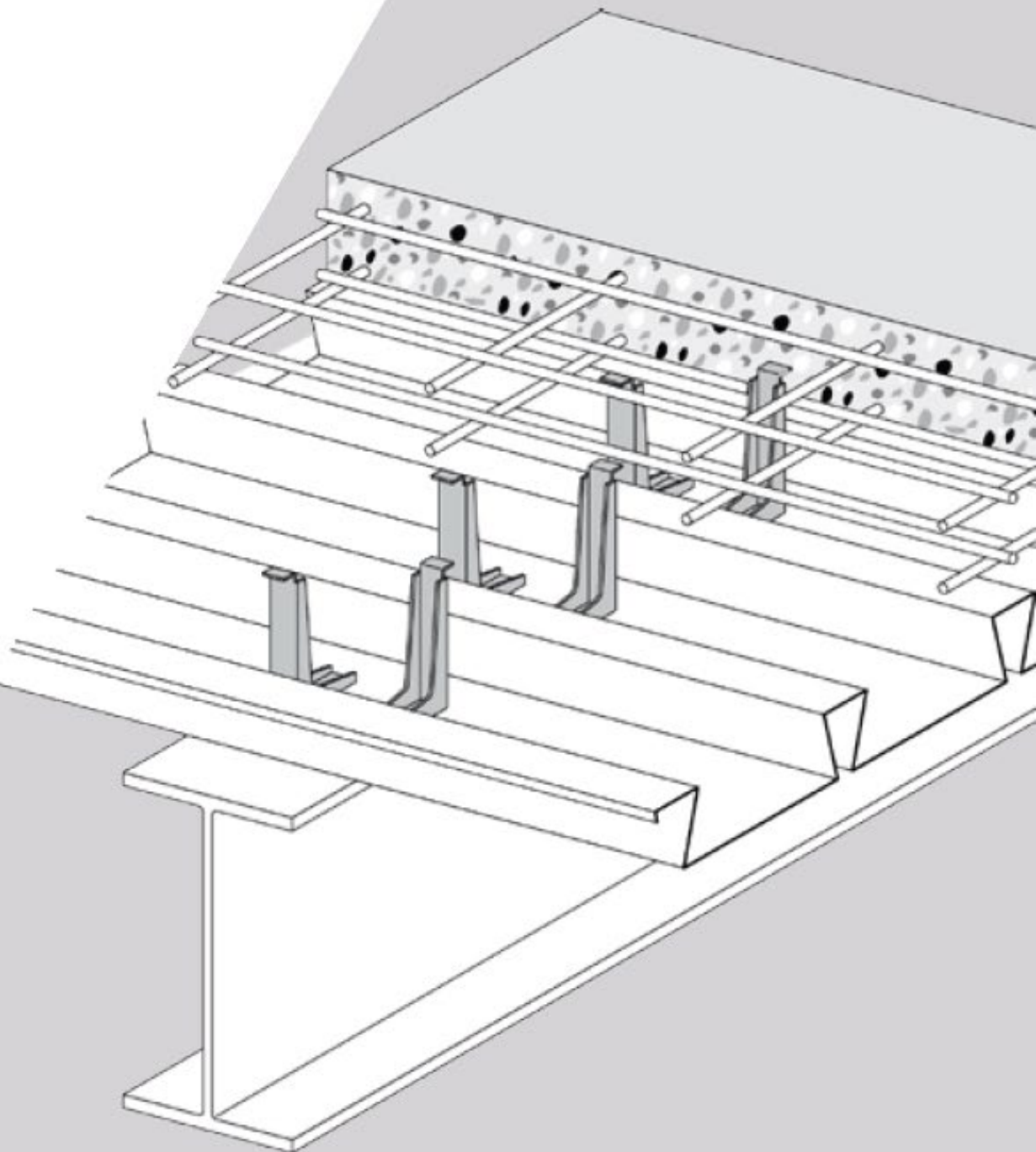




# SHEAR CONNECTOR DESIGN SOFTWARE

Technical specifications



Content according to :

HILTI SHEAR CONNECTOR DESIGN - CALCULATION MODULE\_TECHNICAL SPECIFICATIONS\_DRV/HVB/MT/003-G

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## 1. INTRODUCTION

The calculation module of the Hilti Shear Connector Design software allows the user to perform the design of composite beams using HILTI X-HVB shear connectors according to the rules of the Eurocodes. This document gives the technical specifications for the assumptions, the methods and the calculations carried out by the design module.

The scope of application of the module is defined as follows:

- The beam is assumed to be simply supported;
- The beam is a structural element of a building;
- The cross-section of the steel profile is a doubly symmetric I-section;
- The cross-section is uniform along the beam;
- The connection between the concrete slab and the steel profile is achieved through HILTI X-HVB shear connectors;
- For the checks at Ultimate Limit States (ULS), the type of design (elastic or plastic) depends on the section class. However, the user can impose an elastic design;
- For the checks at Serviceability Limit States (SLS), the elastic deflection and the natural frequency of the beam are calculated.
- The calculations and design checks are carried out according to the relevant Eurocodes (see references in 7). As an option, a specific national annex may be applied. The list of national annexes that are covered in this module is given in paragraph 2.1.1.

## 2. BASIC DATA

### 2.1. Nationally Determined Parameters

#### 2.1.1. National Annexes

National annexes covered in the calculation module are:

Austria	France	Lithuania	Singapore (*)
Belgium	Germany	Luxemburg	Slovakia
Bulgaria	Hungary	Netherlands	Slovenia
Czech Republic	Italy	Poland	Spain
Estonia	Latvia	Portugal	Switzerland
		Romania	United Kingdom

\*) Singapore is not member of the CEN but applies Eurocodes with its own National Annexes.

For Countries not member of the CEN (European Committee for Standardization) and thus not having national Annexes, default parameters are the recommended values of the Eurocode. These countries are:

Qatar	Saudi Arabia	South Africa	Turkey	United Arab Emirates (UAE)
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When no national annex is chosen, recommended values of the Eurocodes will be applied.

Additionally, if the user modifies one of the parameters defined either by the national annex or by the Eurocode, the case will be dealt as a custom parameter.

### 2.1.2. Combination factor $\psi_0$ and category of loaded areas

The category of loaded area is used to determine the value of the combination factor  $\psi_0$  according to the chosen National Annex. Its values are given in Table 1.

	Category of loaded areas					
	A Residential	B Office	C1 Congregation	D1 Shopping	E1 Storage	H Roof
Recommended values of Eurocodes	0,7	0,7	0,7	0,7	1,0	0
Austria	0,7	0,7	0,7	0,7	1,0	0
Belgium	0,7	0,7	0,7	0,7	1,0	0
Bulgaria	0,7	0,7	0,7	0,7	1,0	0,6
Czech Republic	0,7	0,7	0,7	0,7	1,0	0
Estonia	0,7 <sup>(*)</sup>	0,7 <sup>(*)</sup>	0,7 <sup>(*)</sup>	0,7 <sup>(*)</sup>	1,0 <sup>(*)</sup>	0 <sup>(*)</sup>
France	0,7	0,7	0,7	0,7	1,0	0
Germany	0,7	0,7	0,7	0,7	1,0	0
Hungary	0,7	0,7	0,7	0,7	1,0	0
Italy	0,7	0,7	0,7	0,7	1,0	0
Latvia	0,7	0,7	0,7	0,7	1,0	0
Lithuania	0,7	0,7	0,7	0,7	1,0	0
Luxemburg	0,7	0,7	0,7	0,7	1,0	0
Netherlands	0,4	0,5	0,25	0,4	1,0	0
Poland	0,7	0,7	0,7	0,7	1,0	0
Portugal	0,7	0,7	0,7	0,7	1,0	0
Romania	0,7	0,7	0,7	0,7	1,0	0,7
Singapore	0,7	0,7	0,7	0,7	1,0	0,7
Slovakia	0,7	0,7	0,7	0,7	1,0	0
Slovenia	0,7	0,7	0,7	0,7	1,0	0
Spain	0,7 <sup>(*)</sup>	0,7 <sup>(*)</sup>	0,7 <sup>(*)</sup>	0,7 <sup>(*)</sup>	1,0 <sup>(*)</sup>	0 <sup>(*)</sup>
Switzerland	0,7	0,7	0,7	0,7	1,0	0
United Kingdom	0,7	0,7	0,7	0,7	1,0	0
Non-members of CEN <sup>(1)</sup>	0,7 <sup>(*)</sup>	0,7 <sup>(*)</sup>	0,7 <sup>(*)</sup>	0,7 <sup>(*)</sup>	1,0 <sup>(*)</sup>	0 <sup>(*)</sup>

Table 1 : Values of the combination factor  $\psi_0$ .

Note:

- (\*) indicates that the recommended value is used because no national annex is available.
- Parameters in italic and red indicate different values to the recommended values.

- (1) Countries non-members of CEN without National Annexes: Turkey, Qatar, Saudi-Arabia, UAE, South Africa.

The value of the combination factor may also be modified by the user.

**Error Code (see Annex D):**

*Error code 4 is returned if the following condition is not met:*

- $0 \leq \psi_0 \leq 1$

### 2.1.3. Partial factors

Partial factors on actions for ULS combinations are:

$\gamma_G$  for permanent actions

$\gamma_Q$  for variable actions

Partial factors for design resistances are:

$\gamma_{M0}$  for the section resistance of the structural steel

$\gamma_{M1}$  for the element resistance of the structural steel

$\gamma_c$  for the compression resistance of the concrete

$\gamma_V$  for the resistance of shear connectors

$\gamma_S$  for the resistance of reinforcement steel bars

$\gamma_p$  for the resistance of the profiled steel

Values of partial factors, determined according to the chosen National Annex, are given in Table 2.

	$\gamma_G$	$\gamma_Q$	$\gamma_{M0}$	$\gamma_{M1}$	$\gamma_c$	$\gamma_v$	$\gamma_s$	$\gamma_P$
Recommended values of Eurocodes	1,35	1,50	1,00	1,00	1,50	1,25	1,15	1,00
Austria	1,35	1,50	1,00	1,00	1,50	1,25	1,15	1,00
Belgium	1,35	1,50	1,00	1,00	1,50	1,25	1,15	1,00
Bulgaria	1,35	1,50	<i>1,05</i>	<i>1,05</i>	1,50	1,25	1,15	1,00
Czech Republic	1,35	1,50	1,00	1,00	1,50	1,25	1,15	1,00
Estonia	1,35 <sup>(*)</sup>	1,50 <sup>(*)</sup>	1,00	1,00	1,50	1,25	1,15	1,00 <sup>(*)</sup>
France	1,35	1,50	1,00	1,00	1,50	1,25	1,15	1,00
Germany	1,35	1,50	1,00	<i>1,10</i>	1,50	1,25	1,15	<i>1,10</i>
Hungary	1,35	1,50	1,00	1,00	1,50	1,25	1,15	1,00
Italy	<i>1,30</i>	1,50	<i>1,05</i>	<i>1,05</i>	1,50	1,25	1,15	<i>1,05</i>
Latvia	1,35	1,50	1,00	1,00	1,50	<i>1,50</i>	1,15	1,00
Lithuania	1,35	1,50	1,00	1,00	1,50	1,25	1,15	1,00
Luxemburg	1,35	1,50	1,00	1,00	1,50	1,25	1,15	1,00
Netherlands	1,35	1,50	1,00	1,00	1,5	1,25 <sup>(*)</sup>	1,15	1,00 <sup>(*)</sup>
Poland	1,35	1,50	1,00	1,00	1,5	1,25	1,15	1,00
Portugal	1,35 <sup>(*)</sup>	1,50 <sup>(*)</sup>	1,00 <sup>(*)</sup>	1,00 <sup>(*)</sup>	1,50 <sup>(*)</sup>	1,25 <sup>(*)</sup>	1,15 <sup>(*)</sup>	1,00 <sup>(*)</sup>
Romania	1,35	1,50	1,00	1,00	1,50	1,25	1,15	1,00
Singapore	1,35	1,50	1,00	1,00	1,50	1,25	1,15	1,00
Slovakia	1,35	1,50	1,00	1,00	1,50	1,25	1,15	1,00
Slovenia	1,35	1,50	1,00	1,00	1,50	1,25	1,15	1,00
Spain	1,35 <sup>(*)</sup>	1,50 <sup>(*)</sup>	<i>1,05</i>	<i>1,05</i>	1,50	1,25	1,15	1,00 <sup>(*)</sup>
Switzerland	1,35	1,50	1,05	1,05	1,5	1,25	1,15	1,00 <sup>(*)</sup>
United Kingdom	1,35	1,50	1,00	1,00	1,50	1,25	1,15	1,00
Non-members of CEN <sup>(1)</sup>	1,35 <sup>(*)</sup>	1,50 <sup>(*)</sup>	1,00 <sup>(*)</sup>	1,00 <sup>(*)</sup>	1,50 <sup>(*)</sup>	1,25 <sup>(*)</sup>	1,15 <sup>(*)</sup>	1,00 <sup>(*)</sup>

Table 2 : Values of the partial factors.

Note:

- (\*) indicates that the recommended value is used because no national annex is available.
- Parameters in italic and red indicate different values to the recommended values.
- (1) Countries: Turkey, Qatar, Saudi-Arabia, UAE, South Africa.

The values of partial factors may be modified by the user.

**Error Code (see Annex D):**

Error code 3 is returned if the following condition is not met:

- $1 \leq \gamma_i \leq 2$

## 2.2. Type of design

Following types of design are available in the software:

- by default, a plastic design is carried out, if the cross-sections are of Class 1 or 2;
- an elastic design according to EN 1994-1-1 is performed for Class 3 cross-sections;
- the user can impose an elastic design whatever the class of the cross-section is. In this case, resistance checks are carried out by using the Von-Mises criteria.

Class 4 cross-sections are not covered by the software.

## 2.3. Geometrical description of the beam

A beam has to be defined either as an intermediate beam or as an edge beam.

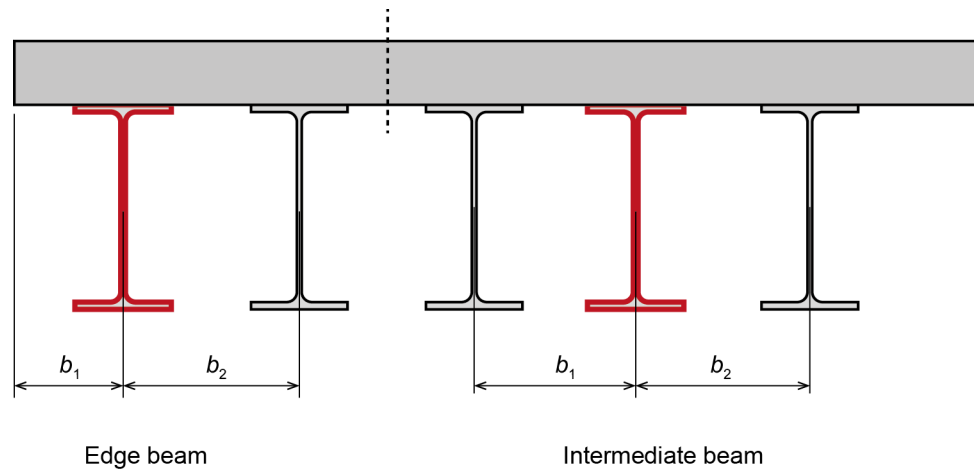


Figure 1 : Edge beam and intermediate beam

The geometry of the beam is defined by (see Figure 1):

- For intermediate beam:
  - $L$  is the beam length
  - $b_1$  is the spacing of the beam to the left beam
  - $b_2$  is the spacing of the beam to the right beam
- For edge beam:
  - $L$  is the beam length
  - $b_1$  is the spacing of the beam to the slab edge
  - $b_2$  is the spacing of the beam to the adjacent beam

### Error Code (see Annex D):

Error code 5 is returned if the following condition is not met:

- $2m \leq L \leq 20m$

Error code 6 is returned if the following conditions are not met:

- $0,5m \leq b_1 \leq 20m$
- $0,5m \leq b_2 \leq 20m$



It is also possible to define the presence of slab opening on one side or on both sides of the beam. A slab opening is defined by the distance of its edge to the beam axis.

The position of the slab openings is defined by:

$d_1$  is the distance of the left hand-side slab opening (if any) to the beam axis

$d_2$  is the distance of the right hand-side slab opening (if any) to the beam axis

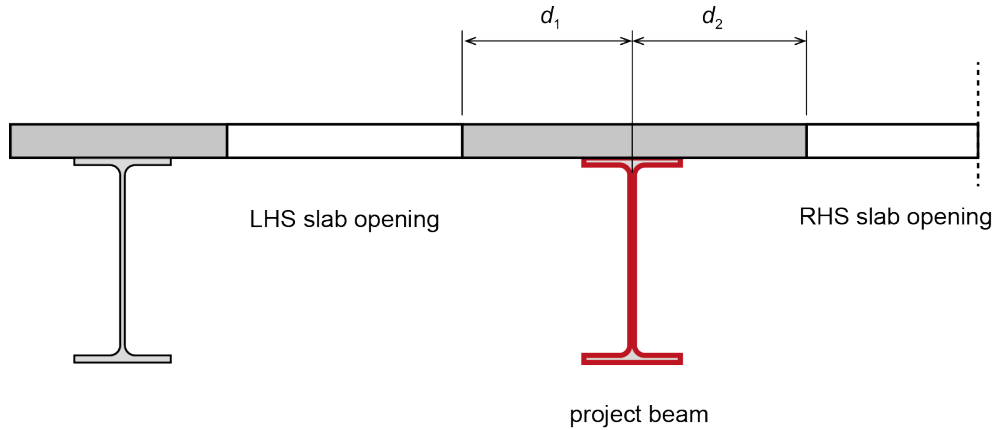


Figure 2 : Definition of slab openings

## 2.4. Steel section

### 2.4.1. Geometry

The steel profile of a beam is defined by its geometrical parameters as follows (Figure 3):

$h_t$  is the total height

$b$  is the width of the flanges

$t_f$  is the thickness of the flanges

$t_w$  is the thickness of the web

$r_1$  is the root radius (only for hot rolled sections)

$r_2$  is the toe radius (only for hot rolled sections)

$a$  is the throat of the fillet weld (only for custom sections)

#### Error Code (see Annex D):

Error code 8 is returned if the following conditions are not met:

- $t_w > 3\text{mm}$
- $t_f > 0$
- $r_1 \geq 0$
- $r_2 \geq 0$
- $b > 2 r_1 + 2 r_2 + t_w$
- $h_t > 2 r_1 + 2 t_f$
- $a \geq 0$

When the user selects a hot rolled section, all values are automatically read in the database except  $a = 0$ . For a custom section, assumed to be a welded section, all parameters are to be defined by the user, except  $r_1$  and  $r_2$  which are assumed equal to 0.

