Centre Technique Industriel de la Construction Métallique

Espace Technologique - L'Orme des Merisiers - Immeuble Apollo - 91193 Saint-Aubin
Tél : +33 (0)1 60 13 83 00 Fax : 33 (0)1 60 13 13 03

Client demandeur :	**HILTI**
Référence et date de commande :	**Framework and Appendix 1 – 22/03/2017**
File :	**DRV/16-RCM-122/002-C**
Document index :	**DRV/HVB/MT/002-C**

HVB SOFTWARE – CALCULATION MODULE

TECHNICAL MANUAL

Date :	29/05/2017	Auteurs :	P-O. MARTIN / T-M. NGUYEN	Nombre total de pages : (y compris celle-ci)	45

Révision	Date	Auteur	Objet
A	24/03/2017	T-M. NGUYEN P-O. MARTIN	First revision / Specifications of the project
B	31/03/2017	P-O. MARTIN	Update of the report according to project last decisions
C	29/05/2017	P-O. MARTIN T-M. NGUYEN	Update of the report -> Technical Manual. Associated to V1.00 Beta version. Modifications in red.
D			
E			

CONTENT

1. INTRODUCTION

The calculation module of the HVB software allows the user to perform the design of composite beams using HILTI 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 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 connectors;
- For the checks at Ultimate Limit States (ULS), only the plastic design is carried out;
- 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 HVB Calculation module are:

- France
- Spain
- Portugal
- Belgium
- Luxemburg
- Italy

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 ψ_0 and category of loaded areas

The category of loaded area is used to determine the value of the combination factor ψ_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
France	0,7	0,7	0,7	0,7	1,0	0
Spain	0,7(*)	0,7(*)	0,7(*)	0,7(*)	1,0(*)	0(*)
Portugal	0,7	0,7	0,7	0,7	1,0	0
Belgium	0,7	0,7	0,7	0,7	1,0	0
Luxemburg	0,7	0,7	0,7	0,7	1,0	0
Italy	0,7	0,7	0,7	0,7	1,0	0

Table 1 : Values of the combination factor ψ_0.

Note: (*) indicates that the recommended value is used because no national annex is available.
The value of the combination factor may also be modified by the user.

Error Code (see [27] §7):

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

- $0 \le \psi_0 \le 1$

2.1.3. Partial factors

Partial factors on actions for ULS combinations are:

γ_G for permanent actions

γ_Q for variable actions

Partial factors for design resistances are:

γ_{M0} for the section resistance of the structural steel

γ_{M1} for the element resistance of the structural steel

γ_c for the compression resistance of the concrete

γ_V for the resistance of shear connectors

γ_S for the resistance of reinforcement steel bars

γ_p for the resistance of the profiled steel

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

	γ_G	γ_Q	γ_{M0}	γ_{M1}	γ_c	γ_V	γ_s	γ_P
Recommended values of Eurocodes	1,35	1,50	1,00	1,00	1,50	1,25	1,15	1,00
France	1,35	1,50	1,00	1,00	1,50	1,25	1,15	1,00
Spain	1,35(*)	1,50(*)	*1,05*	*1,05*	1,50	1,25	1,15	1,00(*)
Portugal	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
Luxemburg	1,35	1,50	1,00	1,00	1,50	1,25	1,15	1,00
Italy	*1,30*	1,50	*1,05*	*1,05*	1,50	1,25	1,15	*1,05*

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 the values which differ from the recommended values.

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

Error Code (see [27] §7):

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

- $1 \le \gamma_i \le 2$

2.2. Geometrical description of the beam

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

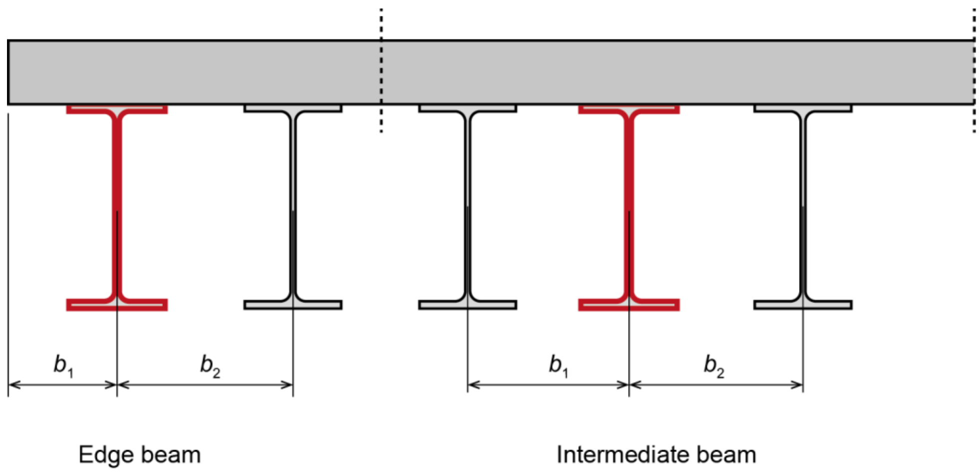


Figure 1 : Edge beam and intermediate beam

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 [27] §7):

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

- *$2m \le L \le 20m$*

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

- *$0{,}5m \le b_1 \le 20m$*
- *$0{,}5m \le b_2 \le 20m$*

For the type of the beam, the user can chose either:

- Primary beam (default)
- Secondary beam

When the option “Primary beam” is chosen, the following additional parameters must be defined:

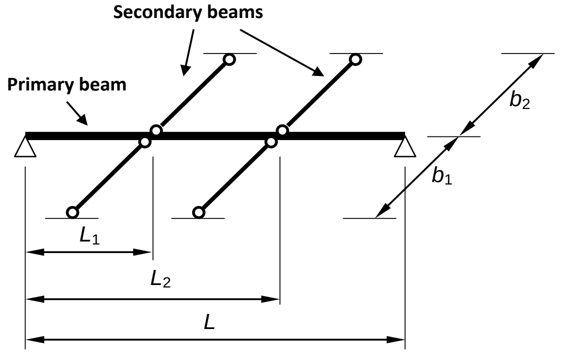
n_s is the number of secondary beams

Dimensions of the steel section of secondary beams (all secondary beams are assumed to have the same cross-section)

Secondary beams are regularly spaced along the primary beam. The location of the secondary beams is defined by the parameter L_i. This parameter is automatically calculated by the program (UI) as follows:

$$L_i = \frac{i}{n_s + 1} L \qquad (1)$$

For intermediate beams, secondary beams are located between the main girder and each adjacent beam, i.e. on each side of the primary beam. For edge beams, secondary beams are located only on the same side than the adjacent beam (see Figure 2).



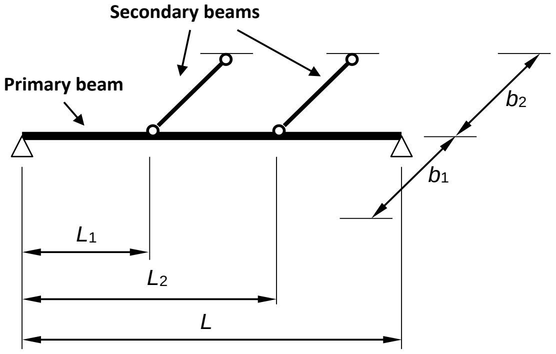


Figure 2: Primary and secondary beams

The linear mass of secondary beams, denoted m_s, is calculated by using the following formulae:

$$m_s = A_s \times \rho_s \tag{2}$$

Where:

A_s is the area of the secondary beams

ρ_s is the density of the steel, taken as: $\rho_s = 7850 \ kg/m^3$

According to 29/03/17 meeting, there will be no secondary beams defined for the geometry of the beam in the interface.

