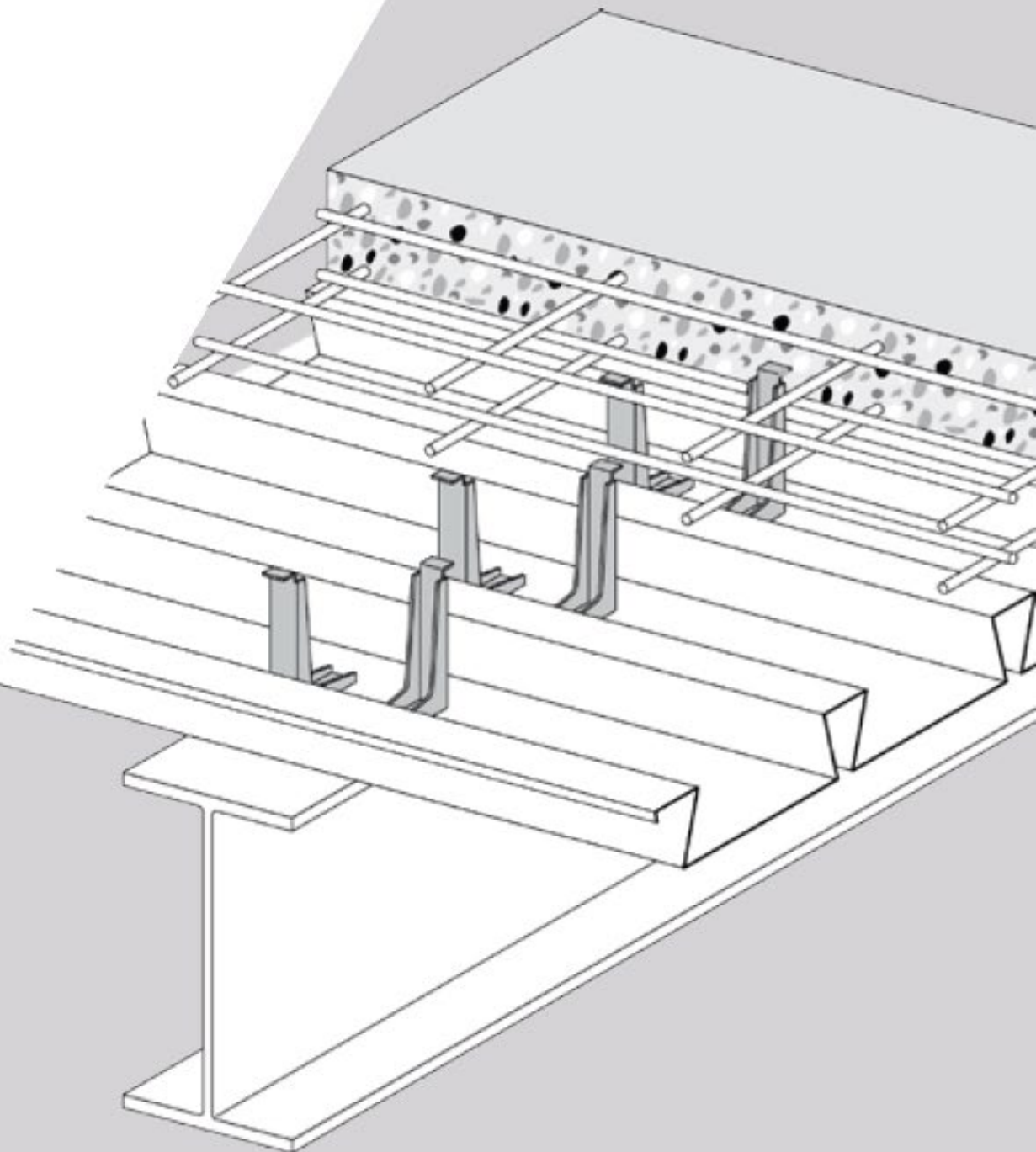




SHEAR CONNECTOR DESIGN SOFTWARE

Technical specifications



Content according to :

HILTI SHEAR CONNECTOR DESIGN - CALCULATION MODULE_TECHNICAL SPECIFICATIONS_DRV/HVB/MT/006-B

Courtesy of :

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1. INTRODUCTION

The calculation module of the Hilti Shear Connector Design software allows the user to perform the design of composite beams using HILTI X-HVB shear connectors according to the rules of the American 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 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 shear connectors;
- For the checks at Ultimate Limit States (ULS), the type of design (elastic or plastic) depends on the section slenderness. However, the user can impose an elastic design;
- The design of the beam is carried out using Load and Resistance Factor Design (LRFD); The Allowable Strength Design method (ASD) is not available in the software;
- For the checks at Serviceability Limit States (SLS), the elastic deflection and the acceleration of the beam under vibrations phenomenon are calculated;
- The calculations and design checks are carried out according to the American Standards. The European Technical Assessment ETA-15/0876 is used for the design of the connection.