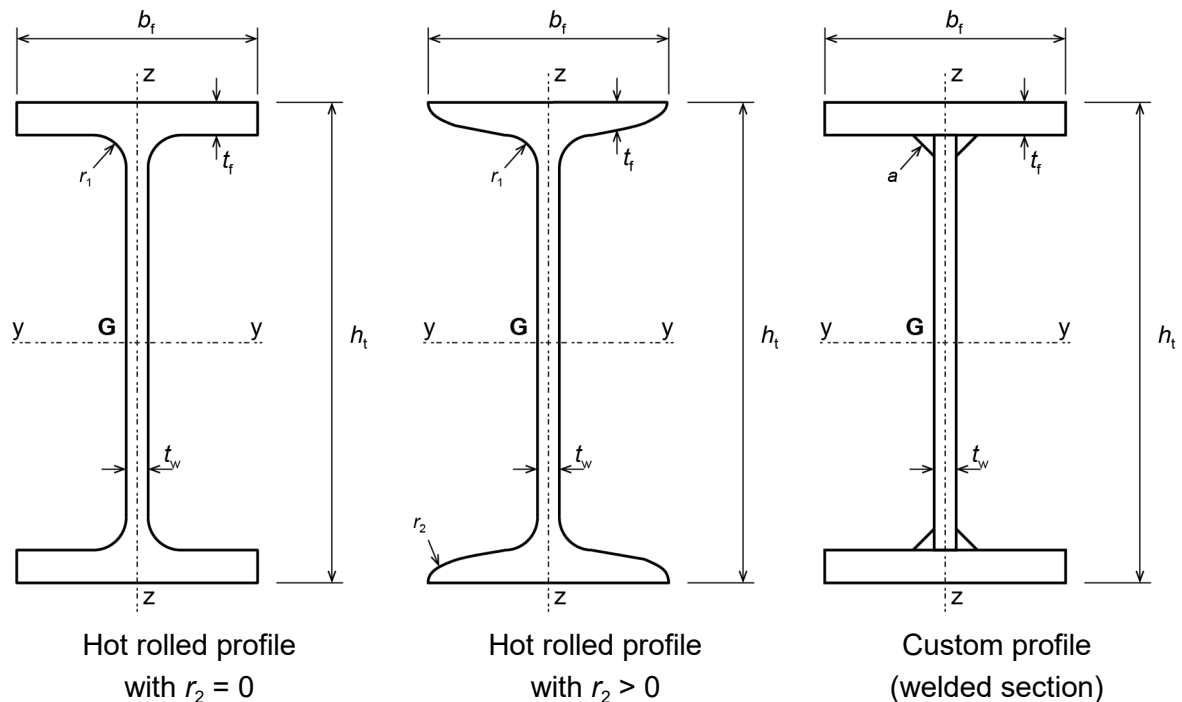


Figure 3: Steel profiles

Analytical formulas that will be used for calculating the section properties are given in Annex A.

The thickness of the base material (i.e. the flange of the section) must fulfil the following conditions (according to [1] Annex B3):

- For solid slabs, the minimum thickness of flanges is 6 mm, i.e.  $t_f \geq 6 \text{ mm}$ . Two profiles IPE100 and IPN100 which have the flanges thickness smaller than 6 mm are also covered;
- For slabs with profiled steel sheeting, the minimum thickness of flanges is 8 mm, i.e.  $t_f \geq 8 \text{ mm}$ . From the version 2019-05, the minimum thickness of flanges can be reduced to 6 mm when the relevant software option is activated (see 2.8.5). This option extended the scope of the ETA report [1].

**Error Code (see Annex D):**

Error code **22** is returned when this condition is not met.

### 2.4.2. Steel grade

The steel grade is to be chosen among the following list (as indicated in the ETA-15/0876 [1]): S235, S275, S355 or custom (the yield strength, defined by the user, should verify:  $170 \leq f_{yk} \leq 355 \text{ MPa}$ ).

**Error Code (see Annex D):**

Error code **9** is returned when the hereunder condition is not met. Error code **10** is returned when the steel grade is not among the authorised list.

The steel properties are calculated according to EN 1993-1-1 [12]:

$E$  is the elasticity modulus:  $E = 210000 \text{ MPa}$

$G$  is the shear modulus:  $G = 80770 \text{ MPa}$

$\rho_{\text{steel}}$  is the steel density:  $\rho_{\text{steel}} = 7850 \text{ kg/m}^3$ ;

$f_{yk}$  is the characteristic yield strength of the steel. For predefined steel grades (S235, S275 and S355), this value is obtained from Table 3 from the maximal thickness ( $t_f$  and  $t_w$ ) of the profile, according to EN 10025-2 [2]. For a custom steel, the characteristic yield strength is constant and equal to the value defined by the user.

Steel grade	S235	S275	S355
$t_f \leq 16$ mm	235	275	355
$16 < t_f \leq 40$ mm	225	265	345
$40 < t_f \leq 63$ mm	215	255	335
$63 < t_f \leq 80$ mm	215	245	325
$80 < t_f \leq 100$ mm	215	235	315
$100 < t_f \leq 150$ mm	195	225	295
$150 < t_f \leq 200$ mm	185	215	285

Table 3: Yield strength of the steel

## 2.5. Concrete slab

Two types of slabs are covered by the module. The user has to choose one of them:

- Solid slab (default)
- Slab with profiled steel sheeting

For both types of slabs, the concrete slab is defined by the following parameters:

- $h$  is the slab thickness. The minimum slab thickness, depending on the connector type and the effect of corrosion, is given in Table 4 (also in ETA-15/0876 [1])
- $\rho_c$  is the density of the concrete (default value 2000 kg/m<sup>3</sup> for normal concrete and 1800 kg/m<sup>3</sup> for light concrete, minimal value is 1750 kg/m<sup>3</sup>).

### Error Codes (see Annex D):

Error code **24** is returned when the condition on the slab thickness is not met.

Error code **13** is returned when the condition on the slab is not met.

X-HVB	Minimum slab thickness $h$ [mm]	
	Concrete coverage not required	Concrete coverage required
40	50	60
50	60	70
80	80	100
95	95	115
110	110	130
125	125	145
140	140	160

Table 4: Minimum slab thickness

The concrete type can be chosen between:

- Normal weight concrete
- Light weight concrete

For the normal weight concrete, the concrete class, as indicated in ETA-15/0876, can be chosen from:

- C20/25
- C25/30
- C30/37
- C35/45
- C40/50
- C45/55
- C50/60

For the light weight concrete, the concrete class, as indicated in ETA-15/0876, can be chosen from:

- LC20/22
- LC25/28
- LC30/33
- LC35/38
- LC40/44
- LC45/50
- LC50/55

The mechanical properties for concrete, given in Table 3.1 of EN 1992-1-1 [7] for the normal weight concrete or in Table 11.3.1 of EN 1992-1-1 for the light weight concrete, are defined by:

$f_{ck}$  is the characteristic value of the compression strength of the concrete “cylinder compressive test at 28 days”. Its value is obtained from the concrete class, see Table 5 or Table 6.

$f_{cd}$  is the design compressive strength of the concrete, calculated by:

$$f_{cd} = f_{ck} / \gamma_c \quad (1)$$

$E_{cm}$  is the mean secant modulus of elasticity of the concrete for short term loading. For normal concrete, its value is obtained in Table 5 from the concrete class. For light weight concrete, its mean secant modulus of elasticity is calculated by using the following formulae:

$$E_{lcm} = E_{cm} \eta_E \quad (2)$$

where:  $\eta_E = (\rho_c / 2200)^2$  and  $E_{cm}$  is obtained in Table 5 from the associated normal weight concrete class.

Normal weight concrete Class	$f_{ck}$ MPa	$E_{cm}$ kN/mm <sup>2</sup>
C20/25	20	30
C25/30	25	31
C30/37	30	33
C35/45	35	34
C40/50	40	35
C45/55	45	36
C50/60	50	37

Table 5: Mechanical properties of the normal weight concrete

Light weight concrete Class	$f_{ck}$ MPa
-----------------------------	-----------------

LC20/22	20
LC25/28	25
LC30/33	30
LC35/38	35
LC40/44	40
LC45/50	45
LC50/55	50

Table 6: Mechanical properties of the light weight concrete

For solid slabs, it will also be possible for the user to define a haunch. Its width will be equal to the top flange width whereas its depth will be defined by the user.

## 2.6. Profiled steel sheeting

The profiled steel sheeting is defined by its geometry and its surface weight:

$h_p$  is the deck depth

$t_p$  is the deck thickness

$b_s$  is the trough spacing

$b_t$  is the top width of the rib

$b_b$  is the bottom width of the rib

$G_{deck}$  is the deck surface weight

$f_{ypk}$  is the yield strength of the steel

$n_{rib}$  is the number of rib stiffener, that can be equal to 0, 1 or 2. The value 0 is used for sheetings without rib stiffeners or for sheetings with stiffeners that can be bent down when the connectors are nailed

$s_{un}$  is the stiffener width (used only if  $n_{rib} \geq 1$ )

$s_{av}$  is the spacing between rib stiffeners (used only if  $n_{rib} = 2$ ).

Following assumptions are considered for rib stiffeners – see Figure 5:

- The distribution of the stiffeners is assumed to be symmetrical with respect to the vertical rib axis;
- Stiffeners are assumed to be identical and the stiffener arrangement is identical for every rib;
- The shape of the stiffener is displayed as triangular or trapezoidal according to the dimensions of the rib.

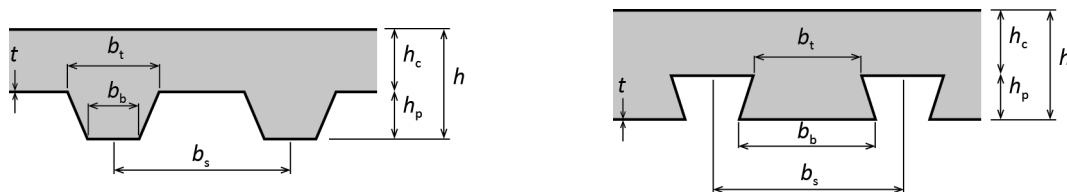


Figure 4: Dimensions of a profiled steel sheeting



Figure 5: Geometry of a rib with one or two stiffeners

The orientation of ribs can be chosen between:

- *Perpendicular* to the beam axis. In this case, the decking may be: *continuous* or *not continuous* on the beam;
- *Parallel* to the beam axis.

The design yield strength of the profiled steel sheeting is obtained from the following relation:

$$f_{ypd} = f_{ypk} / \gamma_p \quad (3)$$

**Error Codes (see Annex D):**

Error code **16** is returned when the following conditions are not met:

- $0 \leq G_{deck} \leq 050 \text{ kg/m}^2$
- $170 \text{ MPa} \leq f_{ypk} \leq 600 \text{ MPa}$

Error code **15** is returned when the following conditions are not met:

- $0,5 \text{ mm} \leq t_p \leq 2,0 \text{ mm}$
- $100 \text{ mm} \leq b_s \leq 600 \text{ mm}$
- $30 \text{ mm} \leq b_t \leq 400 \text{ mm}$
- $30 \text{ mm} \leq b_b \leq 400 \text{ mm}$
- $b_s \geq \max(b_b; b_t)$
- $0 \leq n_{rib} \leq 2$
- $0 \text{ mm} \leq s_{un} \leq 50 \text{ mm}$
- $0 \text{ mm} \leq s_{av} \leq 200 \text{ mm}$
- $b_b > 2 s_{un} + s_{av}$

## 2.7. Reinforcement steel bars

Only the steel grade of reinforcement steel bars is necessary in calculations. It can be chosen from:

- B500 : it covers B500A, B500B, B500C
- B450 : it covers B450B, B450C

The characteristic value of the yield strength of the reinforcement steel, denoted  $f_{yr,k}$ , is given in Table 7.

Steel grade	B450	B500
$f_{yrk}$ [MPa]	450	500

Table 7 : Yield strength of the reinforcement steel

The design yield strength of the reinforcement steel is calculated by:

$$f_{yrd} = f_{yrk} / \gamma_s$$

(4)

**Error Code (see Annex D):**

Error code 17 is returned when the steel grade is not among the authorised list.

## 2.8. Shear connectors

### 2.8.1. General parameters

The shear connection is defined by:

$P_{Rk}$  is the characteristic resistance of the connector

$P_{Rd}$  is the design resistance of the connector

Dimensions used in calculations and for drawings are (see Table 8 for values):

$h_{sc}$  is the total height

$w_b$  is the bottom length

$w$  is the transverse width

The user can choose the connector type from the following list:

- X-HVB 40 (used only for solid slabs)
- X-HVB 50 (used only for solid slabs)
- X-HVB 80
- X-HVB 95
- X-HVB 110
- X-HVB 125
- X-HVB 140

For slabs with profiled steel sheeting, connectors X-HVB 40 and X-HVB 50 are not applicable.

The values of  $P_{Rk}$  and  $P_{Rd}$  are given in Table 8 (according to Table 3 of ETA 15-0876):

X-HVB	Characteristic resistance $P_{Rk}$ [kN]	Design resistance $P_{Rd}$ [kN]	$h_{sc}$ mm	$w$ mm	$w_b$ mm
40	29	23,2	43	24,3	51
50	29	23,2	52	24,3	50
80	32,5	26	80	24,3	50
95	35	28	95	24,3	50
110	35	28	112,5	24,3	51
125	37,5	30	127,5	25,3	51
140	37,5	30	142,5	25,3	51

Table 8 : Properties of connectors

**Error Code (see Annex D):**

The consistency of the selected connector with the previously defined parameter should also be checked.

- first check: control of the minimum slab thickness, according to the requirement of concrete coverage – see Table 4. If the check is negative, a warning message should alert the user about the inconsistency of the values. If this control is negative, the calculation module will send back an error index = 24.
- second check: for slabs with profiled steel sheeting, a second check should be performed regarding the maximum height of the composite decking, see Table 9. If the check is negative,

a warning message should alert the user about the inconsistency of the values. If this control is negative, the calculation module will send back an error index = 25.

X-HVB	Maximum value of $h_p$ (mm)		
	$b_0/h_p \geq 1,8$	$1,0 < b_0/h_p < 1,8$	$b_0/h_p \leq 1,0$
80	45	45	30
95	60	57	45
110	75	66	60
125	80	75	73
140	80	80	80

Table 9 : Maximum height of the profiled steel sheeting

For composite decking perpendicular to the beam axis with connectors parallel with the beam axis, the following additional condition must be fulfilled:

$$b_0/h_p \geq 1,0$$

Where:

$$b_0 = (b_t + b_b)/2 \quad \text{if } b_t \geq b_b$$

$$b_0 = b_t \quad \text{if } b_t < b_b$$

### 2.8.2. Orientation of connectors

For X-HVB connectors, 3 orientations are possible:

- duckwalk
- longitudinal
- transverse

In most of cases, the orientation is directly chosen by the software, according to the geometry and to the connector type:

- for solid slabs with X-HVB 40 or X-HVB 50 connectors, the orientation is duckwalk;
- for solid slabs with other connectors, the orientation is longitudinal;
- for slabs with profiled sheeting parallel with the beams axis, the orientation is longitudinal.

For slabs with profiled steel sheeting transverse to the beam axis, the orientation of connectors must be chosen between:

- Longitudinal with the beam axis;
- Transverse with the beam axis.

### 2.8.3. Degree of connection

Only when a plastic design is performed (see § 2.2), the user has to choose the degree of connection for the calculation. The three possible choices are:

- full connection;
- partial connection;
- partial connection with a user-defined degree of connection.

When the full connection is chosen, the software calculates the number of connectors in order to be at least equal to the minimum plastic resistance of the slab in compression or the steel profile in tension, so that the full plastic bending resistance of the beam can be obtained. The partial connection choice involves a reduced number of connectors, also assessed by the software, and consequently a reduced bending resistance. The partial connection also implies a greater deformation of the loaded composite beam.



For a partial connection, the user has the possibility (release May 2019) to directly impose a degree of connection (third choice). In this case, it is not necessary to fulfil the minimum requirement of Eurocode 4 on the degree of connection. The relevant warning message is then given in the calculation sheet.

#### 2.8.4. Connection arrangement along the beam

The connection between the slab and the steel profile is automatically designed by the software. Two separate approaches are available according to the type of slabs.

For plain slabs or for composite slabs with parallel profiled steel decking, the connection is always assumed to be uniform along the beam. These types of slabs are thus optimised by giving the minimum number of connectors required to fulfil all the Eurocode requirements.

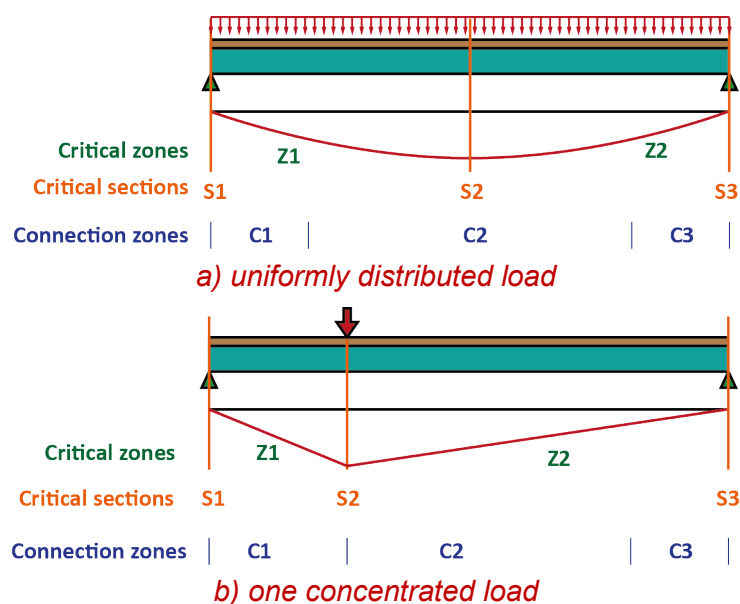
For composite slabs with transverse sheetings, the software tries by default to optimise the connection. In order to minimise the number of connectors that are necessary to fulfil all the Eurocode requirements, it might in this case lead to the definition of one to three connection zones with separate connectors arrangement. The following principles are observed for the definition of connection zones:

- connectors arrangement along a connection zone is always assumed to be uniform;
- the length of a connection zone is at least equal the one fifth of the beam length;
- the limit between two adjacent connection zones is always located between a beam end and the first critical section.

Critical sections are defined in Eurocode 4 as follows:

- cross-sections where the bending moment is maximal;
- cross-sections where concentrated loads are introduced.

Figure 6 below shows examples of load configuration, critical sections and connections zones. Alternatively, for composite slabs with transverse profiled steel sheeting, the user can impose a uniform arrangement of connectors along the beam (i.e. one single connection zone without optimisation). In this case, the total number of connectors proposed by the software might not be the most economical one.



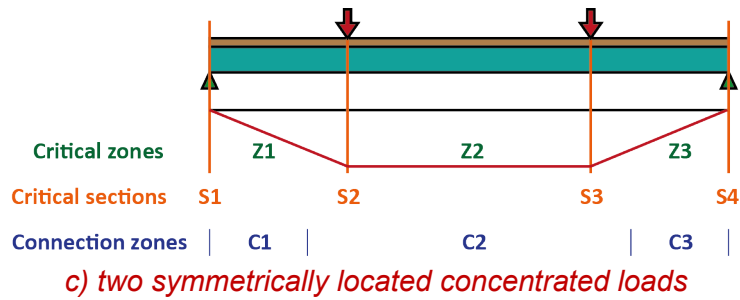


Figure 6 : Configurations of critical sections and connection zones.

