

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




| Class | Type | Limit |
|---|---|---|
| 1 | (1) rolled or (2) welded | c/t ≤ 9ε |
| 2 | | c/t ≤ 14ε |
| 3 | | c/t ≤ 20ε |

- the concrete that encases a web is reinforced by longitudinal bars and stirrups, and/or welded mesh
- the requirements for the ratio $b_c$ / $b$ given in Table 5.2 are fulfilled
- the concrete between the flanges is fixed to the web in accordance with Figure 6.10 by welding the stirrups to the web or by means of bars of at least 6 mm diameter through holes and/or studs with a diameter greater than 10 mm welded to the web




**Key**

1 closed stirrups

2 open stirrups welded to the web

3 stirrups through the web

**Figure 6.10 : Arrangement of stirrups**

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

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

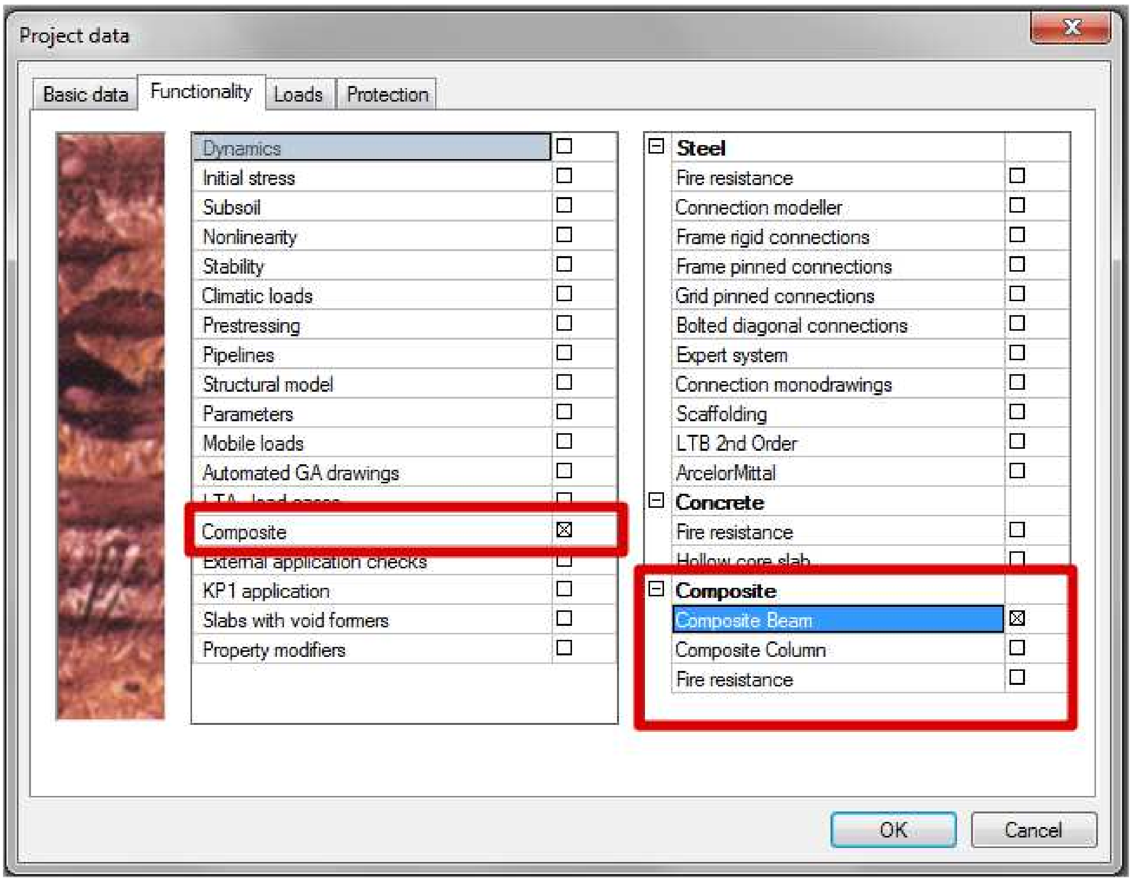
In Scia Engineer:

| Cross-section Classification | Value | Unit | Reference |
|---|---|---|---|
| Design strength of steel | 275,0 | MPa | 3.3 |
| Effective width | 750,00 | mm | 5.4.1.2 |
| Critical section | 0,000 | m | |
| Flange class | Class 1 | | |
| Web class | Class 1 | | |
| Cross-section class | Class 1 | | 5.5 |

# Section 6: Ultimate limit state

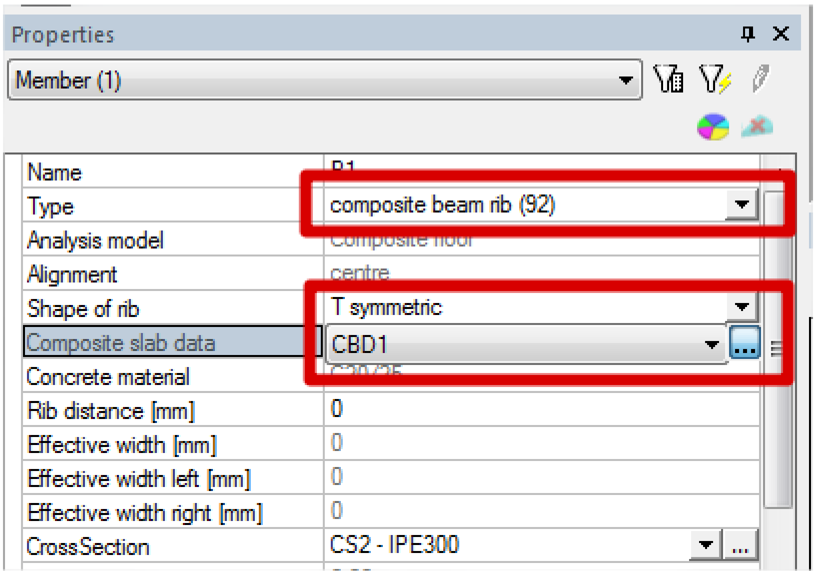
## Composite beam in Scia Engineer

Define a project with “Concrete” and “Steel as materials and activate the “Composite” and “Composite Beam” functionality:

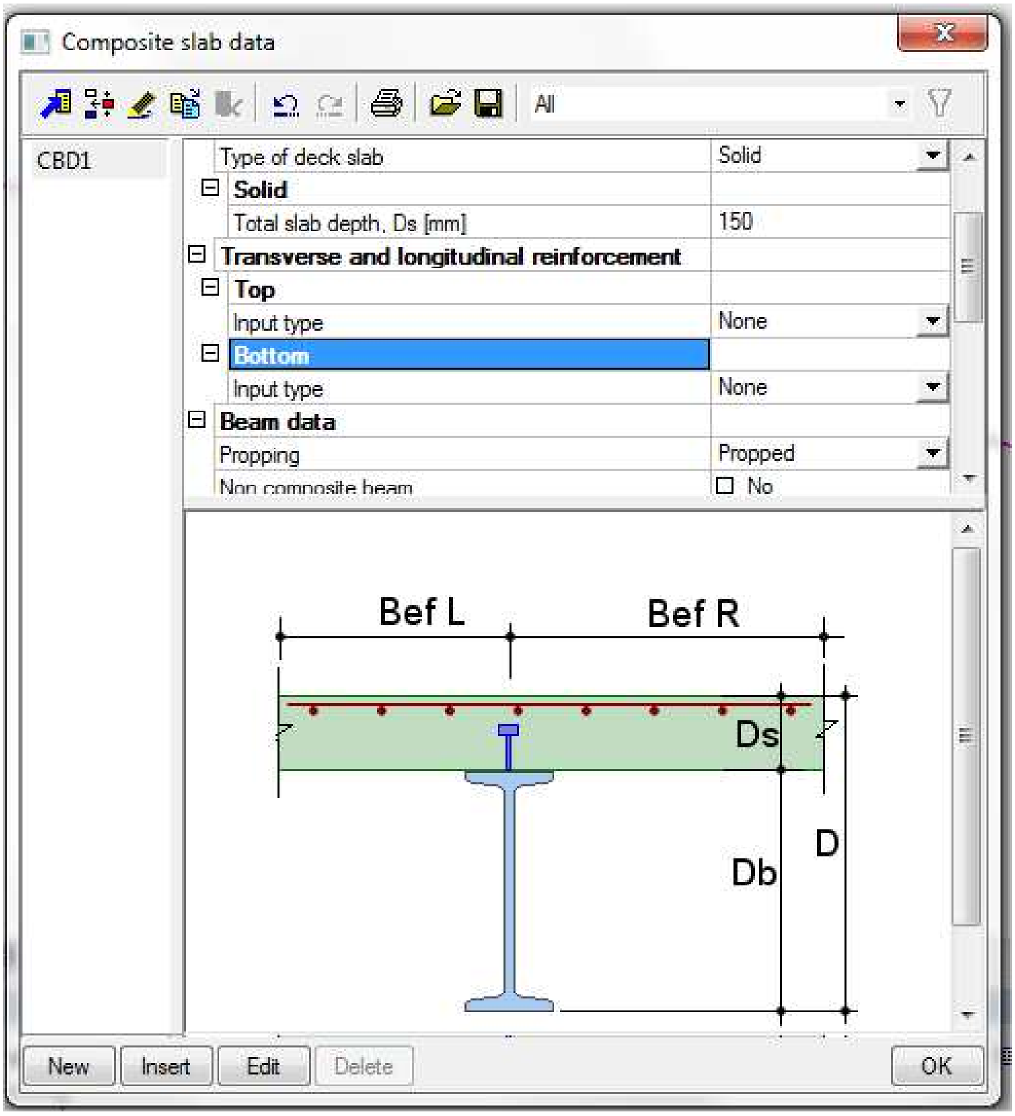



### Drawing a composite beam in Scia Engineer

Define a new composite cross section and draw a beam in Scia Engineer. When drawing a composite beam, the user should indicate that this is a composite beam in Scia Engineer. This can be chosen in the properties window of the beam, by changing the “Type” into “composite beam rib (92)”.




Afterwards the properties of the studs, the longitudinal reinforcement and the form of the slab can be indicated in "Composite slab data":




In below the options you can choose are explained:

**Type of beam**
The beam type options are primary / secondary. This input is used for reference only. The detailing of beams must be such that no significant torsion is transmitted to the beam. If this is not the case, additional calculations will need to be carried out.

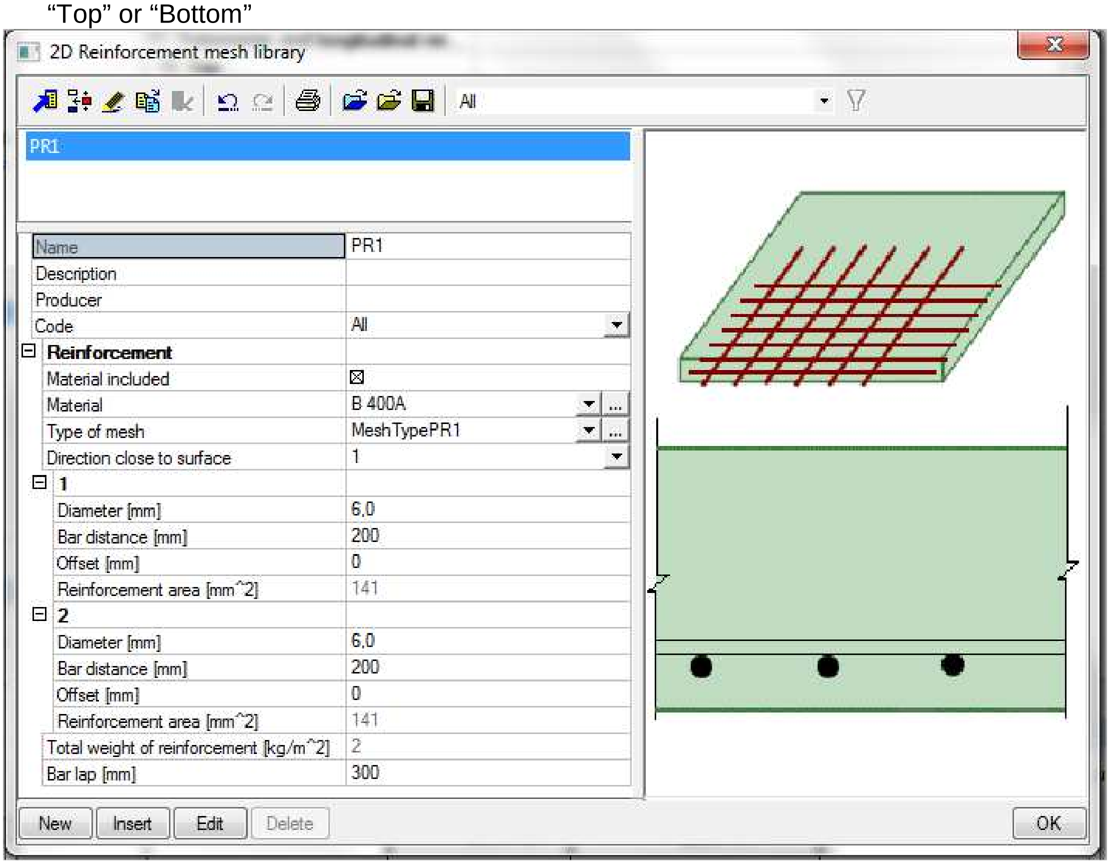
**Type of deck slab**
The types of slab supported are

- Solid
- Haunched solid
- Profiled deck

When the "Profiled deck" option is selected, an additional dialog is activated to help select a profile deck type from the program product libraries.
Also the option "Contribution to longitudinal shear resistance" will appear:
This is an option to include the resistance offered by the profiled sheet in the calculation of the slab longitudinal shear resistance. The sheet continuous option is disabled by default whenever the orientation of profiled sheeting on any one side is parallel.
Please note that this software does NOT design the deck or the slab. For primary beams, the deck direction is set parallel to the beam by default. User can modify the default left and right side rib directions. By default the beams generated by the composite load/floor panel are secondary beams with deck direction perpendicular to the beam. User can change this setting.

**Transverse and longitudinal reinforcement**
You can choose to input:

- None: no reinforcement
- User input: define the reinforcement just below
- Library: Choose the reinforcement from a library. You can adapt it to click on the three dots behind "Top" or "Bottom"



## Beam data
### Propping

Beam props may be introduced at the construction stage to reduce construction stage deflections of the beam or prevent the capacity of the steel beam exceeding before the concrete gains sufficient strength.

When the beam is specified as propped, this software assumes the beam is (theoretically) continuously propped along the whole length of the beam. All construction stage checks are omitted. In particular, steel stresses (before prop removal) are assumed to be zero.