2. REFERENCES

2.1. Technical references

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

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

2.2. Standards

- [2] ANSI/AISC 360-16 – Specifications for Structural Steel Buildings – American Institute for Steel Construction – Chicago – July 2016
- [3] ANSI/ASTM A36/A36M-19 – Standard specification for Carbon Structural Steel
- [4] ANSI/ASTM A572/A572M-18 – Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel
- [5] ASCE/SEI 7-16 – Minimum Design Loads and Associated Criteria for Buildings and Other Structures – American Society of Civil Engineering – 2017
- [6] Murray T.M. *et al.* – AISC Design Guide 11 “Floor Vibrations Due to Human Activity” – American Institute of Steel Construction – 1997
- [7] ACI 318-19 – Building Code Requirements for Structural Concrete – 2019
- [8] AISC Design Guide 3 – Serviceability Design Considerations for Steel Buildings, 2nd edition
- [9] Chen E.Y.L. and Ritchie J.K. – Design of Construction of Composite Floor Systems – Canadian Institute of Steel Construction – 1984

3. SOFTWARE PARAMETERS

3.1. Units

When using the American standards, the XHVB software may be used either with US customary Units (by default) or with metric Units. Table 1 gives the units used for each type of parameters, according to the selected Unit System.

When US Customary system unit is chosen, all values will be displayed (in the interface and in the calculation report) using a decimal notation (no fractional display).

For a project created for one Unit System, it is not possible to switch to the other Unit System.

Parameters	US Customary Units	Metric Units
Length (*)	foot (ft)	meter (m)
Dimension	inch (in.)	millimeter (mm)
Load (*)	pound-force (lbf); kilo pound (kip)	Newton (N); deca Newton (daN); kilo Newton (kN)
Distributed load	Load Unit / Length Unit	Load Unit / Length Unit
Surface load	Load Unit / (Length Unit) ²	Load Unit / (Length Unit) ²
Stress	pound per square inch (psi)	mega Pascal (MPa)
Mass	pound (lb)	kilogram (kg)
Density	pound-mass per cube feet (lb/ft ³)	kilogram per cube meter (kg/m ³)

Table 1 : Unit Management

This reports works by default with the US customary system. Values or Formulas for the metric system are given as a second choice between brackets.

3.2. Type of design

The steel-concrete composite beam can be checked by the software considering either a plastic or an elastic design. By default, following rules apply:

- For cross-section with compact web in bending, a plastic design is performed.
- For cross-section with non-compact web in bending, an elastic design is performed.
- Slender cross-sections are not covered by the software.

Elastic design can nevertheless always be imposed by the User for compact and non-compact sections.

4. BASIC DATA

4.1. Geometrical description of the beam

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

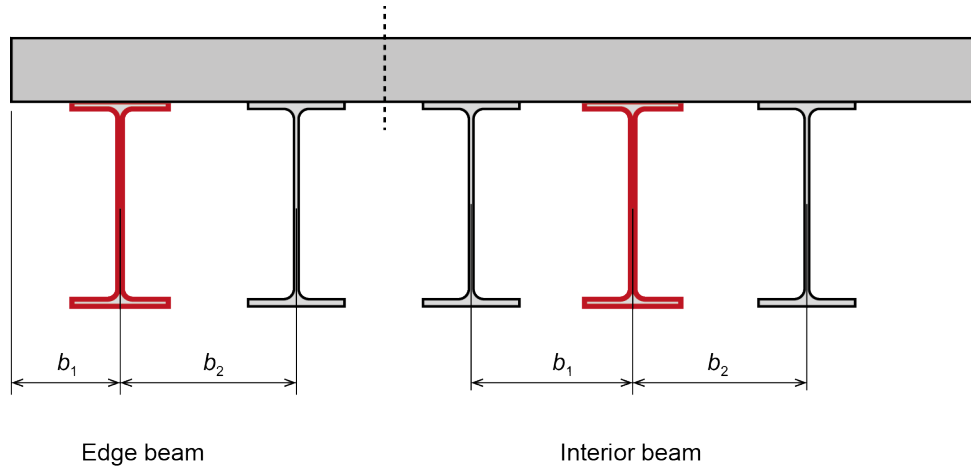


Figure 1 : Edge beam and interior beam

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

- For interior beams:
 - L is the beam length
 - b_1 is the spacing to the left beam
 - b_2 is the spacing to the right beam
- For edge beams:
 - L is the beam length
 - b_1 is the spacing to the slab edge
 - b_2 is the spacing to the adjacent beam

Error Code (see Annex D):

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

- $6,56 \leq L \leq 65,6 \text{ ft.}$ [$2 \leq L \leq 20 \text{ m}$]

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

- $b_{1,min} \leq b_1 \leq 65,6 \text{ ft.}$ [20 m], with $b_{1,min} = 1,64 \text{ ft.}$ [$0,5 \text{ m}$] for interior beams and $b_{1,min} = 0,49 \text{ ft.}$ [$0,15 \text{ m}$] for edge beams.
- $1,64 \leq b_2 \leq 65,6 \text{ ft.}$ [$0,5 \leq b_2 \leq 20 \text{ m}$]