### 2.8.5. Connection rules

The connectors are placed on the beam following the requirements of ETA document [1]. Nevertheless, two scope extensions of the ETA document are introduced in the software, based on HILTI internal judgement. The scope extensions concern:

- the possibility to have flange thickness between 6 and 8 mm, even for composite slabs (see 2.4.1);
- the possibility to have only one row of connectors in profiled deckings with narrow ribs (see 2.9.2).

These scope extensions are both activated when the following conditions are all met:

- Composite slab with profiled steel sheeting;
- Profiled steel sheeting is perpendicular to the beam;
- The selected profiled steel sheeting is HI BOND 55, HI BOND A 55 or Sand 55 Profile sheeting.

If the scope extensions have been used in the calculations, warning messages are displayed in the interface and in the calculation report.

## 2.9. Spacing and positioning of connectors

The spacing between connectors within cross section must fulfil conditions that are indicated in ETA 15-0876.

### 2.9.1. For solid slab with multiple rows of connectors:

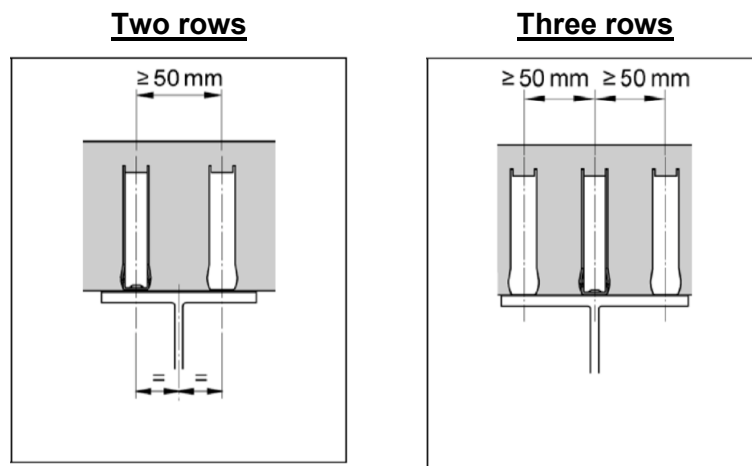


Figure 7 : Spacing of connectors for solid slabs

The flange width, denoted  $b$ , of the steel section must fulfil the following condition:

$$b \geq 50(n_r - 1) + w$$

Where:  $w$  is the transverse width of the connector.

This condition may limit the maximum number of connectors in a row – see § 6.4.1.

### 2.9.2. Slabs with transverse steel decking

- **connectors parallel with the beam axis (single row), without rib stiffeners**

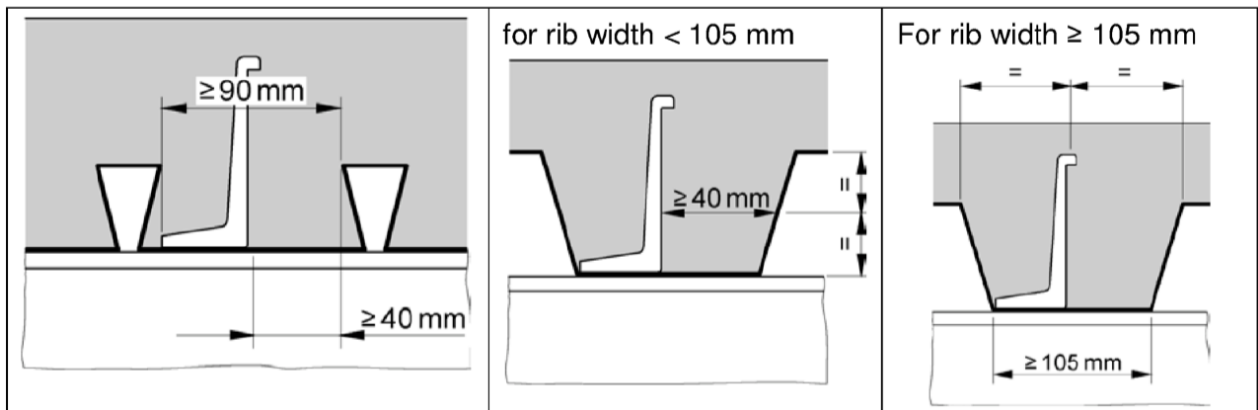


Figure 8 : Spacing of connectors for slabs with transverse decking and connectors parallel with the beam (single row).

For the rib width smaller than  $b_b < 105 \text{ mm}$ , the width at mid-height of the rib must fulfil the following condition:

$$b_0 \geq \max (w_b + 40 \text{ mm}; 90\text{mm})$$

Where:  $w_b$  is the bottom width of the connector.

**Error Code (see Annex D):**

Error code **26** is returned when this condition is not met.

- **connectors parallel with the beam axis (single row), with rib stiffeners**

For sheetings with one rib stiffener (see Figure 9), the following additional condition must be fulfilled:

$$\min\{b_b, b_t\} \geq 2w_b + s_{un}.$$

**Error Code (see Annex D):**

*Error code **28** is returned when this condition is not met.*

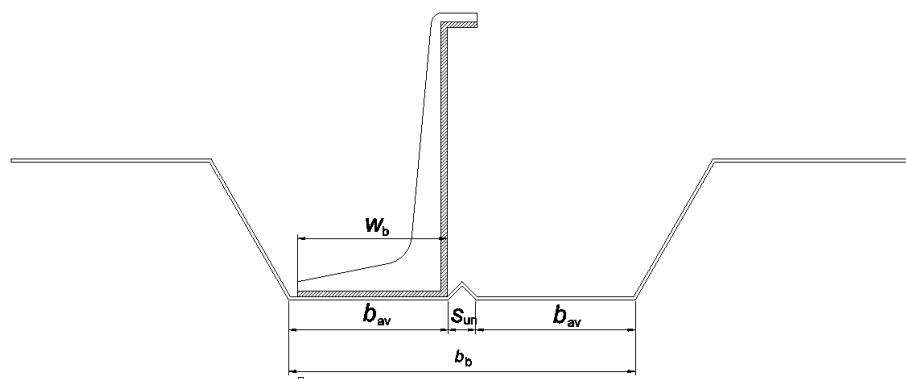


Figure 9 : Transverse sheeting with one rib stiffener and parallel connectors.

For sheetings with two rib stiffeners, two different configurations are possible:

- if the spacing of stiffeners greater or equal to the connector width ( $s_{av} \geq w_b$ , see Figure 10), the connectors are located between the stiffeners.
- if the spacing of stiffeners is smaller than the connector width ( $s_{av} < w_b$ , see Figure 11), the connectors are located outside the stiffeners, if the following additional condition is fulfilled:

$$\min\{b_b, b_t\} \geq 2w_b + 2s_{un} + s_{av}$$

**Error Code (see Annex D):**

Error code **28** is returned when the two locations of the connectors shown on Figures 8 and 9 are not possible.

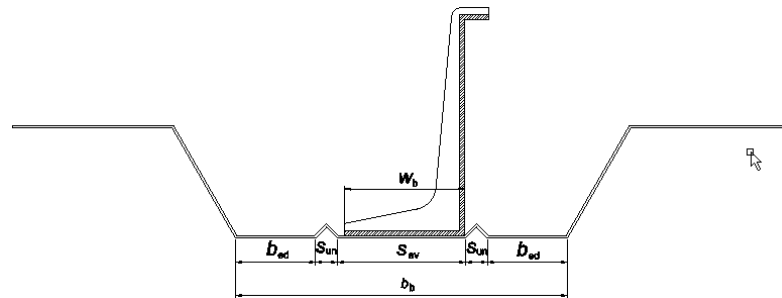


Figure 10 : Transverse sheeting with two rib stiffeners and parallel connectors.

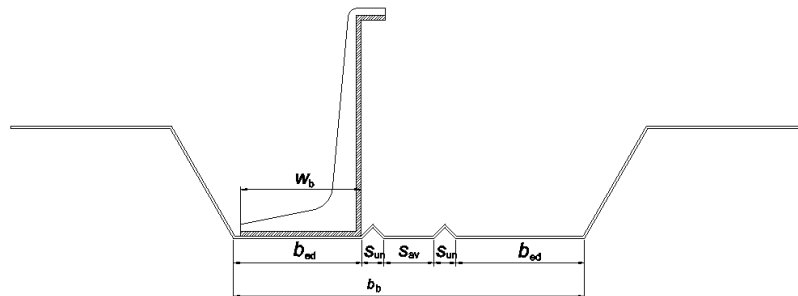
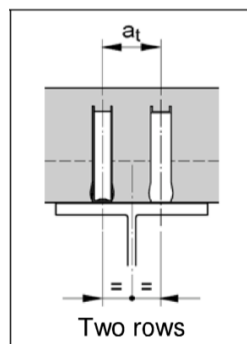


Figure 11 : Transverse sheeting with two rib stiffeners and parallel connectors.

- **connectors parallel with the beam axis (multiple rows), without rib stiffeners**

**Two rows**



**Three rows**

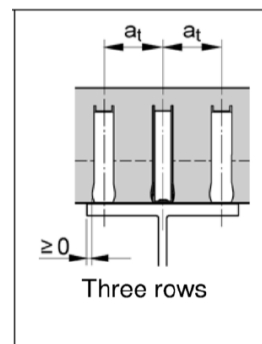


Figure 12 : Spacing of connectors for slabs with transverse decking and connectors parallel with the beam.

The following conditions must be fulfilled:

- For the flange width of the steel section:

$$b \geq (n_r - 1)a_t + w$$

Where:  $a_t \geq 50 \text{ mm}$  for profiled decking with  $b_0/h_p \geq 1,8$

$a_t \geq 100 \text{ mm}$  for other decking

This condition may limit the maximum number of connectors in a row – see § 6.4.1.

- For the bottom width of the rib:

$$b_b \geq 60 \text{ mm}$$

**Error Code (see Annex D):**

Error code **26** is returned when this condition is not met.

▪ **Connectors parallel with the beam axis (multiple rows), with rib stiffeners**

For sheetings with one or two rib stiffeners, the same conditions as the previous case (case of single row) are applied for the case of multiple rows.

▪ **Connectors transverse to the beam axis (single row), without rib stiffener**

Only the deck without rib stiffener is considered in this document.

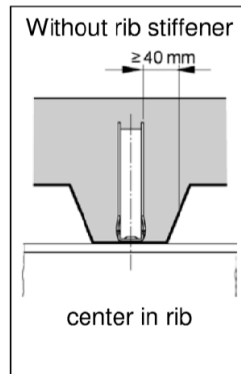


Figure 13 : Spacing of connectors for slabs with transverse decking and connectors transverse to the beam (single row).

The width at mid-height of the rib must fulfil the following condition:

$$b_0 \geq w + 80 \text{ mm}$$

**Error Code (see Annex D):**

Error code **26** is returned when this condition is not met and if  $b_0 < 40 \text{ mm}$  (see conditions for multiple rows).

Error code **26** is also returned if one of the two following conditions is not fulfilled:

- $b_0 > w$
- $b_r > w$

When the 2 following conditions are met:

- $b_0 < w + 80 \text{ mm}$
- $b_b \geq 40 \text{ mm}$

Sometimes, it is not possible to have a single row but it is possible to have multiple rows. So the calculation module starts the design process by considering 2 connectors in a row (see 6.4.1).

From the version 2019-05, the minimum width of ribs for one connector can be reduced when the relevant software option is activated (see 2.8.5). This option extended the

scope of the ETA report [1]. In this case, following conditions are checked:  $b_b \geq 30$  mm and  $b_t \geq 30$  mm.

▪ **Connectors transverse to the beam axis (single row), with rib stiffeners**

For sheetings with one rib stiffener (see Figure 14), according to ETA 15/0876 case b, only the following condition must be fulfilled:

$$\min\{b_b, b_t\} \geq 2w + s_{un}$$

**Error Code (see Annex D):**

Error code **28** is returned when this condition is not met.

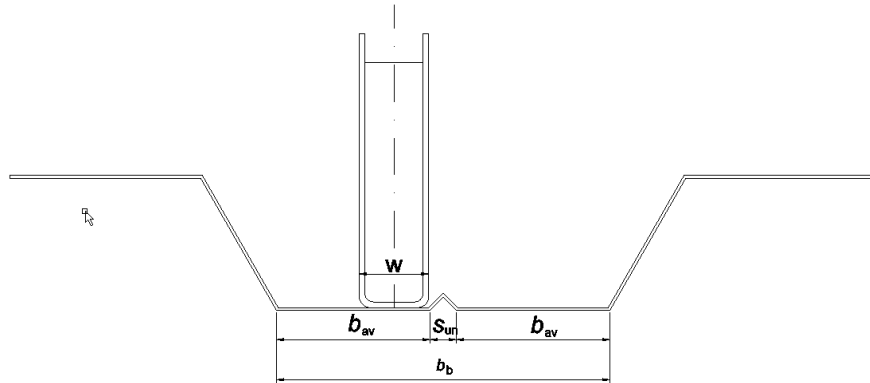


Figure 14 : Transverse sheeting with one rib stiffener and transverse connectors.

According to cases a) and b) given in page 14 of Annex B7 in ETA document [1], case b) should be preferred. It does not influence the final results or drawings of the software. In this configuration, the following warning message is displayed, both in the calculation sheet and in the web interface: “Note: place the H-HVB in contact with the stiffener towards the nearest support in the compression zone of the concrete rib”.

For sheetings with two rib stiffeners, two different locations are possible, according to the spacing between stiffeners:

- if  $s_{av} \geq w$  (see Figure 15), the connector is placed in the rib axis.
- if  $s_{av} < w$  (see Figure 16), the connector is placed outside the two ribs, if the following condition is fulfilled:

$$\min\{b_b, b_t\} \geq 2w + 2s_{un} + s_{av}$$

**Error Code (see Annex D):**

Error code **28** is returned when the previous conditions are not met.

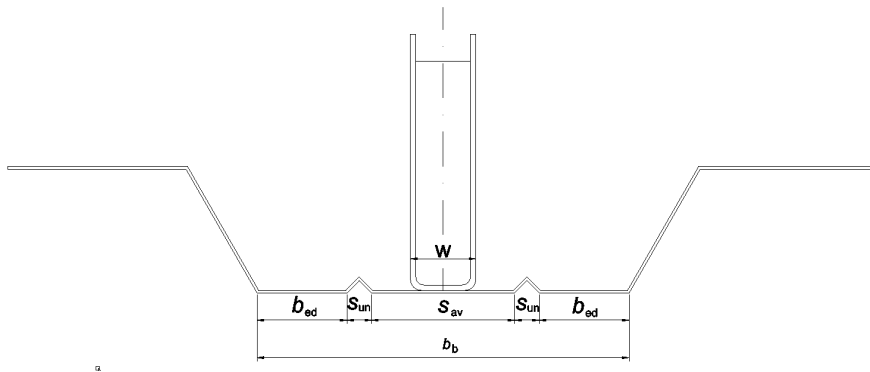


Figure 15 : Transverse sheeting with two rib stiffeners and transverse connectors.

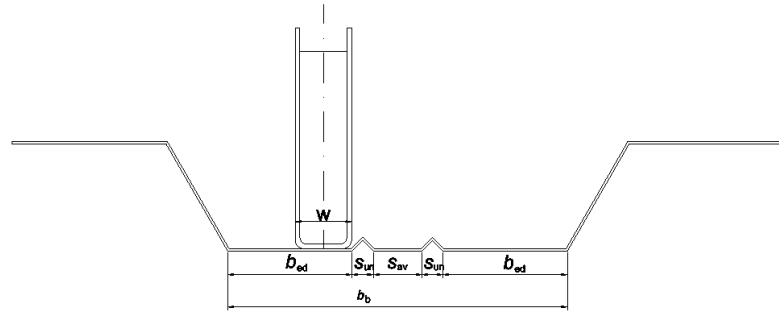


Figure 16 : Transverse sheeting with two rib stiffeners and transverse connectors.

According to cases a) and b) given in page 14 of Annex B7 in ETA document [1], case b) should be preferred. It does not influence the final results or drawings of the software. In this configuration, the following warning message is displayed, both in the calculation sheet and in the web interface: “*Note: place the H-HVB in contact with the stiffener towards the nearest support in the compression zone of the concrete rib*”.

▪ **Connectors transverse to the beam axis (multiple rows), without rib stiffener**

For steel sheeting without rib stiffener the following conditions must be fulfilled:

- For the flange width of the steel section:

$$b \geq (n_r - 1)a_t$$

Where:  $a_t \geq 50 \text{ mm}$  for profiled decking with  $b_0/h_p \geq 1,8$  and with two rows of connectors

$$a_t \geq 100 \text{ mm for other decking or with three rows of connectors}$$

This condition may limit the maximum number of connectors in a row – see § 6.4.1.

- For the bottom width of the rib:

$$b_b \geq 40 \text{ mm}$$

This condition may limit the maximum number of connectors in a row.

- For the bottom width of connectors:

$$a_t \geq 2w_b$$

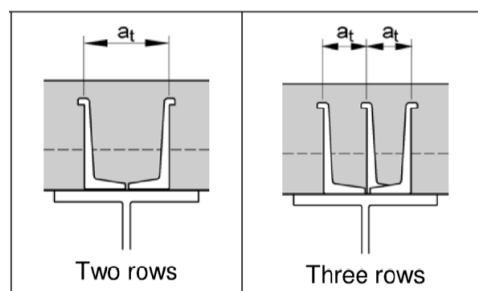


Figure 17 : Spacing of connectors for slabs with transverse decking and connectors transverse to the beam (multiple rows).

▪ **Connectors transverse to the beam axis (multiple rows), with rib stiffeners**

For decks with rib stiffeners, the same conditions as the previous case (single row) are applied for the case of multiple rows.

### 2.9.3. Slabs with parallel steel decking

- **Connectors parallel with the beam axis (single row), without rib stiffeners**

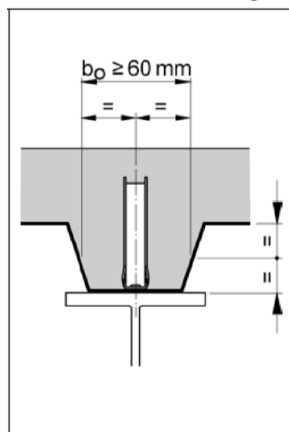


Figure 18 : Spacing of connectors for slabs with transverse decking and connectors transverse to the beam (multiple rows).

The width at mid-height of the rib must fulfil the following condition:

$$b_0 \geq 60 \text{ mm}$$

**Error Code (see Annex D):**

Error code **26** is returned when this condition is not met.

- **Connectors parallel with the beam axis (single row), with rib stiffeners**

For sheetings with one rib stiffener, the software proposes, when possible, the splitting of the rib stiffener, in order to keep the central location of the connector. If the splitting of the rib stiffener is not possible, the configuration with only one row of connectors is not allowed and the minimal number of rows becomes 2.