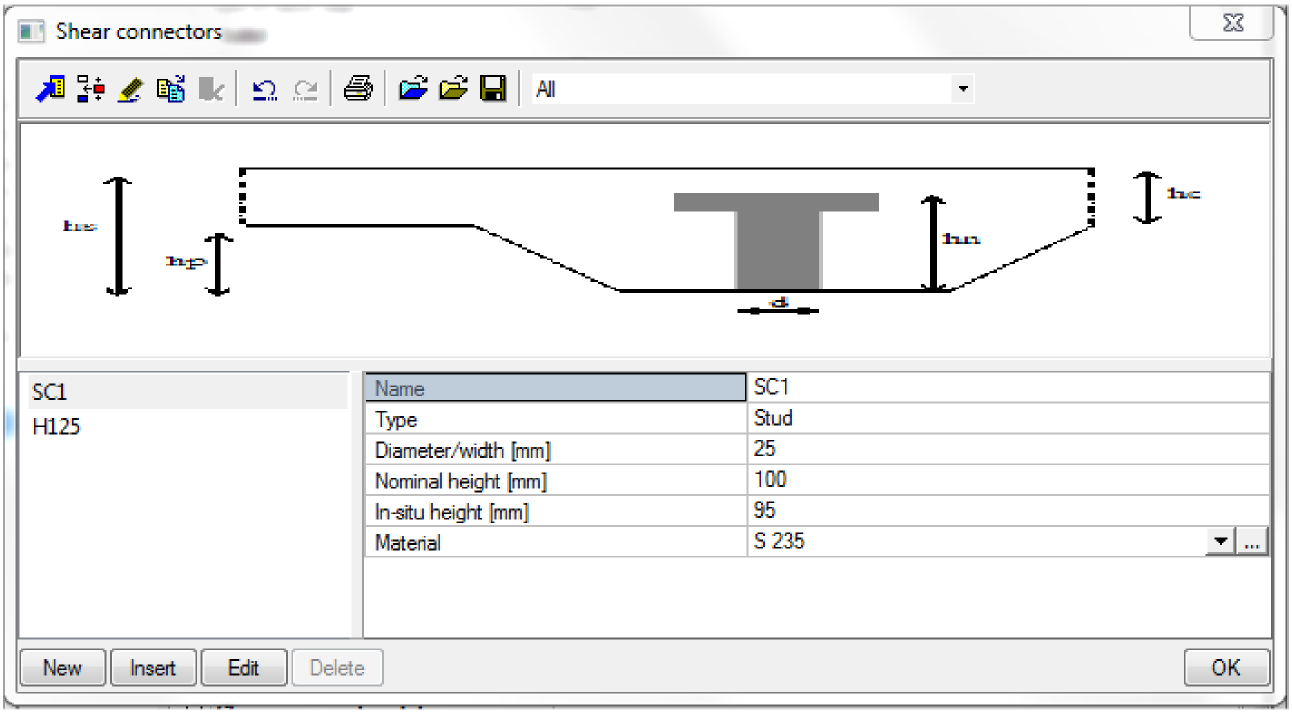
The beam may be propped or un-propped during construction. If the beam is propped during construction, construction stage analysis and design is not necessary. All beams may be designed assuming that at the ultimate limit state the whole of the loading acts on the composite member. If the beam is un-propped, construction stage checks are performed using the properties of the steel beam alone without composite action.

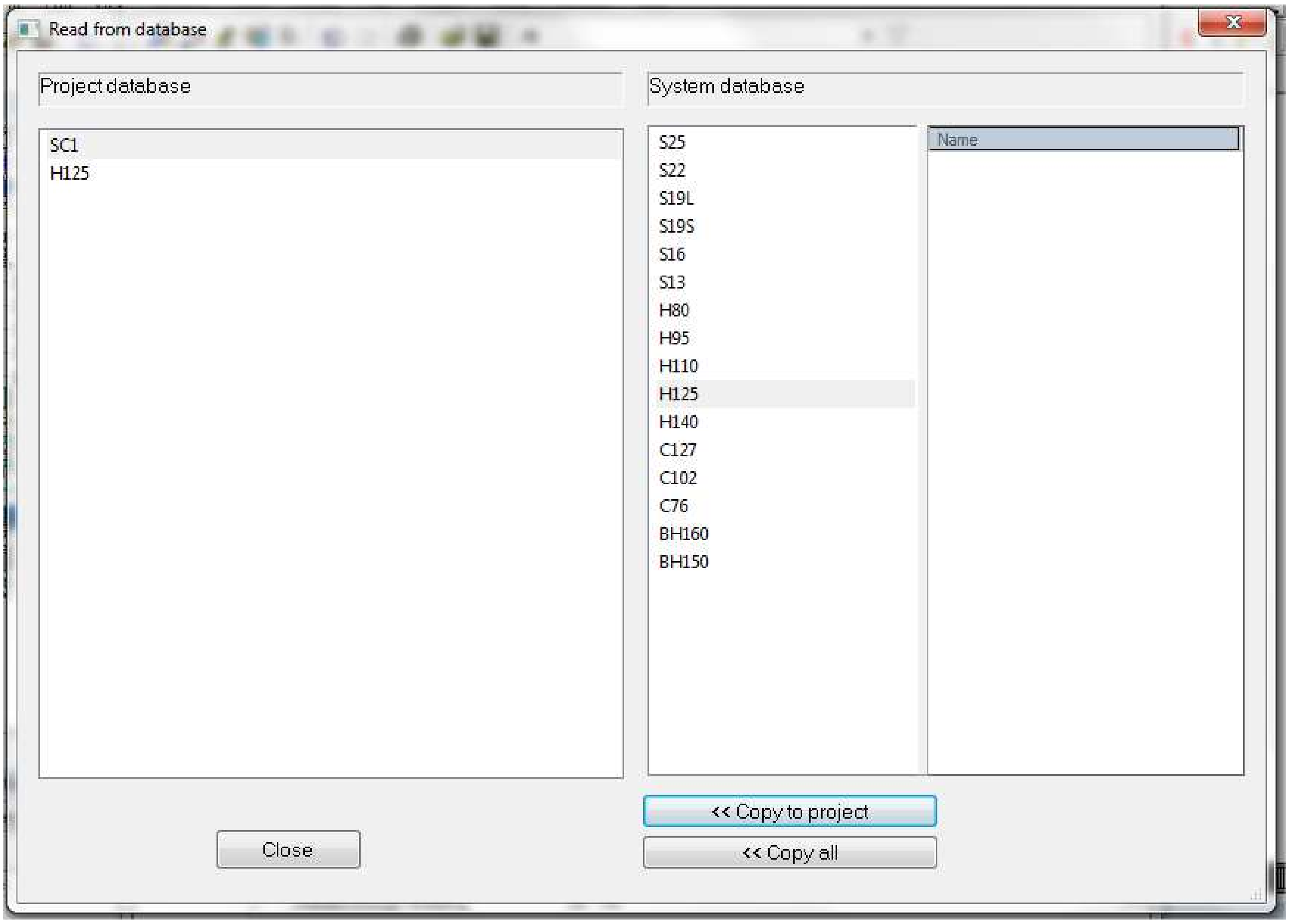
**Non-composite beam:**
The user can design a section as non-composite (ordinary steel beam without composite action) by disabling the shear connector using the non-composite check box.

**Shear connectors:**
Composite construction requires the use of appropriately designed and detailed shear connectors to develop composite action between the steel beam and the concrete slab.
Shear connector data is provided in the form of a library. This can be accessed through “Library -> Composite -> Shear Connectors” on the Main tree.
The shear connectors dialog allows the user to specify the number of shear connectors in a row, transverse spacing, diameter, in situ height and longitudinal spacing. By default, the program assumes a single row of shear connectors. For high loads and/or thick slabs, the connectors may be arranged in pairs.

The longitudinal spacing of shear connectors can be input along the length of the beam. Same spacing is assumed for both hogging & sagging zones.

Pressing the three dots behind “Shear connector data” opens the library of shear connectors. This can be used to choose a standard shear connector or to create a user-defined connector.






The list of shear connectors includes stud connectors with available diameters 13, 16, 19, 22 and 25mm. These are available in standard lengths for which the characteristic resistance values given are taken from BS 5950-3.1. There are two standard lengths for 19 mm diameter studs namely 75mm and 100 mm. The 19 mm x 100 mm long stud is the program default as the most commonly used type. In accordance with the code, 5 mm is deducted from the nominal length of stud connectors to obtain the in-situ length to allow for loss of material in the fusion welding process. The capacity of studs less than standard height is calculated pro-rata to the in situ lengths but longer lengths do not increase capacity.

Other types of shear connectors include Hilti connectors, which are fixed by a mechanical process rather than by welding. Shear capacities are taken from Hilti product literature. Shear connectors may be formed from channels and bar-hoop fabrications as listed. These are profile welded to the top flange of the steel beam and have relatively high capacities proportional to their width. Details and capacities are taken from the former code of practice CP117 and the bridge design code BS 5400 part 4.

It should be noted that a minimum concrete cover of 15mm is required to the top of shear connectors. The number of shear connectors must meet the requirements for the composite strength of the beam and the code requirements for minimum degree of shear connection for the beam span.

Both the moment capacity and the shear connection degree are checked at the position of maximum moment. The moment capacity and the shear connection degree are also calculated at the position of any additional loads. However, only the bending moment capacity is checked at these points. In theory, for satisfactory performance under service loads, the distribution of the shear connectors should be reasonably close to the elastic distribution of longitudinal shear force. However, uniformly spaced connectors routinely resist uniform loads generating parabolic force distribution.

The quantity and distribution of shear connection must be such that they transfer the longitudinal force between the concrete and the steel to develop the required composite resistance at all points along the span.

