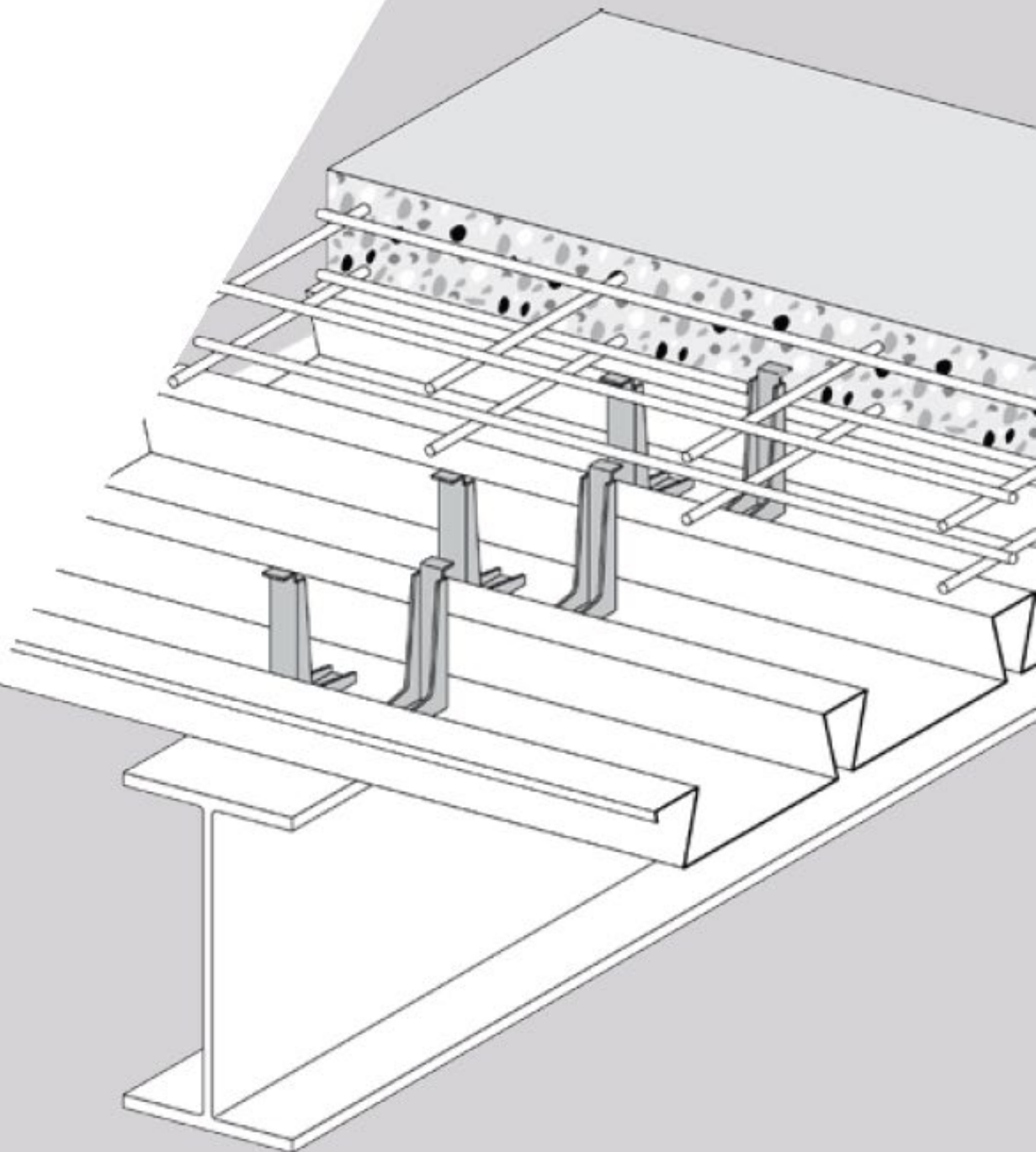




# SHEAR CONNECTOR DESIGN SOFTWARE

Technical specifications



Content according to :

HILTI SHEAR CONNECTOR DESIGN - CALCULATION MODULE\_TECHNICAL SPECIFICATIONS\_DRV/HVB/MT/009-B

Courtesy of :

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## 1. INTRODUCTION

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The calculation module of the HVB software allows the user to perform the design of composite beams using HILTI X-HVB connectors according to the rules of the Australasian (Australia and New-Zealand) standards. 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 for Australasian standards 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 beam is achieved through HILTI X-HVB connectors;
- The design of the beam is carried out at Ultimate Limit States (ULS) and at Serviceability Limit States (SLS);
- For the checks at SLS, the elastic deflection and the vibrations of the beam due to walking activities or crowd synchronized movement are calculated;
- The calculations and design checks are carried out according to the Australasian Standards and European Technical Assessment ETA-15/0876 (see references in §2).

## 2. REFERENCES

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### 2.1. Technical references

- [1] European Technical Assessment ETA-15/0876 of 3 June 2016, Deutsches Institut für Bautechnik, 2016.
- [2] HERA report No SSTR-066 – Design rules for Hilti nailed shear connectors X-HVB – HERA – 12/12/2018
- [3] DRV/HVB/MT/003-F – Hilti Shear Connector Design – Calculation Module – Technical Manual – CTICM 11/06/2019
- [4] CTICM report DRV/HVB/MT/001/Rev Q – HVB Software – Description of DLL and parameters – 04/2020
- [5] Feldmann M. *et al.* – Design of floor structures for human induced vibrations – JRC Scientific and Technical Reports – 2009
- [6] Smith A.L. *et al.* – Design of Floor for Vibrations: A New Approach – SCI Publication P354 – February 2009.

All the requirements regarding the XHVB connectors considered in this Report are based upon the European Technical Agreement [1].

### 2.2. Standards

- [7] AS/NZS 2327:2017 – Composite structures – Composite steel-concrete construction in buildings – Australian/New Zealand Standards – published on 20 December 2017.
- [8] AS/NZS 1170.1:2002 – Structural design actions – Part 1: Permanent, imposed and other actions - Australian/New Zealand Standards – published on 4 June 2002 + amendment n°1
- [9] AS/NZS 1170.0:2002 – Structural design actions – Part 0: General principles - Australian/New Zealand Standards – published on 4 June 2002 + amendments no 1, 2 and 4.

- [10] AS/NZS 1554.1:2014 – Structural steel welding – Part 1: Welding of steel structures - Australian/New Zealand Standards – published on 2 December 2014.
- [11] AS 4100:1998 – Steel structures – Australian Standards – published on 5 June 1998 – Reconfirmed in 2005 – Including amendment no 1.
- [12] AS 3600:2018 – Concrete structures – Australian Standards – published on 29 June 2018.
- [13] NZS 3404: Part 1: 1997 – Steel structures standard – New Zealand Standard – Including Amendments no 1 (2001) and 2 (2007)
- [14] NZS 3101: Part 1: 2006 – Concrete structures standard - New Zealand Standard – Including Amendments no 1, 2 and 3



*Standards for loads, combinations and composite structures are common for Australia and New-Zealand. They differ for steel and concrete structures. When relevant, this document highlights the differences between the national standards.*

For the use of the software, the User chooses the country where his project is built. This report gives the specific rules applied for each country, when relevant.

### 3. BASIC DATA

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#### 3.1. Symbols

As far as possible, symbols and terminology of the Australasian standards are used in this document.

#### 3.2. Units

All calculations are carried out considering the metric unit system.

#### 3.3. Type of design

According to Australasian standards, a plastic design is applied for both compact and non-compact cross-sections. Slender cross-sections are not covered by the software.

### 3.4. Geometrical description of the beam

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

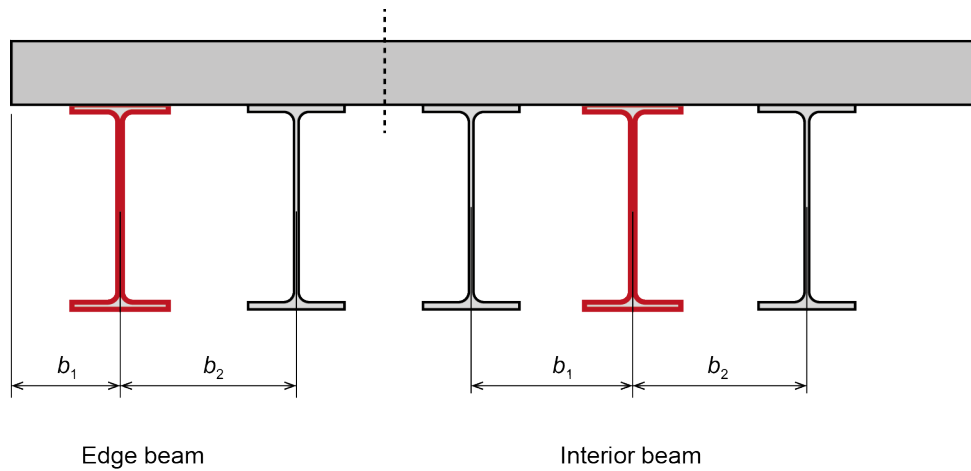


Figure 1 : Edge beam and interior beam

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

- For interior 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 E):**

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:

- $b_{1,\min} \leq b_1 \leq 20m$  where  $b_{1,\min} = 0,5 m$  for interior beams and  $b_{1,\min} = 0,15 m$  for edge beams;
- $0,5m \leq b_2 \leq 20m$

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

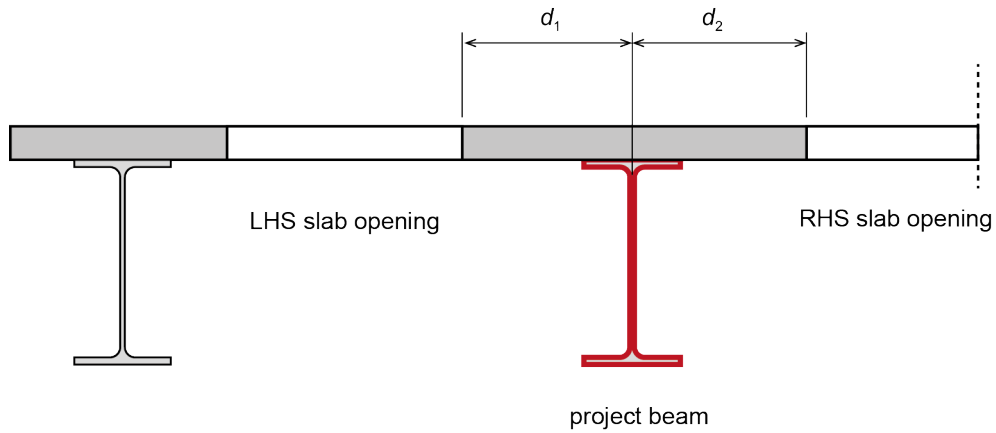


Figure 2 : Definition of slab openings

### 3.5. Steel section

#### 3.5.1. Geometry

By default, the steel profile is defined as a hot-rolled profile to be selected in the profiles database of the software. The User can nevertheless directly define a welded steel section by its geometrical parameters, as follows (Figure 3):

- $h_t$  is the total height
- $b_f$  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 E):**

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

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



For custom profiles, minimum values of the throat of the fillet weld are given in *Table 1*:

Thickness of thickest part $t$ (mm)	Minimum size of fillet weld $a_{\min}$ (mm)
$t_f \leq 3$	$2t/3$
$3 < t_f \leq 7$	3
$7 < t \leq 10$	4
$10 < t \leq 15$	5
$15 \leq t$	6

*Table 1: Minimum size of fillet welds (Table 3.3.5 [10]).*

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.

Section properties are obtained using the analytical formulas given in Annex A.

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

- For solid slabs, the minimum thickness of flanges is 6 mm, i.e.  $t_f \geq 6 \text{ mm}$ . Two European 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.  $t_f \geq 8 \text{ mm}$ . The minimum thickness of flanges can be reduced to 6 mm when the relevant software option is activated (see 3.9.5). This option extended the scope of the ETA report [1]. The scope extension applies for all Standards available in the XHVB software, including the Australasian standards.

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

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



*For solid slabs, all profiles with  $t_f < 6 \text{ mm}$  are out of the scope of ETA and are excluded from the software, except the 2 European profiles IPE 100 and IPN 100.*

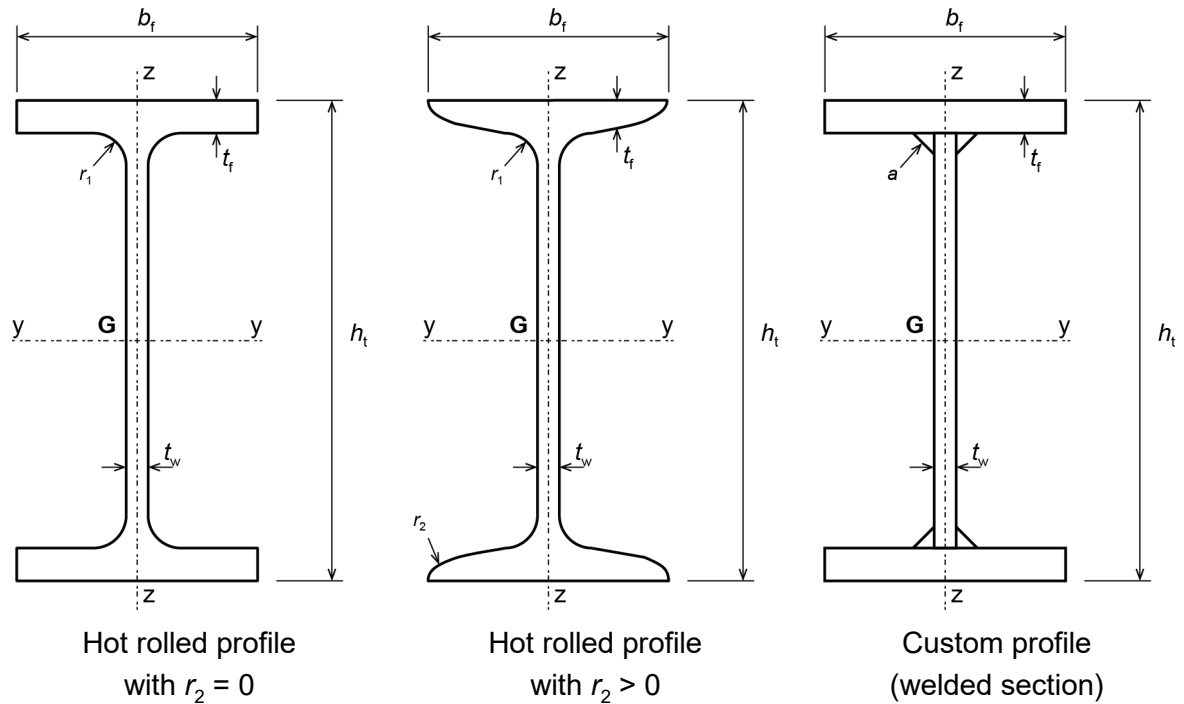


Figure 3: Steel profiles

### 3.5.2. Steel grade



References:

AS 4100 [11]: Section 2      NZS 3404 [13]: Section 2

According to the Australian standard for steel structures [11], the steel grades for hot-rolled profiles should comply with the Australasian standards AS/NZS 3679.1. According to Table 2.1 [11], the steel grade for the profile should be selected by the user among the list given in Table 2. This Table also gives the value of the yield strength  $f_y$ .