2.3. Steel section

2.3.1. Geometry

The steel profile of a beam (either primary or secondary) 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

r_2 is the toe radius

Error Code (see [27] §7):

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

- $t_w > 3mm$
- $t_f > 0$
- $r_1 \geq 0$
- $r_2 \geq 0$
- $b > 2\,r_1 + 2\,r_2 + t_w$
- $h_t > 2\,r_1 + 2\,t_f$

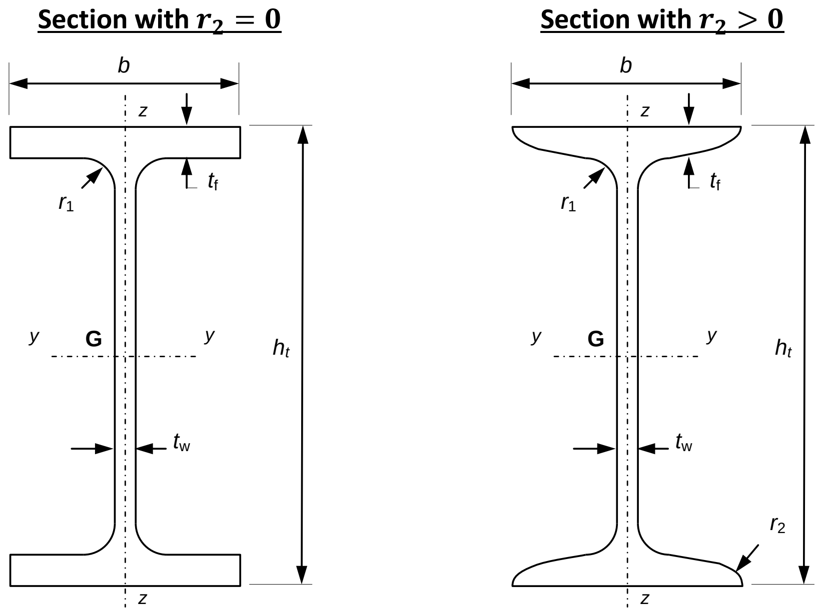


Figure 3: Steel profile

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

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

- For solid slabs, the minimum thickness of flanges is 6 mm, i.e $t_f \geq 6\ mm$. 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 $t_f \geq 8\ mm$

Error Code (see [27] §7):

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

2.3.2. 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: $170 \leq f_{yk} \leq 355$ MPa).

By default, the steel grade is S235.

Error Code (see [27] §7):

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: $E = 210000\ MPa$

G is the shear modulus: $G = 80770\ MPa$

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

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

2.4. Concrete slab

Two types of slabs are covered by the application. 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])

ρ_c is the density of the concrete (default value 2000 kg/m^3 for normal concrete and 1800 kg/m^3 for light concrete, minimal value is 1750 kg/m^3).

Error Codes (see [27] §7):

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:

$$f_{cd} = f_{ck}/\gamma_c \tag{3}$$

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:

$$E_{lcm} = E_{cm}\,\eta_E \tag{4}$$

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/mm²*
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.

2.5. 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 though 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

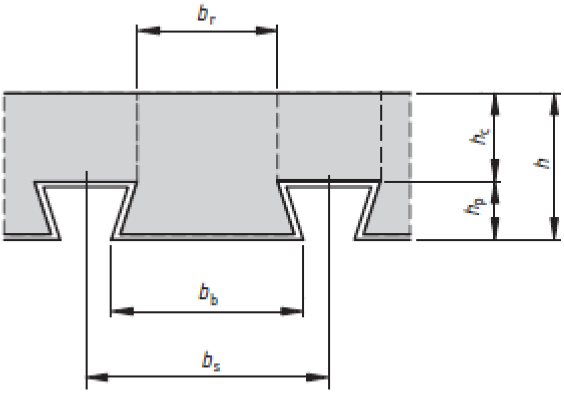


Figure 4: Dimensions of a profiled steel sheeting

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:

$$f_{ypd} = f_{ypk}/\gamma_p \tag{5}$$

Error Codes (see [27] §7):

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

- *50kg/m^2 ≥ G_{deck} ≥ 0*
- *355MPa ≥ f_{ypk} ≥ 170 MPa*

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

- *2mm ≥ t_p ≥ 0,5mm*
- *350mm ≥ b_s ≥ 100mm*
- *200mm ≥ b_t ≥ 30mm*
- *200mm ≥ b_b ≥ 30mm*

b_s + 30mm ≥ max(b_b ; b_t)

2.6. 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:

$$f_{yrd} = f_{yrk}/\gamma_s \tag{6}$$

Error Code (see [27] §7):

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

2.7. Shear connection

2.7.1. 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 chose 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	31	51
125	37,5	30	127,5	31	51
140	37,5	30	142,5	31	51

Table 8 : Properties of connectors

Error Code (see [27] §7):

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 sheetings, 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)		
	$b_0/h_p \geq 1,8$	$1< b_0/h_p < 1,8$	$b_0/h_p \leq 1,8$
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

2.7.2. Orientation of connectors

The user has to choose the orientation of the connectors among 3 possible choices:

- duckwalk (used only for solid slab with X-HVB 40 and X-HVB 50 connectors)
- longitudinal
- transverse

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

For others slabs (solid slabs or slabs with profiled steel sheeting parallel with the beam axis), connectors are always longitudinal with the beam axis.

2.7.3. Degree of connection

The user has to choose the degree of connection for the calculation. The 2 possible choices are:

- full connection;
- partial connection.

2.7.4. Spacing and positioning of connectors

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

- ***For solid slab with multiple rows of connectors:***

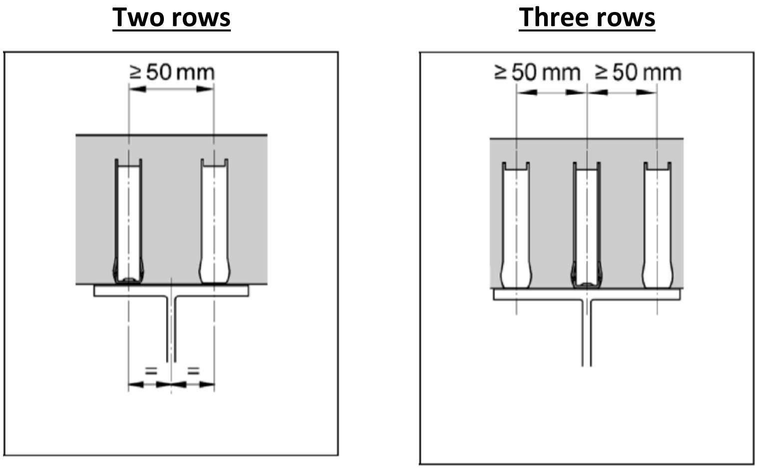


Figure 5 : Spacing of connectors for solid slabs

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.

- ***For slabs with transverse decking and connectors parallel with the beam axis (single row)***

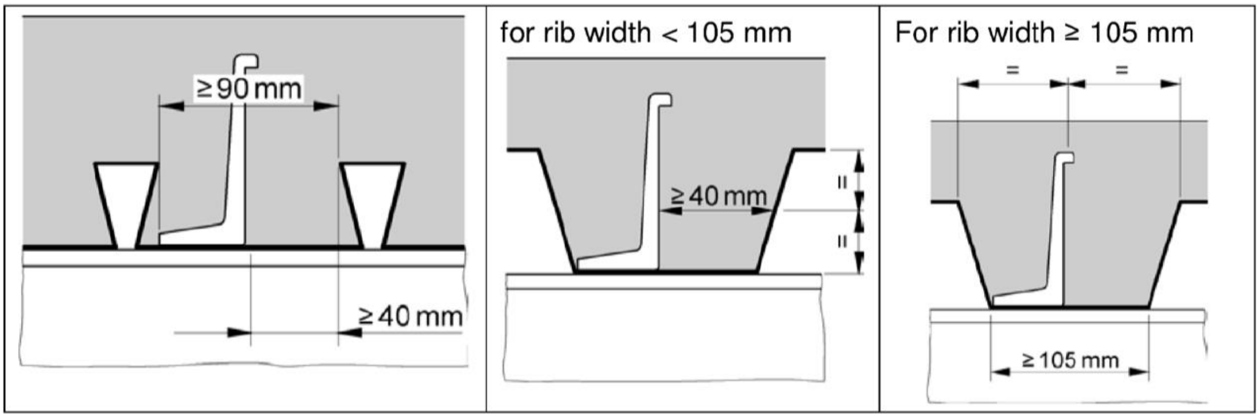


Figure 6 : Spacing of connectors for slabs with transverse decking and connectors parallel with the beam (single row).