The number of shear connectors provided can develop a longitudinal force Rq., The existence of full shear connection becomes evident if this is greater than the smaller of the axial capacity of the concrete slab Rc, and the axial (tensile) capacity of the steel beam Rs. If Rq is less than the minimum of Rc and Rs, then partial shear connection is developed.

The degree of shear connection is defined as the ratio of the number of shear connectors provided to the number required for full shear connection. Limits are placed on the minimum degree of shear connection in BS 5950: Part 3 : Clause 5.5.2., as follows:

- For spans up to 10 metres, the degree of shear connection should not be less than 0.40.
- For spans between 10 and 16 metres, the degree of shear connection should exceed (Span-6)/10.
- For spans greater than 16 metres, partial shear connection is not allowed.

## Loads on a composite beam

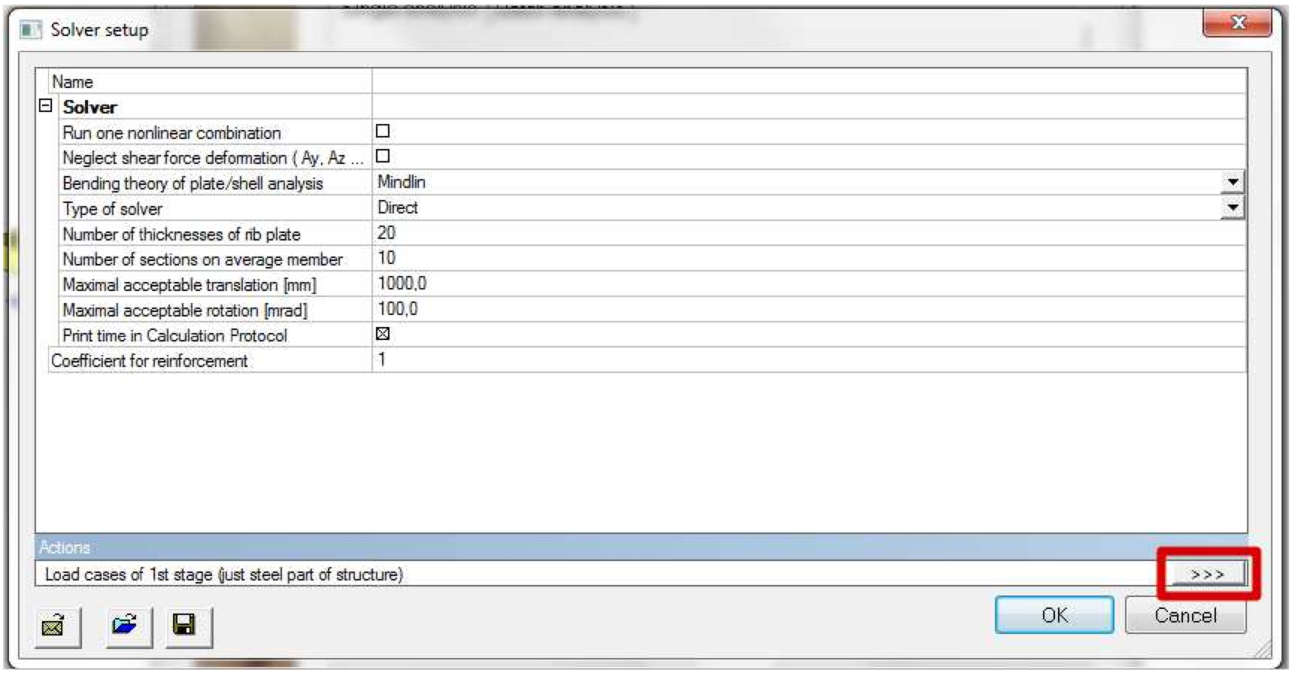
A composite beam consists of 2 construction stages:

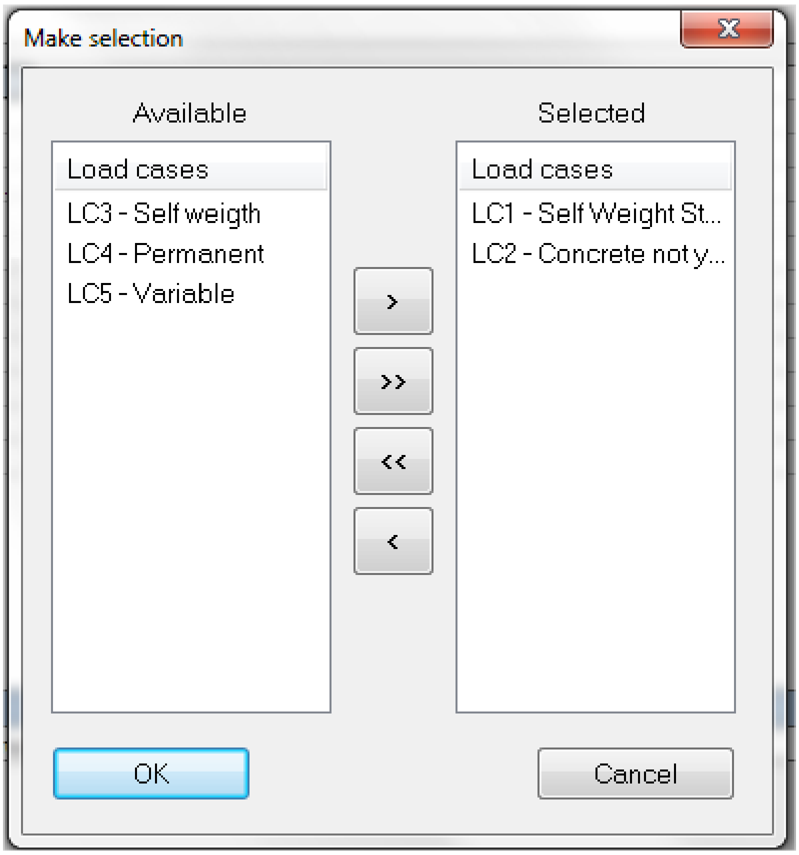
- Stage 1: only steel element
- Stage 2: Concrete will be added.

The loading on the composite beam can be categorized into

- Construction stage permanent load
- Construction stage variable load
- Final stage permanent load
- Final stage variable load

The user must create appropriate load cases representing the above through the standard load cases dialog. No distinction is made between final and construction stage load cases at this stage. This is done just before analysis when the user is required to specify the load cases for construction stage through solver options (“Setup -> Solver” menu):






Based on the load cases appropriate load combinations must be created by the user. A load combination must contain only final stage or only construction stage load cases. If a combination is found to have both it cannot be used for design checks.

During analysis the construction stage combinations are calculated with steel beam section properties and final stage combinations are calculated with composite section properties.

The program requires at least one permanent load case to be specified for final & construction stage load combination in order to proceed with the analysis.

## Analysis

Only linear analysis is supported. Loads can be applied for both stages, construction and final stages. The final stage model will be analyzed with composite beam section properties (uncracked section properties). The construction stage model will be analyzed with steel beam properties.

In the solver setup it is recommended to select "Neglect shear force deformation (Ay, Az >> A)" as the program does not consider composite cross section for shear area 'Ay' & 'Az' calculation.