Australasian steel grades given in Table 2 comply with the scope of application for X-HVB connectors given in the European Technical Agreement [1].

Steel grade	Thickness	Yield strength $f_y$ (MPa)
350	$t_f \leq 11$ mm	360
	$11 < t_f < 40$ mm	340
	$40$ mm $\leq t_f$	330
300	$t_f \leq 11$ mm	320
	$11 < t_f < 17$ mm	300
	$17$ mm $\leq t_f$	280

Table 2: Australasian steel grades

In the New-Zealand standard for steel structures [13] (see § 2.2.1 of the standard), the steel grades for hot-rolled profiles should comply with the requirements of one of the following standards:

- Australasian standard: AS/NZS 3679 Part 1 (see *Table 2*);
- British standard, which is actually the European standard EN 10025 (see *Table 3*);
- Japanese standard JIS G 3101

In the software, when the project is located in New Zealand, only the two first options are available and Japanese standard is not covered.

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: European steel grades according to the British standard*

Whatever the selected country, the custom steel grade is also available: the yield strength, defined by the user, should verify:

$$170 \text{ MPa} \leq f_y \leq 355 \text{ MPa}$$

By default, the steel grade is 350 according to the Australasian standard AS/NZS 3779.

**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 other steel properties are obtained as follows:

$E_s$  is the elastic modulus:  $E_s = 200\,000$  MPa (see § 1.1.7 of [7]);

$G_s$  is the shear modulus:  $G = 80\,000$  MPa (see §1.4 of [11]);

$\rho_s$  is the steel density:  $\rho_s = 7850$  kg/m<sup>3</sup>

### 3.6. Concrete slab

#### 3.6.1. Geometry

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

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

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

$h_{sl}$  is the overall slab thickness. The minimum slab thickness, depending on the connector type and the effect of corrosion, is given in *Table 4*, according to the ETA-15/0876 [1].

According to AS/NZS 2327 § 2.2.1, the overall slab thickness should verify

$$300 \geq h_{sl} \geq 90 \text{ mm} \quad (1)$$

**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 <sup>(1)</sup>
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 from ETA document*



*(1): Values given in Table 4 when concrete coverage is required correspond to the exposure classification A1 of both standards AS and NZ. There are consistent with the scope of use of X-HVB connectors.*

Requirements for the slab thickness given in *Table 4* depend on the presence or not of a concrete coverage of the connector, according to the ETA-15/0876 [1]. The absence of concrete coverage is nevertheless not consistent with the requirements of the Australasian standards. The possibility to use X-HVB connectors without concrete coverage is still available but with a warning message (“The use of X-HVB connectors without concrete coverage does not fulfil the requirements of AS and NZ standards”) given both in the interface and the calculation report.

Additionally, the requirements of ETA-15/0876 [1] for the smaller connectors are less restrictive than the condition given in the Australasian standards (see Formula (1)). It is possible to define a slab that does not fulfill the latter. In this case, a warning message is given in the interface and in the calculation report (“The slab thickness is in the scope of application of X-HVB connectors but does not fulfil the requirement of AS/NZS 2327 § 2.2.1”).

For solid slabs, it is also possible for the User to define a concrete haunch. Its width will be equal to the top flange width whereas its depth  $h_h$  is defined by the user. In the calculations (see § 7.3.1), concrete haunches are assumed to fulfil the requirements of § 3.6.2.7.2 of AS/NZS 2737:

- the side of the haunch should lie outside a line drawn at 45° from the outside edge of the connector;
- the nominal concrete cover from the side of the haunch to the connector should be not less than 50 mm;
- transverse reinforcing bars sufficient to transfer the longitudinal shear are provided in the haunch at not less than 40 mm clear below the surface of the connector that resists uplift.

### 3.6.2. Concrete



*References:*

AS 3600 [12]: § 3.1 NZS 3101 [14]: § 5.1

When the project is located in Australia, the software proposes the concrete strength grades of *Table 5*, according to the Australian standard for concrete structures [12]. These standards strength grades and their main properties are given below in *Table 5* (acc. to Table 3.1.2 of [12]). The properties of concrete are defined by the following parameters:

$\rho_c$  is the density (default value 2400 kg/m<sup>3</sup> for normal concrete and 1800 kg/m<sup>3</sup> for light-weight concrete). Range value is [1750 kg/m<sup>3</sup> – 2000 kg/m<sup>3</sup>] for light-weight concrete and [2100 kg/m<sup>3</sup> – 2800 kg/m<sup>3</sup>] for light weight concrete

In the software, the range of value for concrete density will be [1750 kg/m<sup>3</sup> – 2800 kg/m<sup>3</sup>]. If the value entered by the user is lower than 2000 kg/m<sup>3</sup>, a lightweight concrete is assumed and a message “light weight concrete” is displayed on the interface.

$E_c$  is the mean value of the modulus of elasticity at 28 days;

$f'_c$  is the characteristic compressive (cylinder) strength at 28 days;

$f_{cm}$  is the mean value of the cylinder strength.

Label	$f'_c$ (MPa)	$f_{cm}$ (MPa)	$f_{cmi}$ (MPa)	$E_c$ (MPa)
20 MPa	20	25	22	24 000
25 MPa	25	31	28	26 700
32 MPa	32	39	35	30 100
40 MPa	40	48	43	32 800
50 MPa	50	59	53	34 800

Table 5: Concrete strength grades



*The list of concrete strength grades has been selected in order to comply with the requirements of ETA for X-HVBs [1]. Australasian Standard [7] for composite structures has a larger scope of application.*

*Table 5 is identical to Table 3.1.2 of [12]. Values given in this Table are relevant only for  $\rho_c = 2400 \text{ kg/m}^3$ . These values are not directly used by the DLL. Only the formulas given in this report are used for the assessment of  $f_{cmi}$  and  $E_c$ .*

The modulus of elasticity for concrete  $E_c$  is calculated by the software from the following formula:

$$E_c = \rho_c^{1,5} (0,043 \sqrt{f_{cmi}}) \quad \text{when } f_{cmi} \leq 40 \text{ MPa} \quad (2)$$

$$E_c = \rho_c^{1,5} (0,024 \sqrt{f_{cmi}} + 0,12) \quad \text{when } f_{cmi} > 40 \text{ MPa} \quad (3)$$

where  $f_{cmi}$  is the mean value of the in-situ compressive strength of concrete, that is given in AS/NZS Table 3.6.2.3, by:

$$f_{cmi} = 0,9 (1,2875 - 0,001875 f'_c) f'_c \quad (4)$$

Formulas (2) and (3) are expressed in MPa.



According to the Australian standards, Formula (2) is used for  $f_{cmi} = 40$  MPa. But the values given in Table 3.1.2 of the standards (cf. Table 5) corresponds to Formula (3).

For application in New Zealand, the same predefined list of concrete strength grade is proposed (Table 5). In this case, according to New-Zealand standard for concrete structures [14], the modulus of elasticity is obtained by the formula:

$$E_c = \left( \frac{\rho_c}{2300} \right)^{1,5} (4700 \sqrt{f'_c}) \quad (5)$$

For both countries, it is possible to choose a custom concrete, defined directly by its characteristic compressive strength  $f'_c$ , which should then verify:

$$20 \text{ MPa} \leq f'_c \leq 50 \text{ MPa} \quad (6)$$

### 3.7. 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

$f_{yp}$  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  $n_{rib} = 2$ )



For the profiled steel sheeting "Comflor 80", a specific treatment is applied by the software for the calculation of the shear resistance – see § 7.3.2 – and for the connectors arrangement – see § 3.11.3.

According to the requirements of AS/NZS § 3.6.2.7.3, the minimum width of troughs filled with concrete should fulfil the following condition:

$$\min(b_b; b_t) \geq 50 \text{ mm} \quad (7)$$

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 through axis;
- Stiffeners are assumed to be identical and the stiffener arrangement is identical for every trough;
- The shape of the stiffener is displayed as triangular or trapezoidal according to the dimensions of the trough.

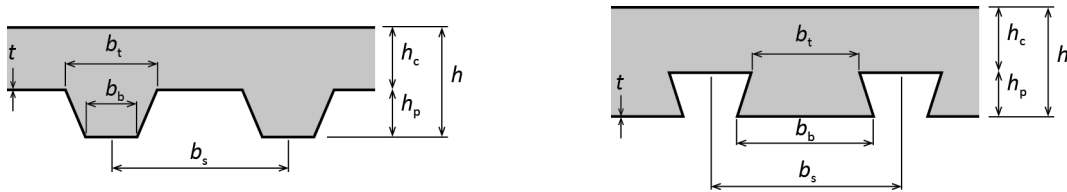


Figure 4: Dimensions of a profiled steel sheeting



a) Rib with one stiffener

b) Rib with two stiffeners

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.

According to AS/NZS 2327 § 2.2.1, the total height above the ribs should fulfil the following conditions:

$$h_c = h_{sl} - h_p \geq h_{c,min} \quad (8)$$

$$h_{sl} \leq 4 h_c \quad (9)$$

where  $h_{c,min}$  is the minimal thickness of concrete above the main flat surface of the top of the ribs of the sheeting, as defined in *Table 6*.

Overall depth of the slab $h_{sl}$	Required depth of concrete above the steel sheeting $h_{c,min}$
$h_{sl} \leq 175$ mm	$h_{c,min} = 40$ mm
$175 < h_{sl} \leq 225$ mm	$h_{c,min} = 45$ mm
$225 < h_{sl} \leq 300$ mm	$h_{c,min} = 50$ mm

Table 6: Minimal thickness of concrete above the profiled steel sheeting (AS/NZS 2327:2017 Table 2.2.1)

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

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

- $0 \leq G_{deck} \leq 50 \text{ kg/m}^2$
- $170 \text{ MPa} \leq f_{ypk} \leq 600 \text{ MPa}$

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

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

Error code **32** is returned when the conditions given in Table 6 are not met.

### 3.8. Steel reinforcement



**References:**

AS 3600 [12]: § 3.2 NZS 3101 [14]: § 5.3

Both Australian and New-Zealand standards refer to Australasian standard AS/NZS 4671 for the properties of steel reinforcement. Australian standard explicitly proposes a list of steel grades for reinforcement (see Table 3.2.1). This list is proposed in the X-HVB software:

- R250 (covering R250N)
- D500 (covering D500L and D500N)

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

Steel grade	R250	D500
$f_{sy}$ [MPa]	250	500

Table 7 : Yield strength of the reinforcement steel

Grade D500 is the default value.

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

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



### 3.9. Shear connection

#### 3.9.1. General parameters

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. Dimensions used in calculations and for drawings of connectors are (see *Table 8* for values):

$h_{sc}$  is the total height

$w_b$  is the bottom length

$w$  is the transverse width

The shear resistance of the connection is defined by:

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

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

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 – According to ETA [1]*



The values of  $P_{Rd}$  and  $P_{Rk}$  in Table 8 verify the equation:  $P_{Rd} = \phi_{sc} P_{Rk}$ , where  $\phi_{sc}$  is defined in § 7.2.

For Comflor 80 steel deck transverse with beam, only X-HVB 140 connectors are possible. Specific values of design resistance are then used (see § 7.3.2).

The consistency of the connectors with the concrete slab and with the profiled steel deck (if any) is checked according to (see error codes below):

- Table 4 for the height of the slab;
- Table 9 for the height of the profiled steel deck.

**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 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.*



*The check of the conditions for the maximal height of the profiled steel deck given in Table 9 is applied without taking into account the potential presence of an upper stiffener.*

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 according to ETA [1]

For composite decking perpendicular to the beam axis with connectors parallel with the beam axis, the following additional condition must be fulfilled (see ETA [1] Note x in Annex B4):

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

Where:  $b_0 = (b_t + b_b)/2$  if  $b_t \geq b_b$   
 $b_0 = b_t$  if  $b_t < b_b$

### 3.9.2. 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.

### 3.9.3. Degree of connection

As a plastic design is performed by the software (see § 3.3), the user has to choose the type of connection for the design and 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 partial connection, the minimum degree of shear connection as given in § 7.5.4 applies. For a partial connection, the user has the possibility (release May 2019) to impose directly a degree of connection (third choice). In this case, it is not necessary to fulfil the minimum requirement on the degree of connection. The relevant warning message is then given in the calculation sheet.

### 3.9.4. 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 design code 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 design code 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 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.

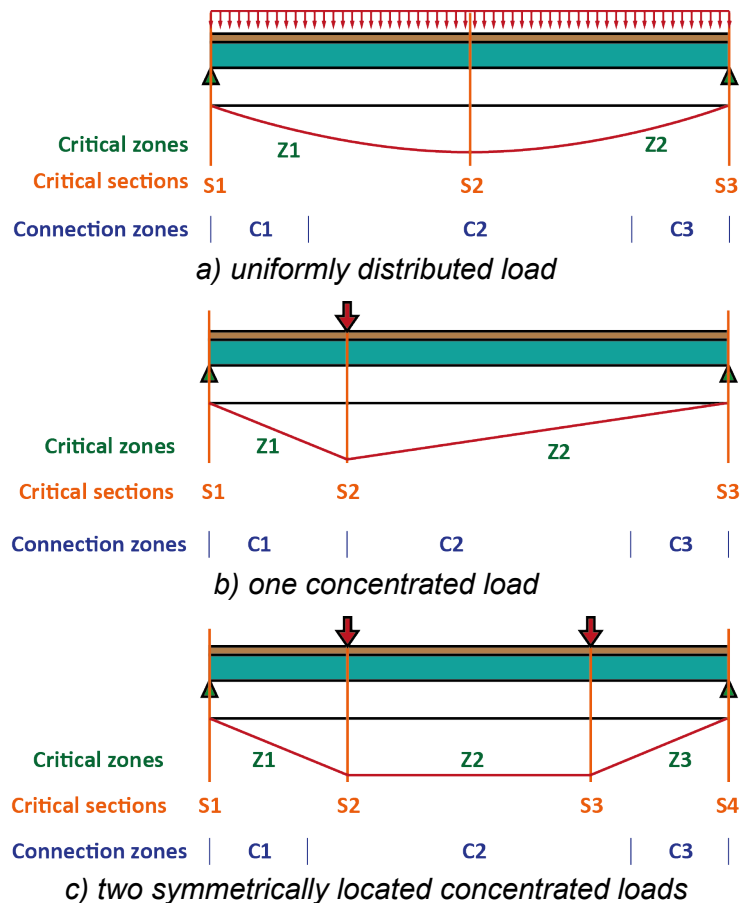


Figure 6 : Configurations of critical sections and connection zones.

### 3.9.5. 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 § 3.5.1);
- the possibility to have only one row of connectors in profiled deckings with narrow ribs (see § 3.11.2).

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.

Both scope extensions also apply with Australasian Standards.

### 3.10. Spacing and positioning of connectors along the beam

According to ETA [1], the maximum centre-to-centre spacing of connectors along the beam shall not exceed four times the total slab thickness or 600 mm:

$$d \leq \min\{4 h; 600 \text{ mm}\} \quad (10)$$



Clause 3.6.4.1 (b) of [7] gives the same requirement.