The dimensions of the split rib are as follows (see Figure 19):

$b_{b,split}$ : width of the rib at the bottom, given by:  $b_{b,split} = b_f$

$b_{t,split}$ : width of the rib at the top, given by:  $b_{t,split} = b_t + (b_f - b_b)$

The splitting of the rib stiffener is possible only if the two following conditions are met:

- $b_f/2 \geq s + w/2$ , where  $s = (b_b + s_{un})/2$
- $b_{0,split} \geq 60$  mm, where  $b_{0,split} = b_0 + (b_f - b_b)$

**Error Code (see Annex D):**

*Error code 28 is returned when all the conditions relative to parallel sheetings with parallel connectors and with one rib stiffener are not met.*

If this solution is possible and is finally considered, the following warning message is displayed, both in the calculation sheet and in the web interface: “*The centric positioning of the connectors within the concrete rib imposes the split of the decking*”

For sheetings with two rib stiffeners, the following additional condition must be fulfilled:

$$S_{qv} \geq w$$

**Error Code (see Annex D):**

*Error code 28 is returned when all the conditions relative to parallel sheetings with parallel connectors and with two rib stiffeners are not met.*



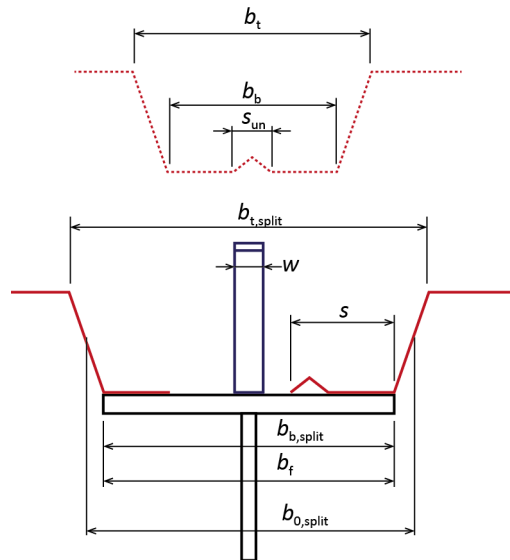


Figure 19 : Splitting of a rib with one rib stiffener (one row of parallel connectors).

▪ **Connectors parallel with the beam axis (multiple rows), without rib stiffener**

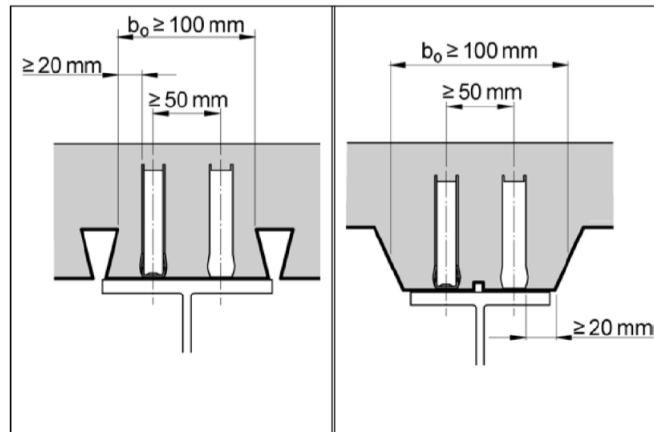


Figure 20 : Spacing of connectors for slabs with decking parallel with the beam (multiple rows).

The following conditions must be fulfilled:

- For the flange width of the steel section:  
 $b \geq w_b + 50 \text{ mm}$
- For the width at mid-height of the rib:  
 $\min\{b_b, b_t\} \geq w + 90 \text{ mm}$   
 $b_o \geq 100 \text{ mm}$

▪ **Connectors parallel with the beam axis (multiple row), with rib stiffeners**

For sheetings with one rib stiffener, the following conditions must be fulfilled:

- For the flange width of the steel section:  
 $b \geq w + \max\{50 \text{ mm}, s_{un} + w\}$

- For the width of the rib:  
 $\min\{b_b, b_t\} \geq w + 40 \text{ mm} + \max\{50 \text{ mm}, s_{un} + w\}$   
 $b_0 \geq 100 \text{ mm}$

**Error Code (see Annex D):**

Error code 28 is returned when all the conditions relative to parallel sheetings with parallel connectors and with one rib stiffener are not met.

For sheetings with two rib stiffeners, two configurations are possible:

- if  $s_{av} \geq w + 50 \text{ mm}$  (see Figure 21), the connectors are located between the stiffeners. The same conditions as the case of sheetings without rib stiffener must be fulfilled.

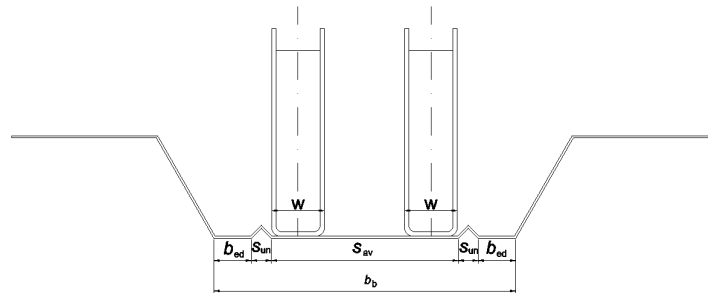


Figure 21 : Sheeting with two rib stiffeners and connectors parallel with the beam (multiple rows).

- if  $s_{av} < w + 50 \text{ mm}$ , the connectors are located outside the stiffeners, if the following conditions are fulfilled:

- For the flange width of the steel section:

$$b \geq w + \max\{50 \text{ mm} - w, s_{av} + 2s_{un}\}$$

- For the width of the rib:

$$\min\{b_b, b_t\} \geq 2w + 40 \text{ mm} + \max\{50 \text{ mm} - w, s_{av} + 2s_{un}\}$$

$$b_0 \geq 100 \text{ mm}$$

**Error Code (see Annex D):**

Error code 28 is returned when all the conditions relative to parallel sheetings with parallel connectors and with two rib stiffeners are not met.

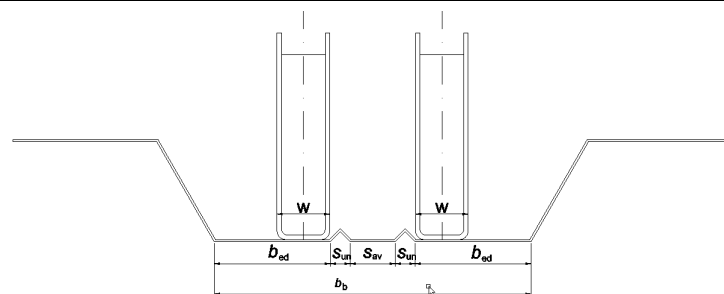


Figure 22 : Sheeting with two rib stiffeners and connectors parallel with the beam (multiple rows).

#### 2.9.4. Slabs with duckwalk positioning (X-HVB-40 and 50) – single row only.

The minimum spacing between 2 connectors is 100 mm.

This condition may limit the maximum number of connection rows – see § 6.4.1.

## 2.10. Loads

### 2.10.1. General definitions

The calculation module allows the user to define elementary variable load cases that will be used in the combinations of actions for ULS or SLS according to EN 1990 [3].

Only gravity loads are considered (downwards).

Up to four elementary load cases are considered within these specifications:

- One permanent load case, denoted **G**
- One live load during construction stage, denoted **Q<sub>c</sub>**
- Up to two live load cases during the final stage, denoted **Q<sub>1</sub>** and **Q<sub>2</sub>**

The dead weight of the steel profile and concrete slab will be automatically calculated and added in the permanent load case.

For each load case, it's possible to define:

- One uniformly distributed surface load, denoted  $q_{surf}$
- Up to ten point loads along the beam, denoted  $P_i$
- Up to three patch loads along the beam, denoted  $q_i$

For the permanent load case **G**, 2 surface loads could be defined:

- the first one associated to the dead loads of the beam and acting during the construction phase;  $q_{surf,d}$  is by default automatically calculated by the program but may be modified by the user;
- the second one  $q_{surf,u}$  associated to additional permanent loads, acting only during the composite stage.

For live load construction stage live loads case, only the uniformly distributed surface load can be defined in the UI.

For the final stage live load Q1, only the uniformly distributed surface load can be defined in the UI.

### 2.10.2. Specific treatment of surface loads

For the check of the beam, and especially the calculation of internal forces, the surface load of each load case is derived either as uniformly distributed loads or as a set of point loads.

For a "intermediate beam", a linear uniformly distributed load along the beam is derived:

$$q_{lin} = q_{surf} (b_1 + b_2)/2 \quad (5)$$

For a "edge beam", this relation is replaced by:

$$q_{lin} = q_{surf} (b_1 + b_2/2) \quad (6)$$

### 2.10.3. Automatic dead load assessment

The dead loads of the beam, of the potential secondary beams, of the slab and of the potential steel profiled sheeting are automatically included in the permanent load case **G**.

The dead load of the beam is treated as an uniformly surface load defined by:

$$\begin{aligned} \text{for intermediate beams : } q_{surf,d} &= 2 \frac{g \rho_{steel} A}{b_1 + b_2} \\ \text{for edge beams : } q_{surf,d} &= 2 \frac{g \rho_{steel} A}{2 b_1 + b_2} \end{aligned} \quad (7)$$

where:  $g$  is the gravity constant:  $g = 9,81 \text{ m/s}^2$

$\rho_{steel}$ : see § 2.4.2

A is the area of the beam profile (see Annex A)

The dead load of a slab is defined as a surface load given by the following equation:

$$q_{\text{slab}} = g \cdot \rho_c \cdot h_{\text{eq}} \quad (8)$$

where:  $\rho_c$ : see § 2.5

$h_{\text{eq}}$ : is the equivalent thickness of the slab defined by:

$h_{\text{eq}} = h$  for plain slabs

$h_{\text{eq}} = h + h_p \frac{b_r + b_b - 2 b_s}{2 b_s}$  for slabs with profiled steel sheeting

$h_p, b_r, b_b, b_s$ : see § 2.6

$h$ : see § 2.5

The dead load of a profiled steel sheeting is defined as a surface load given by:

$$q_{\text{deck}} = g \cdot G_{\text{deck}} \quad (9)$$

where:  $G_{\text{deck}}$ : see § 2.6.

### 2.10.4. Default surface live loads

The default surface live load (for live load case Q1) is defined according to the category of loaded surface and the National Annex – see Table 10. Values are given in EN 1991-1-1 and the associated NA.

	Category of loaded areas					
	<b>A</b> <sup>(1)</sup> Residential	<b>B</b> Office	<b>C</b> <sup>(2)</sup> Congregat.	<b>D</b> <sup>(3)</sup> Shopping	<b>E</b> <sup>(4)</sup> Storage	<b>H</b> Roof
Recommended values of Eurocodes	2,0	3,0	3,0	4,0	7,5	0,4
Austria	2,0	2,0	3,0	4,0	5,0	1,0
Belgium	2,0	3,0	3,0	5,0	7,5	$0,8-A/100 \geq 0,2$
Bulgaria	2,0	3,0	3,0	4,0	7,5	0,75
Czech Republic	1,5	2,5	3,0	5,0	7,5	0,75
Estonia	2,0	3,0	3,0	5,0	7,5	0,4
France	1,5	2,5	2,5	5,0	7,5	0,8
Germany	2,0	2,0	3,0	2,0	6,0	0,4
Hungary	2,0	3,0	3,0	4,0	7,5	0,4
Italy	2,0	2,0	3,0	4,0	6,0	0,5
Latvia	2,0	2,5	2,5	4,0	7,5	0,4
Lithuania	1,5	2,0	3,0	4,0	7,5	0,4
Luxemburg	2,0	3,0	3,0	5,0	7,5	0,4
Netherlands	1,75	2,5	4,0	4,0	5,0	1,0 <sup>(6)</sup>
Poland	2,0	3,0	3,0	4,0	7,5	0,4
Portugal	2,0	3,0	3,0	4,0	7,5	0,4
Romania	1,5	2,5	3,0	4,0	7,5	0,75
Singapore <sup>(7)</sup>	1,5	2,5	3,0	4,0	5,0	0,5
Slovakia	2,0	3,0	3,0	4,0	7,5	0,4
Slovenia	2,0	3,0	3,0	4,0	7,5	0,4
Spain	2,0 <sup>(*)</sup>	3,0 <sup>(*)</sup>	3,0 <sup>(*)</sup>	4,0 <sup>(*)</sup>	7,5 <sup>(*)</sup>	0,4 <sup>(*)</sup>
Switzerland	2,0 <sup>(*)</sup>	3,0 <sup>(*)</sup>	3,0 <sup>(*)</sup>	5,0 <sup>(*)</sup>	7,5 <sup>(*)</sup>	0,4 <sup>(*)</sup>
United Kingdom <sup>(5)</sup>	1,5	2,5	2,5	4,0	5,0	0,6
Non-members of CEN <sup>(**)</sup>	2,0 <sup>(*)</sup>	3,0 <sup>(*)</sup>	3,0 <sup>(*)</sup>	4,0 <sup>(*)</sup>	7,5 <sup>(*)</sup>	0,4 <sup>(*)</sup>

Table 10 : Default surface live load (kN/m<sup>2</sup>)

Notes:

(\*) indicates that the recommended value is used because no national annex is available. Parameters in italic and red indicate the values which differ from the recommended values

(\*\*) Countries non-members of CEN are: Qatar, Saudi-Arabia, South Africa, Turkey, UAE.

(1) For category A, sub category Floors is considered

(2) For category C, sub category C1 is considered.

(3) For category D, sub category D1 is considered.

- (4) For category E, sub category E1 is considered.
- (5) For UK NA: categories A1, B1, C11 and E14 are considered
- (6) For a slope of roof equal to 0
- (7) For Singapore NA: categories A1, B1, C13 and E14

### 3. COMBINATIONS OF LOADS

---

#### 3.1. Ultimate Limit States (ULS)

The ULS combinations are automatically generated according to EN 1990:

- For the case with two variable load cases:

$$\begin{aligned} \gamma_G G + \gamma_Q Q_1 + \gamma_Q \psi_0 Q_2 \\ \gamma_G G + \gamma_Q Q_2 + \gamma_Q \psi_0 Q_1 \end{aligned} \quad (10)$$

- For the case with one variable load case:

$$\gamma_G G + \gamma_Q Q_1 \quad (11)$$

#### 3.2. Serviceability Limit States (SLS)

The SLS combinations for the verification of deflections are automatically generated according to EN 1990:

- For the case with two variable load cases:

$$\begin{aligned} G + Q_1 + \psi_0 Q_2 \\ G + Q_2 + \psi_0 Q_1 \end{aligned} \quad (12)$$

- For the case with one variable load case:

$$G + Q_1 \quad (13)$$

The SLS combinations for the calculation of natural frequencies are:

$$\begin{aligned} G + p_Q Q_1 \\ G + p_Q Q_2 \end{aligned} \quad (14)$$

where:  $p_Q$  is the percentage of the variable load case in the SLS combination.

**Error Code (see Annex D):**

Error code **21** is returned if the following condition is not met:

- $0 \leq p_Q \leq 50 \%$

## 4. GLOBAL ANALYSIS

### 4.1. Design points

Shear forces, bending moments and deflections are calculated at design points along the beam. Initially, design points are regularly spaced along the beam with the spacing of  $L/50$  between two consecutive design points. An additional design point may be added at each point load if the last one is not located at existing design points.

### 4.2. Critical sections

ULS verifications are carried out at critical sections (EN 1994-1-1 [22] §6.1.1 (4)) where:

- The bending moment is maximum
- At supports
- At point load locations

### 4.3. Calculation of internal forces, moments and deflections for basic loads

The calculation of internal forces and moments is described hereafter for each individual point load and patch load. Any surface load will be considered with these 2 methods according to § 2.10.2.

#### 4.3.1. Point load

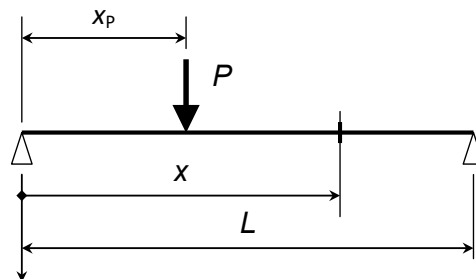


Figure 23 : Point load.

The reactions at supports are calculated by:

$$R_L = -P (L - x_p) / L \quad \text{at the Left support}$$

$$R_R = -P x_p / L \quad \text{at the Right support}$$

where:  $P$  is the applied point load;

$x_p$  is the abscissa of the point load from the left support

The shear force in a section located at the abscissa  $x$  is calculated by:

$$V(x) = R_L \quad \text{if } x < x_p$$

$$V(x) = -R_R \quad \text{if } x > x_p$$

The bending moment in a section located at the abscissa  $x$  is calculated by:

$$M(x) = -R_L x \quad \text{if } x < x_p$$

$$M(x) = -R_R (L - x) \quad \text{if } x > x_p$$

The deflection in a section located at the abscissa  $x$  is calculated by:

$$w(x) = \frac{F}{6EI} [L^2 - (L - x_p)^2 - x^2](L - x_p)x \quad \text{if } x < x_p$$

$$w(x) = \frac{F}{6EI} [L^2 - (L - x)^2 - x_p^2](L - x)x_p \quad \text{if } x > x_p$$

#### 4.3.2. Patch load

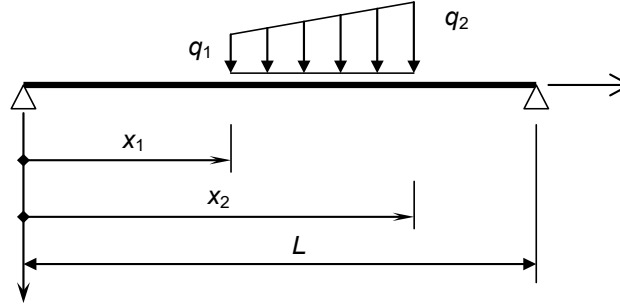


Figure 24 : Patch load.

The reactions at supports are calculated by:

$$R_L = \left[ q_1 \left( \frac{x_1 + x_2}{2L} - 1 \right) + \frac{q_2 - q_1}{2} \left( \frac{x_1 + 2x_2}{3L} - 1 \right) \right] (x_2 - x_1) \quad \text{at the Left support}$$

$$R_R = \left[ -q_1 \left( \frac{x_1 + x_2}{2L} \right) - \frac{q_2 - q_1}{2} \left( \frac{x_1 + 2x_2}{3L} \right) \right] (x_2 - x_1) \quad \text{at the Right support}$$

Where  $x_1$ ,  $q_1$ ,  $x_2$  and  $q_2$  define the distributed load as shown in *Figure 24*.