## Composite check

First a summary of some inputted properties will be shown:

| Result Summary | Value | Unit |
|---|---|---|
| Member | B1 | |
| Cross-section | IPE300 | |
| Steel grade | S 235 | |
| Concrete grade | C20/25 | |
| Critical Combination | CO1/1 | |
| Position | 0,000 | m |
| Utilisation | 0,95 | - |
| Status | Adequate | |

Afterwards the Design strength of steel, the inputted effective width (inputted by user) and the classification of the cross-section in show in the detailed preview:

| Cross-section Classification | Value | Unit | Reference |
|---|---|---|---|
| Design strength of steel | 235,0 | MPa | 3.3 |
| Effective width | 400 | mm | 5.4.1.2 |
| Critical section | 0,000 | m | |
| Flange class | Class 1 | | |
| Web class | Class 1 | | |
| Cross-section class | Class 1 | | 5.5 |

## 6.1. Beams - general

Typical cross-sections:

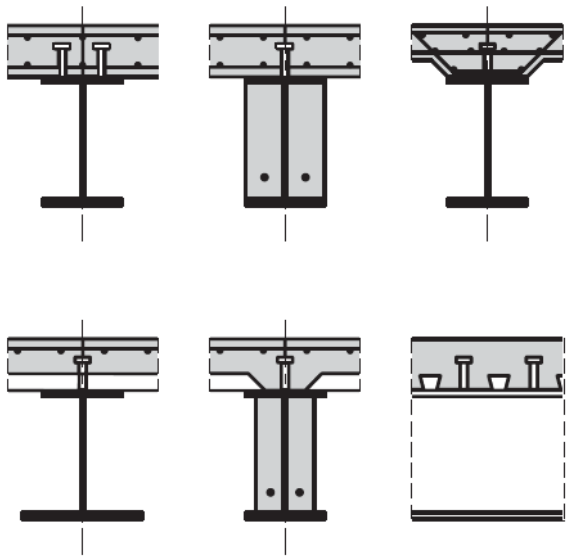


Figure 6.1 : Typical cross-sections of composite beams

Composite beams shall be checked for the following checks, described in the next paragraphs

- resistance of critical cross-sections
- resistance to lateral-torsional buckling
- resistance to shear buckling
- transverse forces on webs
- resistance to longitudinal shear

## 6.6. Shear connection

### Design resistance

The design shear resistance of a headed stud automatically welded in accordance with EN 145555 should be determined from (see also En 1994-1-1, 6.6.3.1):

$$P_{Rd} = \frac{0{,}8 f_u \pi d^2/4}{\gamma_v}$$

Or:

$$P_{Rd} = \frac{0{,}29 \alpha d^2 \sqrt{f_{ck} E_{cm}}}{\gamma_v}$$

With:

Whichever is smaller:

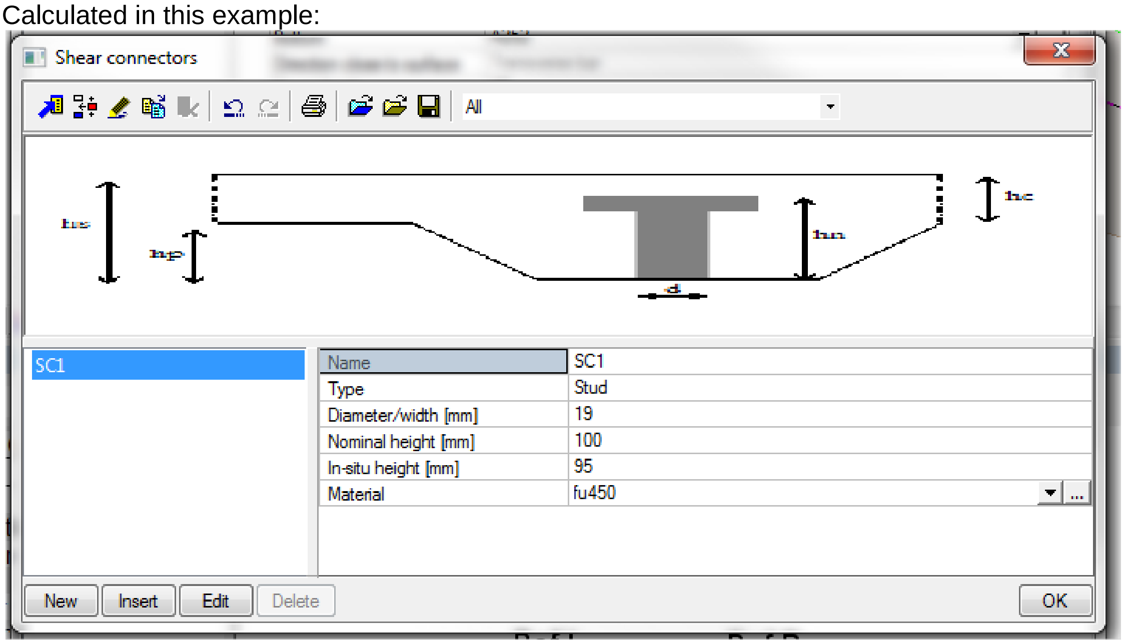
$\alpha = 0{,}2\left(\frac{h_{sc}}{d} + 1\right)$ for $3 \leq h_{sc}/d \leq 4$

$\alpha = 1.0$ for $h_{sc}/d > 4$

where:

| | |
|---|---|
| $\gamma_V$ | is the partial factor, by default 1,25, but could be changed in the Natioanl Annex. |
| $d$ | is the diameter of the shank of the stud, 16 mm $\leq d \leq$ 25 mm; |
| $f_u$ | is the specified ultimate tensile strength of the material of the stud but not greater than 500N/mm²; |
| $f_{ck}$ | is the characteristic cylinder compressive strength of the concrete at the age considered, of density not less than 1750 kg/m3; |
| $h_{sc}$ | is the overall nominal height of the stud. |

Calculated in this example:



$$P_{Rd} = \frac{0{,}8 \cdot 450 \cdot \pi \cdot 19^2/4}{1.25} = 81656N = 81.656kN$$

Or:

$h_{sc}$ / d = 100 / 19 > 4 => $\alpha$ = 1.0

$$P_{Rd} = \frac{0{,}29 \cdot 1.00 \cdot 19^2\sqrt{25 \cdot 31000}}{1{,}25} = 73730N = 73{,}730\ kN$$

| Shear connector | Value | Unit | Reference |
|---|---|---|---|
| Length of zone under consideration | 6,000 | m | |
| Longitudinal spacing of connectors | 300 | mm | |
| No. of connectors in the zone | 20 | | |
| Design resistance of each shear connector | 74,29 | kN | 6.6.3.1 |
| Reduction factor due to profile deck | 0,8 | - | 6.6.4.2 |
| Effective design resistance | 63,15 | kN | |
| Combined resistance of shear connectors | 1263,00 | kN | |
| Minimum degree of shear connection | 0,4 | - | |
| Actual degree of shear connection | 0,5 | - | |

## Reduction factor due to profile deck

There are 2 formulas given in the Eurocode, one for sheeting with ribs parallel to the supporting beams (EN1994-1-1, formula (6.22)) and one sheeting with ribs transverse to the supporting beams (EN1994-1-1, formula (6.23)).

In the example above we are in the second case, so in this manual this formula is taken into account, but the principle of the other one is exactly the same.

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

And $n_r$ is the number of stud connectors in one rib at a beam intersection, not to exceed 2 in computations.

Other symbols are explained below in the figure:

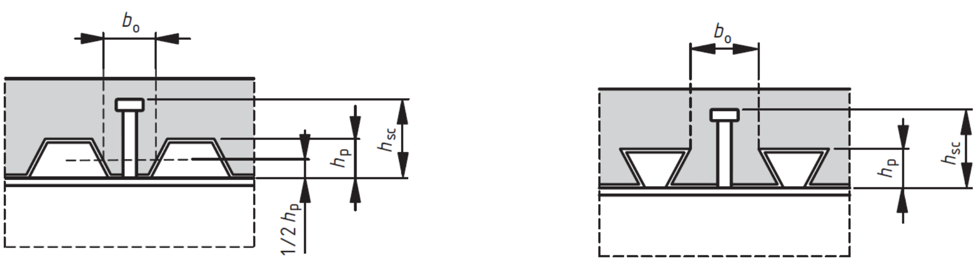



**Figure 6.13 : Beam with profiled steel sheeting transverse to the beam**

The upper limits for $k_t$ can be taken from the table below:

**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 |
|---|---|---|---|
| $n_r = 1$ | ≤ 1,0 | 0,85 | 0,75 |
| | > 1,0 | 1,0 | 0,75 |
| $n_r = 2$ | ≤ 1,0 | 0,70 | 0,60 |
| | > 1,0 | 0,8 | 0,60 |

So in the example here: $n_r$ = 1 and the thickness of the sheet < 1mm, so $k_t$ = 0,85.