### 3.11. Spacing and positioning of connectors perpendicularly to the beam axis

#### 3.11.1. Solid slab with multiple rows of connectors

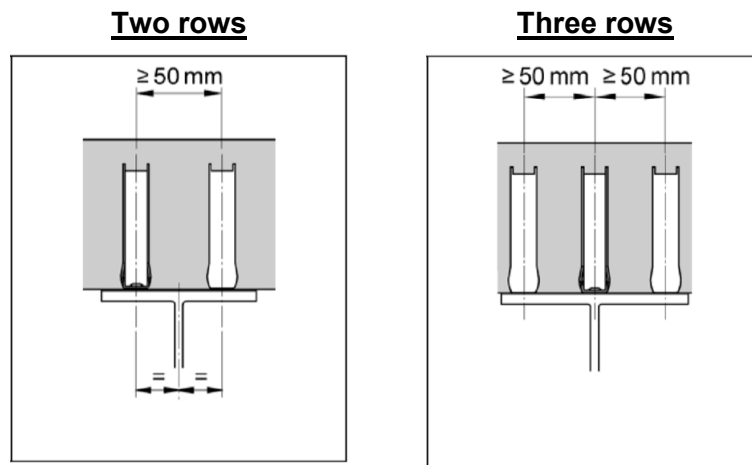


Figure 7 : Spacing of connectors for solid slabs

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

$$b \geq 50 (n_r - 1) + w \quad (11)$$

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

This condition may limit the maximum number of connectors in a row – see § 7.5.1.

### 3.11.2. Slabs with transverse steel decking and connectors parallel to the beam axis

- **Single row, without rib stiffeners**

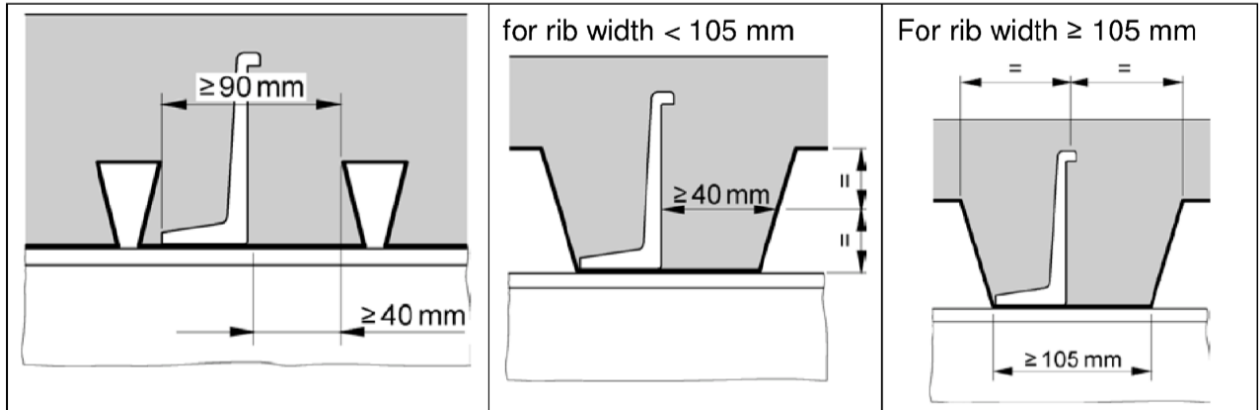


Figure 8 : Spacing of connectors for slabs with transverse decking and connectors parallel with the beam (single row).

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

$$b_0 \geq \max\{w_b + 40 \text{ mm}; 90 \text{ mm}\} \quad (12)$$

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

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

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

- **Single row, with rib stiffeners**

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

$$\min\{b_b, b_t\} \geq 2w_b + s_{un} \quad (13)$$

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

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

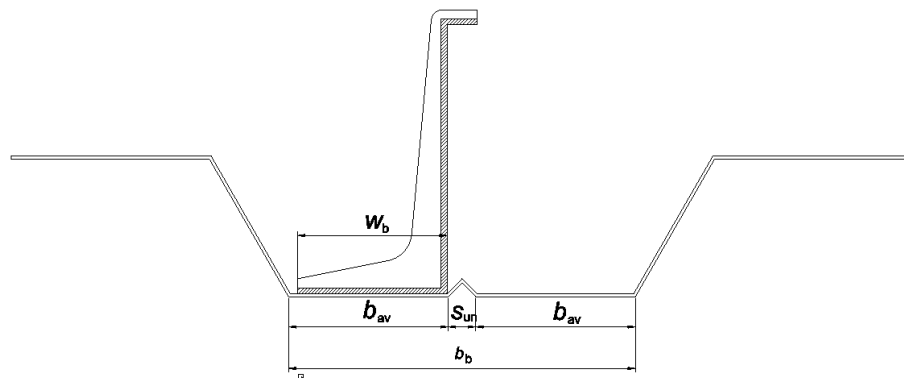


Figure 9 : Transverse sheeting with one rib stiffener and parallel connectors.

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:

$$\min\{b_b, b_t\} \geq 2w_b + 2s_{un} + s_{av} \tag{14}$$

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

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

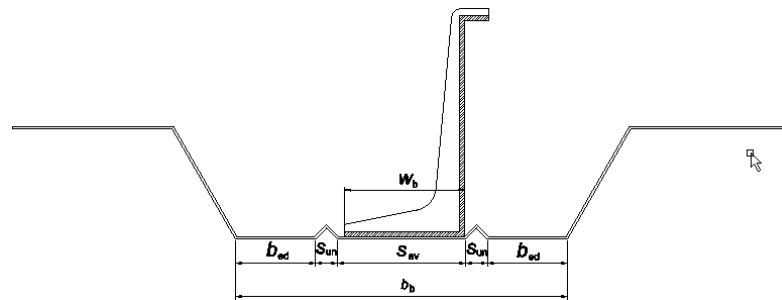


Figure 10 : Transverse sheeting with two rib stiffeners and parallel connectors.

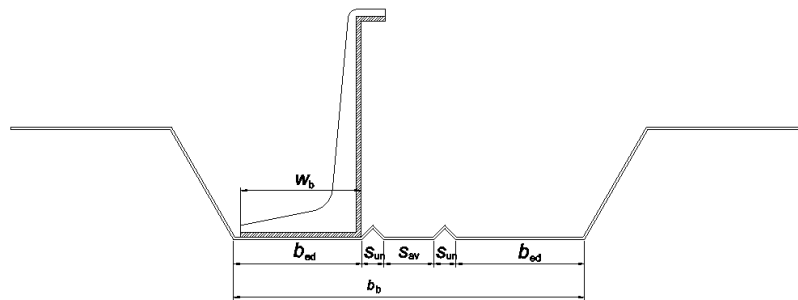


Figure 11 : Transverse sheeting with two rib stiffeners and parallel connectors.

▪ **Multiple rows, without rib stiffener**

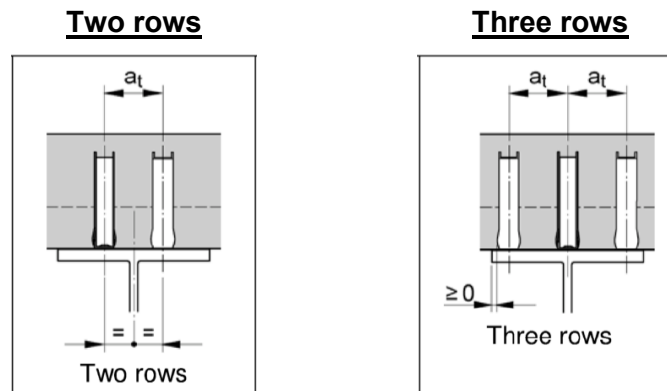


Figure 12 : Spacing of connectors for slabs with transverse decking and connectors parallel with the beam.

The following conditions must be fulfilled:

- For the flange width of the steel section:

$$b_f \geq (n_r - 1)a_t + w \quad (15)$$

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 § 7.5.1.

- For the bottom width of the rib:

$$b_b \geq 60 \text{ mm} \quad (16)$$

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

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

- **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.

### 3.11.3. Slabs with transverse steel decking and connectors transverse to the beam axis

- **Single row, without rib stiffener**

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

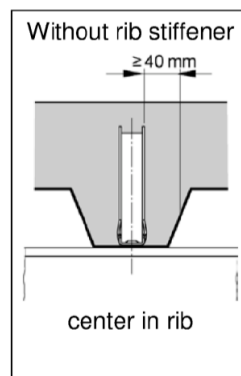


Figure 13 : Spacing of connectors for slabs with transverse decking and connectors transverse to the beam (single row).

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

$$b_0 \geq w + 80 \text{ mm} \quad (17)$$

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

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

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

- $b_b > w$
- $b_r > w$

When the 2 following conditions are met, it is not possible to have a single row but it is possible to have multiple rows:



$$\begin{aligned} b_0 &< w + 80 \text{ mm} \\ b_b &\geq 40 \text{ mm} \end{aligned} \quad (18)$$

In this case, the calculation module starts the design process by considering 2 connectors in a row (see § 7.5.1).

The minimum width of ribs for one connector can be reduced when the relevant software option is activated (see § 3.9.5). This option extended the scope of the ETA report [1]. In this case, following conditions are checked:

$$\begin{aligned} b_t &\geq 30 \text{ mm} \\ b_b &\geq 30 \text{ mm} \end{aligned} \quad (19)$$

▪ **Single row, with rib stiffeners**

For sheetings with one rib stiffener (see *Figure 14*), according to ETA report [1] case b, only the following condition must be fulfilled:

$$\min\{b_b, b_t\} \geq 2w + s_{un} \quad (20)$$

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

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

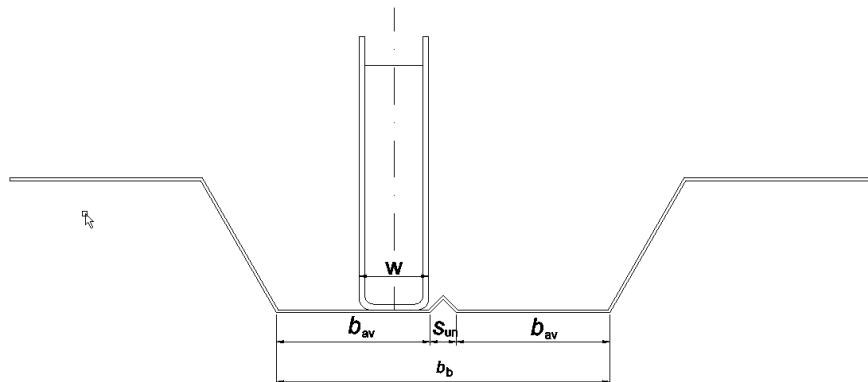


Figure 14 : Transverse sheeting with one rib stiffener and transverse connectors.

According to cases a) and b) given in page 14 of Annex B7 in ETA report [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:

$$\min\{b_b, b_t\} \geq 2w + 2s_{un} + s_{av} \quad (21)$$

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

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

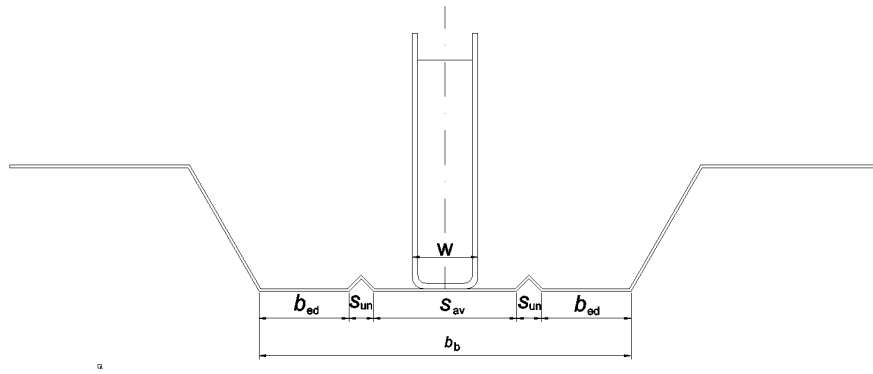


Figure 15 : Transverse sheeting with two rib stiffeners and transverse connectors.

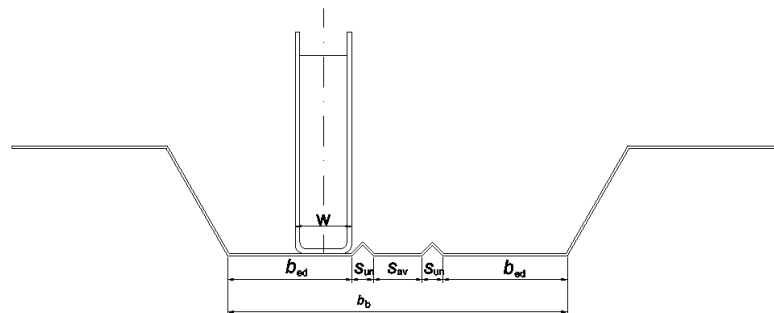


Figure 16 : Transverse sheeting with two rib stiffeners and transverse connectors.

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*”.

▪ **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:

$$b_f \geq (n_r - 1)a_t \tag{22}$$

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 § 7.5.1.

- For the bottom width of the rib:

$$b_b \geq 40 \text{ mm} \tag{23}$$

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

- For the bottom width of connectors:

$$a_t \geq 2w_b$$

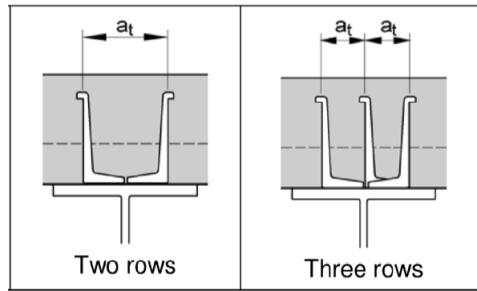


Figure 17 : Spacing of connectors for slabs with transverse decking and connectors transverse to the beam (multiple rows).

- **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.

For Comflor 80 deck, when  $n_r = 3$  or 4, the condition (23) is used with  $a_t = 65 \text{ mm}$  (according to [2]).

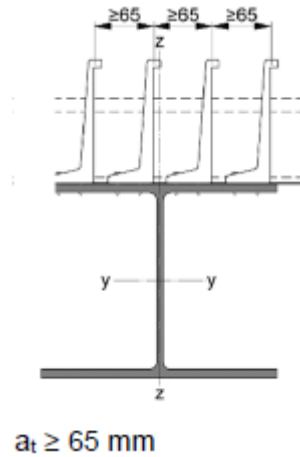


Figure 18 : Specific positioning of 4 transverse connectors with Comflor 80 deck

### 3.11.4. Slabs with parallel steel decking and connectors parallel to the beam axis

- **Single row, without rib stiffener**

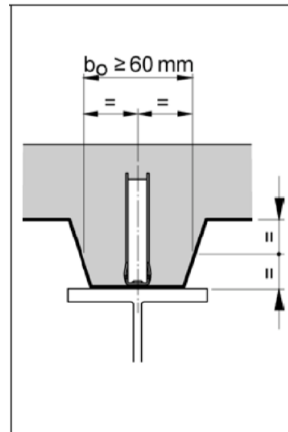


Figure 19 : Spacing of connectors for slabs with transverse decking and connectors transverse to the beam (multiple rows).