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

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

Where: w_b is the bottom width of the connector.

Error Code (see [27] §7):
Error code 26 is returned when this condition is not met.

- ***For slabs with transverse decking and connectors parallel with the beam axis (multiple rows)***

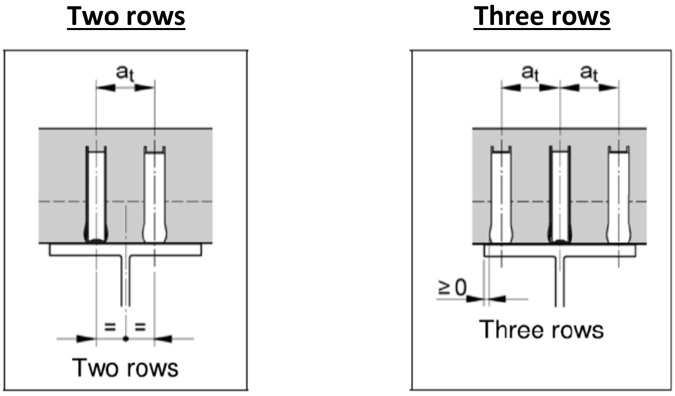


Figure 7 : Spacing of connectors for slabs with transverse decking and connectors parallel with the beam.

The following conditions must be fulfilled:

- For the flange width of the steel section:

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

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

$a_t \geq 100\ mm$ 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:

$$b_b \geq 60\ mm$$

> ***Error Code (see [27] §7):***
>
> *Error code 26 is returned when this condition is not met.*

- ***For slabs with transverse decking and connectors transverse to the beam axis (single row)***

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

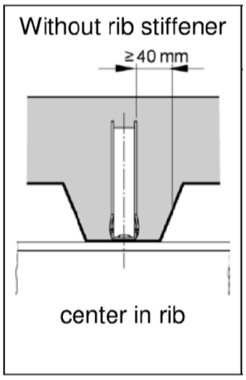


Figure 8 : Spacing of connectors for slabs with transverse decking and connectors transverse to the beam (single row).

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

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

> ***Error Code (see [27] §7):***
>
> *Error code 26 is returned when this condition is not met <u>and</u> if b_b < 40 mm (see conditions for multiple rows).*
>
> *Error code 26 is also returned if one of the 2 following conditions is not fulfilled:*
>
> - $b_b > w$
> - $b_r > w$

When the 2 following conditions are met:

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

Sometimes, it is not possible to have a single row but it is possible to have multiple row. So when the software designs the number of connectors, it starts the design process by considering 2 connectors in a row (see 6.4.1).

- ***For slabs with transverse decking and connectors transverse to the beam axis (multiple rows)***

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

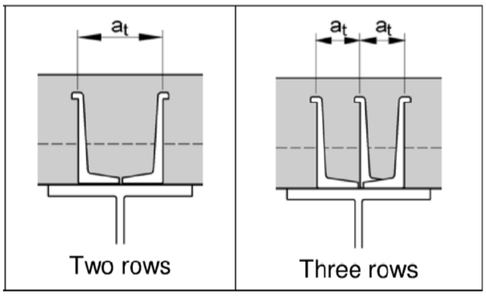


Figure 9 : Spacing of connectors for slabs with transverse decking and connectors transverse to the beam (multiple rows).

The following conditions must be fulfilled:

- For the flange width of the steel section:

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

Where: $a_t \geq 50\ mm$ for profiled decking with $b_0/h_p \geq 1{,}8$ and with two rows of connectors
$a_t \geq 100\ mm$ 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:

$$b_b \geq 40\ mm$$

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

- For the bottom width of connectors:

$$a_t \geq 2w_b$$

- ***For slabs with decking parallel with the beam axis (single row)***

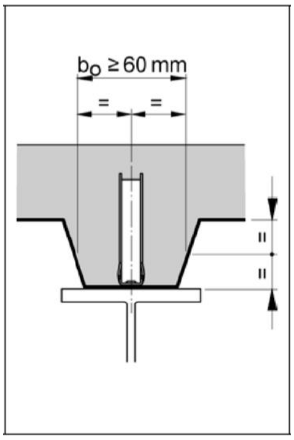


Figure 10 : Spacing of connectors for slabs with transverse decking and connectors transverse to the beam (multiple rows).

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

$$b_0 \geq 60\ mm$$

Error Code (see [27] §7):   *Error code 26 is returned when this condition is not met.*

- ***For slabs with decking parallel with the beam axis (multiple rows)***

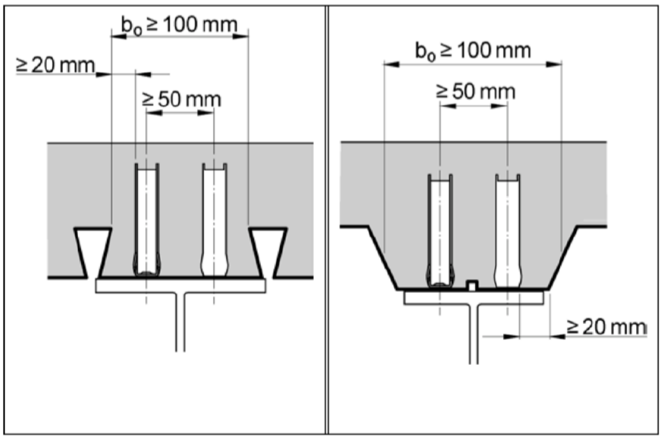


Figure 11 : Spacing of connectors for slabs with decking parallel with the beam (multiple rows).

The following conditions must be fulfilled:

- For the flange width of the steel section:

$$b \geq w_b + 50\ mm$$

- For the width at mid-height of the rib:

$$b_0 \geq \max(100\ mm; w_b + 90\ mm)$$

> ***Error Code (see [27] §7):***
> *Error code 26 is returned when this condition is not met.*

- ***For 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.

2.8. Loads

2.8.1. 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 for elementary load cases are considered within these specifications:

- One permanent load case, denoted $\boldsymbol{G}$
- one live load during construction stage, denoted $\boldsymbol{Q}_c$
- Up to two live load cases during the final stage, denoted $\boldsymbol{Q}_1$ and $\boldsymbol{Q}_2$

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 $\boldsymbol{G}$, 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.

2.8.2. 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 "secondary intermediate beam", a linear uniformly distributed load along the beam is derived:

$$q_{lin} = q_{surf}\,(b_1 + b_2)/2 \tag{7}$$

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

$$q_{lin} = q_{surf}\,(b_1 + b_2/2) \tag{8}$$

For a "primary beam" defined without secondary beams, the same 2 previous equations are applied, considering the option intermediate or edge beam.

For a "primary beam" defined with one or several secondary beams, the surface load applied on the concrete slab (q_{surf}) is transferred by secondary beams to the primary beam. Considering a "primary intermediate beam", a point load is derived at the location of the i-th secondary beam (note: no secondary beams in the first version of the HVB software):

$$P_{s,i} = \left[q_{surf}\,(L_{i+1} - L_{i-1})/2\right](b_1 + b_2)/2 \tag{9}$$

Where: $L_0 = 0$ and $L_{n_s+1} = L$

For the global equilibrium of the applied forces, point loads are also applied at both supports:

$$P_{s,0} = \left[q_{surf}\,L/(2(ns+1))\right](b_1 + b_2)/2 \tag{10}$$

For a "primary edge beam", these 2 equations are replaced respectively by the 2 following ones:

$$P_{s,i} = \left[q_{surf}\,(L_{i+1} - L_{i-1})/2\right](b_1 + b_2/2) \tag{11}$$

$$P_{s,0} = \left[q_{surf}\,L/(2(ns+1))\right](b_1 + b_2/2) \tag{12}$$

2.8.3. Automatic dead load assessment

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

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

$$\text{for intermediate beams : } \quad q_{\text{surf,d}} = 2\,\frac{g\,\rho_{\text{steel}}\,A}{b_1 + b_2}$$