The shear force in a section located at the abscissa  $x$  is calculated by:

$$V(x) = R_L \quad \text{if } x < x_1$$

$$V(x) = R_L + \left[ q_1 + \frac{q_2 - q_1}{2} \left( \frac{x - x_1}{x_2 - x_1} \right) \right] (x - x_1) \quad \text{if } x_1 \leq x \leq x_2$$

$$V(x) = -R_R \quad \text{if } x > x_2$$

The bending moment in a section located at the abscissa  $x$  is calculated by:

$$M(x) = -R_L x \quad \text{if } x < x_1$$

$$M(x) = -R_L x - \left[ 3q_1 + (q_2 - q_1) \left( \frac{x - x_1}{x_2 - x_1} \right) \right] \frac{(x - x_1)^2}{6} \quad \text{if } x_1 \leq x \leq x_2$$

$$M(x) = -R_R (L - x) \quad \text{if } x > x_2$$

The deflection in a section located at the abscissa  $x$  is calculated by:

$$w(x) = \frac{1}{EI} \left( R_L \frac{x^3}{6} + A_1 x + B_1 \right) \quad \text{if } x < x_1$$

$$w(x) = \frac{1}{EI} \left( -R_R \frac{x^3}{6} + R_R \frac{Lx^2}{2} + A_3 x + B_3 \right) \quad \text{if } x > x_2$$

$$w(x) = \frac{1}{EI} (w_0(x) + A_2 x + B_2) \quad \text{if } x_1 \leq x \leq x_2$$



Where:

$$\begin{aligned}
 p &= (q_2 - q_1)/(x_2 - x_1) \\
 w_0(x) &= R_L \frac{x^3}{6} - \frac{1}{120}(x - x_1)^4 [5q_1 + p(x - x_1)] \\
 B_1 &= 0 \\
 B_2 &= R_L \frac{x_1^3}{3} + w'_0(x_1)x_1 - w_0(x_1) \\
 B_3 &= w_0(x_2) + B_2 - \left[ -R_R \frac{x_2^3}{3} + R_R L \frac{x_2^2}{2} + w'_0(x_2)x_2 \right] \\
 A_3 &= \frac{1}{L} \left( R_R \frac{L^3}{3} - B_3 \right) \\
 A_2 &= A_3 - \left( w'_0(x_2) - R_R \frac{x_2^2}{2} + R_R L x_2 \right) \\
 A_1 &= w'_0(x_1) + A_2 + R_L \frac{x_1^2}{2}
 \end{aligned}$$

#### 4.4. Precambering

In the previous formulas for the calculation of the deflection at each design cross-section, the second moment of area will be calculated for the composite stage considering the following assumptions:

When a preambering has been defined by the user, the following deflection is added in each cross-section:

$$w_{0,x} = -4 w_0 \left( 1 - \frac{x}{L} \right) \frac{x}{L} \quad (15)$$

Where:  $w_0$  is the preambering.

The preambering deflection is not considered when assessing the deflections used for the natural frequency.

#### 4.5. Influence of the connectors slip

The influence of the connector slip on the beam deflection will be treated according to the simplified version provided by the clause 5.2.2 (6) of ENV 1994. For beams with partial connection (plastic design only – see § 2.2), all deflections are increased by the following factor:

$$\begin{aligned}
 &\text{for propped beams during construction : } k_{pc} = 1 + 0,5(1 - \eta) \left[ \frac{\delta_a}{\delta_c} - 1 \right] \\
 &\text{for unpropped beams during construction : } k_{pc} = 1 + 0,3(1 - \eta) \left[ \frac{\delta_a}{\delta_c} - 1 \right]
 \end{aligned} \quad (16)$$

where:  $\eta$  is the degree of connection of the beam (see § 6.4.5)

$\delta_a$  is the maximum deflection of the beam considering only the steel beam flexural stiffness

$\delta_c$  is the maximum deflection of the beam considering the composite action with a full connection.

Note: The specific formulas given by the French DAN of ENV 1994 are not considered in this project.

## 5. VERIFICATIONS AT THE CONSTRUCTION STAGE

### 5.1. General

Verifications at the construction stage are carried out only when the beam is unproped.

Only the ULS verifications for the steel beam (without slab) will be carried out. It includes:

- Bending resistance of sections
- Shear resistance of sections
- Resistance of sections to M-V interaction
- Shear buckling resistance
- Shear buckling resistance – M-V interaction
- Lateral torsional buckling (LTB) resistance

All verification criteria (except for LTB resistance) are calculated at each design point along the beam. In the calculation report, the maximum value of each criterion will be displayed.

### 5.2. ULS verifications

#### 5.2.1. Classification of the cross-section

The classification of the cross-section is carried out according to the Table 5.2 of EN 1993-1-1.

The class of the cross-section is the highest class of the uniformly compressed flange and the web in bending:

- For the compressed flange:
  - Class 1, if  $0,5(b - t_w - 2r_1)/t_f \leq 9\varepsilon$
  - Class 2, if  $0,5(b - t_w - 2r_1)/t_f \leq 10\varepsilon$
  - Class 3, if  $0,5(b - t_w - 2r_1)/t_f \leq 14\varepsilon$

Where:  $\varepsilon = \sqrt{235/f_y}$ ;  $f_y$  is expressed in N/mm<sup>2</sup>

- For the web in bending:
  - Class 1, if  $(h_t - 2t_f - 2r_1)/t_w \leq 72\varepsilon$
  - Class 2, if  $(h_t - 2t_f - 2r_1)/t_w \leq 83\varepsilon$
  - Class 3, if  $(h_t - 2t_f - 2r_1)/t_w \leq 124\varepsilon$

**Error code (see Annex D):**

*If the cross-section is of class 4, other calculations will not be performed and an error message will be sent back to the UI (error code = 11).*

### 5.2.2. Bending resistance of sections

The criterion for the bending resistance of sections is calculated according to EN 1993-1-1 §6.2.5:

$$\Gamma_M = M_{Ed} / M_{c,Rd} \leq 1 \quad (17)$$

where:  $M_{Ed}$  is the maximum design bending moment along the beam

$M_{c,Rd}$  is the design bending resistance given by:

- For sections of class 1 or 2:

$$M_{c,Rd} = W_{pl,y} f_y / \gamma_{M0} \quad (18)$$

- For sections of class 3:

$$M_{c,Rd} = W_{el,y} f_y / \gamma_{M0} \quad (19)$$

$W_{pl,y}$ ,  $W_{el,y}$ : see Annex A

If the elastic design is imposed by the user (see §2.2), the criterion for bending (17) is replaced by the following one, whatever the class of the cross-section is:

$$\Gamma_M = \sigma_{Ed,max} / (f_y / \gamma_{M0}) \leq 1 \quad (20)$$

where:  $\sigma_{Ed,max} = \frac{M_{Ed}}{W_{el,y}}$

### 5.2.3. Shear resistance of sections

The criterion for the shear resistance of sections is calculated according to EN 1993-1-1 §6.2.6:

$$\Gamma_V = V_{Ed} / V_{c,Rd} \leq 1 \quad (21)$$

where:  $V_{Ed}$  is the maximum design shear force along the beam

$V_{c,Rd}$  is the design shear resistance:

$$V_{c,Rd} = \frac{A_v f_{yk}}{\sqrt{3} \gamma_{M0}} \quad (22)$$

$A_v$  is the shear area of the cross-section (see Annex A):

For hot rolled profile sections (i.e. defined from the profile database), the shear area is obtained by  $A_v = A_{v,z}$  (see Annex A).

For custom profiles, the shear area is obtained by the formula:

$$A_v = \eta (h_t - 2 t_f) t_w \quad (23)$$

$h_t$ ,  $t_f$ ,  $t_w$ : see § 2.4.1

$\eta$  is a factor for shear resistance as defined in EN 1993-1-5 [17] § 5.1:

	$\eta$		$\eta$
Recommended values of Eurocodes	1,20	Luxemburg	1,20
Austria	1,20	Netherlands	1,20
Belgium	1,20	Poland	1,20
Bulgaria	1,20	Portugal	1,20
Czech Republic	1,20	Romania	1,20
Estonia	1,20	Singapore	1,00
France	1,20	Slovakia	1,20
Germany	1,20	Slovenia	1,20
Hungary	1,20	Spain	1,20
Italy	1,20	Switzerland	1,20
Latvia	1,20	United Kingdom	1,00
Lithuania	1,20	Non-members of CEN <sup>(1)</sup>	1,20

Table 11 : Value of the factor  $\eta$

- Parameters in italic and red indicate different values to the recommended values.
- (\*\*) Countries: Turkey, Qatar, Saudi-Arabia, UAE, South Africa.

If the elastic design is imposed by the user (see §2.2), the criterion for the shear resistance (21) is replaced by the stress criterion:

$$\Gamma_V = \tau_{Ed} / \tau_y \leq 1 \quad (24)$$

where:  $\tau_{Ed}$  is the maximum shear stress in the web, given by:  $\tau_{Ed} = V_{Ed} / A_v$   
 $\tau_y$  is the elastic design shear resistance:

$$\tau_y = \frac{f_{yk}}{\sqrt{3} \gamma_{M0}} \quad (25)$$

$A_v$  is the shear area of the cross-section, calculated as follows, for both hot rolled profiles and custom sections:

$$A_v = (h_t - 2 t_f) t_w \quad (26)$$

$h_t, t_f, t_w$ : see § 2.4.1

#### 5.2.4. Bending moment and Shear force Interaction

When the elastic calculation is not directly imposed by the user, the influence of the plastic moment resistance on the shear force is considered if the latter is greater than a half of the plastic shear resistance, i.e.  $V_{Ed} / V_{pl,Rd} \geq 0,5$ , whatever the class of the cross-section is.

In this case, the reduced moment resistance is calculated by:

$$M_{V,Rd} = \left( W_{pl,y} - \frac{\rho A_w^2}{4 t_w} \right) \frac{f_y}{\gamma_{M0}} \leq M_{c,Rd} \quad (27)$$

$$A_w = (h_t - 2 t_f) t_w \quad (28)$$

$$\rho = \left( \frac{2 V_{Ed}}{V_{pl,Rd}} - 1 \right)^2 \quad (29)$$

The interaction criterion is:

$$\Gamma_{MV} = M_{Ed} / M_{V,Rd} \leq 1 \quad (30)$$

$\Gamma_{MV} = \Gamma_M$  when  $V_{Ed} / V_{pl,Rd} < 0,5$ .

If the elastic design is imposed by the user (see §2.2), the criterion for the MV interaction (30) is replaced by the following equivalent stress criterion:

$$\Gamma_{MV} = \frac{\text{Max}\{\sigma_{eq}; \sigma_{Ed,max}\}}{(f_y / \gamma_{M0})} \leq 1,0 \quad (31)$$

where:  $\sigma_{Ed,max}$ : see § 5.2.2

$\sigma_{eq}$  is the equivalent Von Mises stresses at the top (or bottom) of the web , obtained by:

$$\sigma_{eq} = \sqrt{\sigma_{w,Ed}^2 + 3\tau_{Ed}^2}$$

$\tau_{Ed}$ : see § 5.2.3

$\sigma_{w,Ed}$  is the normal stress in the web obtained by:

$$\sigma_{w,Ed} = \frac{M_{Ed}}{I_y} \left( \frac{h_t}{2} - t_f \right)$$

$I_y$ : see Annex A

### 5.2.5. Web resistance to shear buckling

Only unstiffened webs are considered in this document.

The web resistance to shear buckling must be calculated according to EN 1993-1-5 §5 when then ratio  $h_w/t_w$  exceeds  $72 \varepsilon/\eta$  , where  $h_w = h_t - 2 t_f$ .

The verification criterion is:

$$\Gamma_{Vb} = V_{Ed} / V_{b,Rd} \leq 1 \quad (32)$$

The contribution of the flange to the shear buckling resistance is neglected. The shear buckling resistance is then calculated by:

$$V_{b,Rd} = \frac{\chi_w h_w t_w f_{yk}}{\sqrt{3} \gamma_{M1}} \quad (33)$$

$\chi_w$  is the reduction factor, given by:

$$\begin{aligned} \text{if } \bar{\lambda}_w < \frac{0,83}{\eta}: \quad \chi_w &= \eta \\ \text{if } \bar{\lambda}_w \geq \frac{0,83}{\eta}: \quad \chi_w &= \frac{0,83}{\bar{\lambda}_w} \end{aligned} \quad (34)$$

$$\text{where } \bar{\lambda}_w = \frac{h_w}{86,4 t_w \varepsilon}$$

When the web is compact (i.e. for  $h_w/t_w \leq 72 \varepsilon/\eta$ ),  $\Gamma_{vb} = 0$ .

It is to be noted that within the scope of the software (Class 1 and 2 cross-sections only), most of the cross-sections won't have slender webs.

### 5.2.6. Web resistance to shear buckling – Interaction M-V

When the criterion for web resistance to shear buckling, calculated in paragraph 5.2.5, is greater than 0,5, i.e.  $\Gamma_{vb} > 0,5$ , the influence of the interaction between bending moment and shear force must be taken into account according to EN 1993-1-5 §7.1.

The verification criterion is:

$$\Gamma_{MVb} = \frac{M_{Ed}}{M_{pl,Rd}} + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right) (2\Gamma_{vb} - 1)^2 \leq 1 \quad (35)$$

Where:  $M_{f,Rd}$  is the design plastic moment resistance of the section consisting of the effective area of the flanges (here gross area, as the cross-section are not in Class 4)

$M_{pl,Rd}$  is the design plastic moment resistance of the section consisting of the effective area of the flanges and the fully effective web irrespective of its section class (here the plastic resistance of the gross cross-section as the latter in in Class 1 or 2).

### 5.2.7. Resistance to Lateral Torsional Buckling

The criterion for LTB resistance is calculated according to EN 1993-1-1 §6.3.2:

$$\Gamma_{MT} = M_{Ed} / M_{b,Rd} \leq 1 \quad (36)$$

where:  $M_{Ed}$  is the maximum design bending moment along the beam

$M_{b,Rd}$  is the design LTB resistance, calculated by:

$$M_{b,Rd} = \chi_{LT} W_{pl,y} f_y / \gamma_{M1} \text{ for Class 1 and Class 2 cross-sections}$$

$$M_{b,Rd} = \chi_{LT} W_{el,y} f_y / \gamma_{M1} \text{ for Class 3 cross-sections or when the elastic design is imposed by the user (see § 2.2)}$$

$\chi_{LT}$  is the reduction factor, calculated according to the general method (EN 1993-1-1 §6.3.2.2):

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \leq 1,0$$

$$\Phi_{LT} = 0,5 \left[ 1 + \alpha_{LT} (\bar{\lambda}_{LT} - 0,2) + \bar{\lambda}_{LT}^2 \right]$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl,y} f_y}{M_{cr}}}$$

$\alpha_{LT}$  is the imperfection factor that depends on the LTB curve:

Cross-section	Limits	Reduction curve	$\alpha_{LT}$
Hot rolled (i.e. from the profile database)	$h_t/b_f \leq 2$	a	0,21
	$h_t/b_f > 2$	b	0,34
Custom profile	$h_t/b_f \leq 2$	c	0,49
	$h_t/b_f > 2$	d	0,76

 Table 12 : Imperfection factor  $\alpha_{LT}$ 

$M_{cr}$  is the elastic critical moment, calculated by:

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L^2} \left[ \sqrt{\frac{I_w}{I_z} + \frac{GI_t L^2}{\pi^2 EI_z} + (C_2 z_g)^2} - C_2 z_g \right]$$

$z_g = +h_t/2$  : loads are assumed to be applied on the top flange of the section

$C_1$  and  $C_2$  are given in Table 13.

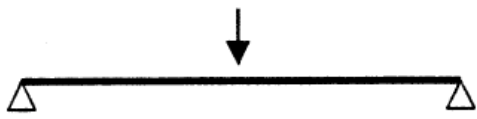

Loading	$C_1$	$C_2$
	1,35	0,59
	1,13	0,45

 Table 13 : Factors  $C_1$  and  $C_2$ 

When the French National Annex is selected, the reduction factor  $\chi_{LT}$  is calculated by the following formula:

$$\chi_{LT} = \frac{1}{f} \times \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}}$$

with:  $\chi_{LT} \leq 1$  and  $\chi_{LT} < \frac{1}{\bar{\lambda}_{LT}^2}$

where:  $f = 1 - 0,5(1 - k_c)[1 - 2(\bar{\lambda}_{LT} - 0,8)^2]$

$$k_c = \frac{1}{\sqrt{C_1}}$$

### 5.3. SLS verifications

No SLS checks at construction stage.

The software will provide the deflection under SLS combination.

## 6. VERIFICATION AT FINAL STAGE

The ULS verifications for the composite beam include:

- Bending resistance of sections
- Shear resistance of sections
- Resistance of sections to M-V interaction
- Shear buckling resistance
- Shear buckling resistance – M-V interaction

The SLS verifications include:

- Total deflection
- Deflexion under variable load cases
- Natural frequency of the beam

All verification criteria are calculated at each design point along the beam. In the calculation report, the maximum value of each criterion is displayed.

### 6.1. Effective width of the concrete slab

The effective width of the concrete slab is determined according to EN 1994-1-1 §5.4.1.2:

- For  $x \leq L/4$ :  $b_{\text{eff}}(x) = b_e[\beta + 4(1 - \beta)x/L] + b_0$
- For  $x \geq 3L/4$ :  $b_{\text{eff}}(x) = b_e[\beta + 4(1 - \beta)(L - x)/L] + b_0$
- Otherwise:  $b_{\text{eff}}(x) = b_e + b_0$

where:  $b_e = \min\{L/8; b_1/2\} + \min\{L/8; b_2/2\}$  for an intermediate beam

$b_e = \min\{L/8; b_1\} + \min\{L/8; b_2/2\}$  for an edge beam

$\beta = 0,55 + 0,025 L/b_e \leq 1,0$

$b_0 = 0$  according to EN 1994-1-1 §5.4.1.2 (9)

When slab openings are defined by the user (see § 2.3), following formulas are used:

$b_e = \min\{L/8; b_1/2; d_{\text{so1}}\} + \min\{L/8; b_2/2; d_{\text{so2}}\}$  for an intermediate beam

$b_e = \min\{L/8; b_1; d_{\text{so1}}\} + \min\{L/8; b_2/2; d_{\text{so2}}\}$  for an edge beam

### 6.2. Design resistance of the connectors

The design horizontal shear resistance of a connector is obtained as follows:

- **For solid slabs:** the design resistance  $P_{\text{Rd}}$  is directly obtained from the Table 8.
- **For slabs with decking transverse to the beam axis:**
  - **Connector longitudinal with the beam:** the design resistance  $P_{\text{Rd,t}}$  is obtained from the formulae given in Table 4 of ETA-15/0876:

$$P_{\text{Rd,t}} = k_{t,l} P_{\text{Rd}}$$

$$\text{where: } k_{t,l} = \frac{0,66}{\sqrt{n_r}} \frac{b_0}{h_p} \left( \frac{h_{sc}}{h_p} - 1 \right) \leq 1 \quad (37)$$

- **Connector transverse with the beam:** the design resistance  $P_{\text{Rd,t}}$  is directly obtained from the formulae given in Table 4 of ETA-15/0876:

$$P_{\text{Rd,t}} = 0,89 k_{t,t} P_{\text{Rd}}$$

$$\text{where: } k_{t,t} = \frac{1,18}{\sqrt{n_r}} \frac{b_0}{h_p} \left( \frac{h_{sc}}{h_p} - 1 \right) \leq 1 \quad (38)$$



- **For slabs with decking parallel to the beam axis:** the design resistance  $P_{Rd,l}$  is directly obtained from the formulae given in Table 5 of ETA-15/0876:

$$P_{Rd,l} = k_l P_{Rd}$$

$$\text{where: } k_l = 0,6 \frac{b_0}{h_p} \left( \frac{h_{sc}}{h_p} - 1 \right) \leq 1 \quad (39)$$

For formulas (37) to (39):

$$b_0 = (b_r + b_b)/2$$

$b_r, b_b, h_p$ : see § 2.6

$h_{sc}$ : see Table 8

$n_r$ : see § 2.8.1

For connectors X-HVB 80 to 140, a reduction factor is applied to the design resistance of the connector if the flange thickness is less than 8 mm:

$$P_{Rd,red} = \frac{t_f}{8} P_{Rd} \geq 23 \text{ kN} \quad (40)$$

This reduction is applied for plain slabs as covered by the European Technical Assessment report [1] but also for slabs with profiled steel sheeting when relevant (see § 2.4.1). In the latter case, which is outside the scope of [1], a notification is given in the calculation report.