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

$$b_0 \geq 60 \text{ mm} \quad (24)$$

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

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

- **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 20):

$$\begin{aligned} b_{b,\text{split}}: & \text{ width of the rib at the bottom, given by: } & b_{b,\text{split}} &= b_f \\ b_{t,\text{split}}: & \text{ width of the rib at the top, given by: } & b_{t,\text{split}} &= b_t + (b_f - b_b) \end{aligned}$$

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

$$b_f/2 \geq s + w/2, \text{ where } s = (b_b + s_{\text{un}})/2 \quad (25)$$

$$b_{0,\text{split}} \geq 60 \text{ mm, where } b_{0,\text{split}} = b_0 + (b_f - b_b) \quad (26)$$

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

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:

$$s_{av} \geq w \quad (27)$$

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

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

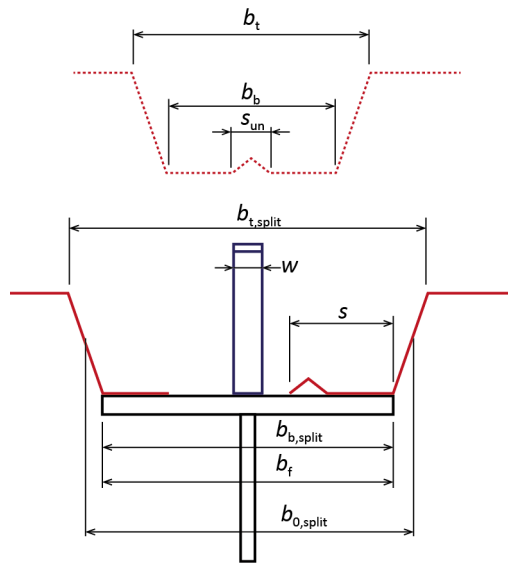


Figure 20 : Splitting of a rib with one rib stiffener (one row of parallel connectors).

▪ **Multiple rows, without rib stiffener**

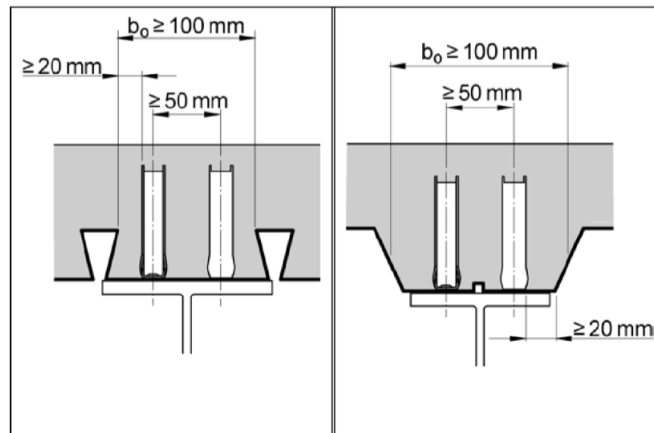


Figure 21 : Spacing of connectors for slabs with decking parallel with the beam (multiple rows).

The following conditions must be fulfilled:

- For the flange width of the steel section:

$$b_f \geq w_b + 50 \text{ mm} \tag{28}$$

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

$$\min\{b_b, b_t\} \geq w + 90 \text{ mm} \tag{29}$$

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

▪ **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:

$$b_f \geq w + \max\{50 \text{ mm}, s_{un} + w\} \quad (30)$$

- For the width of the rib:

$$\min\{b_b, b_t\} \geq w + 40 \text{ mm} + \max\{50 \text{ mm}, s_{un} + w\} \quad (31)$$

$$b_0 \geq 100 \text{ mm} \quad (32)$$

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

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 22), the connectors are located between the stiffeners. The same conditions as the case of sheetings without rib stiffener must be fulfilled.

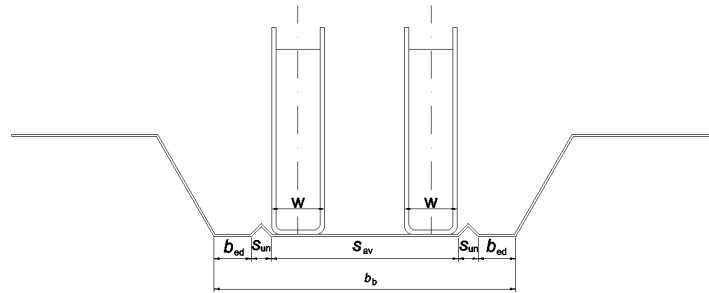


Figure 22 : Sheeting with two rib stiffeners and connectors parallel with the beam (multiple rows).

- 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:

$$b_f \geq w + \max\{50 \text{ mm} - w, s_{av} + 2 s_{un}\} \quad (33)$$

- For the width of the rib:

$$\min\{b_b, b_t\} \geq 2 w + 40 \text{ mm} + \max\{50 \text{ mm} - w, s_{av} + 2 s_{un}\} \quad (34)$$

$$b_0 \geq 100 \text{ mm} \quad (35)$$

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

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

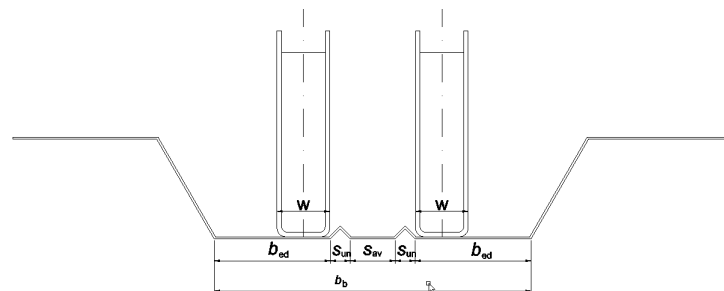


Figure 23 : Sheeting with two rib stiffeners and connectors parallel with the beam (multiple rows).

### 3.11.5. 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 § 7.5.1.

## 3.12. Loads

### 3.12.1. General definitions

The calculation module allows the user to define elementary variable load cases (according to AS/NZS 1170.1) that are used in the combinations of actions for ULS or SLS (according to AS/NZS 1170.0).

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, **Q<sub>1</sub>** and **Q<sub>2</sub>**

The dead weight of the steel profile and concrete slab is automatically calculated and added in the permanent load case. It can be overwritten by the user.

For each load case, it is 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 **Q<sub>1</sub>**, only the uniformly distributed surface load can be defined in the UI.

### 3.12.2. 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 interior beam”, a linear uniformly distributed load along the beam is derived:

$$q_{in} = q_{surf} (b_1 + b_2)/2 \quad (36)$$

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

$$q_{in} = q_{surf} (b_1 + b_2/2) \quad (37)$$

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):

$$P_{s,i} = [q_{surf} (L_{i+1} - L_{i-1})/2] (b_1 + b_2)/2 \quad (38)$$

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:

$$P_{s,0} = [q_{surf} L/(2(n_s + 1))] (b_1 + b_2)/2 \quad (39)$$

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

$$P_{s,i} = [q_{surf} (L_{i+1} - L_{i-1})/2] (b_1 + b_2)/2 \quad (40)$$

$$P_{s,0} = [q_{surf} L/(2(n_s + 1))] (b_1 + b_2)/2 \quad (41)$$

### 3.12.3. 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:

$$\text{for interior beams :} \quad q_{surf,d} = 2 \frac{g \rho_s A}{b_1 + b_2} \quad (42)$$

$$\text{for edge beams :} \quad q_{surf,d} = 2 \frac{g \rho_s A}{2 b_1 + b_2} \quad (43)$$

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

$\rho_s$ : see § 3.5.2

$A$  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:

$$q_{\text{slab}} = g \rho_c h_{\text{eq}} \quad (44)$$

where:  $\rho_c$ : see § 3.6.2

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

$$h_{\text{eq}} = h_{\text{sl}} \quad \text{for plain slabs}$$

$$h_{\text{eq}} = h_c + h_p \frac{b_t + b_b}{2 b_s} \quad \text{for composite slabs}$$

$h_p, b_t, b_b, b_s$ : see § 3.7

$h_c$ : see Formula (8)

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

$$q_{\text{deck}} = g G_{\text{deck}} \quad (45)$$

where:  $G_{\text{deck}}$ : see § 3.7

### 3.12.4. Default surface live loads

The default surface live load (for live load case  $Q_1$ ) is defined according to the type of activity or occupancy of loaded area - see *Table 10* (from the values given in Table 3.1 of AS/NZS 1170.1).

Type of activities/occupancies	Uniform load (kN/m <sup>2</sup> )
A - Domestic and residential activities	2,0
B - Offices and work areas	3,0
C – Areas where people may congregate	3,0
D – Shopping areas	4,0
E – Warehousing and storage areas	5,0
F - Roofs	Formula (46)

*Table 10 : Default surface live load*

The default uniform live load on roofs is obtained by (according to case R2 (i) in Table 3.2 of AS/NZS 1170.1):

$$q = \frac{1,8}{A} + 0,12 \geq 0,25 \text{ kN/m}^2 \quad (46)$$

where  $A$  is calculated according to § 3.12.5.

### 3.12.5. Live load reduction

When the associated option is activated, the value of the Live load considered in the calculation is reduced by the reduction factor  $\psi_a$  obtained as follows, according to AS/NZS 1170.1 § 3.4.2 (b):

$$\psi_a = 0,3 + \frac{3}{\sqrt{A}} \quad (47)$$

but :  $0,5 \leq \psi_a \leq 1,0$

where:  $A$  is the sum of the areas supported by the composite beam (in  $m^2$ ), given by:

for interior beams:  $A = L (b_1 + b_2)/2$

for edge beams:  $A = L (b_1 + b_2/2)$

## 4. COMBINATIONS OF LOADS

---

### 4.1. Ultimate Limit States (ULS)

The ULS combinations are automatically generated according to AS/NZS 1170.0 §4.2.2 as follows:

- when only one live load is defined

$$\begin{aligned} &1,35 G \\ &1,2 G + 1,5 Q \end{aligned} \tag{48}$$

- when two live loads are defined (second interpretation of the standard [7])

$$\begin{aligned} &1,35 G \\ &1,2 G + 1,5 Q_1 + 1,5 Q_2 \end{aligned} \tag{49}$$



*It is possible for the User to modify the combinations factors.*

For the construction stage, if the load case  $Q_c$  has been defined, following ULS combinations is considered:

$$\begin{aligned} &1,35 G \\ &1,2 G + 1,5 Q_c \end{aligned} \tag{50}$$

### 4.2. Serviceability Limit States (SLS)

The SLS combinations for the verification of deflections are automatically generated as follows, when only one live loads case is defined:

$$\begin{aligned} \text{total deflection:} & \quad G + Q_1 \\ \text{incremental deflection:} & \quad p_G G + 1,0 Q_1 \end{aligned} \tag{51}$$

When two live loads cases are defined:

$$\begin{aligned} \text{total deflection:} & \quad G + Q_1 + Q_2 \\ \text{incremental deflection:} & \quad p_G G + 1,0 (Q_1 + Q_2) \end{aligned} \tag{52}$$

By default, the fraction of permanent loads is:  $p_G = 0$ .

The combinations for the calculation of the fundamental natural frequency are automatically generated as follows:

$$\begin{aligned} \text{One live loads case:} & \quad G + p_Q Q_1 \\ \text{Two live loads cases:} & \quad G + p_Q (Q_1 + Q_2) \end{aligned} \tag{53}$$

By default, the fraction of live loads is:  $p_Q = 0,20$ .



*It is possible for the User to modify the fraction of the permanent load used for the incremental deflection check and the fraction of live loads used for the calculation of the fundamental natural frequency.*

## 5. GLOBAL ANALYSIS

### 5.1. 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.

### 5.2. Critical sections

ULS verifications are carried out at critical sections (AS/NZS § 3.5.3) where:

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

### 5.3. 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 two methods according to § 3.12.2.

#### 5.3.1. Point load

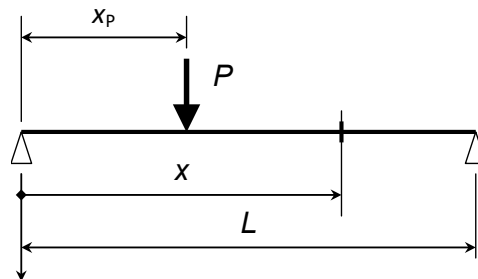


Figure 24 : Point load.

The reactions at supports are calculated by:

$$\begin{aligned} R_L &= -P (L - x_p) / L && \text{at the Left support} \\ R_R &= -P x_p / L && \text{at the Right support} \end{aligned}$$

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:

$$\begin{aligned} V(x) &= R_L && \text{if } x < x_P \\ V(x) &= -R_R && \text{if } x > x_P \end{aligned}$$

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

$$\begin{aligned} M(x) &= -R_L x && \text{if } x < x_P \\ M(x) &= -R_R(L - x) && \text{if } x > x_P \end{aligned}$$

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

$$\begin{aligned} w(x) &= \frac{F}{6EIL} [L^2 - (L - x_P)^2 - x^2](L - x_P)x && \text{if } x < x_P \\ w(x) &= \frac{F}{6EIL} [L^2 - (L - x)^2 - x_P^2](L - x)x_P && \text{if } x > x_P \end{aligned}$$

### 5.3.2. Patch load

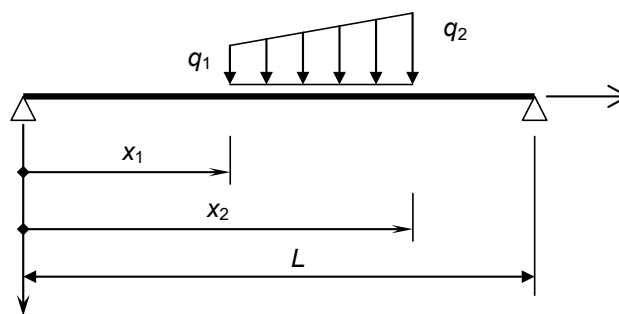


Figure 25 : Patch load.

The reactions at supports are calculated by:

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

Where  $x_1$ ,  $q_1$ ,  $x_2$  and  $q_2$  define the distributed load as shown in *Figure 25*.

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

$$\begin{aligned} V(x) &= R_L && \text{if } x < x_1 \\ V(x) &= R_L + \left[ q_1 + \frac{q_2 - q_1}{2} \left( \frac{x - x_1}{x_2 - x_1} \right) \right] (x - x_1) && \text{if } x_1 \leq x \leq x_2 \\ V(x) &= -R_R && \text{if } x > x_2 \end{aligned}$$

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

$$\begin{aligned} M(x) &= -R_L x && \text{if } x < x_1 \\ M(x) &= -R_L x - \left[ 3q_1 + (q_2 - q_1) \left( \frac{x - x_1}{x_2 - x_1} \right) \right] \frac{(x - x_1)^2}{6} && \text{if } x_1 \leq x \leq x_2 \\ M(x) &= -R_R(L - x) && \text{if } x > x_2 \end{aligned}$$

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

$$\begin{aligned} w(x) &= \frac{1}{EI} \left( R_L \frac{x^3}{6} + A_1 x + B_1 \right) && \text{if } x < x_1 \\ w(x) &= \frac{1}{EI} \left( -R_R \frac{x^3}{6} + R_R \frac{Lx^2}{2} + A_3 x + B_3 \right) && \text{if } x > x_2 \\ w(x) &= \frac{1}{EI} (w_0(x) + A_2 x + B_2) && \text{if } x_1 \leq x \leq x_2 \end{aligned}$$

Where:

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

#### 5.4. 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 preambering has been defined by the user, the following deflection is added in each cross-section:

$$w_{0,x} = -4 w_0 \frac{x}{L} \left( 1 - \frac{x}{L} \right) \quad (54)$$

Where:  $w_0$  is the preambering.