The effective design resistance is: 0,85 x 72,29kN = 63,15 kN.
In total there are 20 connectors, so the combined resistance of shear connectors is: 20 x 63,15 kN = 1263,00 kN

| **Shear connector** | **Value** | **Unit** | **Reference** |
|---|---|---|---|
| Length of zone under consideration | 6,000 | m | |
| Longitudinal spacing of connectors | 300 | mm | |
| No. of connectors in the zone | 20 | | |
| Design resistance of each shear connector | 74,29 | kN | 6.6.3.1 |
| Reduction factor due to profile deck | 0,8 | - | 6.6.4.2 |
| Effective design resistance | 63,15 | kN | |
| Combined resistance of shear connectors | 1263,00 | kN | |
| Minimum degree of shear connection | 0,4 | - | |
| Actual degree of shear connection | 0,5 | - | |

## Degree of shear connection

For composite beams in buildings, the headed shear connectors may be considered as ductile when the minimum degree of shear connection given in EN 1994-1-1, 6.6.1.2 is achieved.
Fore headed shear connectors with:

$h_{sc} \geq 4d$ and 16 mm $\leq d \leq$ 25 mm

The degree of shear connection is defined by the ratio: $\eta = n / n_f$

For a steel-section with equal flanges, the degree of shear connection may be determined from:

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

$L_e > 25$: $\eta \geq 1$

In EN 1994-1-1, 6.6.1.2, other formulas are given for the cases without equal flanges.

There is a simplification method given for headed stud connectors, if the following conditions are satisfied:

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

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

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

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

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

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

In this case the degree of shear connection may be determined from:

$L_e \leq 25$: $\quad \eta \geq 1 - \left(\frac{355}{f_y}\right)(1{,}00 - 0{,}04 L_e) \qquad \eta \geq 0{,}4$

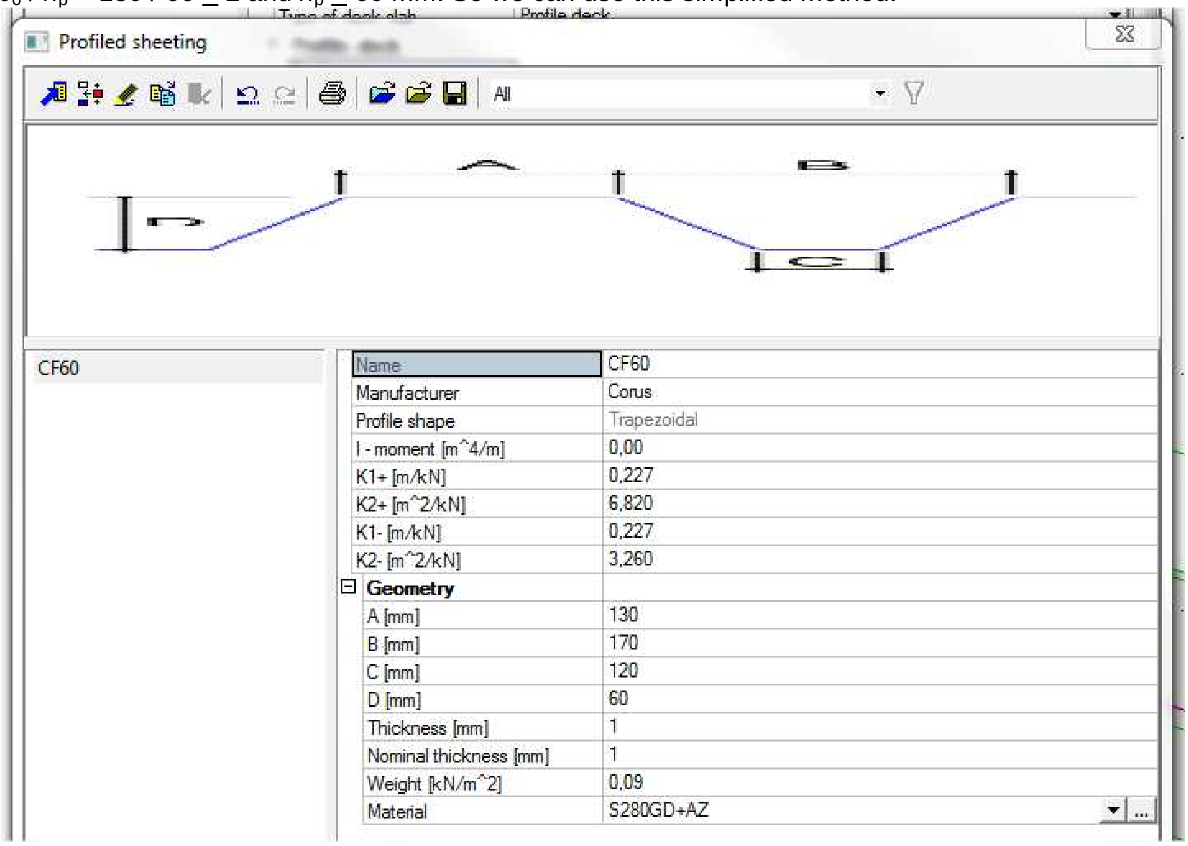
$L_e > 25$: $\quad \eta \geq 1$

In the example :

$h_0 = 60$ mm

$b_0 = 130$ mm

$b_0 / h_p = 130 / 60 \geq 2$ and $h_p \leq 60$ mm. So we can use this simplified method.



$L_e \leq 25$: $\eta \geq 1 - \left(\frac{355}{275}\right)(1{,}00 - 0{,}04 \cdot 12) = 0{,}328$ and $\eta \geq 0{,}4$

**So η = 0,4**

| Shear connector | Value | Unit | Reference |
|---|---|---|---|
| Length of zone under consideration | 6,000 | m | |
| Longitudinal spacing of connectors | 300 | mm | |
| No. of connectors in the zone | 20 | | |
| Design resistance of each shear connector | 74,29 | kN | 6.6.3.1 |
| Reduction factor due to profile deck | 0,8 | - | 6.6.4.2 |
| Effective design resistance | 63,15 | kN | |
| Combined resistance of shear connectors | 1263,00 | kN | |
| Minimum degree of shear connection | 0,4 | - | |
| Actual degree of shear connection | 0,5 | - | |

The present degree of shear connection is the following ratio: $\eta = N_c / N_{c,f}$

With:

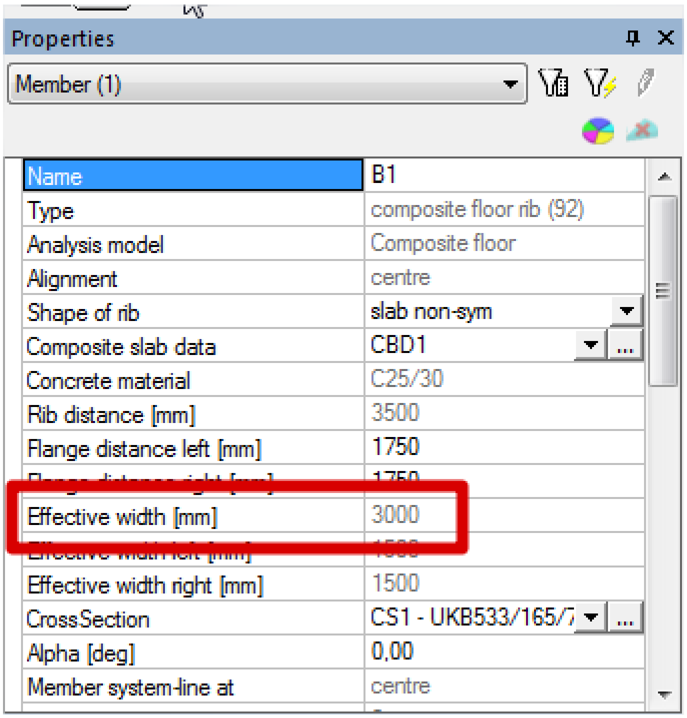
$N_c$ the reduced value of the compressive force in the concrete flange (i.e. the force transferred nu the shear connectors).

$N_{c,f}$ the compressive force in the concrete flange at full shear connection (i.e. the lesser of the compressive resistance of the concrete and the tensile resistance of the steel beam).