For custom steel (see § 2.4.2), an additional reduction factor is applied to the resistance of the connector:

$$\alpha_{BM,red} = 0,95 \quad (41)$$

This reduction factor will be applied for each custom steel, where  $f_{yk} < 235 \text{ MPa}$ .

### 6.3. Participating depth of the concrete slab

The participating depth of the concrete slab considered in the calculation of plastic resistance is given by:

$$\begin{aligned} &\text{▪ for plain slabs :} && e_{part} = \eta h \\ &\text{▪ for slabs with steel sheeting :} && e_{part} = \eta (h - h_p) \end{aligned} \quad (42)$$

Where:  $\eta$  is the degree of connection (see § 2.8.3);  $\eta = 1$  for full connection.

See following chapters for the calculation of the degree of connection for partial connection.

### 6.4. Connection in plastic design

#### 6.4.1. Principles

For plastic design, the shear connectors are distributed equally and uniformly along the beam. The number of connectors is determined according to the option of degree connection that can be chosen between:

- Full connection
- Partial connection

The number of connectors is calculated for the shortest segment between supports and the section where the bending moment is maximum.

The number of connectors per each row of connection is denoted  $n_r$ . The final output of the module will be the number of rows of connection and the number  $n_r$  of connectors per row. The software always begins by considering  $n_r = 1$ . In some specific cases, the initial number can be switched to 2 (see 2.9). If the requirements for full or partial connection are not met, the module will try to increase the number of connectors per row, until the requirements are fulfilled. The maximum number of connectors per row is 3.

#### 6.4.2. Design strategies

The final number of connectors that is selected by the software depends upon the following parameters:

- User option for full or partial connection
- the values of the SLS and ULS criteria.

Table 14 gives the software strategies according to the value or state of these parameters.

ULS criteria for resistance	SLS criteria for deflection	User option	
		Full connection	Partial connection
Full = not OK	all cases	Results are displayed for the maximum number of connectors that can be located on the beam	
Full = OK	Full = not OK	Results are displayed considering the minimum number of connectors with full connection	Results are displayed considering the minimum number of connectors with partial connection where the ULS criteria are met
Full = OK Partial = OK	Full = OK Partial = Not OK	Results are displayed considering the minimum number of connectors with full connection	
Full = OK Partial = Not OK	Full = OK Partial = OK		
Partial = OK	Partial = OK	Results are displayed considering the minimum number of connectors with full connection	Results are displayed considering the minimum number of connectors with partial connection where both SLS and ULS criteria are met

Table 14 : Software strategies for the assessment of the number of connectors

#### 6.4.3. Number of connectors for full connection

For a critical cross-section (see § 4.2), the number of connectors is obtained when the resistance of all the connectors between the critical cross section and the closest support is equal to the minimum plastic resistance of the slab and of the profile. The process is detailed hereafter taking into account the type of slab.

The location of the  $i$ -th critical cross-section is denoted  $x_c$ .

##### a) Slab with decking transverse to the beam axis

The number of ribs between the critical cross-section and the closest support is obtained by:

$$n_{\text{ribs}} = \frac{\min\{x_c; L - x_c\}}{b_s} \quad (43)$$

where:  $b_s$ : see § 2.6

At the first trial, the degree of connection is then calculated by:

$$\eta = \frac{n_{\text{ribs}}(n_r P_{\text{Rd,t}})}{\min\{N_{\text{a,Rd}}; N_{\text{c,Rd}}\}}$$

Where:  $N_{\text{a,Rd}}$  is the design axial resistance of the steel profile, calculated by:

$$N_{\text{a,Rd}} = A f_y / \gamma_{M0}$$

$N_{\text{c,Rd}}$  is the design compression resistance of the concrete slab, calculated by:

$$N_{\text{c,Rd}} = (h - h_p) \times b_{\text{eff}}(x_c) \times 0,85 f_{\text{cd}}$$

The number of connectors, denoted  $n_f$ , is then determined according to the value of  $\eta$ :

- If  $\eta < 1$ , the full connection is not possible. The module will switch to the partial connection option
- If  $1 \leq \eta < 2$ :  $n_f = n_{\text{ribs}}$
- If  $2 \leq \eta < 3$ :  $n_f = n_{\text{ribs}}/2$  (A connector is placed at every two ribs)
- etc...

#### b) Slab with decking parallel with the beam axis

$$n_f = \frac{\min\{N_{\text{a,Rd}}; N_{\text{c,Rd}}\}}{n_r P_{\text{Rd,l}}}$$

#### c) Solid slab

$$n_f = \frac{\min\{N_{\text{a,Rd}}; N_{\text{cs,Rd}}\}}{n_r P_{\text{Rd}}}$$

Where:  $N_{\text{cs,Rd}} = h \times b_{\text{eff}}(x_c) \times 0,85 f_{\text{cd}}$

The number of connectors  $n_f$  must fulfil the requirements of connectors positioning as defined in § 2.9. If this control is negative, an error code will be sent back to the UI.

### 6.4.4. Minimum number of connectors for partial connection

#### a) Minimum degree of connection

The minimum degree of connection, denoted  $\eta_{\text{min}}$ , is calculated according to EN 1994-1-1 §6.6.1.2:

- If  $L \leq 25 \text{ m}$ :

$$\eta_{\text{min}} = 1 - (355/f_y)(0,75 - 0,03L) \geq 0,4$$

- Otherwise:

$$\eta_{\text{min}} = 1$$

#### b) Slab with decking transverse to the beam axis

At the first trial, the number of connectors is calculated by assuming the degree of connection equal to  $\eta_{\text{min}}$ :

$$n_0 = \eta_{\text{min}} \times n_f$$

The minimum number of connectors, denoted  $n_{\min}$ , is then determined as follows:

- If  $n_{\text{ribs}} < n_0$ , the partial connection is not possible. An errorcode will be provided.
- If  $n_0 \leq n_{\text{ribs}} < 2n_0$ :  $n_{\min} = n_{\text{ribs}}$  (A connector is placed at each rib)
- If  $2n_0 \leq n_{\text{ribs}} < 3n_0$ :  $n_{\min} = n_{\text{ribs}}/2$  (A connector is placed at every two ribs)
- etc...

**Error code (see Annex D):**

*If the partial connection is not possible, other calculations will not be performed and an error message will be sent back to the UI (error code = 27).*

### c) Other slabs

The minimum number of connectors is calculated by:

$$n_{\min} = \eta_{\min} n_f$$

#### 6.4.5. Determination of the number of connectors for partial connection

**Step 1:** ULS and SLS verifications with the full connection. If the resistance and deformation criteria are not checked, they will neither be with the partial connection. Otherwise, continue Step 2.

**Step 2:** ULS and SLS verifications with the partial connection. If the resistance and deformation criteria are checked, the number of connectors for partial connection is equal to:

$$n = n_{\min}$$

Otherwise, continue Step 3.

**Step 3:** Increase the number of connectors until the resistance and deformation criteria are checked.

## 6.5. Connection in elastic design

### 6.5.1. Longitudinal shear flow

Under composite stages, the longitudinal shear flow in the connection between the concrete and the steel profile (at Point 5 in Figure 25) is calculated for each type of actions:

$$v_{\text{Ed},j} = \frac{V_{\text{Ed},j} \cdot \sum E_i S_i}{EI_{y,j}}$$

where:

$$\sum E_i S_i = \frac{E_{\text{cm}}}{n_{\text{eq},j}} b_{\text{eff}}(x) (h - h_p) \left( \frac{h + h_p}{2} + h_t - z_{\text{ENA},j} \right)$$

The total longitudinal shear flow is calculated by cumulating the shear flow due to short-term actions and the one due to long-term actions:

$$v_{\text{Ed}} = v_{\text{Ed},l} + v_{\text{Ed},s}$$

### 6.5.2. Number of connectors

*Even in elastic design, the connectors are assumed to be uniformly distributed along the beam.* The number of connectors along the beam is thus defined conservatively considering the highest longitudinal shear flow.

The minimum number of connectors by length unit along the beam is thus given by:

$$n_{f,min} = \frac{v_{Ed,max}}{P_{Rd}}$$

According to the requirements about the spacing of connector (see Page 3 of [1]), the minimum number of connectors must fulfil the following condition:

$$\min\{4h ; 600 \text{ mm}\} \geq \frac{1}{n_{f,min}} \text{ and } \frac{1}{n_{f,min}} \geq 100 \text{ mm}$$

where:  $P_{Rd}$  is the design shear resistance of the connector (see § 6.2);

$v_{Ed,max}$  is the is the maximum longitudinal shear flow along the beam.

**Error code (see Annex D):**

*If the second condition is not fulfilled, the connection between the concrete and the steel profile is not possible. An error message will be sent back to the UI (error code = 29).*

In order to fulfil the first condition,  $n_{f,min}$  may be chosen as:

$$n_{f,min} = \max\left\{\frac{v_{Ed,max}}{P_{Rd}} ; \frac{1}{\min\{4h ; 600 \text{ mm}\}}\right\}$$

**Slab with decking transverse to the beam axis**

The number of ribs by length unit is calculated by:

$$n_{ribs} = \frac{1}{b_s} \quad (44)$$

where:  $b_s$  is the spacing of two adjacent ribs

The required geometrical condition is defined by:

$$\eta_{req} = \frac{n_{ribs}}{n_{f,min}}$$

This value must fulfil the following condition:

$$\eta_{req} \geq 1$$

**Error code (see Annex D):**

*If the condition is not fulfilled, the connection between the concrete and the steel profile is not possible. An error message will be sent back to the UI (error code = 29).*

The number of connectors, denoted  $n_f$ , is then determined according to the value of  $\eta_{req}$ :

- If  $1 \leq \eta_{req} < 2$ :  $n_f = n_{ribs}$  (A connector is placed at each rib)
- If  $2 \leq \eta_{req} < 3$ :  $n_f = n_{ribs}/2$  (A connector is placed at every two ribs)
- etc...

**Slab with decking parallel with the beam axis or Solid slab**

The number of connectors to be chosen is the first integer greater than  $n_{f,min} \times L$ .

## 6.6. ULS verification principles

### 6.6.1. Classification of the cross-section

The classification of the cross-section is carried out according to the Table 5.2 of EN 1993-1-1.

The class of the cross-section is the highest class of the uniformly compressed flange and the web in bending:

- For the flange: it has to be classified only when the PNA is located in the fillets or web, i.e.  $N_{pl,f,Rd} > N_c$ 
  - Class 1, if  $0,5(b - t_w - 2r_1)/t_f \leq 9\varepsilon$
  - Class 2, if  $0,5(b - t_w - 2r_1)/t_f \leq 10\varepsilon$
  - Class 3, if  $0,5(b - t_w - 2r_1)/t_f \leq 14\varepsilon$
- For the web: it has to be classified only when the PNA is located in the web, i.e.  $N_{pl,w,Rd} \geq N_c$ 
  - Class 1, if  $(h_t - 2t_f - 2r_1)/t_w \leq 396\varepsilon/(13\alpha - 1)$
  - Class 2, if  $(h_t - 2t_f - 2r_1)/t_w \leq 456\varepsilon/(13\alpha - 1)$
  - Class 3:
    - if  $(h_t - 2t_f - 2r_1)/t_w \leq 42\varepsilon/(0,6 + 0,33\psi)$  when  $\psi > -1$
    - if  $(h_t - 2t_f - 2r_1)/t_w \leq 62\varepsilon(1 - \psi)\sqrt{-\psi}$  when  $\psi \leq -1$

Where:  $\alpha = \frac{h_t - t_f - r_1 - z_{PNA,a}}{h_t - 2t_f - 2r_1}$ . It should be noted that:  $\alpha > 0,5$ .

$z_{PNA,a}$  is the distance between the plastic neutral axis and the bottom fiber of the lower flange;

$\psi$  is the ratio between the elastic stress at the bottom of the web and the elastic stress at the top of the web, this latter being in compression.

**Error code (see Annex D):**

*If the cross-section is of class 4, other calculations will not be performed and an error message will be sent back to the UI (error code = 11).*

## 6.6.2. Calculation of elastic stresses

When an elastic design is performed (see § 2.2), the elastic stresses are calculated for the following points (see Figure 25).

- Point 1 located on the top of the concrete;
- Point 2 located on the lower face of the upper flange;
- Point 3 located on upper face of the bottom flange;
- Point 4 located on the bottom of the steel profile;
- Point 5 located on the interface between the profile and the slab.

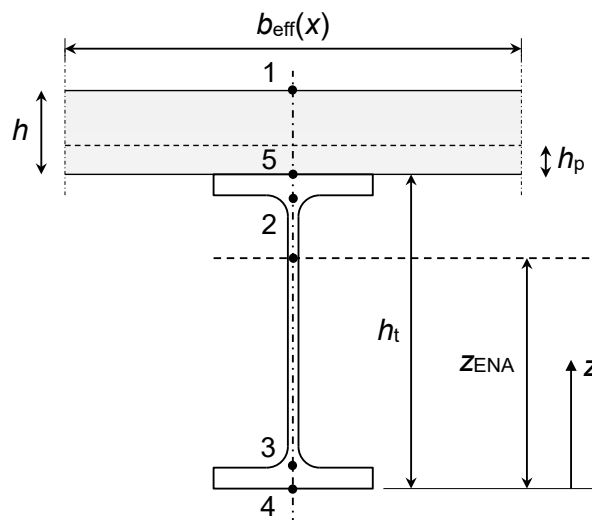


Figure 25: Points for the calculation of the elastic stresses

The elastic normal stresses under bending moments are calculated independently for each load case, according to the following principles:

- under self-weight:
  - if the beam is propped during construction, the normal stresses are obtained considering a composite stage with the long-term modular ratio;
  - if the beam is not propped, the normal stresses are calculated considering the steel profile only, as described in § 5.2.2.
- under other permanent loads: the normal stresses are obtained considering a composite stage with the long-term modular ratio;
- under live loads: the normal stresses are obtained considering a composite stage with the short-term modular ratio.

The modular ratios for the composite stages are calculated as follows:

- For long-term actions:  $n_{eq,l} = (1 + \psi_L \phi_t) E_a / E_{cm} \approx 3 E_a / E_{cm}$  (see clause 5.4.2.2 (2) of EN 1994-1-1)
- For short-term actions:  $n_{eq,s} = E / E_{cm}$

where:  $E$ : Young's modulus for steel (see § 2.4.2)

$E_{cm}$ : mean secant modulus of elasticity for concrete (see § 2.5)

### Calculations of elastic stresses under composite stages

All sections along the beam length are under the positive bending moment (the top of the concrete slab is always in compression). At each section, the following bending moments are calculated:

- Part of the bending moment under ULS combination of loads due to long-term actions in composite stage:  $M_{c,l,Ed}$
- Part of the bending moment under ULS combination of loads due to short-term actions in composite stage:  $M_{c,s,Ed}$

#### Concrete:

The normal stress in the concrete under the bending moment  $M_{c,j,Ed}$  is calculated by:

- If  $z_{ENA,j} \geq h_t + h_p$ , the elastic neutral axis (ENA) lies within the concrete:

$$\sigma_{c,j}(z) = \frac{1}{n_{eq,j}} \times \frac{M_{c,j,Ed}}{I_{y,j}} \times (z - z_{ENA,j}) \quad \text{for} \quad z_{ENA,j} \leq z$$

$$\sigma_{c,j}(z) = 0 \quad \text{for} \quad z_{ENA,j} > z \geq h_t + h_p$$

- If  $z_{ENA,j} < h_t + h_p$ , ENA lies within the steel sheeting or steel profile:

$$\sigma_{c,j}(z) = \frac{1}{n_{eq,j}} \times \frac{M_{c,j,Ed}}{I_{y,j}} \times (z - z_{ENA,j}) \quad \text{for} \quad z \geq h_t + h_p$$

The total normal compression stress is calculated by cumulating the stresses due to long-term actions and to short-term actions:

$$\sigma_c = \sigma_{c,l} + \sigma_{c,s}$$

The normal compression stress in the concrete slab is maximum for Point 1 ( $z = h + h_t$  – see Figure 25).

$z_{ENA,j}$  and  $I_{y,j}$ : see Annex B

#### Steel profile:

The normal stress in the steel profile under the bending moment  $M_{c,j,Ed}$  at the final stage is calculated by:

$$\sigma_{a,c,j}(z) = \frac{M_{c,j,Ed}}{I_{y,j}} \times (z_{ENA,j} - z) \quad \text{for} \quad h_t \geq z$$

### **Calculation of the elastic normal stresses under self-weight:**

If the beam is propped during construction, the self-weight acts on the beam under the composite stage and previous formulas are applied.

If the beam is unpropped during the construction stage, the bending moment  $M_{a,Ed}$  is resisted only by the steel profile and the normal stresses in the steel profile are calculated by:

$$\sigma_{a,a}(z) = \frac{M_{a,Ed}}{I_{y,a}} \times \left( \frac{h_t}{2} - z \right) \quad \text{for} \quad h_t \geq z$$

where :  $I_{y,a}$  is the second moment of area about the strong axis of the steel profile (see Annex A);

$M_{a,Ed}$  is the total bending moment due to short-term and long-term actions during the construction stage.

According to EN 1994-1-1 § 6.2.1.5 (3), the total stress is defined as the sum of the stress due to actions at the construction stage and the stress due to actions at the final stage:

$$\sigma_a(z) = \sigma_{a,a}(z) + \sigma_{a,c,l}(z) + \sigma_{a,c,s}(z)$$

The normal stress in the steel profile is maximum for Point 4 ( $z = 0$ ).

## **6.7. ULS verification for plastic design**

The same verifications for the section resistance as presented in paragraph 5.2 are carried out:

- Bending resistance of sections: the plastic moment resistance is calculated according to Annex C by taking the reduction factor  $\rho = 0$ ;
- Shear resistance of sections
- Resistance of sections – M-V interaction
- Web resistance to the shear buckling
- Web resistance to the shear buckling – M-V interaction

## **6.8. ULS verifications for elastic design**

### **6.8.1. Resistance to bending moment**

For Class 3 cross-sections or when the elastic design is imposed (see § 2.2), the two following criteria are checked:

#### **Normal compression stresses in the concrete:**

$$\Gamma_{M,c} = \frac{\sigma_1}{f_{cd}} \leq 1,0$$

where:  $f_{cd}$  is the design compression strength of the concrete:

$$f_{cd} = 0,85 \frac{f_{ck}}{\gamma_c}$$

#### **Normal tension stress in the steel profile:**

$$\Gamma_{M,a} = \frac{\sigma_4}{f_{yd}} \leq 1,0$$

where:  $f_{yd}$  is the design yield strength of the steel profile:



$$f_{yd} = \frac{f_y}{\gamma_{M0}}$$

### 6.8.2. Resistance to shear force

The same criterion as for construction stage is applied (see § 5.2.3). The contribution of the slab is ignored.