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

#### 5.5. Influence of the connectors slip



**References:**  
AS/NZS 2327 § 3.5.5

The influence of the connector slip on the beam deflection is dealt with according to the simplified method provided by Formulas 3.10.3.2 (1) and (2) of AS/NZS 2327. For beams with partial connection, all deflections are increased by one of the following factor:

$$\text{for propped beams during construction : } k_{pc} = 1 + 0,5 (1 - \beta_{cr} \beta) \left( \frac{\delta_s}{\delta_c} - 1 \right) \quad (55)$$

$$\text{for unpropped beams during construction : } k_{pc} = 1 + 0,3 (1 - \beta_{cr} \beta) \left( \frac{\delta_s}{\delta_c} - 1 \right) \quad (56)$$

where:  $\beta$  is the degree of connection of the beam (see § 7.5.6);

$\beta_{cr}$  is the creep deflection coefficient:  $\beta_{cr} = 1$ ;

$\delta_s$  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.

## 6. VERIFICATIONS AT THE CONSTRUCTION STAGE

### 6.1. General

Verifications at the construction stage are carried out only when the beam is unpropped. The beam is checked for Ultimate State of Strength, which include the following verifications:

- Section bending moment capacity
- Member capacity (resistance to lateral torsional buckling)
- Web resistance to vertical shear
- Interaction of vertical shear and bending

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.

No SLS checks apply at construction stage. The software will nevertheless provide the deflection under SLS combination.

### 6.2. Strength verifications

#### 6.2.1. Capacity factors



*References:*

*AS 4100 [11]: Table 3.4      NZS 3404 [13]: Table 3.3*

Design capacity for	§	Capacity factor
Bending moment capacity	6.2.3	$\phi_M = 0,9$
Member capacity	6.2.4	$\phi_{Mb} = 0,9$
Shear resistance of the web	6.2.5	$\phi_v = 0,9$
Shear and bending interaction	6.2.6	$\phi_{vm} = 0,9$

*Table 11: Capacity factors*

These values are predefined and it is possible to modify them in the settings of the project.

## 6.2.2. Section slenderness



References:

AS 4100 [11]: § 5.2.2 NZS 3404 [13]: § 5.2.2

The slenderness of the upper flange (in uniform compression) is obtained by:

$$\lambda_{ef} = \frac{(b_f - t_w)/2}{t_f} \sqrt{\frac{f_y}{250}} \quad (57)$$

The section plasticity and yield slenderness limits ( $\lambda_{efp}$  and  $\lambda_{efy}$  respectively) of the flange are given in *Table 12* (from Table 5.2 of [11] for Australia and from Table 5.2 of [13] for New Zealand).

Type of profile	Plasticity limit $\lambda_{efp}$	Yield limit $\lambda_{efy}$
Profiles from the database (hot-rolled)	9	16
Custom profiles (built-up heavily welded)	8	14

Table 12: Slenderness limits for flanges acc. to Australian and New Zealand standards



Differently from Eurocodes, the slenderness is assessed by taking into account the clear width of the panel from the face of the supporting plate. It thus includes the fillets.

The slenderness of the web (bending, with compression at one edge and tension at the other) is defined by:

$$\lambda_{ew} = \frac{h_t - 2 t_f}{t_w} \sqrt{\frac{f_y}{250}} \quad (58)$$

The section plasticity and yield slenderness limits ( $\lambda_{ewp}$  and  $\lambda_{ewy}$  respectively) of the web are given in *Table 13*.

Type of profile	Plasticity limit $\lambda_{ewp}$	Yield limit $\lambda_{ewy}$
from Table 5.2 of [11] for Australia		
all sections	82	115
from Table 5.2 of [13] for New Zealand		
all section	82	180 (*)

(\*) acc. to note (5) in Table 5.2 of [13] for doubly symmetric I section

Table 13: Slenderness limits for flanges acc. to Australian and New Zealand standards

The section slenderness  $\lambda_s$  and the slenderness limits  $\lambda_{sp}$  and  $\lambda_{sy}$  are defined as follows:

- if  $\frac{\lambda_{ew}}{\lambda_{ewy}} \geq \frac{\lambda_{ef}}{\lambda_{efy}}$

$$\lambda_s = \lambda_{ew}$$

$$\lambda_{sp} = \lambda_{ewp}$$

$$\lambda_{sy} = \lambda_{ewy}$$

- if  $\frac{\lambda_{ew}}{\lambda_{ewy}} < \frac{\lambda_{ef}}{\lambda_{efy}}$ 
  - $\lambda_s = \lambda_{ef}$
  - $\lambda_{sp} = \lambda_{efp}$
  - $\lambda_{sy} = \lambda_{efy}$

The section is classified as follows:

- the section is compact if:  $\lambda_s \leq \lambda_{sp}$
- the section is non-compact if:  $\lambda_{sp} < \lambda_s \leq \lambda_{sy}$
- the section is slender if:  $\lambda_s > \lambda_{sy}$

Slender cross-sections are excluded from the scope of the X-HVB software (similarly as Class 4 cross-sections with Eurocodes and slender section with US standards).

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

*If a cross-section is classified as slender, other calculations is not performed and an error message is displayed in the UI (error code = 11).*

### 6.2.3. Section bending moment capacity



**References:**

AS 4100 [11]: § 5.2.1 NZS 3404 [13]: § 5.2.1

The criterion for the verification of section to the bending moment about the principal axis is:

$$\Gamma_{Ms} = M^*/(\phi_M M_s) \leq 1.0 \quad (59)$$

where  $M^*$  is the design bending moment;

$\phi_M$  is the capacity factor (strength reduction factor) for bending resistance – see *Table 11*;

$M_s$  is the nominal section moment capacity, obtained by:

$$M_s = Z_e f_y \quad (60)$$

$Z_e$  is the effective section modulus, calculated according to the section slenderness.

For compact section,  $Z_e = S$  where  $S$  is the plastic section modulus.

For a non-compact section,  $Z_e$  is given by:

$$Z_e = Z + (S - Z) \frac{\lambda_{sy} - \lambda_s}{\lambda_{sy} - \lambda_{sp}} \quad (61)$$

where  $S$  and  $Z$  are the plastic and elastic section moduli respectively.



*It is assumed here that for all profiles of the database, the plastic section modulus  $S$  is always lower that 1,5 times the elastic modulus  $Z$ .*



#### 6.2.4. Member capacity (resistance to lateral torsional buckling)



References:

AS 4100 [11]: § 5.6.1.1      NZS 3404 [13]: § 5.6.1.1  
(considering an open section with equal flanges)

The criterion for the verification of the steel beam to lateral torsional buckling is:

$$\Gamma_{LT} = M_m^*/(\phi_{Mb} M_b) \leq 1.0 \quad (62)$$

where  $\phi_{Mb}$  is the capacity factor for LTB resistance – see Table 11;

$M_b$  is the nominal member moment capacity, obtained by:

$$M_b = \alpha_m \alpha_s M_s \leq M_s \quad (63)$$

$\alpha_m$  is a moment modification factor, given by:

$$\alpha_m = \frac{1,7 M_m^*}{\sqrt{(M_2^*)^2 + (M_3^*)^2 + (M_4^*)^2}} \leq 2,5 \quad (64)$$

$M_m^*$  is the maximum bending moment in the segment;

$M_2^*$ ,  $M_4^*$  are the design bending moments at the quarter point of the segment (i.e. the beam);

$M_3^*$  is the design bending moment at the midpoint of the segment (i.e. the beam);

$\alpha_s$  is a slenderness reduction factor, given by:

$$\alpha_s = 0,6 \left( \sqrt{\left(\frac{M_s}{M_o}\right)^2 + 3} - \frac{M_s}{M_o} \right) \quad (65)$$

$M_s$  is the nominal section moment capacity as obtained in 6.2.3;

$M_o$  is the reference buckling moment given by:

$$M_o = \sqrt{\frac{\pi^2 E_s I_y}{L_e^2} \left( G_s J + \frac{\pi^2 E_s I_w}{L_e^2} \right)} \quad (66)$$

$E_s$ ,  $G_s$ : elastic moduli (see § 3.5.2)

$I_y$  is the second moment of area of the steel cross-section about the minor principal axis;

$I_w$  is the warping constant, see Annex A;

$J$  is the torsion constant, see Annex A;

$L_e$  is the effective length, given by:

$$L_e = k_t k_l k_r L \quad (67)$$

$L$  is the beam length

$k_t$  is the twist restraint factor:  $k_t = 1$

$k_l$  is the load height factor:  $k_l = 1$

$k_r$  is a lateral rotation restraint factor:  $k_r = 1$



For the values of  $k_t$ ,  $k_l$  and  $k_r$ , the beam is assumed to be laterally restrained at both ends.

### 6.2.5. Shear capacity of webs



References:

AS 4100 [11]: § 5.11 NZS 3404 [13]: § 5.11

The shear force is resisted by the profile web. The verification criterion is:

$$\Gamma_{VS} = V^*/(\phi_V V_V) \leq 1.0 \quad (68)$$

where  $V^*$  is the design shear force in the web;

$\phi_V$  is the capacity factor (strength reduction factor) for shear resistance – see *Table 11*;

$V_V$  is the nominal shear capacity of the web.

As the steel profiles are bi-symmetrical, a uniform shear stress distribution may be assumed and the nominal shear capacity is obtained from the geometrical slenderness of the web as follows.

- Compact webs:

The web of the profile is compact when the following condition is fulfilled:

$$\frac{d_p}{t_w} \leq \frac{82}{\sqrt{f_y}} \quad (69)$$

where:  $d_p$  is the clear transverse dimension of the web panel, given by:

$$d_p = h_t - 2(t_f + r_1) \text{ for hot-rolled profiles}$$

$$d_p = h_t - 2t_f \text{ for built-up sections}$$

$t_w$  is the thickness of the web.

For compact webs, the nominal shear capacity is obtained by the following formula:

$$V_V = V_W = 0,6 A_w f_y \quad (70)$$

where:  $A_w$  is the gross sectional area of the web, given by:  $A_w = (h_t - 2t_f)t_w$

- Non-compact webs:

The web of the profile is non-compact when the condition (69) is not fulfilled. In this case, the nominal shear capacity is given by (shear buckling resistance, assuming unstiffened webs):

$$V_V = V_b = \alpha_V V_W \leq V_W \quad (71)$$

where:  $\alpha_V$  is a reduction factor calculated by:

$$\alpha_V = 82^2 \frac{250}{f_y} \left( \frac{t_w}{d_p} \right)^2 \quad (72)$$

### 6.2.6. Interaction of shear and bending



**References:**

AS 4100 [11]: § 5.12 NZS 3404 [13]: § 5.12

The shear and bending interaction is verified through the following criterion:

$$\Gamma_{vms} = V^*/(\phi_{vm} V_{vm}) \leq 1.0 \quad (73)$$

where  $V^*$  is the design shear force in the web;

$\phi_{vm}$  is the capacity factor (strength reduction factor) for shear resistance – see Table 11;

$V_{vm}$  is obtained by:

$$V_{vm} = V_v \quad \text{for } M^* \leq 0,75 \phi_{vm} M_s \quad (74)$$

$$V_{vm} = V_v \left( 2,2 - \frac{1,6 M^*}{\phi_{vm} M_s} \right) \quad \text{for } 0,75 \phi_{vm} M_s \leq M^* \leq \phi_{vm} M_s \quad (75)$$

$V_v$  is the nominal shear capacity of the web – see § 6.2.5;

$M_s$  is the nominal section moment capacity – see § 6.2.3;

$M^*$  is the design bending moment in the cross-section.



*The second method of §5.12 of the standard is applied. It is a more general rule.*

## 7. 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

The SLS verifications include:

- Total deflection
- Incremental deflexion
- Control of the vibration comfort

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

### 7.1. Effective width of the concrete slab

The effective width of the concrete slab is determined according to AS/NZS 2327 [7] § 3.4.2.1:

$$\begin{aligned} \text{for } x \leq L/4 & \quad b_{\text{eff}}(x) = b_e[\beta + 4(1 - \beta)x/L] + b_0 \\ \text{for } x \leq 3L/4 & \quad b_{\text{eff}}(x) = b_e[\beta + 4(1 - \beta)(L - x)/L] + b_0 \\ \text{otherwise} & \quad b_{\text{eff}}(x) = b_e + b_0 \end{aligned} \quad (76)$$

where  $b_e$  is given by:

for an interior beam  $b_e = \min\left\{\frac{L}{8}; \frac{b_1}{2}\right\} + \min\left\{\frac{L}{8}; \frac{b_2}{2}\right\}$

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

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

$$b_0 = 0.$$

When slab openings are defined by the user (see § 3.6.1), following formulas are used:

for an interior beam  $b_e = \min\left\{\frac{L}{8}; \frac{b_1}{2}; d_{so,1}\right\} + \min\left\{\frac{L}{8}; \frac{b_2}{2}; d_{so,2}\right\}$

for an edge beam  $b_e = \min\left\{\frac{L}{8}; b_1; d_{so,1}\right\} + \min\left\{\frac{L}{8}; \frac{b_2}{2}; d_{so,2}\right\}$

## 7.2. Capacity factors



References: AS/NZS 2137 [7]: Table 1.4.3

Design capacity for	§	Capacity factor
Steel in tension or compression	Annex C	$\phi_s = 0,9$
Concrete in compression	Annex C	$\phi_c = 0,8$
Shear of the cross-section	7.7.2 and 7.7.3	$\phi_v = 0,9$
Shear connectors	7.3	$\phi_{sc} = 0,8$
Longitudinal shear in concrete slab	7.9.3	$\phi_{sr} = 0,63$

Table 14: Capacity factors acc. to AS/NZS 2327

These values are predefined but it is possible to modify them in the settings of the project.