$$\text{for edge beams : } \quad q_{\text{surf,d}} = 2\,\frac{g\,\rho_{\text{steel}}\,A}{2\,b_1 + b_2} \tag{13}$$

where: g is the gravity constant: g = 9,81 m/s2

ρ_{steel}: see § 2.3.2

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

For primary beams, the dead load of the secondary beams is defined with point loads. For a "primary intermediate beam", the point load at the location of the i-th secondary beam is given by (not used in the first version):

$$P_i = g\,m_s\,(b_1 + b_2)/2 \tag{14}$$

where: m_s, b_1, b_2: see § 2.2

For a "primary edge beam", this equation is replaced by (idem):

$$P_i = g\,m_s\,(b_1 + b_2/2) \tag{15}$$

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

$$q_{slab} = g\ \rho_c\ h_{eq} \tag{16}$$

where: ρ_c: see § 2.4

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

$h_{eq} = h$ for plain slabs

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

h_p, b_r, b_b, b_s: see § 2.5

h: see § 2.4

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

$$q_{deck} = g\ G_{deck} \tag{17}$$

where: G_{deck}: see § 2.5

2.8.4. 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					
	A (*) Residential	**B** Office	**C (*)** Congregat.	**D (*)** Shopping	**E (*)** Storage	**H** Roof
Recommended values of Eurocodes	2,0	3,0	3,0	4,0	7,5	0,4
France	1,5	2,5	2,5	5,0	7,5	0,8
Spain	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
Belgium	2,0	3,0	3,0	5,0	7,5	0,8-A/100 ≥ 0,2
Luxemburg	2,0	3,0	3,0	5,0	7,5	0,4
Italy	2,0	2,0	3,0	4,0	6,0	0,5

For category A, sub category Floors is considered.
For category C, sub category C1 is considered.
For category D, sub category D1 is considered.
For category E, sub category E1 is considered.
(*): recommended value is used

Table 10 : Default surface live load (kN/m²)

Mapping with the UI:

A routine is provided with the DLL that calculates this default surface load - see [27].

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:

$$\begin{gathered}\gamma_G \boldsymbol{G} + \gamma_Q \boldsymbol{Q_1} + \gamma_Q \psi_0 \boldsymbol{Q_2} \\ \gamma_G \boldsymbol{G} + \gamma_Q \boldsymbol{Q_2} + \gamma_Q \psi_0 \boldsymbol{Q_1}\end{gathered} \tag{18}$$

- For the case with one variable load case:

$$\gamma_G \boldsymbol{G} + \gamma_Q \boldsymbol{Q_1} \tag{19}$$

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:

$$\begin{gathered}\boldsymbol{G} + \boldsymbol{Q_1} + \psi_0 \boldsymbol{Q_2} \\ \boldsymbol{G} + \boldsymbol{Q_2} + \psi_0 \boldsymbol{Q_1}\end{gathered} \tag{20}$$

- For the case with one variable load case:

$$\boldsymbol{G} + \boldsymbol{Q_1} \tag{21}$$

The SLS combinations for the calculation of natural frequencies are:

$$\begin{gathered}\boldsymbol{G} + p_Q \boldsymbol{Q_1} \\ \boldsymbol{G} + p_Q \boldsymbol{Q_2}\end{gathered} \tag{22}$$

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

Error Code (see [27] §7):

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 § 2.8.2.

4.3.1. Point load

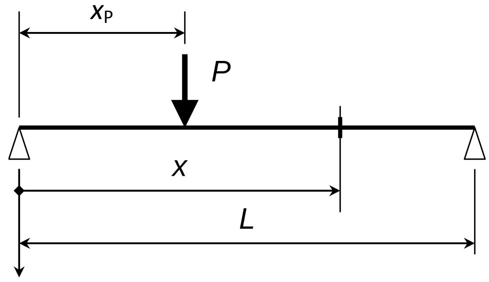


Figure 12 : Point load.

The reactions at supports are calculated by:

$R_L = -P\,(L - x_P)\,/\,L$ at the Left support

$R_R = -P\,x_P\,/\,L$ 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:

$V(x) = R_L$ if $x < x_P$

$V(x) = -R_R$ if $x > x_P$

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

$M(x) = -R_L x$ if $x < x_P$

$M(x) = -R_R(L - x)$ if $x > x_P$

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

$$w(x) = \frac{F}{6EIL}[L^2 - (L - x_P)^2 - x^2](L - x_P)x \qquad \text{if } x < x_P$$

$$w(x) = \frac{F}{6EIL}[L^2 - (L - x)^2 - x_P^2](L - x)x_P \qquad \text{if } x > x_P$$

4.3.2. Patch load

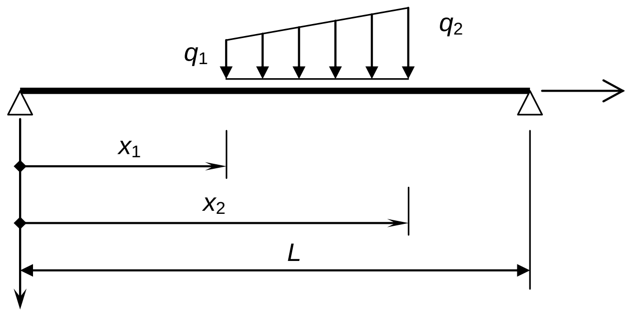


Figure 13 : Patch load.

The reactions at supports are calculated by:

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

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

Where x_1, q_1, x_2 and q_2 define the distributed load as shown in *Figure 13*.

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

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

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

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

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

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

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

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

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

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

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

And, if $x_1 \le x \le x_2$:

$$w(x) = \frac{1}{EI}\left\{\left[p\frac{x^3}{120} + (\alpha - 2px_1)\frac{x^2}{72} + \left(\frac{px_2^2 - 2\alpha x_2}{6} - R_R\right)\frac{x}{6} + \frac{1}{2}\left(R_R L + \alpha\frac{x_2^2}{2}\right)\right]x^2 + A_2 x + B_2\right\}$$

Where:

$$p = (q_2 - q_1)/(x_2 - x_1)$$
$$\alpha = -p\,x_1 + q_1 + 2\,q_2$$
$$B_1 = 0$$
$$B_2 = R_L \frac{x_1^3}{3} + w'(x_1)x_1 - w(x_1)$$
$$B_3 = w(x_2) + B_2 - \left[-R_R \frac{x_2^3}{3} + R_R L \frac{x_2^2}{2} + w'(x_2)x_2\right]$$
$$A_3 = \frac{1}{L}\left(R_R \frac{L^3}{3} - B_3\right)$$
$$A_2 = A_3 - \left(w'(x_2) - R_R \frac{x_2^2}{2} + R_R L x_2\right)$$
$$A_1 = w'(x_1) + A_2 + R_L \frac{x_1^2}{2}$$

4.4. Precamber

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

$$w_{0,x} = -4\,w_0 \left(1 - \frac{x}{L}\right) \frac{x}{L} \qquad (23)$$

Where: w_0 is the precambering.

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

4.5. Influence of the connectors slip

Decision during the 29/03/2017 meeting.

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, all deflections will be increased by the following factor:

$$\text{for propped beams during construction : } k_{pc} = 1 + 0{,}5\,(1-\eta)\left[\frac{\delta_a}{\delta_c} - 1\right]$$
$$\text{for unpropped beams during construction : } k_{pc} = 1 + 0{,}3\,(1-\eta)\left[\frac{\delta_a}{\delta_c} - 1\right] \qquad (24)$$

where: η is the degree of connection of the beam (see § 6.4.5)

δ_a is the maximum deflection of the beam considering only the steel beam flexural stiffness

δ_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 is 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 unpropped.

The weight of the concrete is considered as a variable action in the load case q.