### 6.8.3. Resistance to MV interaction

For Class 3 cross-section, the following criterion is checked (clause 7.1 (1) of EN 1993-1-5 according to clause 6.6.2.4 (3) of EN 1994-1-1):

$$\begin{aligned} \Gamma_{MV} &= 0 & \text{if } M_{Ed} \leq M_{f,Rd} \\ \Gamma_{MV} &= \frac{M_{Ed}}{M_{pl,Rd}} + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right) \left(2 \frac{V_{Ed}}{V_{Rd}} - 1\right)^2 \leq 1 & \text{if } M_{Ed} > M_{f,Rd} \end{aligned}$$

where:  $M_{f,Rd}$  is the plastic bending resistance of the cross-section constituted only by the effective flanges and the slab;

$M_{pl,Rd}$  is the plastic bending resistance of the composite cross-section constituted with the effective flange and the gross section of the web, whatever its class.

When the elastic design is imposed by the user (see § 2.2), the following criterion is checked, whatever the class of the cross-section is:

$$\Gamma_{MV} = \frac{\sigma_{eq,max}}{f_{yd}} \leq 1,0$$

Where  $\sigma_{eq,max}$  is the maximum equivalent Von-Mises stresses, calculated at Point 3 for  $z = (h_t - h_w)/2$

### 6.8.4. Resistance to shear buckling

The same criterion as for the construction is applied. The contribution of the slab is ignored.

### 6.8.5. Resistance to the interaction of the shear buckling to the bending moment

Following criterion is checked when the shear force is greater than half the resistance to shear buckling:

$$\begin{aligned} \Gamma_{MV} &= 0 & \text{if } M_{Ed} \leq M_{f,Rd} \\ \Gamma_{MV} &= \frac{M_{Ed}}{M_{pl,Rd}} + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right) \left(2 \frac{V_{Ed}}{V_{b,Rd}} - 1\right)^2 \leq 1 & \text{if } M_{Ed} > M_{f,Rd} \end{aligned}$$

## 6.9. SLS verifications

### 6.9.1. General

According to EN 1994-1-1 §7.3.1 (2), the elastic design can be used for the SLS verifications that include:

- Deflections of the composite beam: the same verifications are carried out as presented in paragraph 5.3;
- Natural frequency of the composite beam

The second moment of area of the composite section (instead of the one of the steel section) is used in these calculations.

For unpropped beams, the deflection under the dead loads (in permanent load case) is obtained considering the steel part only (without composite effect). If the user has defined additional loads in the permanent load case, the effect on deflection is calculated considering the composite effect with the long term modular ratio.

For fully propped beams, the deflection under permanent load case is obtained considering the composite effect with the long term modular ratio.

For the composite stage live loads, the deflections are calculated considering the composite effect with the short term modular ratio.

For the calculation of the natural frequency, all deflections are calculated considering the composite effect with the short term modular ratio.

The modular ratio to be used for the calculations of composite stage deflections is given in § 6.6.2.

### 6.9.2. Position of the Elastic Neutral Axis (ENA)

The elastic properties of the composite cross-section used for the calculation of deflections are given in Annex B.

### 6.9.3. Calculation of the natural frequency

The eigen frequency of the composite beam (in Hz), is assessed by the Rayleigh method, expressed by the following general formula:

$$f = \frac{1}{2\pi} \sqrt{g \frac{\sum P_i |w_i|}{\sum P_i w_i^2}} \quad (45)$$

where:  $P_i$  is the applied load at design point no  $i$ , obtained by:

$$P_i = P_{Gi} + p_q P_{Qij}$$

$P_{Gi}$  is the applied load at design point no  $i$  for permanent load case G

$P_{Qij}$  is the applied load at design point no  $i$  for permanent load case  $Q_j$

$p_q$ : see § 3.2

$w_i$  is the deflection of the beam at design point no  $i$  under the applied loads  $P_i$ .

For beams with partial connection, the deflections  $w_i$  includes the effect of connectors slip, according to 4.5. Finally, the following criterion is checked:

$$f \geq f_{lim}$$

Where  $f_{lim}$  is the minimum frequency as defined by the User.

## 6.10. Longitudinal shear resistance

### 6.10.1. Introduction

The calculation module will assess the transverse reinforcement required to carry the longitudinal from the concrete to the connectors.

Following assumptions will be considered:

- For beams, with plain slabs, two layers of reinforcement are assumed. The connectors go through the bottom layer but not through the top one. Calculations are performed for 2 shear areas (a-a and b-b). See *Figure 26 a*).
- For slabs with profiled steel sheeting, either longitudinal or perpendicular, only one layer is assumed (*Figure 26 b*) to *d*). Calculations are performed for one shear area (a-a).
- For perpendicular profiled sheeting, the contribution of the sheeting to the longitudinal shear resistance is taken into account when the sheeting is continuous over the beam flange (See *Figure 26 c*).

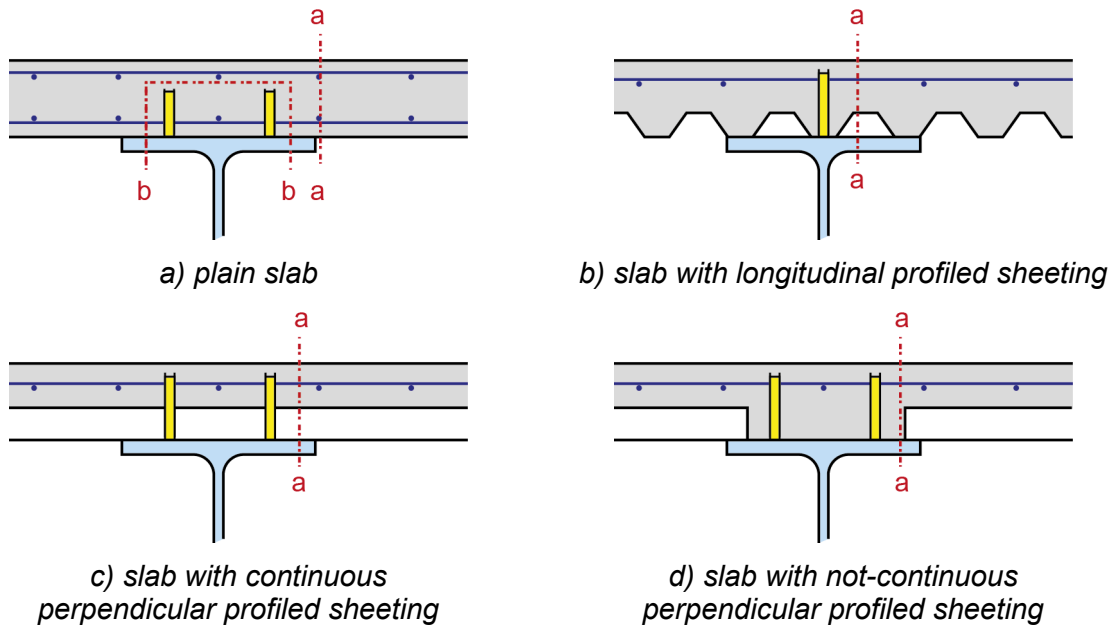


Figure 26: Transverse reinforcement configurations and shear areas

#### 6.10.2. Minimum transverse reinforcement ratio

The minimum transverse reinforcement ratio is obtained according to EN 1992-1-1 §9.2.2 (5):

$$\rho_{w,min} = \frac{0,08 \sqrt{f_{ck}}}{f_{yrk}} \quad (46)$$

where:  $f_{ck}$ : see § 2.5

$f_{yrk}$ : see § 0

### 6.10.3. Transverse reinforcement in plastic design

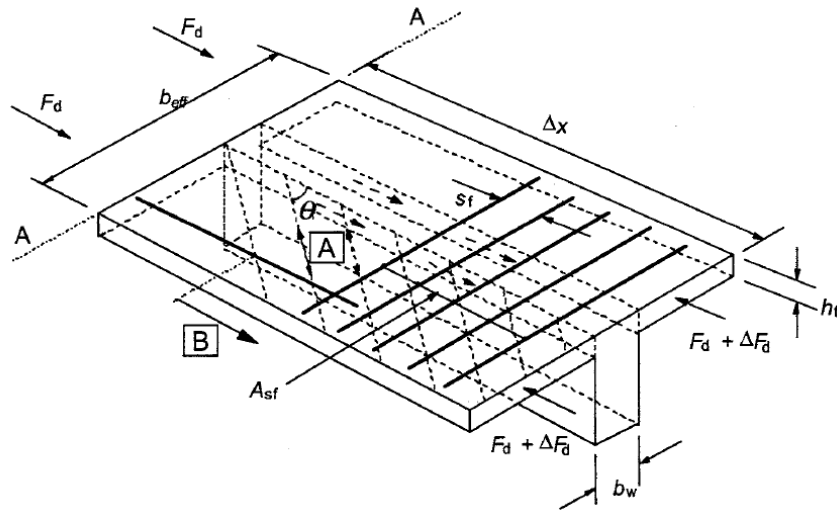


Figure 27 : Transverse reinforcement.

The calculation is carried out for a segment between each critical section (see § 4.2) and the closest support. For plain slabs, it is performed for the 2 shear areas, where the most unfavourable results are kept.

Following steps are applied:

#### Calculation of the longitudinal shear to be transferred by the reinforcement through the shear area

The longitudinal shear  $\Delta F_d$  between the critical section and the closest support that should be transferred by the reinforcement through the shear area is given by:

$$\text{For a shear area a-a: } \Delta F_d = N_{Rd,slab} \frac{\max\{b_{eff,left}(x_c); b_{eff,right}(x_c)\}}{b_{eff}(x_c)} \quad (47)$$

$$\text{For a shear area b-b: } \Delta F_d = N_{Rd,slab} \quad (48)$$

where:  $x_c$  is the location of the critical section;

$N_{Rd,slab}$  is the plastic resistance in compression of the slab at the critical section, given by:

$$N_{Rd,slab} = 0.85 \frac{b_{eff}(x_c) e_{part}(x_c) f_{ck}}{\gamma_c} \quad (49)$$

$b_{eff}(x_c)$  is the effective width of concrete slab at the critical section (see § 0);

$b_{eff,left}(x_c)$  and  $b_{eff,right}(x_c)$  are the part of this effective width on the LHS and RHS respectively, with:

$$b_{eff}(x_c) = b_{eff,left}(x_c) + b_{eff,right}(x_c)$$

$e_{part}(x_c)$  see § 6.3

$f_{ck}$  see § 2.5

#### Check of the concrete strut under compression and calculation of the orientation of the strut

According to 6.2.4 (4) of EN 1992-1-1, the resistance of the concrete strut in compression is checked by the following formula:

$$v_{Ed} = \frac{\Delta F_d}{n_{fs} \Delta x h_f} \leq v_{Rd} = v f_{cd} \sin \theta_f \cos \theta_f \quad (50)$$

where:  $\Delta x$  is the shear length.

$h_f$  is the height of the concrete slab ( $h_f = h$  for plain slabs and  $h_f = h - h_p$  for slabs with profiled steel sheeting).

$n_{fs}$  the number of faces in the shear area:  $n_{fs} = 2$  for the shear area b-b of an intermediate beam with plain slab, and  $n_{fs} = 1$  for any other case.

$f_{cd}$  see § 2.5

$v$  is a strength reduction factor for concrete cracked in shear, given by equation 6.6N of EN 1992-1-1:

$$v = 0,6 \left[ 1 - \frac{f_{ck}}{250} \right] \quad (51)$$

$\theta_f$  is the orientation of the concrete strut under compression, which is calculated by the following equation:

$$\theta_f = \frac{1}{2} \arcsin \left[ \frac{2 v_{Ed}}{v f_{cd}} \right] \text{ but } \sqrt{\frac{8}{15}} \leq \frac{2 v_{Ed}}{v f_{cd}} \leq 1 \quad (52)$$

and:

$$\arctan \left( \frac{1}{2} \right) = 26,56^\circ \leq \theta_f \leq \arctan(1) = 45^\circ \quad (53)$$

The shear length  $\Delta x$  is obtained as follows:

- for a critical section associated to a concentrated load, the shear length is the distance to the closest support,
- for a critical section associated to the maximum bending of the ULS combination, the shear length is the shortest distance between a support and the cross-section where the compression force in the slab is obtained. For partial connection, this distance is equal to the distance to the closest support. But in full connection, the shear length is lower than the distance to the relevant support (see Figure 28).

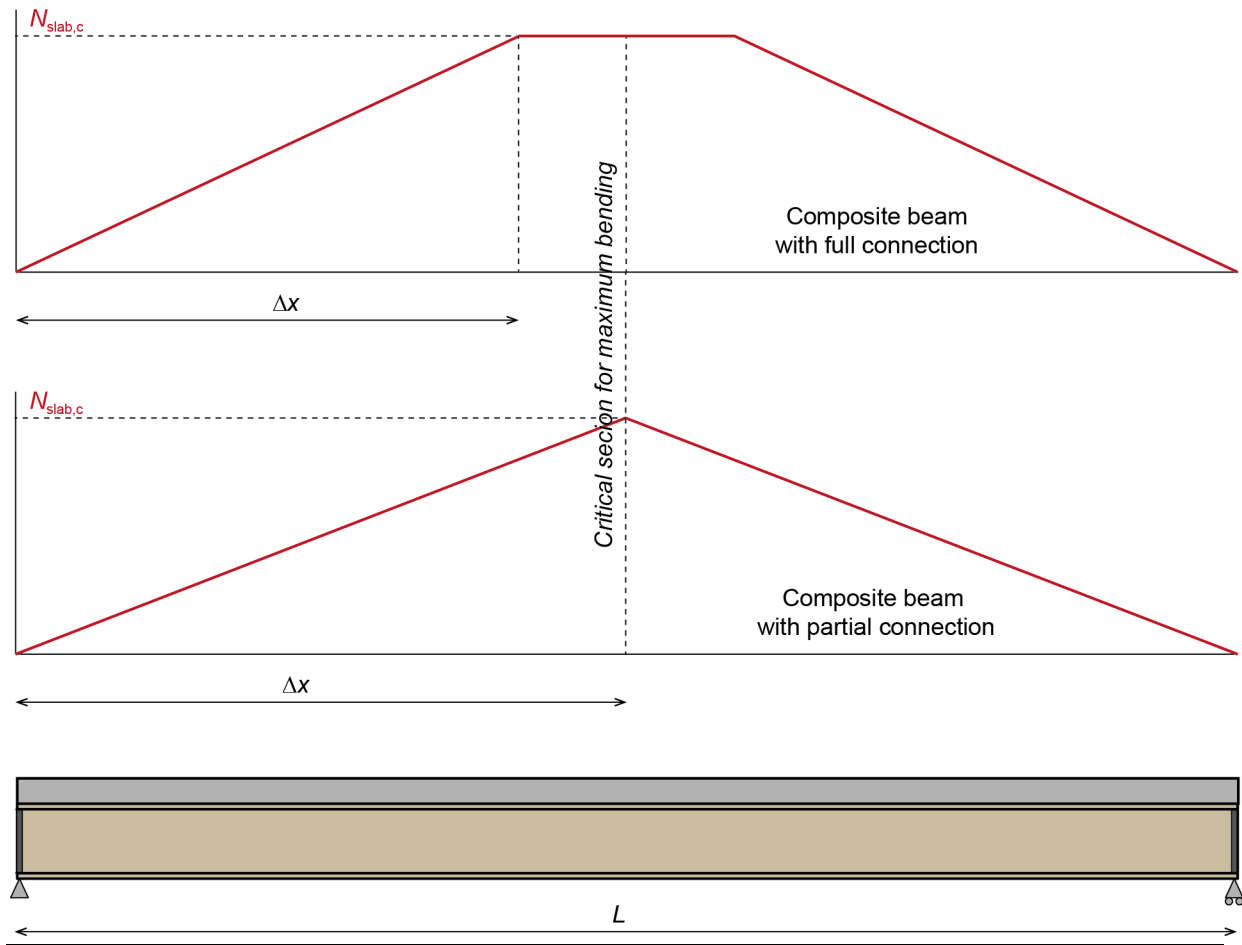


Figure 28 : Shear length

The criterion for the resistance of the concrete strut under compression is finally assessed, with the last orientation obtained with the previous process:

$$\Gamma_{\text{cstrut}} = \frac{V_{\text{Ed}}}{V_{\text{Rd}}} \leq 1 \quad (54)$$

### Assessment of the transverse reinforcement

For each shear area, the transverse reinforcement required is obtained by:

$$\left( \frac{A_{\text{sf}}}{s_{\text{f}}} \right) \geq \frac{v_{\text{Ed}} h_{\text{f}}}{f_{\text{yd}} \cot \theta_{\text{f}}} \quad (55)$$

where:  $f_{\text{yd}}$ : see § 2.7

$v_{\text{Ed}}$  and  $\theta_{\text{f}}$  are the shear stress and the orientation of the strut obtained at the previous step for the resistance of concrete.

For slabs with profiled steel sheeting, with only one layer of transverse reinforcement, this previous equation directly gives the transverse reinforcement required for the slab.

For plain slabs, the calculation performed for the shear area b-b gives directly the design of the bottom layer of reinforcement  $A_{\text{b}}/s_{\text{b}}$ . The calculation performed for the shear area a-a gives  $A_{\text{f}}/s_{\text{f}} = A_{\text{t}}/s_{\text{t}} + A_{\text{b}}/s_{\text{b}}$ .

### Influence of continuous perpendicular profiled steel sheeting

For slabs with continuous perpendicular profiled steel sheeting, the previous equation (55) is replaced by:

$$\left( \frac{A_{sf}}{s_f} \right) \geq \max \left\{ 0; \frac{v_{Ed} h_f}{f_{yd} \cot \theta_f} - A_{pe} \frac{f_{yp,d}}{f_{yd}} \right\} \quad (56)$$

where:  $f_{yp,d}$ : see § 2.6

$A_{pe}$  is the area of the profiled sheeting per length unit, calculated by:

$$A_{pe} = \frac{t_p}{b_s} \left[ b_s + b_b - b_r + \sqrt{(b_b - b_r)^2 + 4 h_p^2} \right] \quad (57)$$

$t_p$ ,  $h_p$ ,  $b_b$ ,  $b_s$ ,  $b_r$ : see § 2.6

### Control of the minimum reinforcement criterion

At the end of the process, it is checked that the transverse reinforcement obtained by calculation are greater than the minimal requirement, i.e.

For plain slabs: 
$$\left( \frac{A_b}{s_b} \right) \geq \frac{\rho_{w,min} h_f}{2} \quad \text{and} \quad \left( \frac{A_t}{s_t} \right) \geq \frac{\rho_{w,min} h_f}{2}$$

For slabs with profiled steel sheeting: 
$$\left( \frac{A_{sf}}{s_f} \right) \geq \rho_{w,min} h_f$$

#### 6.10.4. Transverse reinforcement in elastic design

When the cross-section is of Class 3 or when the elastic design is imposed by the user, the transverse reinforcement are designed according to 6.10.3, except for the calculation of the longitudinal shear stress, which is obtained as follows:

- For the shear area a-a:

$$v_{L,j,Ed} = \frac{V_{Ed,j} \cdot \sum E_i S_i}{n_{fs} \cdot E I_{y,j} \cdot h_f} \times \frac{\max\{b_{eff,left}(x_c); b_{eff,right}(x_c)\}}{b_{eff}(x_c)}$$

- For the shear area b-b:

$$v_{L,j,Ed} = \frac{V_{Ed,j} \cdot \sum E_i S_i}{n_{fs} \cdot E I_{y,j} \cdot h_f}$$

where:  $x_c$  is the location of the critical section;

$b_{eff}(x_c)$  is the effective width of concrete slab at the critical section;

$b_{eff,left}(x_c)$  and  $b_{eff,right}(x_c)$  are the part of this effective width on the LHS and RHS respectively, with:

$$b_{eff}(x_c) = b_{eff,left}(x_c) + b_{eff,right}(x_c)$$

$h_f$  is the height of the concrete slab ( $h_f = h$  for solid slabs and  $h_f = h - h_p$  for slabs with profiled steel sheeting)

$n_{fs}$  the number of faces in the shear area:  $n_{fs} = 2$  for the shear area b-b of an intermediate beam with solid slab, and  $n_{fs} = 1$  for any other case

$$\sum E_i S_i = \frac{E_{cm}}{n_{eq,j}} b_{eff}(x) \left( \frac{h + h_p}{2} + h_t - z_{ENA,j} \right)$$

The total longitudinal shear stress:

$$v_{L,Ed} = v_{L,s,Ed} + v_{L,l,Ed}$$

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## Annex A : PROPERTIES OF THE STEEL SECTION

The section properties are calculated using following analytical formulae. In these formulae:

- $r_c = r_1$  for hot rolled cross-sections;
- $r_c = 0$  for custom welded cross-sections.