To determine the degree of shear connection present in the beam, first the axial forces in the steel and concrete are required ($N_{pl,a}$ and $N_{c,f}$ respectively), as shown in the figure below:


In the example in Scia Engineer:

$b_{eff}$ has been inputted in Scia Engineer manually and is equal to 3,00 m:




The design compressive strength of concrete for persistent and transient design situations is:

$$f_{cd} = \frac{f_{ck}}{\gamma_c} = \frac{25}{1.5} = 17.7\ N/mm^2$$

**Compressive resistance of the concrete flange** is for 1 row of studs:

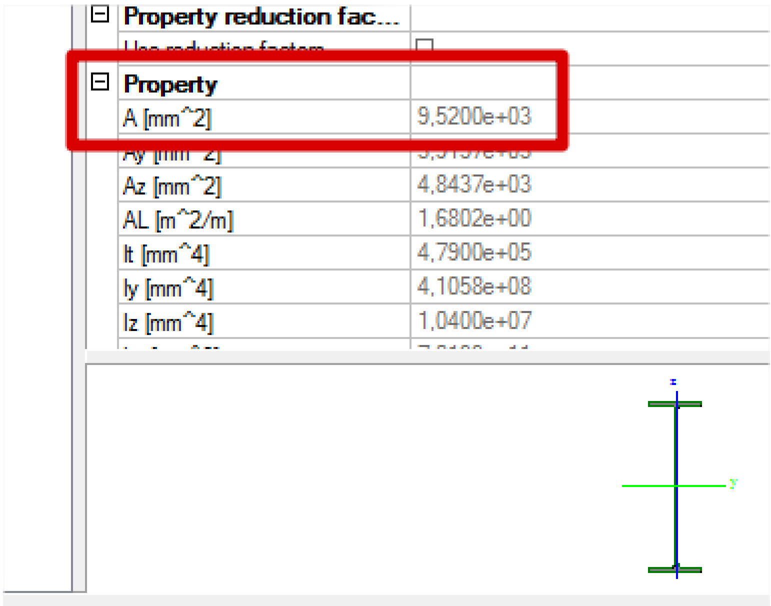
$$N_{c,Rd} = 0{,}85 f_{cd} b_{eff} h_c$$

And $h_c$ = 130 mm – 60 mm = 70mm

$$N_{c,Rd} = 0{,}85 f_{cd} b_{eff} h_c = 0{,}85 \cdot 16.7 \cdot 3000 \cdot 70 = 2980950\ N = \mathbf{2980,95\ kN}$$

**Tensile resistance of the steel member**:

$$N_{pl,a} = f_y A_a = 275 \cdot 9{,}52 \cdot 10^3 = 2618000\ N = \mathbf{2618\ kN}$$




The tensile resistance of the steel member is the lesser of the two, (2980,95 kN and 2618 kN) so $Nc_{,f}$ = 2618 kN.

The actual degree of shear connection:

$\eta = N_c / N_{c,f} = 1252 / 2618 = 0.48$

| Shear connector | Value | Unit | Reference |
|---|---|---|---|
| Length of zone under consideration | 6,000 | m | |
| Longitudinal spacing of connectors | 300 | mm | |
| No. of connectors in the zone | 20 | | |
| Design resistance of each shear connector | 74,29 | kN | 6.6.3.1 |
| Reduction factor due to profile deck | 0,8 | - | 6.6.4.2 |
| Effective design resistance | 63,15 | kN | |
| Combined resistance of shear connectors | 1263,00 | kN | |
| Minimum degree of shear connection | 0,4 | - | |
| Actual degree of shear connection | 0,5 | - | |

## Resistance to vertical shear (6.2.2)

### Resistance to shear

This value can be taken as the resistance value for shear for the steel member. So the maximum shear capacity in the composite output, is exactly the same as in the steel code check:

Composite check (final stage)

| Check for shear resistance | Value | Unit | Reference |
|---|---|---|---|
| Shear force | 0,00 | kN | |
| Shear area | 5485 | mm^2 | 6.2.2.3 |
| Shear capacity | 870,84 | kN | |
| Unity Check | 0,00 | - | |
| Status | Adequate | | |

Steel code check (construction stage):

**Shear check (Vz)**
According to article EN 1993-1-1 : 6.2.6. and formula (6.17)

| Table of values | | |
|---|---|---|
| Vc,Rd | 870.84 | kN |
| Unity check | 0.11 | - |

### Resistance to bending and shear

Also the influence of the vertical shear on the resistance to bending may be taken into account by a reduced design steel strength (1 - ρ) $f_{yd}$ with:

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

In this example is $V_{Ed}$ / $V_{Rd}$ < 0,5 and is it not necessary to take this reduction factor into account.

## Resistance to bending 6.2.1.2 and 6.2.1.3

When having two shear connectors per trough, article EN 1994-1-1, 6.2.1.2. will be used.

### Two shear connectors

Typical plastic stresses:

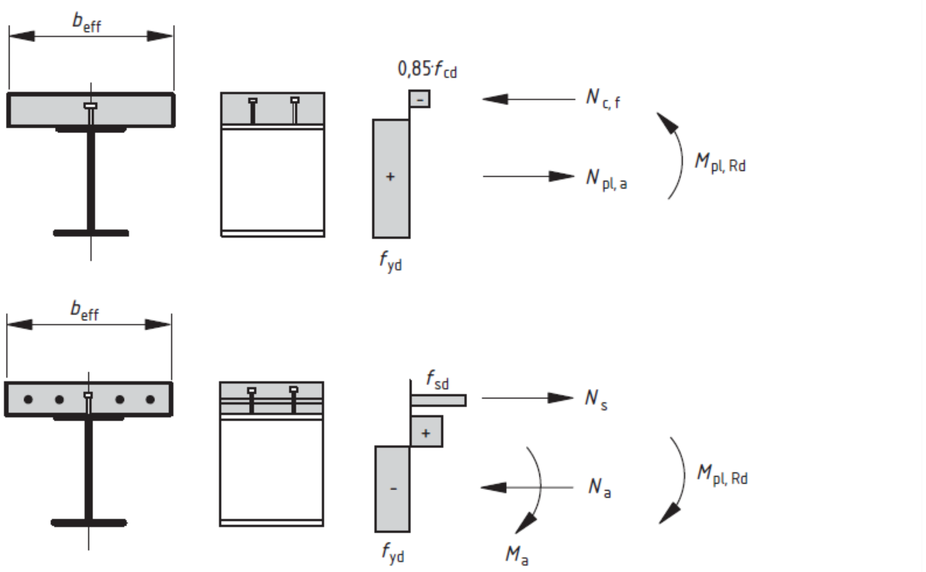


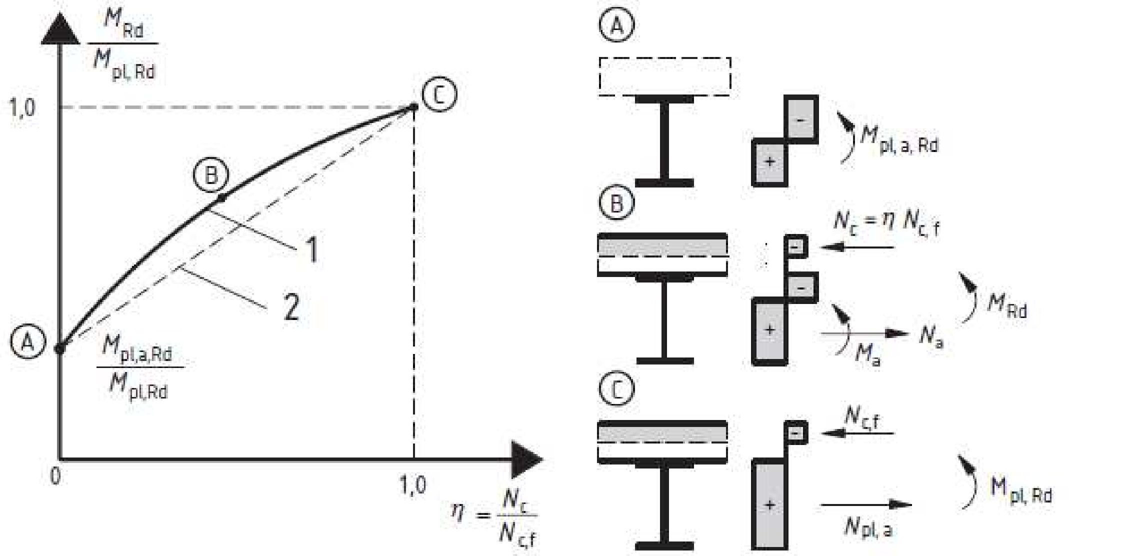

**Figure 6.2 : Examples of plastic stress distributions for a composite beam with a solid slab and full shear connection in sagging and hogging bending**

A reduction of the yield strength $f_y$ should be taken into account for steel grades S420 or S460 as described in EN 1994-1-1 §6.2.1.2(2).