The ULS verifications for the steel beam (without slab) 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
- Lateral torsional buckling (LTB) resistance

The SLS verifications for the steel beam include:

- Total deflection
- Deflexion under variable load cases

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 $0{,}5(b - t_w - 2r_1)/t_f \leq 10\varepsilon$

  Where: $\varepsilon = \sqrt{235/f_y}$; f_y is expressed in N/mm^2
- For the web in bending:
  - Class 1, if $\frac{h_t - 2t_f - 2r_1}{t_w} \leq 72\varepsilon$
  - Class 2, if $(h_t - 2t_f - 2r_1)/t_w \leq 83\varepsilon$

Since only the plastic design is covered in this document, the class of the cross-section must be Class 1 or 2.

Error code:

If the class of the cross-section is 3 of 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:

$$\Gamma_{\mathrm{M}} = M_{\mathrm{Ed}} / M_{\mathrm{c,Rd}} \leq 1 \tag{25}$$

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

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

$$M_{\mathrm{c,Rd}} = W_{\mathrm{pl,y}}\, f_y / \gamma_{\mathrm{M0}} \tag{26}$$

$W_{\mathrm{pl,y}}$: see Annex A

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:

$$\Gamma_{\mathrm{V}} = V_{\mathrm{Ed}} / V_{\mathrm{c,Rd}} \leq 1 \tag{27}$$

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

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

$$V_{\mathrm{c,Rd}} = \frac{A_{\mathrm{v}}\, f_{\mathrm{yk}}}{\sqrt{3}\, \gamma_{\mathrm{M0}}} \tag{28}$$

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 $A_{\mathrm{v}} = A_{\mathrm{v,z}}$ (see Annex A).

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

$$A_{\mathrm{v}} = \eta\,(h_{\mathrm{t}} - 2\,t_{\mathrm{f}})\,t_{\mathrm{w}} \tag{29}$$

h_t, t_f, t_w: see § 2.3.1

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

	η
Recommended values of Eurocodes	1,20
France	1,20
Spain	1,20
Portugal	1,20
Belgium	1,20
Luxemburg	1,20
Italy	1,20

Table 11 : Value of the factor η

5.2.4. Bending moment and Shear force Interaction

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. $V_{Ed} / V_{pl,Rd} \geq 0{,}5$.

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

$$M_{V,Rd} = \left(W_{pl,y} - \frac{\rho \, A_w{}^2}{4 \, t_w} \right) \frac{f_y}{\gamma_{M0}} \leq M_{c,Rd} \qquad (30)$$

$$A_w = (h_t - 2 \, t_f) \, t_w \qquad (31)$$

$$\rho = \left(\frac{2 \, V_{Ed}}{V_{pl,Rd}} - 1 \right)^2 \qquad (32)$$

The interaction criterion is:

$$\Gamma_{MV} = M_{Ed} / M_{V,Rd} \leq 1 \qquad (33)$$

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

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:

$$\Gamma_{Vb} = V_{Ed} / V_{b,Rd} \leq 1 \qquad (34)$$

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

$$V_{b,Rd} = \frac{\chi_w \, h_w \, t_w \, f_{yk}}{\sqrt{3} \, \gamma_{M1}} \qquad (35)$$

χ_w is the reduction factor, given by:

$$\text{if } \bar{\lambda}_w < \frac{0{,}83}{\eta} : \quad \chi_w = \eta$$

$$\text{if } \bar{\lambda}_w \geq \frac{0{,}83}{\eta} : \quad \chi_w = \frac{0{,}83}{\bar{\lambda}_w} \qquad (36)$$

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

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:

$$\Gamma_{MVb} = \frac{M_{Ed}}{M_{pl,Rd}} + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right)(2\,\Gamma_{Vb} - 1)^2 \leq 1 \qquad (37)$$

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:

$$\Gamma_{MT} = M_{Ed} \;/\; M_{b,Rd} \leq 1 \qquad (38)$$

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

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

$$M_{b,Rd} = \chi_{LT} W_{pl,y}\, f_y / \gamma_{M1}$$

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

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

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

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

α_{LT} is the imperfection factor that depends on the LTB curve:

Cross-section	Limits	Reduction curve	α_{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 α_{LT}

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

$$M_{\mathrm{cr}} = C_1 \frac{\pi^2 E I_z}{L^2} \left[\sqrt{\frac{I_w}{I_z} + \frac{G I_t L^2}{\pi^2 E I_z} + \left(C_2 z_{\mathrm{g}}\right)^2} - C_2 z_{\mathrm{g}} \right]$$

$z_{\mathrm{g}} = +h_t/2$: loads are assumed to be applied on the top flange of the section

C_1 and C_2 are given in Table 13.

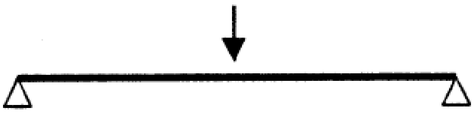
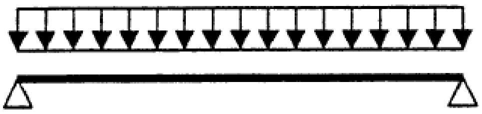
Loading	C_1	C_2
	1,35	0,59
	1,13	0,45

Table 13 : Factors C1 and C2

When the French National Annex is selected, the reduction factor χ_{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: $f = 1 - 0{,}5\,(1 - k_c)\left[1 - 2(\bar{\lambda}_{LT} - 0{,}8)^2\right]$

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

5.3. SLS verifications

No SLS checks at construction stage.

The software will provide the deflection under SLS combination.

6. VERIFICATION AT FINAL STAGE

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

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

$b_{\mathrm{e}} = \min\{L/8\,;\, b_1\} + \min\{L/8\,;\, b_2/2\}$ for an edge beam

$\beta = 0{,}55 + 0{,}025\, L/b_{\mathrm{e}} \leq 1{,}0$

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

6.2. Design resistance of the connector

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

- **For solid slabs**: the design resistance P_{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 $P_{\mathrm{Rd,t}}$ is obtained from the formulae given in Table 4 of ETA-15/0876:

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

$$\text{where: } k_{t,l} = \frac{0{,}66}{\sqrt{n_r}} \frac{b_0}{h_p}\left(\frac{h_{sc}}{h_p} - 1\right) \leq 1 \tag{39}$$

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

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

$$\text{where: } k_{t,t} = \frac{1{,}18}{\sqrt{n_r}} \frac{b_0}{h_p}\left(\frac{h_{sc}}{h_p} - 1\right) \leq 1 \tag{40}$$

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

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

$$\text{where: } k_l = 0{,}6 \frac{b_0}{h_p}\left(\frac{h_{sc}}{h_p} - 1\right) \leq 1 \tag{41}$$

For formulas (39) to (41):

$b_0 = (b_r + b_b)/2$

b_r, b_b, h_p: see § 2.5

h_{sc}: see Table 8

n_r: see § 2.7.1

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:

$$P_{Rd,red} = \frac{t_f}{8} P_{Rd} \geq 23\,kN \tag{42}$$

For custom steel (see § 2.3.2), an additional reduction factor is applied to the resistance of the connector:

$$\alpha_{BM,red} = 0{,}95 \tag{43}$$

This reduction factor will be applied for each custom steel, where f_{yk} < 235 MPa (see decision in email MdS 24/03/17).

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:

- for plain slabs : $e_{part} = \eta\, h$
- for slabs with steel sheeting : $e_{part} = \eta\, (h - h_p)$ (44)

Where: η is the degree of connection (see § 2.7.3); η = 1 for full connection.
See following chapters for the calculation of the degree of connection for partial connection.