$A$  Section area

$$A = 2b_f t_f + (h - 2t_f)t_w + (4 - \pi)r_c^2$$

$I_y$  Second moment of area about the strong axis

$$I_y = \frac{1}{12} \left[ b_f h^3 + (b_f - t_w)(h - 2t_f)^3 \right] + 4kr_c^4 + (4 - \pi)r_c^2 \left[ \frac{h}{2} - t_f - r_c + \frac{2r_c}{3(4 - \pi)} \right]^2$$

with:

$$k = \frac{1}{3} - \frac{\pi}{16} - \frac{1}{9(4 - \pi)}$$

$I_z$  Second moment of area about the weak axis

$$I_z = \frac{1}{12} \left[ 2t_f b_f^3 + (h - 2t_f)t_w^3 \right] + 4kr_c^4 + (4 - \pi)r_c^2 \left[ \frac{t_w}{2} + r_c - \frac{2r_c}{3(4 - \pi)} \right]^2$$

$W_{pl,y}$  Plastic modulus for bending about the strong axis

$$W_{pl,y} = \frac{t_w h^2}{4} + t_f(b_f - t_w)(h - t_f) + (4 - \pi)r_c^2 \left( \frac{h}{2} - t_f \right) + \frac{(3\pi - 10)r_c^3}{3}$$

$W_{pl,z}$  Plastic modulus for bending about the weak axis

$$W_{pl,z} = \frac{t_f b_f^2}{2} + \frac{t_w^2}{4}(h - 2t_f) + (4 - \pi)r_c^2 \left( \frac{t_w}{2} + t_f \right) - \frac{2r_c^3}{3}$$

$W_{el,y}$  Elastic modulus for bending about the strong axis

$$W_{el,y} = \frac{2I_y}{h}$$

$W_{el,z}$  Elastic modulus for bending about the weak axis

$$W_{el,z} = \frac{2I_z}{b_f}$$

$A_{v,y}$  Shear area for a shear force perpendicular to the web

$$A_{v,y} = 2b_f t_f$$

$A_{v,z}$  Shear area for a shear force parallel to the web

$$A_{v,z} = A - 2b_f t_f + t_f(t_w + 2r_c)$$

$I_t$  Torsional constant

$$\begin{aligned} I_t &= \frac{2}{3} b_f t_f^3 \left[ 1 - 0,63 \frac{t_f}{b_f} \left( 1 - \frac{t_f^4}{12b_f^4} \right) \right] + \frac{1}{3} t_w^3 (h - 2t_f) \\ &+ 2 \frac{t_w}{t_f} \left( 0,1 \frac{r_c}{t_f} + 0,15 \right) \left[ \frac{(t_f + r_c)^2 + t_w(r_c + t_w/4)}{t_f + 2r_c} \right]^4 \end{aligned}$$

$I_w$  Warping constant

$$I_w = 0,25 I_z (h - t_f)^2$$

## Annex B : ELASTIC PROPERTIES OF A COMPOSITE BEAM

Considering a load case under composite stage, with a modular ratio equal to  $n_{eq,j}$ , the location of the elastic neutral axis ( $z_{ENA,j}$ , measured from the lower fiber of the bottom flange) and the second moment of area of the composite ( $I_{y,j}$ ) cross-section are obtained as follows.

### **Composite section with steel profiled sheeting, ENA located in the steel profile:**

Location of the elastic neutral axis:

$$z_{ENA,j} = \frac{A h_t/2 + b_{eff}(h - h_p)[h_t + (h + h_p)/2]/n_{eq,j}}{A + b_{eff}(h - h_p)/n_{eq,j}}$$

Criterion to check the assumption:  $z_{ENA,j} \leq h_t + h_p$

Second moment of area:

$$I_{y,j} = I_{y,a} + A(h_t/2 - z_{ENA,j})^2 + \frac{b_{eff}(h - h_p)^3}{12n_{eq,j}} + \frac{b_{eff}(h - h_p)}{n_{eq,j}} [h_t + (h + h_p)/2 - z_{ENA,j}]^2$$

Where:  $I_{y,a}$  and  $A$  are the second moment of area and the section area of the steel profile, given in Annex A;

$j$  is the index for the type of actions:

$j = l$  for long-term actions;

$j = s$  for short-term actions;

$n_{eq,j}$  is the modular ratio, for short-term or long-terms actions.

### **Composite section with steel profiled sheeting, ENA located in the concrete slab:**

Location of the elastic neutral axis:

$$z_{ENA,j} = d_{n,j} - \sqrt{d_{n,j}^2 - \frac{n_{eq,j}A}{b_{eff}} h_t - (h + h_t)^2}$$

Where:  $d_{n,j} = n_{eq,j} A/b_{eff} + (h + h_t)$

Criterion to check the assumption:  $z_{ENA,j} > h_t + h_p$

Second moment of area:

$$\begin{aligned} I_{y,j} &= I_{y,a} + A \left( \frac{h_t}{2} - z_{ENA,j} \right)^2 + \frac{b_{eff}(h + h_t - z_{ENA,j})^3}{12n_{eq,j}} \\ &+ \frac{b_{eff}(h + h_t - z_{ENA,j})}{4n_{eq,j}} (h + h_t - z_{ENA,j})^2 \end{aligned}$$

**Composite section with a plain slab, ENA located in the steel profile:**

Location of the elastic neutral axis:

$$z_{ENA,j} = \frac{A h_t/2 + b_{eff}h(h_t + h_h + h/2)/n_{eq,j} + b_f h_h(h_t + h_h/2)/n_{eq,j}}{A + b_{eff}h/n_{eq,j} + b_f h_h/n_{eq,j}}$$

Criterion to check the assumption:  $z_{ENA,j} \leq h_t$

Second moment of area:

$$\begin{aligned} I_{y,j} &= I_{y,a} + A \left( \frac{h_t}{2} - z_{ENA,j} \right)^2 + \frac{b_{eff}h^3}{12n_{eq,j}} + \frac{b_{eff}h}{n_{eq,j}} \left( h_t + h_h + \frac{h}{2} - z_{ENA,j} \right)^2 + \frac{b_f h_h^3}{12n_{eq,j}} \\ &+ \frac{b_f h_h}{n_{eq,j}} \left( h_t + h_h/2 - z_{ENA,j} \right)^2 \end{aligned}$$

**Composite section with a plain slab, ENA located in the concrete haunch, if any:**

Location of the elastic neutral axis:

$$z_{ENA,j} = d_{n,j} - \sqrt{d_{n,j}^2 - \frac{n_{eq,j}A}{b_f} h_t - \frac{b_{eff}h}{b_f} (2h_h + 2h_t + h) - (h_h + h_t)^2}$$

Where:  $d_{n,j} = n_{eq,j}A/b_{eff} + b_{eff}h/b_f + (h_h + h_t)$

Criterion to check the assumption:  $z_{ENA,j} \leq h_t + h_h$

Second moment of area:

$$\begin{aligned} I_{y,j} &= I_{y,a} + A \left( \frac{h_t}{2} - z_{ENA,j} \right)^2 + \frac{b_{eff}h^3}{12n_{eq,j}} + \frac{b_{eff}h}{n_{eq,j}} \left( h_t + h_h + \frac{h}{2} - z_{ENA,j} \right)^2 + \frac{b_f(h_h + h_t - z_{ENA,j})^3}{12n_{eq,j}} \\ &+ \frac{b_f(h_h + h_t - z_{ENA,j})}{4n_{eq,j}} (h_h + h_t - z_{ENA,j})^2 \end{aligned}$$

**Composite section with a plain slab, ENA located in the concrete slab:**

Location of the elastic neutral axis:

$$z_{ENA,j} = d_{n,j} - \sqrt{d_{n,j}^2 - \frac{n_{eq,j}A}{b_{eff}} h_t - (h_h + h_t + h)^2} > h_t + h_h$$

Where:  $d_{n,j} = n_{eq,j}A/b_{eff} + (h_h + h_t + h)$

Criterion to check the assumption:  $z_{ENA,j} > h_t + h_h$

Second moment of area:

$$\begin{aligned} I_{y,j} &= I_{y,a} + A \left( \frac{h_t}{2} - z_{ENA,j} \right)^2 + \frac{b_{eff}(h_h + h_t + h - z_{ENA,j})^3}{12n_{eq,j}} \\ &+ \frac{b_{eff}(h_h + h_t + h - z_{ENA,j})}{4n_{eq,j}} (h_h + h_t + h - z_{ENA,j})^2 \end{aligned}$$

## Annex C : PLASTIC BENDING RESISTANCE OF THE COMPOSITE CROSS-SECTION

The bending resistance to bending of a cross section is assessed considering the participating depth of the slab (see § 6.3).

### Plastic Neutral Axis in the concrete slab

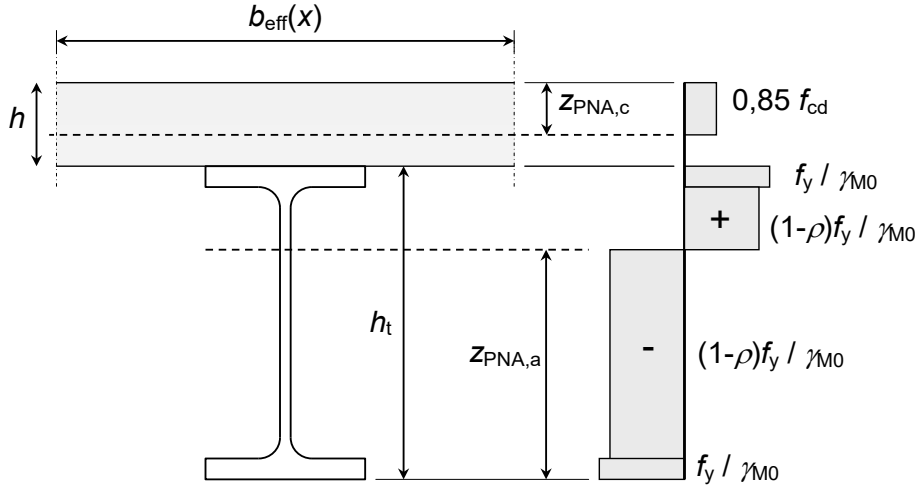


Figure 29: Plastic stresses at ultimate limit state

The position of the plastic neutral axis (PNA) in the concrete slab is calculated by (see Figure 29):

$$z_{PNA,c} = \frac{N_{pl,a,Rd}}{0,85 \times f_{cd} \times b_{eff}(x)} \leq h - h_p$$

where:  $b_{eff}(x)$  is the effective width of the concrete slab at the abscissa  $x$

$h_p = 0$  for solid slabs

$N_{pl,a,Rd}$  is the design plastic resistance to axial force of the steel section, given by:

$$N_{pl,a,Rd} = [A - \rho A_{v,z}] f_y / \gamma_{M0}$$

The bending resistance of the cross section is calculated by:

$$M_{Rd} = N_c \left( h + \frac{h_t - z_{PNA,c}}{2} \right)$$

Where:  $N_c$  is the resulting compression force in the concrete slab, determined by:

$$N_c = 0,85 \times z_{PNA,c} \times f_{cd} \times b_{eff}(x)$$

### Plastic Neutral Axis in the steel section

1) If  $N_{pl,a,Rd} \geq N_c \geq N_{pl,f,Rd}$ , PNA is located in the top flange:

$$z_{PNA,a} = h_t - \frac{N_{pl,a,Rd} - N_c}{2b f_y / \gamma_{M0}}$$

Where:  $N_{pl,f,Rd} = (A - 2bt_f)(1 - \rho) f_y / \gamma_{M0}$

The moment resistance of the section is calculated by:

$$M_{Rd} = (h_t - z_{PNA,a}) b z_{PNA,a} f_y / \gamma_{M0} + M_c$$

2) If  $N_{pl,w,Rd} \geq N_c$ , PNA is located in the web:

$$z_{PNA,a} = \frac{h_t}{2} + \frac{N_c}{2t_w(1-\rho)f_y/\gamma_{M0}}$$

Where:  $N_{pl,w,Rd} = (h_t - 2t_f - 2r_1)t_w(1-\rho)f_y/\gamma_{M0}$

The moment resistance of the section is calculated by:

$$M_{Rd} = \left[ (1-\rho)W_{pl,y} + bt_f(h_t - t_f)\rho - \frac{1}{4}(1-\rho)t_w \left( \frac{N_c}{t_w(1-\rho)f_y/\gamma_{M0}} \right)^2 \right] \frac{f_y}{\gamma_{M0}} + M_c$$

3) If  $N_{pl,f,Rd} > N_c > N_{pl,w,Rd}$ , PNA is located in the fillets. The position of the PNA and the associated moment resistance are calculated by using an iterative procedure.

## Annex D : ERROR CODES MANAGEMENT

ErrorCode	Description	Reference
1	The National Annex Selected is not valid. Please check project settings to verify. [error code: 1]	
2	The value of Loaded Categories is not correct. [error code: 2]	
3	Partial factor values should be between 1 and 2. Please modify the values of partial factors in the project settings. [error code: 3]	§ 2.1.3
4	Combination factor value should be between 0 and 1. Please modify the value of the combination factor in the "General" tab. [error code: 4]"	§ 2.1.2
5	Beam Length must be between 2 and 20 meters. Please modify the value of the beam length in the "Geometry" tab. [error code: 5]	§ 2.2
6	Left spacing and right spacing must be between 0.5 and 20 meters. Please modify the value of the beam spacings in the "Geometry" tab. [error code: 6]	§ 2.2
7	Beam Location should be Intermediate or Edge Beam and Slab Type should be Solid or with Profiled Sheeting. [error code: 7]	
8	Dimension of the steel section are not correct. Please check and modify the dimensions of the steel profile in the "Materials" tab: - $t_w \geq 3 \text{ mm}$ - $t_f > 6 \text{ mm}$ - $r_1 \geq 0$ - $r_2 \geq 0$ - $b > 2r_1 + 2r_2 + t_w$ - $h_t > 2r_1 + 2t_f$ [error code: 8]	§ 2.4.1
9	Value of Steel Strength not correct. ( $170 \leq f_y \leq 355 \text{ MPa}$ ). Please modify the value of the yield strength in the "Materials" tab. [error code: 9]"	§ 2.4.2
10	Value of Steel Grade not correct. [error code: 10]	
11	The class of section is out of scope (Class = 1 or 2 for the plastic design). Please reduce the slenderness of the profile plate or reduce the steel grade in the "Materials" tab. [error code: 11]	§ 5.2.1

ErrorCode	Description	Reference
12	The value of Slab Thickness is not correct or Concrete Type is not correct or Concrete Class is not correct. Please increase the slab thickness ( $\geq 50$ mm) in the "Slab" tab. [error code: 12]	
13	The value of concrete density should be between 1750 and 5000 kg/m <sup>3</sup> . Please modify the concrete density in the "Materials" tab. [error code: 13]"	§ 2.5
14	Haunch height is only valid for Solid slab and $hh \geq 0$ . [error code: 14]	
15	Dimensions of the steel profile sheeting is not correct. Please check and correct the dimensions of the profiled sheeting in the "Slab" tab: - $0.5 \leq t \leq 2$ mm - $100 \leq bs \leq 350$ mm - $30 \leq bb$ or $bt \leq 200$ mm - $bs + 30 \geq \min(bb, bt)$ [error code: 15]	§ 2.6
16	Sheeting orientation is not correct or Deck weight is not correct or Yield Strength is not correct. Please check and correct the weight and yield strength of the profiled sheeting in the "Slab" tab: - $0 \leq G_{deck} \leq 50$ kg/m <sup>2</sup> - $170 \leq f_{ypk} \leq 355$ Mpa [error code: 16]"	§ 2.6
17	The Value of Steel Grade for reinforcement steel bars is not correct. [error code: 17]	§ 2.7
18	Connector Orientation, or connector type or connection degree is not correct. [error code: 18]	
19	At least one load case is required. [error code: 19]	
20	Construction type, precamber or deflection limits not correct. Deflection limits should be $> 0$ . Please check the value of the precamber in the "Deformations" tab. [error code: 20]	
21	Percentage Live load should be between 0-50. [error code: 21]	§ 3.2
22	The flange thickness of the steel section is smaller than the value given in Annex B3 of ETA 15/0876. Please increase the flange thickness in the "Materials" tab. [error code: 22]	§ 2.4.1



ErrorCode	Description	Reference
23	<p>The thickness of the steel profiled sheeting is greater than the value given in Annex B4 of ETA 15/0876.</p> <p>Please reduce the thickness of the profiled sheeting in the "Slab" tab:</p> <ul style="list-style-type: none"> <li>- <math>t \leq 2.0</math> mm for X-HVB 80, X-HVB 95 and X-HVB 110</li> <li>- <math>t \leq 1.5</math> mm for XHVB 125 and X-HVB 140</li> </ul> <p>[error code: 23]</p>	
24	<p>The slab thickness is smaller than the value given in Annex B4 of ETA 15/0876 (minimum slab thickness).</p> <p>Please increase the slab thickness in the "Slab" tab or choose a smaller connector in the "Shear Connection" tab.</p> <p>[error code: 24]</p>	<p>§ 2.5</p> <p>§ 2.8.1</p>
25	<p>The decking height is greater than the value given in Annex B4 of ETA 15/0876.</p> <p>Please reduce the height of the profiled sheeting in the ""Slab"" tab or increase the size of the connector in the "Shear connection" tab.</p> <p>[error code: 25]</p>	§ 2.8.1
26	<p>The dimensions of the ribs are not satisfied according to Annexes B5-B8 of ETA 15/0876.</p> <p>Please change the dimensions of the profiled sheeting ribs in the "Slab" tab or the size of the connector in the "Shear connection" tab.</p> <p>[error code: 26]</p>	§ 2.9
27	<p>The partial connection is not possible. The user should choose other connectors, steel section or concrete slab.</p> <p>First increase the size of the connector in the "Shear connector" tab. Nevertheless it may be necessary to review the global design of the beam.</p> <p>[error code: 27]</p>	
28	<p>The presence of rib stiffeners is not compatible with the nailing of connectors.</p> <p>Change the profiled steel sheeting or the orientation of the connectors.</p> <p>[error code: 28]</p>	§ 2.9