## 7.3. Design resistance of the connector

### 7.3.1. General rule

The design and characteristic horizontal shear resistances of a connector are obtained as follows:

- **For solid slabs with or without concrete haunches** (see § 3.6.1): the design resistance  $P_{Rk}$  and  $P_{Rd}$  are directly obtained from the *Table 8*.
- **For slabs with decking transverse to the beam axis:**
  - **Connector longitudinal with the beam:** the characteristic resistance  $P_{Rk,t}$  and the design resistance  $P_{Rd,t}$  are obtained from the formulae given in Table 4 of ETA-15/0876:

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

$$P_{Rk,t} = k_{t,l} P_{Rk}$$

(77)

$$\text{where: } k_{t,l} = \frac{0,66 b_0}{\sqrt{n_r} h'_p} \left( \frac{h_{sc}}{h'_p} - 1 \right) \leq 1$$

- **Connector transverse with the beam:** the characteristic resistance  $P_{Rk,t}$  and the design resistance  $P_{Rd,t}$  are obtained from the formulae given in Table 4 of ETA-15/0876:

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

$$P_{Rk,t} = k_{t,t} P_{Rk}$$

(78)

$$\text{where: } k_{t,t} = \frac{1,18 b_0}{\sqrt{n_r} h'_p} \left( \frac{h_{sc}}{h'_p} - 1 \right) \leq 1$$

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

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

$$P_{Rk,l} = k_{t,l} P_{Rk}$$

(79)

$$\text{where: } k_l = 0,6 \frac{b_0}{h'_p} \left( \frac{h_{sc}}{h'_p} - 1 \right) \leq 1$$

In Formulas (77) to (79):

- $P_{Rk}$  and  $P_{Rd}$  are given in Table 8;
- $h'_p$  is the height of the profiled steel deck, obtained by:  $h'_p = h_p$
- $b_0$  is given by:
 
$$b_0 = (b_t + b_b)/2 \quad \text{if } b_t > b_b$$

$$b_0 = b_t \quad \text{if } b_t \leq b_b$$
- $b_b, b_t$ : see § 3.7;
- $h_{sc}$ : see Table 8
- $n_r$  is the number of X-HVB connector in a row (without limitation).



The capacity design factor in AS/NZS 2327 for connectors is set to  $\phi_{sc} = 0,8$  (see Table 14). It is thus consistent with the values of  $P_{Rk}$  and  $P_{Rd}$  given in Table 8.

For connectors X-HVB 80 to 140, a reduction factor is applied to both characteristic and design resistances of the connector if the flange thickness  $t_f$  is less than 8 mm:

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

$$P_{Rk,red} = \frac{t_f}{8} P_{Rk} \geq \frac{23 \text{ kN}}{\phi_{sc}}$$

(80)

This reduction is applied for plain slabs as covered by the ETA report [1] but also for slabs with profiled steel sheeting when relevant (see § 3.5.1). In the latter case, which is outside the scope of [1], a notification is given in the calculation report.

For custom steel (see § 3.5.2) with lower yield strengths (see condition (82)), an additional reduction factor is applied to both characteristic and design resistances of the connector:

$$\alpha_{\text{BM,red}} = 0,95 \quad (81)$$

The reduction factor  $\alpha_{\text{BM,red}}$  is applied when the following condition is fulfilled:

$$f_y < 235 \text{ MPa} \quad (82)$$



Condition (82) is consistent with the lower steel grades available with Australian and New Zealand standards for steel structures.

### 7.3.2. Specific rules for Comflor 80 deck

When used with Comflor 80 profiled steel deck, a specific calculation rule is used, based on experimental evidences as described in [2]. Comflor 80 can be used only with X-HVB 140 connectors. When the deck is perpendicular to the beam axis, the values of  $P_{\text{Rk}}$  and  $P_{\text{Rd}}$  given in *Table 15* is directly considered for the calculations.

Specific reductions of Formulas (80) and (81) are used when relevant.

Number of rows $n_r$	Characteristic resistance $P_{\text{Rk}}$ [kN]	Design resistance $P_{\text{Rd}}$ [kN]
1	26,52	21,22
2	26,52	21,22
3	24,00	19,20
4	22,00	17,60

Table 15 : Resistance of X-HVB 140 connectors used with Comflor 80 decks – According to [2]

### 7.4. Participating depth of the concrete slab

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

$$\begin{aligned} \text{for plain slabs:} & \quad e_{\text{part}} = \beta h \\ \text{for composite slabs:} & \quad e_{\text{part}} = \beta (h - h_p) \end{aligned} \quad (83)$$

Where:  $\beta$  is the degree of connection (see § 3.9.3);  $\beta = 1$  for full connection.

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

### 7.5. Connection in plastic design

#### 7.5.1. Principles

For plastic design, the connection between the slab and the steel profile is automatically designed by the software (see §3.9.3 and §3.9.4).

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 § 3.11.3). 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. For a Comflor 80 deck transverse with the beam, when the connectors is placed transversally to the beam, the maximum number of connectors is set to 4.

### 7.5.2. 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 16 gives the software strategies according to the value or state of these parameters.

ULS criteria for resistance	SLS criteria	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 LRFD 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 16 : Software strategies for the assessment of the number of connectors

### 7.5.3. Number of connectors for full connection

For a critical cross-section (see § 5.2), 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:

$$n_{\text{ribs}} = \frac{\min \{x_c; L - x_c\}}{b_s} \quad (84)$$

where:  $b_s$ : see § 3.7

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

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

Where:  $N_{\text{a,n}}$  is the nominal compressive strength of the steel profile, calculated by:

$$N_{\text{a,n}} = \varphi_s A f_y$$

$N_{\text{c,n}}$  is the nominal compressive strength of the concrete slab, calculated by:

$$N_{\text{c,n}} = 0,85 \varphi_c f'_c (h - h_p) b_{\text{eff}}$$

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

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

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

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

#### c) Solid slab

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

Where:  $N_{\text{cs,n}} = 0,85 \varphi_c f'_c h b_{\text{eff}}(x_c)$

The number of connectors  $n_f$  must fulfil the requirements of connectors positioning as defined in § 3.10 and 3.11. If this control is negative, an error code will be sent back to the interface.

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

#### a) Minimum degree of connection

The minimum degree of connection, denoted  $\beta_{\text{min}}$ , is calculated according to AS/NZS 2327 § 3.5.8.3:



$$\begin{aligned} \text{If } L \leq 25 \text{ m:} & \quad \beta_{\min} = 1 - (355/f_y)(0,75 - 0,03L) \geq 0,4 \\ \text{otherwise:} & \quad \beta_{\min} = 1 \end{aligned} \quad (85)$$

### 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  $\eta_{\min}$ :

$$n_0 = \beta_{\min} n_f$$

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 error code 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 E):**

*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:

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

## 7.5.5. 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.

## 7.5.6. Degree of shear connection

The degree of shear connection of the composite beams is actually defined by:

$$\beta = \frac{n_{rs} P_{Rd,t}}{\min \{N_{a,n}; N_{c,n}\}} \quad (86)$$

where  $n_{rs}$  is the total number of connectors between the critical section and the support.

## 7.6. ULS verifications principles

### 7.6.1. Cross-section slenderness



*References:*

AS/NZS 2327 § 3.4.3 and Table 3.4.3.3

For a design in partial connection, a part of steel profile is in compression. The slenderness of the portion of the cross-section in compression is assessed by the software as follows. The location of the plastic neutral axis (PNA) of the composite cross-section is calculated first (see Annex C). The portion of the cross-section in compression is then assessed.

If the PNA is located in or below the upper flange, the latter is assumed in uniform compression. Its slenderness  $\lambda_{ef}$  is obtained by Formula (57). The section plasticity and yield slenderness limits ( $\lambda_{efp}$  and  $\lambda_{efy}$  respectively) of the flange in uniform compression are given in *Table 17* (from Table 3.4.3.3 of AS/NZS 2327 [7]).

Type of profile	Plasticity limit $\lambda_{efp}$	Yield limit $\lambda_{efy}$
Profiles from the database (hot-rolled)	9	16
Custom profiles (built-up heavily welded)	8	14

*Table 17: Slenderness limits for flanges acc. to Australasian standard*

The upper flange is classified as follows:

- compact if:  $\lambda_{ef} \leq \lambda_{efp}$
- non-compact if:  $\lambda_{efp} < \lambda_{ef} \leq \lambda_{efy}$
- slender if:  $\lambda_{ef} > \lambda_{efy}$

According to the notes of Table 3.4.3.3 in AS/NZ 2327, the web is classified without consideration of the presence of the slab. The web slenderness  $\lambda_{ew}$  is obtained by Formula (58). The section plasticity and yield slenderness limits ( $\lambda_{ewp}$  and  $\lambda_{ewy}$  respectively) of the web are calculated as follows (from Table 3.4.3.3 of AS/NZS 2327 [7]).

$$\lambda_{ewp} = \frac{111}{4,7 r_p - 1} \quad \text{for } 0,5 \leq r_p \leq 1,0 \quad (87)$$

$$\lambda_{ewp} = \frac{41}{r_p} \quad \text{for } r_p < 0,5$$

$$\lambda_{ewy} = \frac{60}{r_e} \quad \text{for } 0 \leq r_e \leq 1,0 \quad (88)$$

In Formula (87),  $r_p$  is the ratio of the distance from the PNA to the compression edge of the web to the depth of the web. In Formula (88),  $r_e$  is the ratio of the distance from the elastic neutral axis (ENA) to the compression edge of the web to the depth of the web. Both  $r_p$  and  $r_e$  are assessed considering the composite gross cross-section.

In partly in compression, the web is classified as follows:

- compact if:  $\lambda_{ew} \leq \lambda_{ewp}$
- non-compact if:  $\lambda_{ewp} < \lambda_{ew} \leq \lambda_{ewy}$
- slender if:  $\lambda_{ew} > \lambda_{ewy}$

AS/NZS 2327 does not allow to have slender elements. So that if either the upper flange or the web is slender, an error code is sent back to the interface and the calculations are stopped.

Non-compact cross-sections are not allowed for partial connection as mentioned in AS/NZS 2327 §3.6.1.1.

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

*For partial connection, if a cross-section is classified as non-compact, other calculations are not performed and an error message is displayed in the UI (error code = 33).*

**7.6.2. ULS plastic design and effective portion of the steel beam**



**References:**

AS/NZS 2327 § 3.4.3 and Table 3.4.3.3

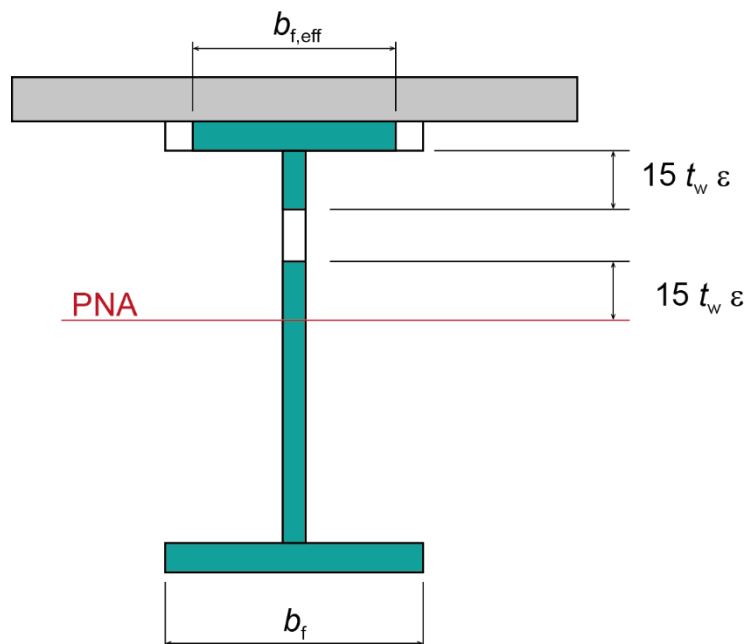
According to AS/NZS standards, a plastic design of the beams is always carried out, considering an effective cross-section. The effective portion of the steel beam is assessed as follows:

- all elements of the section in tension are fully effective;
- all compact elements (upper flange and web) in compression are fully effective;
- if the upper flange (in compression) is classified as non-compact, its effective width is obtained as follows:

$$b_{f,eff} = t_w + 2 \lambda_{efp} t_f \tag{89}$$

- if a part of the web is in compression and is classified as non-compact, its effective portion is obtained by removing the ineffective part at the centre of the portion in compression (see Figure 26). Both remaining effective parts have a width of  $15 t_w \varepsilon$ , where  $\varepsilon$  is defined by:

$$\varepsilon = \sqrt{\frac{250}{f_y}} \tag{90}$$



*PNA = plastic neutral axis of the gross composite steel cross-section*

Figure 26 : Effective portion of a non-compact steel section for plastic design of composite beams

## 7.7. Strength verifications

### 7.7.1. Section bending moment capacity



**References:**  
AS/NZS 2327 § 3.5.2 and 3.5.4

The criterion for the verification of section to the bending moment about the principal axis is:

$$\Gamma_{Mc} = M^*/M_R \leq 1.0 \quad (91)$$

where  $M^*$  is the design bending moment;

$M_R$  is the design moment capacity, obtained according to Annex C

### 7.7.2. Shear capacity



**References:**  
AS/NZS 2327 § 3.5.5

The same criterion as in § 6.2.5 is applied, using the capacity factor  $\phi_v$  of *Table 14*. The contribution of the slab is ignored. The resistance to shear buckling is also applied to composite sections.

For a partial connection ( $\beta < 1$ ), the nominal shear capacity is calculated by:

$$V_{V,\beta} = \alpha V_V \quad (92)$$

Where:  $\alpha = (1 - \beta)/(0.76 \lambda_{sd} + 0.92) + \beta$

$$\lambda_{sd} = h_{sl}/(h_{sl} + h_p)$$

### 7.7.3. Interaction of shear and bending moment



**References:**  
AS/NZS 2327 § 3.5.6

The shear and bending interaction is verified through the following criterion:

$$\Gamma_{vmc} = \left(\frac{M^*}{M_R}\right)^3 + \left(\frac{V^*}{V_R}\right)^6 \leq 1.0 \quad (93)$$

where  $V^*$  is the design shear force;

$M^*$  is the design bending moment;

$M_R$  is the design moment capacity of the section, see § 7.7.1;

$V_R$  is the shear capacity of the section, given by  $V_R = \phi_v V_V$  of the section;

$V_V$  is the design shear capacity of the section, see § 6.2.5;

$\phi_v$  is the capacity factor (strength reduction factor) for shear resistance – see *Table 14*;

## 7.8. SLS verifications

### 7.8.1. General

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, their 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.

The vibrations check needs to calculate the fundamental natural frequency of the composite beam. The later is evaluated by using the Rayleigh method in which all deflections are calculated considering the composite effect with the short term modular ratio.

According to AS/NZS 2327 § 3.10.3.2 and § 3.10.3.3, the modular ratio used for the calculations of composite stage deflections are defined as follows:

$$\text{short term actions : } n_{\text{eq}} = \frac{E_s}{E_c} \quad (94)$$

$$\text{long term actions (*) : } n_{\text{eq}} = 3 \frac{E_s}{E_c} \quad (95)$$

(\*): modular ratio for long term actions includes the effect of creep;

where:  $E_s, E_c$  : see § 3.5.2 and § 3.6.2 respectively.



*Formula (94) is a simplified approach based on engineering judgement. It is also adopted for Eurocodes and North American standards.*

### 7.8.2. Deflection limits

According to AS/NZS 2327, the deflection of the beam should be checked for:

- total deflection, under short term and long term loading effects;
- incremental deflection, which occurs after a chosen stage in the life of the structure.

See § 4.2 for the definition of SLS combination of loads for the calculation of deflections.

AS/NZS 2327 provides informative deflection limits in Annex B of the Standards. The total deflection is always checked with a common limit of 1/250 of the span length. According to the type of finishes, a check of the incremental deflection is also recommended, with a typical limit of 1/500 of the span length.

In the software:

- the total check is always checked, with a default limit of L/250;
- the check of the incremental deflection is optional, with a default limit of L/500.