6.4. Number of connectors

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 software 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 2.7.4). If the requirements for full or partial connection are not met, he 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:

$$n_{ribs} = \frac{\min\{x_c\,;L - x_c\}}{b_s} \quad (45)$$

where: b_s: see § 2.5

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

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

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

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

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

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

The number of connectors, denoted n_{f}, is then determined according to the value of η:

- If $\eta < 1$, the full connection is not possible. An error code will be provided.
- If $1 \leq \eta < 2$: $n_{\mathrm{f}} = n_{\mathrm{ribs}}$
- If $2 \leq \eta < 3$: $n_{\mathrm{f}} = n_{\mathrm{ribs}}/2$ (A connector is placed at every two ribs)
- etc…

b) Slab with decking parallel with the beam axis

$$n_{\mathrm{f}} = \frac{\min\{N_{\mathrm{a,Rd}}; N_{\mathrm{c,Rd}}\}}{n_r P_{\mathrm{Rd,l}}}$$

c) Solid slab

$$n_{\mathrm{f}} = \frac{\min\{N_{\mathrm{a,Rd}}; N_{\mathrm{cs,Rd}}\}}{n_r P_{\mathrm{Rd}}}$$

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

The number of connectors n_{f} must fulfil the requirements of connectors positioning as defined in § 2.7.4. 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 η_{min}, is calculated according to EN 1994-1-1 §6.6.1.2:

- If $L \leq 25\ m$:

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

- Otherwise:

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

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 η_{min}:

$$n_0 = \eta_{\mathrm{min}} \times n_{\mathrm{f}}$$

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

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

c) Other slabs

The minimum number of connectors is calculated by:

$$n_{\mathrm{min}} = \eta_{\mathrm{min}} n_{\mathrm{f}}$$

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:

$$n = n_{\min}$$

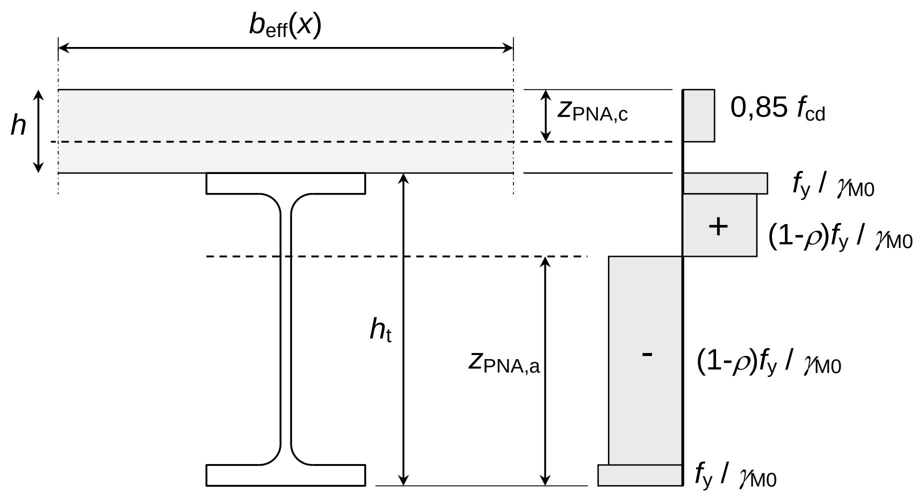
Otherwise, continue Step 3.

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

6.5. Moment resistance

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

6.5.1. PNA in the concrete slab



The position of the PNA in the concrete slab is calculated by:

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

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

$h_{\text{p}} = 0$ for solid slabs

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

$$N_{\text{pl,a,Rd}} = \left[A - \rho A_{\text{v,z}}\right] f_{\text{y}}/\gamma_{M0}$$

The moment resistance of the concrete slab is calculated by:

$$M_{\text{c}} = N_{\text{c}}\left(h + \frac{h_{\text{t}} - z_{\text{PNA,c}}}{2}\right)$$

Where: N_{c} is the resulting compression force in the concrete slab, determined by:

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

6.5.2. PNA in the steel section

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

$$z_{\mathrm{PNA,a}} = h_{\mathrm{t}} - \frac{N_{\mathrm{pl,a,Rd}} - N_c}{2b\, f_{\mathrm{y}}/\gamma_{M0}}$$

Where: $N_{\mathrm{pl,f,Rd}} = (A - 2bt_{\mathrm{f}})(1-\rho)\, f_{\mathrm{y}}/\gamma_{M0}$

The moment resistance of the section is calculated by:

$$M_{\mathrm{Rd}} = \left(h_{\mathrm{t}} - z_{\mathrm{PNA,a}}\right) b z_{\mathrm{PNA,a}}\, f_{\mathrm{y}}/\gamma_{M0} + M_{\mathrm{c}}$$

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

$$z_{\mathrm{PNA,a}} = \frac{h_{\mathrm{t}}}{2} + \frac{N_c}{2t_{\mathrm{w}}(1-\rho)\, f_{\mathrm{y}}/\gamma_{M0}}$$

Where: $N_{\mathrm{pl,w,Rd}} = (h_{\mathrm{t}} - 2t_{\mathrm{f}} - 2r_1)t_{\mathrm{w}}(1-\rho)\, f_{\mathrm{y}}/\gamma_{M0}$

The moment resistance of the section is calculated by:

$$M_{\mathrm{Rd}} = \left[(1-\rho)W_{\mathrm{pl,y}} + bt_{\mathrm{f}}(h_{\mathrm{t}} - t_{\mathrm{f}})\rho - \frac{1}{4}(1-\rho)t_{\mathrm{w}}\left(\frac{N_c}{t_{\mathrm{w}}(1-\rho)\, f_{\mathrm{y}}/\gamma_{M0}}\right)^2\right]\frac{f_{\mathrm{y}}}{\gamma_{M0}} + M_{\mathrm{c}}$$

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

6.6. ULS verifications

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. $N_{\mathrm{pl,f,Rd}} > N_{\mathrm{c}}$
  - Class 1, if $0{,}5(b - t_w - 2r_1)/t_f \leq 9\varepsilon$
  - Class 2, if $0{,}5(b - t_w - 2r_1)/t_f \leq 10\varepsilon$
- For the web: it has to be classified only when the PNA is located in the web, i.e. $N_{\mathrm{pl,w,Rd}} \geq N_{\mathrm{c}}$
  - Class 1, if $(h_t - 2t_f - 2r_1)/t_w \leq 396\varepsilon/(13\alpha - 1)$
  - Class 2, if $(h_t - 2t_f - 2r_1)/t_w \leq 456\varepsilon/(13\alpha - 1)$

  Where: $\alpha = \frac{h_{\mathrm{t}} - t_{\mathrm{f}} - r_1 - z_{\mathrm{PNA,a}}}{h_{\mathrm{t}} - 2t_{\mathrm{f}} - 2r_1}$. It should be noted that: $\alpha > 0{,}5$.

Since only the plastic design is covered in this document, the class of the cross-section must be Class 1 or 2.

6.6.2. ULS verifications

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 the paragraph 6.5 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

6.7. SLS verifications

6.7.1. 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 lase, 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 are defined as follows:

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

As a simplification, it is considered here that $(1 + \psi_L \phi_t) \approx 3$.

6.7.2. Position of the Elastic Neutral Axis (ENA)

The position of the Elastic Neutral Axis (ENA) is calculated by:

$$z_{\mathrm{ENA}} = \frac{A\, h_t/2 + b_{\mathrm{eff}}(x) \times (h - h_p)\left[h_t + (h + h_{\mathrm{p}})/2\right]/n_{\mathrm{eq}}}{A + b_{\mathrm{eff}}(x) \times (h - h_p)/n_{\mathrm{eq}}}$$

The second moment of area of the composite section is calculated by:

$$I_{\mathrm{y}} = I_{\mathrm{y,a}} + A(h_t/2 - z_{\mathrm{ENA}})^2 + \frac{b_{\mathrm{eff}}(x) \times (h - h_p)^3}{12 n_{\mathrm{eq}}} + \frac{b_{\mathrm{eff}}(x) \times (h - h_p)}{n_{\mathrm{eq}}}\left[h_t + (h + h_{\mathrm{p}})/2 - z_{\mathrm{ENA}}\right]^2$$

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

6.7.3. Calculation of the natural frequency

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

$$f = \frac{1}{2\pi}\sqrt{g\frac{\sum P_{\mathrm{i}}\,|w_{\mathrm{i}}|}{\sum P_{\mathrm{i}}\, w_{\mathrm{i}}^2}} \qquad (46)$$

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.