The method for the resistance moment for sections with partial shear connection is described in EN 1994-1-1 §6.2.1.3.

## One shear connector

When having only one shear connector, a simplified method could be used as described in EN 1994-1-1, 6.2.1.3. This one will be used in the example of Scia Engineer.




**Key**

1 plastic theory

2 simplified method

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

And as a simplification a conservative value of $M_{Rd}$ may be determined by the straight line AC in the figure above:

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

This formula is also used in Scia Engineer:

$\eta = N_c / N_{c,f} = 1252 / 2618 = 0.48$

For full shear connection $N_{pl,a}$ (2618 kN) < $N_{c,f}$ (2981) and so the plastic neutral axis of the composite section lies within the concrete.

Therefore, the design plastic resistance moment of the composite section with full shear connection can be determined from:

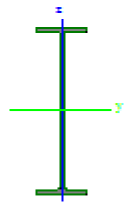
$$M_{pl,Rd} = N_{pl,a}\left[\frac{h_a}{2} + h_s - \frac{x_c}{2}\right]$$

Where:

$$x_c = \left(\frac{N_{pl,a}}{N_{c,f}}\right) \cdot h_c = \left(\frac{2618}{2981}\right) \cdot 70 = 61.5mm$$

$$M_{pl,Rd} = 2618\left[\frac{529.1}{2} + 130 - \frac{61.5}{2}\right] = 952428.4\ kNmm = 952.43\ kNm$$

| | |
|---|---|
| **Geometry** | |
| Formcode | 1 - I sections |
| H [mm] | 529,10 |
| B [mm] | 165,90 |
| t [mm] | 13,60 |
| s [mm] | 9,70 |
| R [mm] | 12,70 |

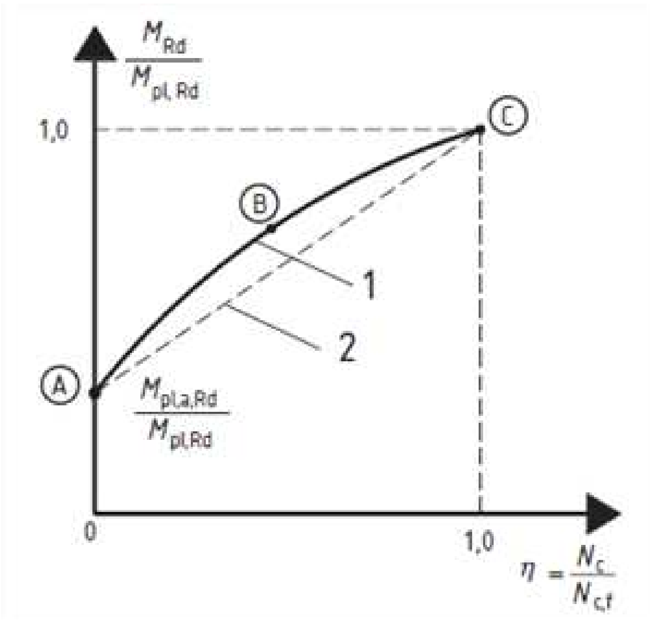


And $M_{pl,a,Rd} = W_{ply} \times f_y$ = 1,8106e+0 x 275 = 497915000Nmm = 497,92 kNm

The design resistance moment:

$$M_{Rd} = M_{pl,a,Rd} + \left(M_{pl,Rd} - M_{pl,a,Rd}\right)\frac{N_c}{N_{cf}} = 497.92kNm + (952.43kNm - 497.92kNm) \cdot 0.48$$
$$= \mathbf{716.08\ kNm}$$

In Scia Engineer this will be calculated with the curved form, so the design bending moment will be a bit higher:



| Check for Bending | Value | Unit | Reference |
|---|---|---|---|
| Bending moment | 569,58 | kNm | |
| Depth of compression zone in concrete | 29,00 | mm | |
| Depth of plastic neutral axis | 157,81 | mm | |
| Location of neutral axis | Web | | |
| Plastic moment capacity | 827,97 | kNm | 6.2.1.2/6.2.1.3 |
| Elastic moment capacity | 642,76 | kNm | 6.2.1.5 |
| Effect of vertical shear on bending capacity | No | | |
| Effective bending capacity | 827,97 | kNm | |
| Unity Check | 0,69 | - | |
| Status | Adequate | | |

## 6.7 Composite columns and composite compression members

First of all the steel contribution ratio δ should fulfill the following condition:

$$0{,}2 \le \delta \le 0{,}9$$

Where δ is defined in 6.7.3.3(1):

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

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

- Resistance of the member
- Resistance to local buckling
- Introduction of loads
- Resistance to shear between steel and concrete elements

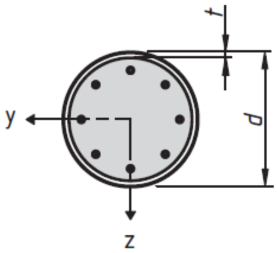
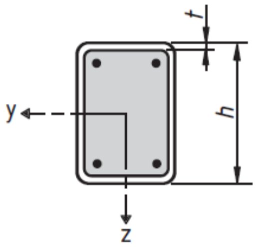
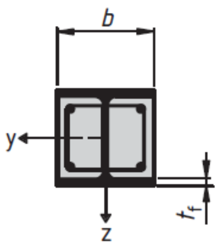
All those checks are explained in the next paragraphs.

2 methods are described:

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

The local buckling for a steel section fully encased may be neglected, as the maximum values of table 6.3 are not exceeded.

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

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

## 6.7.3. Simplified method of design

### 6.7.3. General

This method is limited to members of

- Doubly symmetrical  cross-sections
- Uniform cross-section of the member length

The method is not applicable of the structural steel component consists of two or more unconnected sections.

The slenderness $\bar{\lambda}$ should fulfill the following condistion:

$$\bar{\lambda} \leq 2{,}0 \qquad \text{EN 1994-1-1 (6.28)}$$

With

$$\bar{\lambda} = \sqrt{\frac{N_{pl,Rk}}{N_{cr}}} \qquad \text{EN 1994-1-1 (6.39)}$$

$$N_{pl,Rk} = A_a f_{yd} + 0{,}85 A_c f_{cd} + A_s f_{sd} \qquad \text{EN 1994-1-1(6.30)}$$

And $N_{cr}$ is the critical normal force for the relevant buckling mode. For the determination of $N_{cr}$, the characteristic value of the effective flexural stiffness $(EI)_{eff}$ of a cross section of a composite column should be calculated from:

$$(E\,I)_{eff} = E_a I_a + E_s I_s + K_e E_{cm} I_c \qquad \text{EN 1994-1-1(6.40)}$$

With:

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

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

To take into account the influence of long-term effects on the effective elastic flexural stiffness, the modules of elasticity of concrete $E_{cm}$ should be reduced to the value $E_{c,eff}$:

$$E_{c,eff} = E_{cm} \frac{1}{1+(N_{G,Ed}/N_{Ed})\varphi_t} \qquad \text{EN 1994-1-1(6.41)}$$

Where:

$\varphi_t$ the creep coefficient, calculated following EN 1994-1-1 5.4.2.2(2)

$N_{Ed}$ Total design normal force

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

### 6.7.3.2. Resistance of cross-section

The **plastic resistance to compression *$N_{pl,Rd}$*** of a composite cross-section:

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

This expression applies for concrete encased and partially concrete encased steel sections. For other sections, the coefficient 0,85 may be replaced by 1,0.

Where $V_{a,Ed} > 0{,}5\ V_{pl,a,Ed}$ 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).

For a simplification $V_{Ed}$ may be assumed to act on the structural steel section alone and this has been implemented with this simplification in Scia Engineer.

# References

[1] EN 1994-1-1: 2004, Design of composite steel and concrete structures – Part 1-1: General rules and rules for buildings

[2] Designers' Guido to EN 1994-1-1, Eurocode 4: , Design of composite steel and concrete structures