### 7.8.3. Vibrations



*References:  
AS/NZS 2327 § 6.4.2.2*

The floor vibration check is carried out according to AS/NZS 2327 [7] § 6.4.2 and [6]. At first, the fundamental natural frequency ( $f_n$ ) is calculated using the Rayleigh method. Then, specific criterion is applied for two types of excitation (when relevant, according to the options selected by the User):

- Walking activities;
- Synchronised crowd movement.

#### 7.8.4. Vibrations checks for walking activities

The checking method applied by the software depends on the value of the fundamental natural frequency:

- Low frequency floors:  $f_n \leq 10$  Hz
- High frequency floors:  $f_n > 10$  Hz

##### Low frequency floors

The total weighted root-mean-squared (rms) acceleration is calculated assuming a resonant response to one of the harmonics of walking frequency as follows:

$$a_{w,rms} = \mu_e \mu_r \frac{0,1Q}{2\sqrt{2}M \xi} W \rho \leq a_0 \quad (96)$$

Where:

- $\mu_e$  mode shape factor at the point of excitation;  $\mu_e = 1,0$
- $\mu_r$  mode shape factor at the point of response;  $\mu_r = 1,0$
- $\xi$  critical damping ratio
- $Q$  weight of a person;  $Q = 746$  N (76 kg x 9,81 m/s<sup>2</sup>)
- $W$  frequency weighting factor, given in §7.6 of [6] or in *Figure 27*

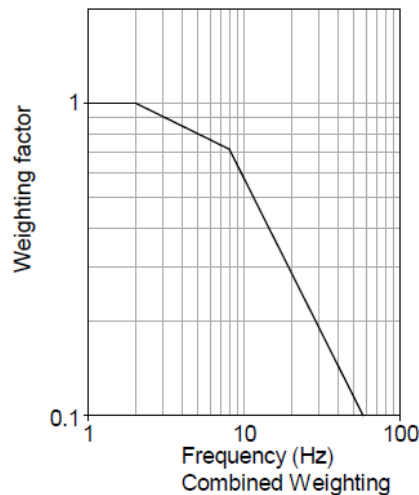


Figure 27 : Frequency weighting factor (Figure 7.5 of [6])

$M$  modal mass, calculated by:

$$M = mL_{\text{eff}}S$$

$m$  floor mass per unit area including dead load and variable load (expressed in kg/m<sup>2</sup>), calculated by:

$$m = (G + 0,2Q)/g$$

$L_{\text{eff}}$  effective floor length, calculated by:

$$L_{\text{eff}} = 1,09(1,10)^{n_y-1} \left( \frac{EI_b}{mbf_n^2} \right)^{1/4} \leq n_y L$$

$n_y$  number of bays in the direction of the beam;  $n_y = 1$

- $EI_b$  dynamic flexural rigidity of the composite beam (expressed in N.m<sup>2</sup>)
- $L$  length of the beam
- $b$  beam spacing, taken as:  $b = \max\{b_L ; b_R\}$
- $f_n$  fundamental frequency
- $S$  effective floor width, calculated by:

$$S = \eta(1,15)^{n_x-1} \left( \frac{EI_s}{mf_n^2} \right)^{1/4} \leq n_x L_x$$

- $L_x$  span of the primary beam (beam in the other direction), taken as:  $L_x = b_L + b_R$
- $n_x$  number of bays in the direction of the primary beam;  $n_x = 1$
- $\eta$  factor that accounts for the influence of floor frequency of the response of the slab, calculated by:
  - $\eta = 0,5$  when  $f_n < 5\text{Hz}$
  - $\eta = 0,21f_n - 0,55$  when  $5\text{ Hz} \leq f_n \leq 6\text{ Hz}$
  - $\eta = 0,71$  when  $f_n > 6\text{Hz}$
- $EI_s$  dynamic flexural rigidity of the slab (expressed in N.m<sup>2</sup>)
- $a_0$  the acceleration limit; given in *Table 18*, according to in ISO 2631:

Activity	Acceleration limit $a_0/g$ (%)		
	1 Hz	4-8Hz	40 Hz
Offices, Residences	1.0	0.5	2.5
Shopping Malls, Dining and Dancing	3.0	1.5	7.0
Rythmic activities	10.0	5.0	25.0
<i>For frequency ranges 1-4 Hz and 8-40 Hz, acceleration limits is obtained by linear interpolation</i>			

*Table 18 : Recommended peak acceleration for human comfort for vibrations due to human activities.*

### **High frequency floors**

If the fundamental frequency is greater than 10 Hz, the rms acceleration should be calculated by:

$$a_{w,rms} = 2\pi\mu_e \mu_r \frac{186}{Mf_n^{0,3}} \frac{Q}{700} \frac{1}{\sqrt{2}} W \leq a_0 \tag{97}$$

### 7.8.5. Vibration check for synchronised crowd movement

When floors are subject to dancing and jumping activities characterized by synchronized crowd movement (as defined in option by the User), the simplified approach of § 6.4.3.1.2 of AS/NZ 2327 is considered in the calculation module (“design to avoid resonance”). The check criterion is:

$$f_1 > 8,4 \text{ Hz} \quad (98)$$



*The far more complex approach “to withstand the anticipated dynamic loads” of the standard is not considered in the module.*

### 7.8.6. Calculation of the fundamental natural frequency

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

$$f = \frac{1}{2 \pi} \sqrt{g \frac{\sum P_i |w_i|}{\sum P_i w_i^2}} \quad (99)$$

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

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

$$g = 9,81 \text{ m/s}^2$$

The dynamic modulus of elasticity of concrete is taken as:  $E_{c,dyn} = 1,1 E_c$



## 7.9. Longitudinal shear resistance

### 7.9.1. 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 28 a*).
- For slabs with profiled steel sheeting, either longitudinal or perpendicular, only one layer is assumed (*Figure 28 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 28 c*).

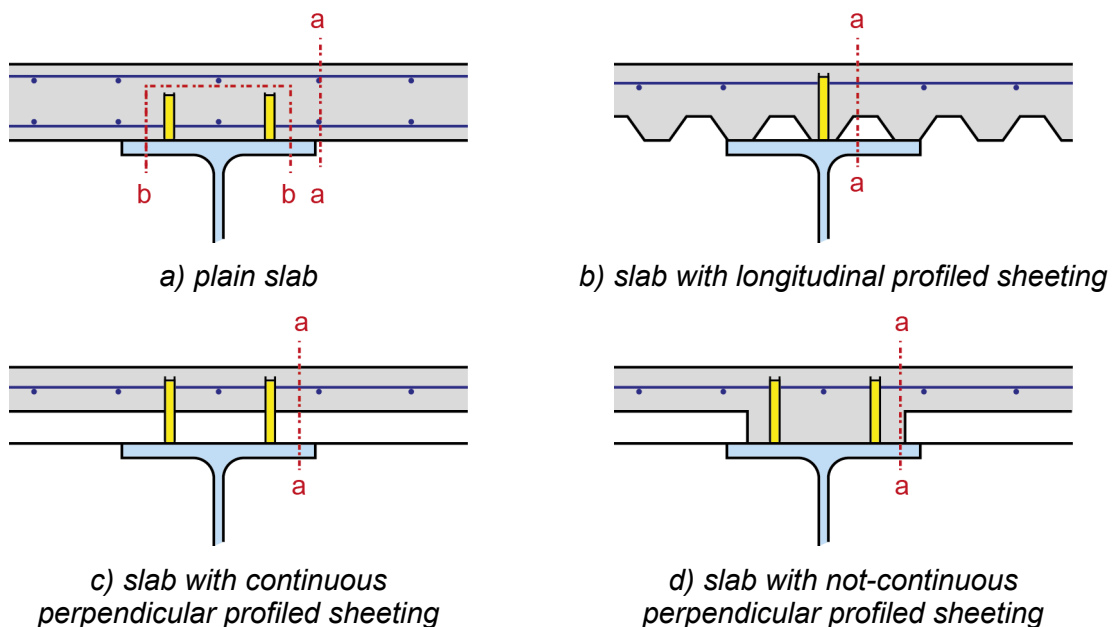


Figure 28: Transverse reinforcement configurations and shear areas

### 7.9.2. Minimum transverse reinforcement ratio



References:  
AS/NZS 2327 § 3.8.6

The minimum area of transverse reinforcement required by AS/NZS 2327 (see § 3.8.6 of the standard) is given by:

$$\rho_{\min} = 0,08 \frac{\sqrt{f'_c}}{f_{sy}} \quad (100)$$

where :  $f'_c$  see § 3.6.2

$f_{sy}$  see § 3.8

The cross-sectional area of transverse reinforcement per unit length of beam should also fulfil the following condition:

$$A_{ts} \geq 0,8 \frac{d_s \sigma_1}{f_{sy}} \quad (101)$$

where :  $d_s$  see definition after Formula (105)

$$\sigma_1 = 1 \text{ MPa}$$

When two layers of transverse reinforcement are used, the lower layer should represent at least 50% ( $A_b \geq 0,5 A_{ts}$ ).

### 7.9.3. Design of transverse reinforcement



References:  
AS/NZS 2327 § 3.8.4

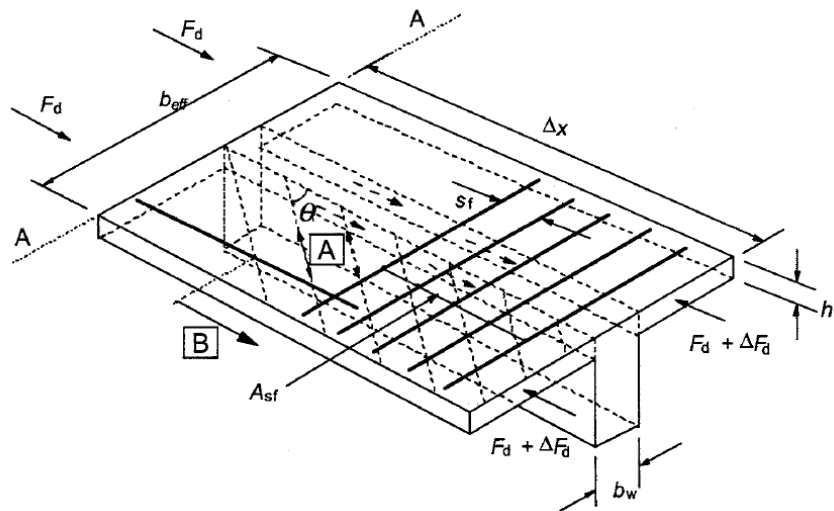


Figure 29 : Transverse reinforcement

The calculation is carried out for a segment between each critical section (see § 5.2) 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:

$$\text{Shear area a-a:} \quad \Delta F_d = \frac{N_{c,d}}{b_{\text{eff}}(x_c)} \max\{b_{\text{eff, left}}(x_c); b_{\text{eff, right}}(x_c)\} \quad (102)$$

$$\text{Shear area b-b:} \quad \Delta F_d = N_{c,d} \quad (103)$$

where:  $x_c$  is the location of the critical section;  
 $N_{c,d}$  resulting compression force in the concrete slab at the critical section, given by:

$$N_{c,d} = 0,85 \phi_c f'_c e_{\text{part}}(x_c) b_{\text{eff}}(x_c) \quad (104)$$

$b_{\text{eff}}(x_c)$  is the effective width of concrete slab at the critical section (see § 7.1);  
 $b_{\text{eff, left}}(x_c)$  and  $b_{\text{eff, right}}(x_c)$  are the part of this effective width on the LHS and RHS respectively, with:

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

$e_{\text{part}}(x_c)$  see § 7.4

$\phi_c$  see Table 14

$f'_c$  see § 3.6.2

### Calculation of the longitudinal shear force per unit length through the shear area

The design longitudinal shear force per unit length of the beam on the particular shear plane is determined by (see § 3.8.2 of AS/NZS 2327):

$$v_{Lp}^* = \frac{\Delta F_d}{n_{sp} s} \quad (105)$$

where:  $n_{sp}$  is the number of shear plane through which the shear force is transferred;  $n_{sp} = 2$  for the shear area b-b of an intermediate beam with plain slab, and  $n_{sp} = 1$  for any other case;

$s$  is the shear length, which 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 30).

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

According to § 3.8.3 of AS/NZS 2327, the resistance of the concrete compression strut is checked by the following formula:

$$\frac{v_{Lp}^*}{d_s} \leq \phi_c f'_c v \sin \theta_f \cos \theta_f \quad (106)$$

where:  $d_s$  is the height of the concrete slab ( $d_s = h_{sl}$  for plain slabs and  $d_s = h_{sl} - h_p$  for slabs with profiled steel sheeting).

$\phi_c$  see Table 14

$f'_c$  see § 3.6.2

$v$  is a strength reduction factor given by:

$$v = 0,6 \left( 1 - \frac{f'_c}{250} \right) \quad (107)$$

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

$$\theta_f = \frac{1}{2} \arcsin \left( \frac{2 v_{Lp}^*}{v d_s \phi_c f'_c} \right) \quad \text{but } 1 \leq \cotg \theta_f \leq 2 \quad (108)$$



Formula (105) is coming from Equation 3.8.4 (2) of AS/NZS 2327. The latter is obviously false and following corrections have been made:

- introduction of  $v$  factor which is missing in AS/NZS 2327;
- unit consistent formula: the left part is divided by  $d_s$  and not the right one.