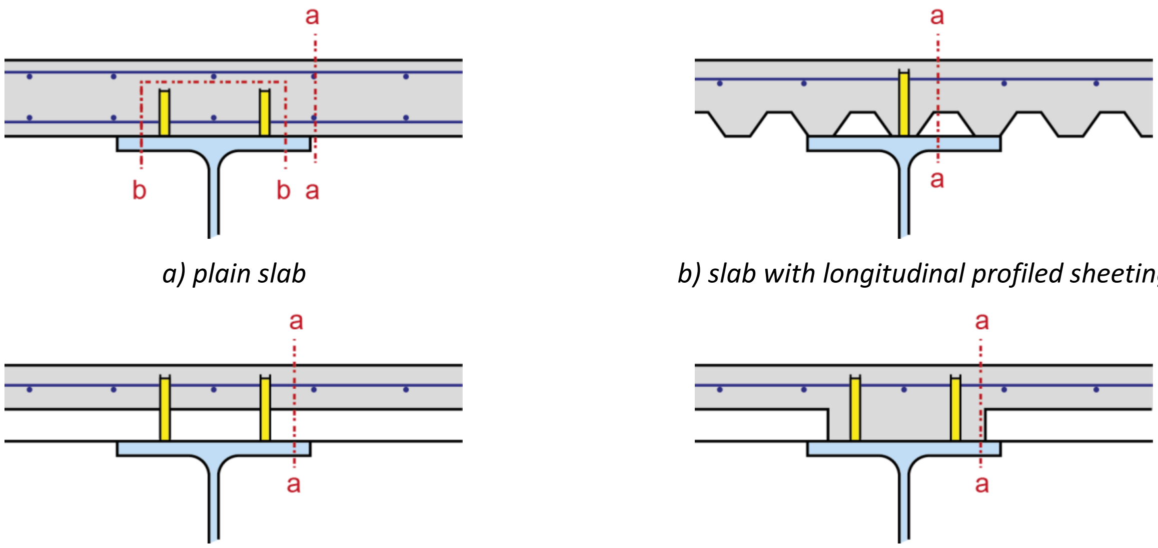
6.8. Longitudinal shear resistance

6.8.1. Introduction

The HVB software 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 trough the top one. Calculations are performed for 2 shear areas (a-a and b-b). See *Figure 14 a)*.
- For slabs with profiled steel sheeting, either longitudinal or perpendicular, only one layer is assumed (*Figure 14 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 14 c)*.



a) plain slab

b) slab with longitudinal profiled sheeting

c) slab with continuous perpendicular profiled sheeting

d) slab with not-continuous perpendicular profiled sheeting

Figure 14: Transverse reinforcement configurations and shear areas

6.8.2. Minimum transverse reinforcement ratio

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

$$\rho_{w,\min} = \frac{0{,}08\sqrt{f_{ck}}}{f_{yrk}} \tag{47}$$

where: f_{ck}: see § 2.4

f_{yrk}: see § 0

6.8.3. Design of transverse reinforcement

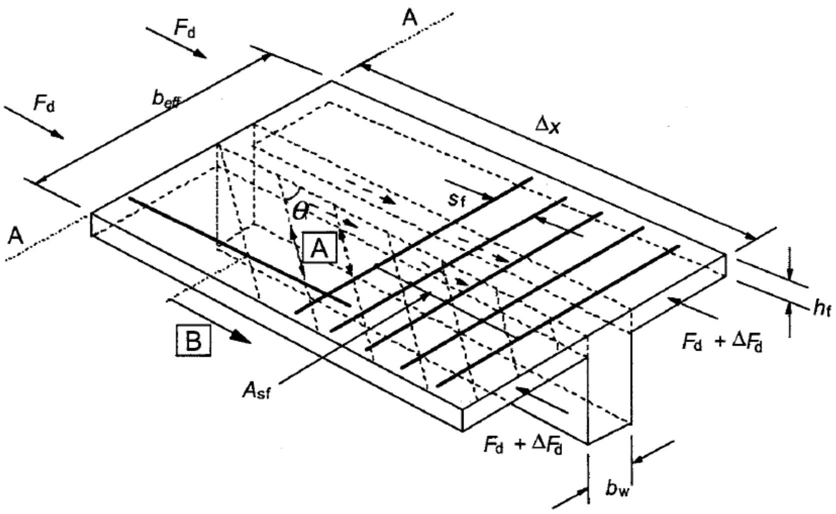


Figure 15 : Transverse reinforcement.

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 ΔF_d between the critical section and the closest support that should be transferred by the reinforcement through the shear area is given by:

For a shear area a-a:

$$\Delta F_d = N_{Rd,slab} \frac{\max\{b_{eff,left}(x_c)\,;\,b_{eff,right}(x_c)\}}{b_{eff}(x_c)} \tag{48}$$

For a shear area b-b:

$$\Delta F_d = N_{Rd,slab} \tag{49}$$

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:

$$N_{Rd,slab} = 0.85\,\frac{b_{eff}(x_c)\,e_{part}(x_c)\,f_{ck}}{\gamma_c} \tag{50}$$

$b_{eff}(x_c)$ is the effective width of concrete slab at the critical section (see § 6.1);

$b_{eff,left}(x_c)$ 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 § 2.4

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:

$$v_{Ed} = \frac{\Delta F_d}{n_{fs}\ \Delta x\ h_f} \leq v_{Rd} = \nu\, f_{cd} \sin\theta_f \cos\theta_f \qquad (51)$$

where: Δx 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 § 2.4

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

$$\nu = 0{,}6\left[1 - \frac{f_{ck}}{250}\right] \qquad (52)$$

θ_f is the orientation of the concrete strut under compression, which is calculated by the following equation:

$$\theta_f = \frac{1}{2}\arcsin\left[\frac{2\,v_{Ed}}{\nu\, f_{cd}}\right] \text{ but } \sqrt{\frac{8}{15}} \leq \frac{2\,v_{Ed}}{\nu\, f_{cd}} \leq 1 \qquad (53)$$

and:

$$\arctan\left(\frac{1}{2}\right) = 26{,}56° \leq \theta_f \leq \arctan(1) = 45° \qquad (54)$$

The shear length Δx 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 16).

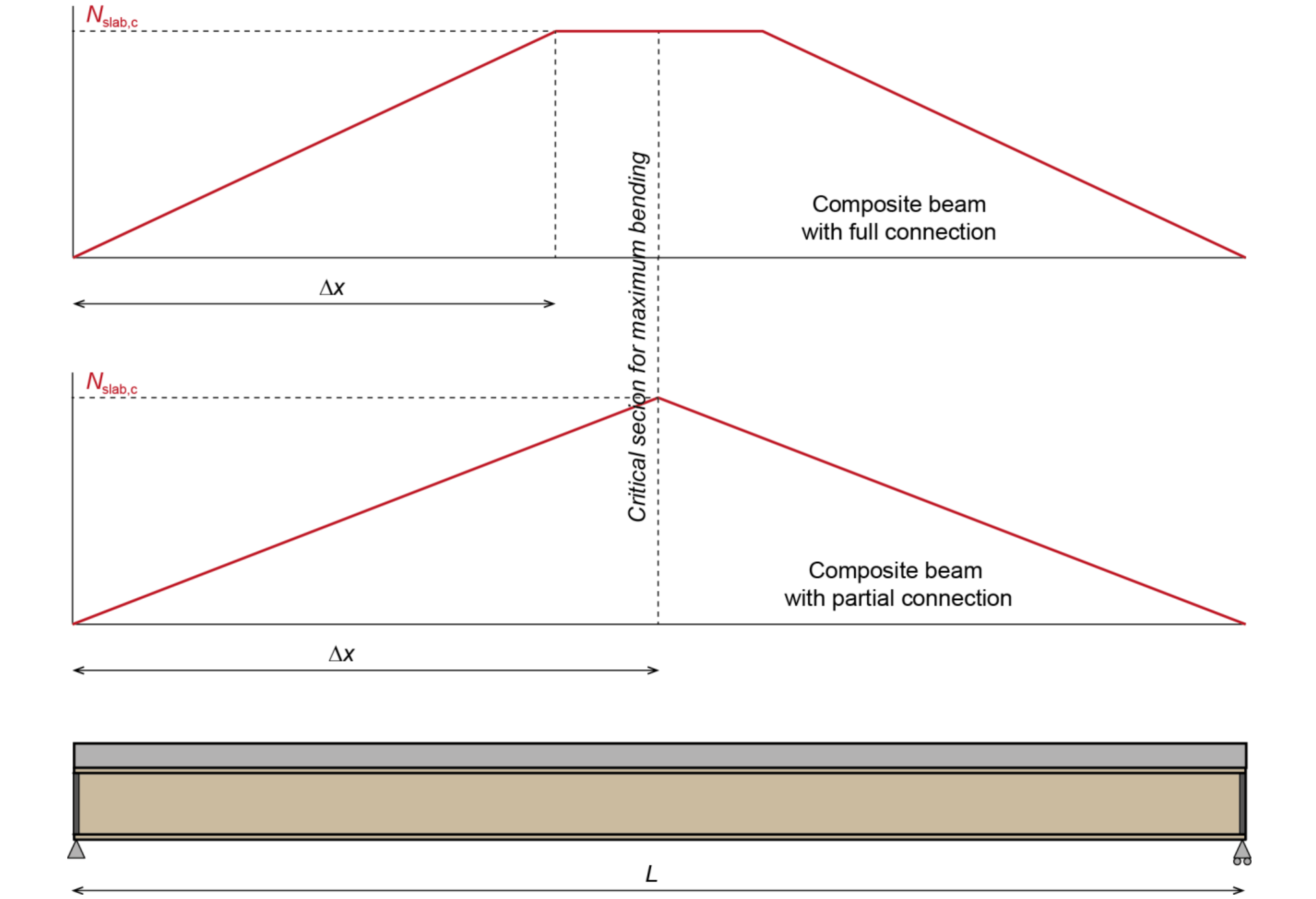


Figure 16 : Shear length

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

$$\Gamma_{\text{cstrut}} = \frac{v_{\text{Ed}}}{v_{\text{Rd}}} \leq 1 \qquad (55)$$

Assessment of the transverse reinforcement

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

$$\left(\frac{A_{\text{sf}}}{s_{\text{f}}}\right) \geq \frac{v_{\text{Ed}}\, h_{\text{f}}}{f_{\text{yd}} \cot \theta_{\text{f}}} \qquad (56)$$

where: f_{yd}: see § 0

v_{Ed} and θ_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_b/s_b. The calculation performed for the shear area a-a gives $A_f/s_f = A_t/s_t + A_b/s_b$.

Influence of continuous perpendicular profiled steel sheeting

For slabs with continuous perpendicular profiled steel sheeting, the previous equation (56) is replaced by:

$$\left(\frac{A_{sf}}{s_f}\right) \geq \max\left\{0\,; \frac{v_{Ed}\, h_f}{f_{yd}\cot\theta_f} - A_{pe}\frac{f_{yp,d}}{f_{yd}}\right\} \qquad (57)$$

where: f_{ypd}: see § 2.5

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

$$A_{pe} = \frac{t_p}{b_s}\left[b_s + b_b - b_r + \sqrt{(b_b - b_r)^2 + 4\,h_p{}^2}\right] \qquad (58)$$

t_p, h_p, b_b, b_s, b_r: see § 2.5

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

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

7. REFERENCES

[1] European Technical Assessment ETA-15/0876 of 3 June 2016, Deutsches Institut für Bautechnik, 2016.

[2] EN 10025-2 – Hot rolled products of structural steels. Part 2: Technical delivery conditions for non-alloy structural steels. March 2005.

[3] EN 1990 – Basic of structural design. AFNOR. March 2003.

[4] NF EN 1990/NA – Bases de calcul des structures. Annexe Nationale Française à la NF EN 1990:2003. AFNOR. Décembre 2011.

[5] N 1990:2002/AN-LU:2011 – Bases de calcul des structures. Annexe Nationale Luxembourgeoise. ILNAS. Septembre 2011.

[6] UNI-EN 1990 – Criteri generali di progettazione strutturale. Italiano Appendice nazionale. Settembre 2010.

[7] EN 1992-1-1 – Design of concrete structures. Part 1-1: General rules and rules for buildings. AFNOR. October 2005.

[8] NF EN 1992-1-1/NA – Calcul des structures en béton. Partie 1-1 : Règles générales et règles pour les bâtiments. Annexe Nationale Française à la NF EN 1992-1-1:2005. AFNOR. Mars 2016.

[9] EN 1992-1-1:2004/AN-LU:2011 – Calcul des structures en acier. Partie 1-1 : Règles générales et règles pour les bâtiments. Annexe Nationale Luxembourgeoise. ILNAS. Septembre 2011

[10] AN/UNE-EN 1992-1-1 – Eurocódigo 2: Proyecto de estructuras de hormigón. Parte 1-1: Reglas generales y reglas para edificios. Octubre 2015.

[11] UNI-EN 1992-1-1 – Eurocodice 2: Progettazione delle strutture di calcestruzzo. Part 1-1: Regole generali e regole per gli edifici. Settembre 2010.

[12] EN 1993-1-1 – Design of steel structures. Part 1-1: General rules and rules for buildings. AFNOR. October 2005.

[13] NF EN 1993-1-1/NA – Calcul des structures en acier. Partie 1-1 : Règles générales et règles pour les bâtiments. Annexe Nationale Française à la NF EN 1993-1-1:2005. AFNOR. Août 2013.

[14] EN 1993-1-1:2005/AN-LU:2011 – Calcul des structures en acier. Partie 1-1 : Règles générales et règles pour les bâtiments. Annexe Nationale Luxembourgeoise. ILNAS. Septembre 2011.

[15] AN/UNE-EN 1993-1-1 – Eurocódigo 3: Proyecto de estructuras de acero. Parte 1-1: Reglas generales y reglas para edificios. Febrero 2013.

[16] UNI-EN 1993-1-1 – Eurocodice 3: Progettazione delle strutture di acciacio. Part 1-1: Regole generali e regole per gli edifici. Settembre 2010.

[17] EN 1993-1-5 – Design of steel structures. Part 1-5: Plated structural elements. AFNOR. March 2007.

[18] NF EN 1993-1-5/NA – Calcul des structures en acier. Partie 1-5 : Plaques planes. Annexe Nationale Française à la NF EN 1993-1-5:2007. AFNOR. Octobre 2007.

[19] EN 1993-1-5:2006/AN-LU:2011 – Calcul des structures en acier. Partie 1-5 : Plaques planes. Annexe Nationale Luxembourgeoise. ILNAS. Septembre 2011.

[20] AN/UNE-EN 1993-1-1 – Eurocódigo 3: Proyecto de estructuras de acero. Parte 1-5: Placas planas cargadas en su plano. Febrero 2013.

[21] UNI-EN 1993-1-5 – Eurocodice 3: Progettazione delle strutture di acciacio. Part 1-5: Elementi strutturali a lastra. Febbraio 2011.

[22] EN 1994-1-1 – Design of composite steel and concrete structures. Part 1-1: General rules and rules for buildings. AFNOR. June 2005.

[23] NF EN 1994-1-1/NA – Calcul des structures mixtes acier-béton. Partie 1-1 : Règles générales et règles pour les bâtiments. Annexe Nationale Française à la NF EN 1994-1-1:2005. AFNOR. Avril 2007.

[24] EN 1994-1-1:2004/AN-LU:2011 – Calcul des structures mixtes acier-béton. Partie 1-1 : Règles générales et règles pour les bâtiments. Annexe Nationale Luxembourgeoise. ILNAS. Septembre 2011.

[25] AN/UNE-EN 1994-1-1 – Eurocódigo 4: Proyecto de estructuras mixtas de acero y hormigón. Parte 1-1: Reglas generales y reglas para edificación. Anejo Nacional Español. Febrero 2013.

[26] UNI-EN 1994-1-1 – Eurocodice 4: Progettazione delle strutture di acciacio-calcestruzzo. Part 1-1: Regole generali e regole per gli edifici. Settembre 2010.

[27] CTICM report DRV/HVB/MT/001/Rev G – HVB Software – Description of DLL and parameters – 05/2017

Annex A : Properties of the steel section

The following section properties are calculated using analytical formulae:

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

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

I_w Warping constant

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