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

$$\Gamma_{c, \text{strut}} = \frac{v_{Lp}^*/d_s}{\phi_c f'_c v \sin \theta_f \cos \theta_f} \leq 1 \quad (109)$$

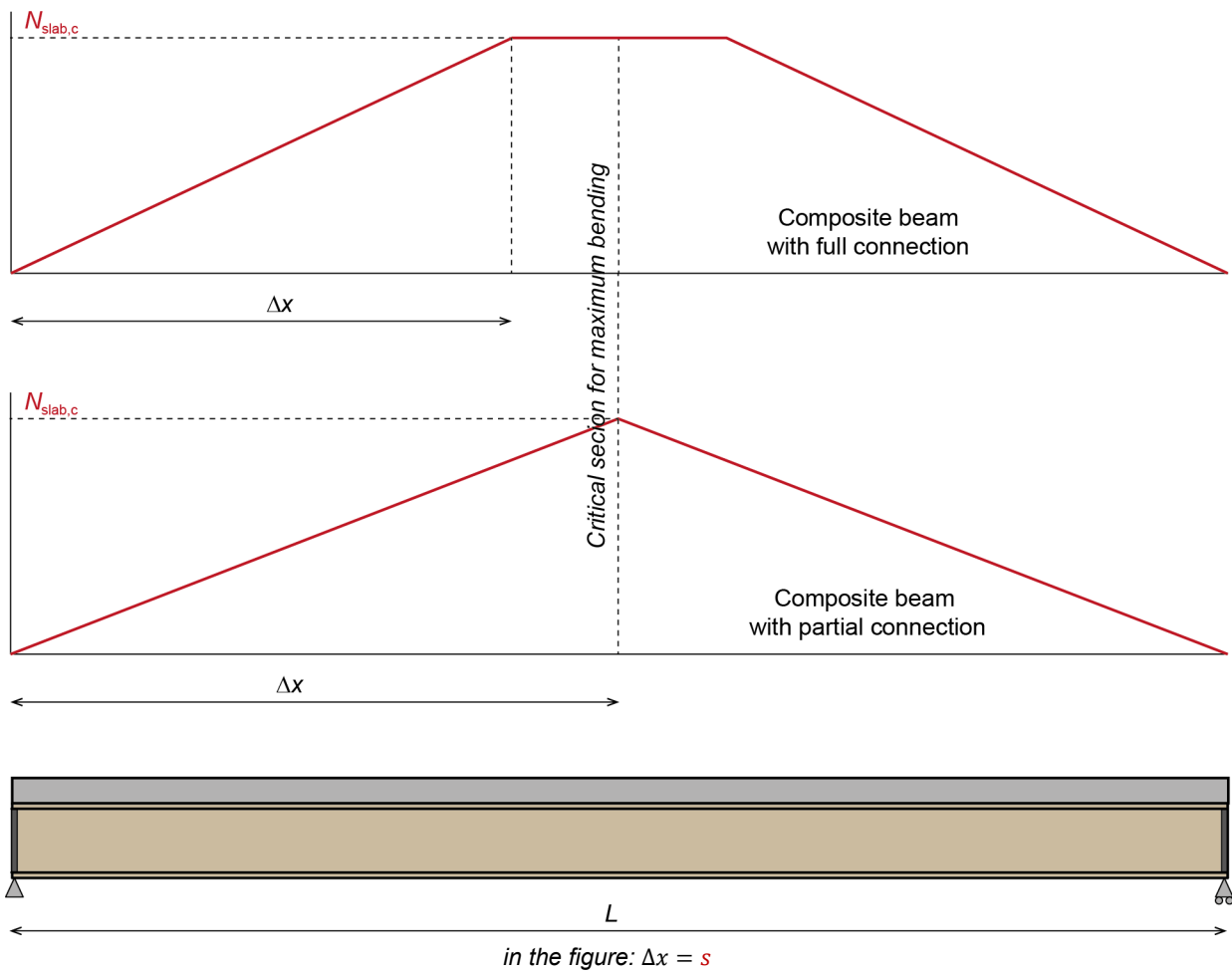


Figure 30 : Shear length

### Assessment of the transverse reinforcement

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

$$A_{ts} \geq \frac{v_{Lp}^*}{\phi_{sr} f_{sy} \cot \theta_f} \quad (110)$$

where:  $f_{sy}$  see § 3.8

$\phi_{sr}$  see Table 14

$\theta_f$  is the orientation of the strut obtained at the previous step for the resistance of concrete;

$A_{ts}$  is the cross-sectional area of transverse reinforcement per unit length of beam

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  $A_b$ . The calculation performed for the shear area a-a gives  $A_{ts} = A_t + A_b$ .

### Influence of continuous perpendicular profiled steel sheeting

For slabs with continuous perpendicular profiled steel sheeting, the previous equation (109) is replaced by:

$$A_{ts} \geq \max \left\{ 0; \frac{v_{Lp}^*}{\phi_{sr} f_{sy} \cot \theta_f} - A_{pe} \frac{\phi_s f_{yp}}{\phi_{sr} f_{sy}} \right\} \quad (111)$$

where:  $f_{yp}$  see § 3.7

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

$$A_{pe} = \frac{t_p}{b_s} \left( b_s + b_b - b_t + \sqrt{(b_b - b_t)^2 + 4 h_p^2} \right) \quad (112)$$

$t_p, h_p, b_s, b_b, b_t$ : see § 3.7



*Formula (110) is not provided directly by AS/NZS but is an adaptation of the equation proposed by EN 1994-1-1 and already used in the X-HVB software in Eurocode version. For continuous perpendicular profiled steel sheetings, AS/NZS proposes an equation 2.7.2. (5) which comprises the same terms than in Eurocode but is also limited by a term that depends on the parameters of studs welded through the deck, which is obviously not applicable for XHVB.*

### 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 according to § 7.9.2, i.e.

$$\text{plain slabs:} \quad A_b \geq \frac{1}{2} \rho_{\min} d_s ; A_t \geq \frac{1}{2} \rho_{\min} d_s ; A_b \geq 0,5 A_{ts} \quad (113)$$

$$\text{composite slabs:} \quad A_{ts} \geq \rho_{\min} d_s \quad (114)$$

## Annex A : PROPERTIES OF THE STEEL SECTION

The following section properties are calculated using analytical formulae:

**A** Section area

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

**$I_x$**  Second moment of area about the strong axis

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

with:

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

**$I_y$**  Second moment of area about the weak axis

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

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

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

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

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

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

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

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

$$W_{el,y} = \frac{2I_x}{b_f}$$

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

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

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

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

**J** Torsional constant

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

**$I_w$**  Warping constant

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

## Annex B : ELASTIC PROPERTIES OF A COMPOSITE BEAM

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

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

Location of the elastic neutral axis:

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

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

Second moment of area:

$$\begin{aligned} I_{x,j} &= I_{x,a} + A(h_t/2 - y_{ENA,j})^2 + \frac{b_{eff}(h - h_p)^3}{12 n_{eq,j}} \\ &+ \frac{b_{eff}(h - h_p)}{n_{eq,j}} [h_t + (h + h_p)/2 - y_{ENA,j}]^2 \end{aligned}$$

Where:  $I_{x,a}$  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:

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

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

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

Second moment of area:

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

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

Location of the elastic neutral axis:

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

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

Second moment of area:

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

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

Location of the elastic neutral axis:

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

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

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

Second moment of area:

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

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

Location of the elastic neutral axis:

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

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

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

Second moment of area:

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



## Annex C : PLASTIC FLEXURAL STRENGTH OF THE COMPOSITE CROSS-SECTION

The plastic flexural strength of a cross section is assessed considering the participating depth of the slab (see § 7.4).

### Capacity factors

Following capacity factors are used for the calculation of the plastic moment capacity of the composite cross-section:

$\phi_s$  capacity factor for steel under axial force – see *Table 14*

$\phi_c$  capacity factor for concrete in compression – see *Table 14*

### Plastic Neutral Axis in the concrete slab

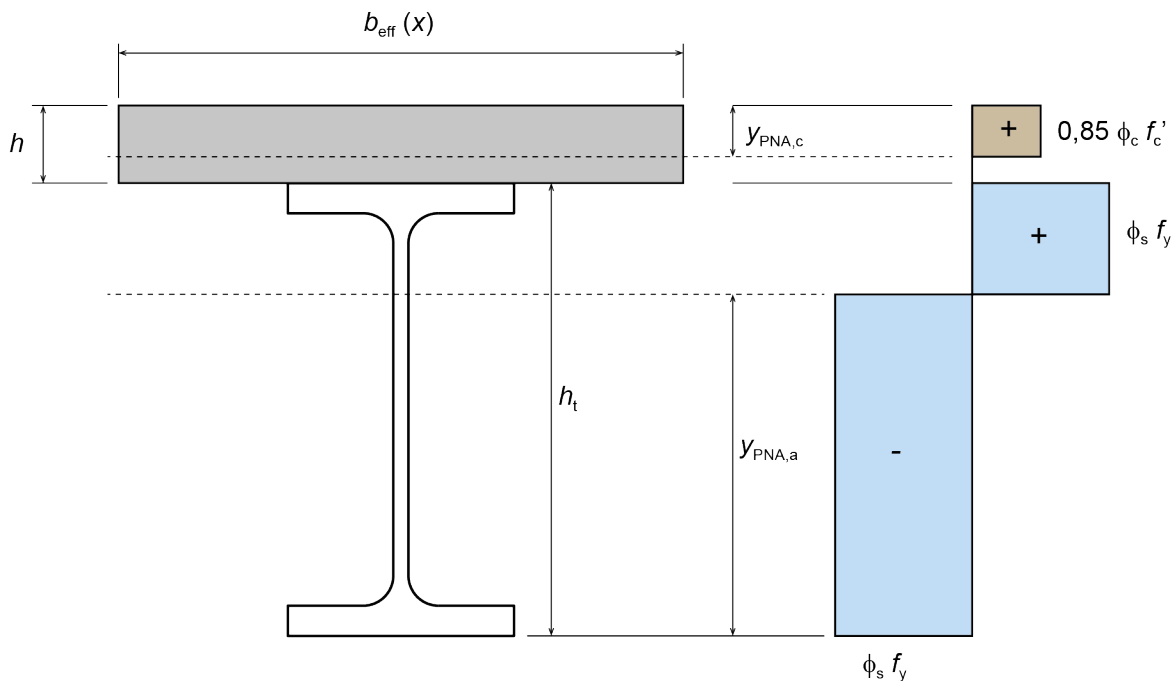


Figure 31: Plastic stresses at limit state of yielding

The position of the plastic neutral axis (PNA) in the concrete slab is calculated by (see *Figure 31*):

$$y_{PNA,c} = \frac{N_{y,d}}{0,85 \phi_c f'_c b_{eff}(x)} \leq h - h_p$$

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

$h_p = 0$  for solid slabs

$N_{y,d}$  is the design axial force resisted by the steel section at yield:

$$N_{y,d} = A \phi_s f_y$$

The moment capacity of the composite section is calculated by:

$$M_R = N_{c,d} \left( h + \frac{h_t - y_{PNA,c}}{2} \right)$$

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

$$N_{c,d} = 0,85 \phi_c f'_c y_{PNA,c} b_{eff}(x)$$

**Plastic Neutral Axis in the steel section**

1) If  $N_{y,d} \geq N_{c,d} \geq N_{y,f}$ , PNA is located in the top flange:

$$y_{PNA,a} = h_t - \frac{N_{y,d} - N_{c,d}}{2 b \phi_s f_y}$$

Where:  $N_{y,f} = (A - 2 b t_f) f_y$

The plastic moment capacity of the section is calculated by:

$$M_R = (h_t - y_{PNA,a}) b y_{PNA,a} \phi_s f_y + M_c$$

2) If  $N_{y,w} \geq N_{c,d}$ , PNA is located in the web:

$$y_{PNA,a} = \frac{h_t}{2} + \frac{N_{c,d}}{2 t_w \phi_s f_y}$$

Where:  $N_{y,w} = (h_t - 2t_f - 2r_1) t_w \phi_s f_y$

The moment capacity of the composite section of the section is calculated by:

$$M_R = \left[ W_{pl,x} - \frac{1}{4} t_w \left( \frac{N_{c,d}}{t_w \phi_s f_y} \right)^2 \right] \phi_s f_y + M_c$$

3) If  $N_{y,f} > N_{c,d} > N_{y,w}$ , PNA is located in the fillets. The position of the PNA and the associated nominal flexural strength are calculated by using an iterative procedure.

## Annex D FLEXURAL RIGIDITY OF SLAB

The Elastic Neutral Axis of the slab is calculated by:

$$y_{\text{slab}} = \frac{\frac{A_c}{n_{\text{eq,dyn}}} y_c + A_d y_{\text{db}}}{\frac{A_c}{n_{\text{eq,dyn}}} + A_d}$$

Where:

$A_c$  concrete area per unit width of slab ( $m^2/m$ ), determined by:  $A_c = h_c \times 1m$

$h_c$  effective thickness of concrete slab that accounts for concrete in deck troughs, determined by:

$$h_c = h_s - h_p + h_p \frac{b_t + b_b}{2b_s}$$

$y_c$  position of ENA of the concrete part below the top of slab, calculated by:  $y_c = h_c/2$

$n_{\text{eq,dyn}}$  dynamic modular ratio, determined by:  $n_{\text{eq,dyn}} = E_s/E_{c,dyn}$

$A_d$  decking area per unit width, calculated by:

$$A_d = \frac{t_p}{b_s} [b_s - b_t + b_b + 2l_r]$$

$t_p$  deck thickness

$$l_r = \sqrt{\left(\frac{b_t - b_b}{2}\right)^2 + h_p^2}$$

$y_d$  position of ENA of the deck below the top of slab, calculated by:  $y_{\text{db}} = h_s - h_p + y_d$

$$y_d = h_p \frac{b_b + l_r}{b_s - b_t + b_b + 2l_r}$$

The second moment of area of the slab is calculated by:

$$I_{\text{slab}} = \frac{I_c + A_c(y_c - y_{\text{slab}})^2}{n_{\text{eq,dyn}}} + I_d + A_d(y_{\text{db}} - y_{\text{slab}})^2$$

Where:

$I_c$  second moment of area per unit width of concrete, determined by:  $I_c = h_c^3/12$

$I_d$  second moment of area per unit width of deck, determined by:

$$\begin{aligned} I_d &= b_b \frac{t_p^3}{12} + b_b t_p (h_p - y_d)^2 + (b_s - b_t) \frac{t_p^3}{12} + (b_s - b_t) t_p y_d^2 + \left( l_r \frac{t_p^3}{12} + t_p \frac{l_r^3}{12} \right) \\ &\quad - \left( l_r \frac{t_p^3}{12} - t_p \frac{l_r^3}{12} \right) \left[ 2 \left( \frac{b_t - b_b}{2l_r} \right)^2 - 1 \right] + 2 \left( \frac{h_p}{2} - y_{\text{db}} \right)^2 l_r t_p \end{aligned}$$

## Annex E : ERROR CODES MANAGEMENT

Error Code	Description	Reference
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]	
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]"	
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]	§ 3.4
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]	§ 3.4
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]	§ 3.5.1
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]"	§ 3.5.2
10	Value of Steel Grade not correct. [error code: 10]	§ 3.5.2
11	The class of section is out of scope (Slender class when checking LRFD of the steel beam). Please reduce the slenderness of the profile plate or reduce the steel grade in the "Materials" tab. [error code: 11]	§ 6.2.1

Error Code	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 ( $\geq 2$ in. or 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]"	§ 0
14	Haunch height is only valid for Solid slab and $h_h \geq 0$ . [error code: 14]	
15		§ 3.7
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]"	§ 3.7
17	The Value of Steel Grade for reinforcement steel bars is not correct. [error code: 17]	
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]	
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]	§ 3.5.1

Error Code	Description	Reference
23	The thickness of the steel profiled sheeting is greater than the value given in Annex B4 of ETA 15/0876. Please reduce the thickness of the profiled sheeting in the "Slab" tab: - $t \leq 2.0$ mm for X-HVB 80, X-HVB 95 and X-HVB 110 - $t \leq 1.5$ mm for XHVB 125 and X-HVB 140 [error code: 23]	

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>§ 3.6</p> <p>§ 3.9.1</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>	<p>§ 3.9.1</p>
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>	<p>§ 3.11.2</p>
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>	<p>§ 7.5.4</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>	<p>§ 3.11.2</p>
29	<p>The connection between the concrete and the steel profile is not possible. Increase the connector and slab strength.</p> <p>[error code: 29]</p>	
30	<p>The concrete coverage of connectors does not fulfill the requirement of AISC 360-16 I3. 2c.1.</p> <p>Increase the depth of the slab.</p> <p>[error code 30]</p>	<p>§ 3.9.1</p>

Error Code	Description	Reference
31	<p>The height of connectors above the profiled deck (at least 1 ½ in. or 38 mm) does not fulfill the requirement of AISC 360-16 I3. 2c.1.</p> <p>Increase the height of the connector or reduce the height of the profiled deck.</p> <p>[error code 31]</p>	

32	The total depth of the slab does not fulfil requirements of AS/NZS. Reduce the height of the slab. [error code 32]	§ 3.6.1 § 3.7
33	Non-compact section is not allowed for partial connection. [error code 33]	§7.6.1