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

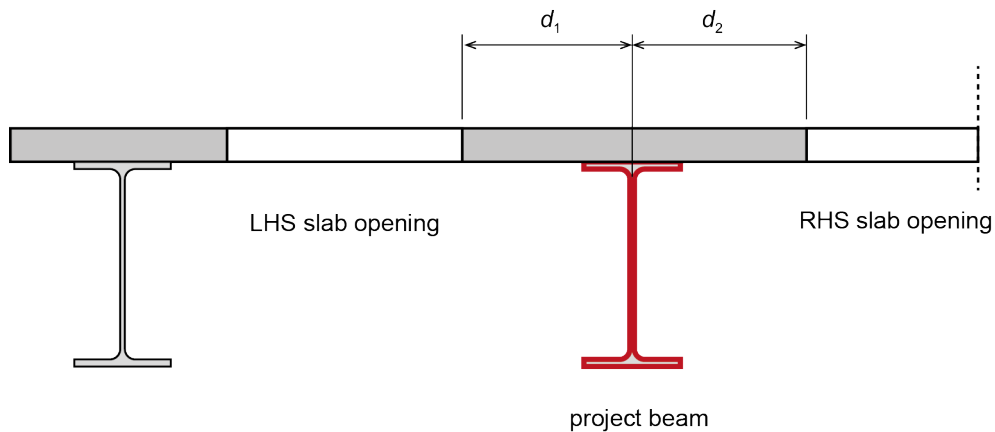


Figure 2 : Definition of slab openings

4.2. Steel section

4.2.1. Geometry

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

- h is the total height
- b is the width of the flanges
- t_f is the thickness of the flanges
- t_w is the thickness of the web
- r_1 is the root radius (only for hot rolled sections)
- r_2 is the toe radius (only for hot rolled sections)
- a is the throat of the fillet weld (only for custom sections)

Error Code (see Annex D):

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

- $t_w > 0,118 \text{ in. } [3 \text{ mm}]$
- $t_f > 0$
- $r_1 \geq 0$
- $r_2 \geq 0$
- $b > 2 r_1 + 2 r_2 + t_w$
- $h_t > 2 r_1 + 2 t_f$
- $a \geq 0$

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

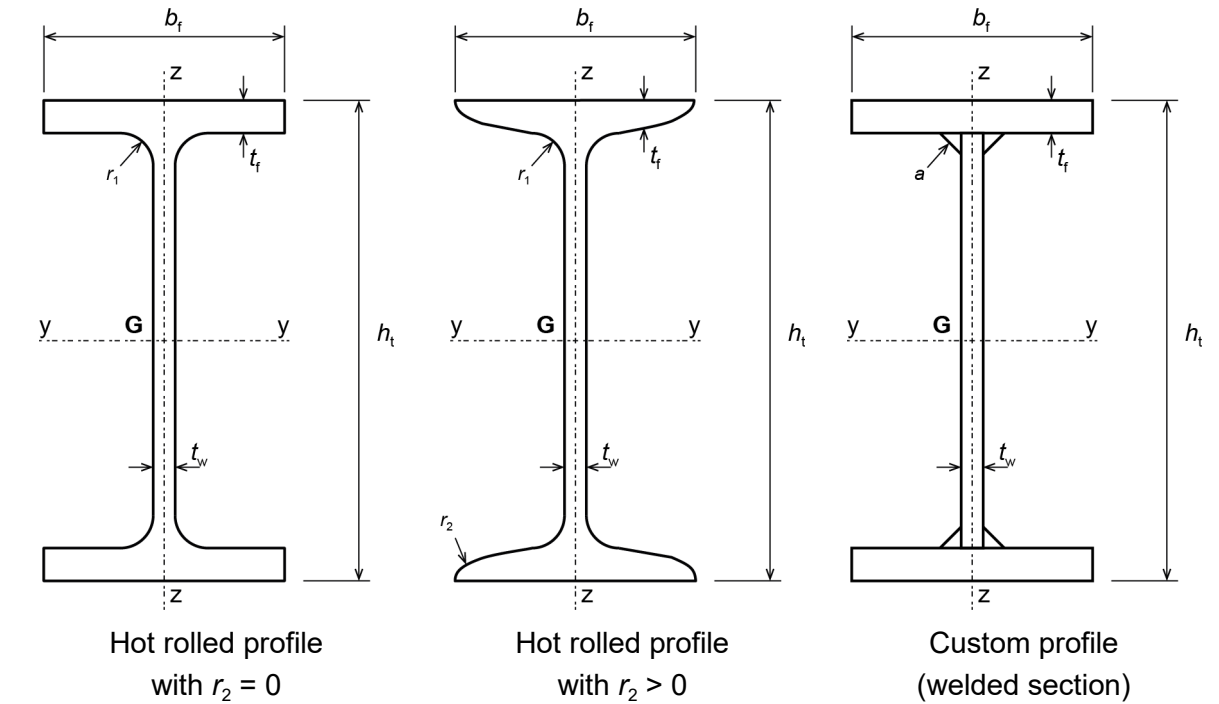


Figure 3: Steel profiles

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

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

- For solid slabs, the minimum thickness of flanges is 0,236 in. [6 mm], i.e. $t_f \geq 0,236$ in. [6 mm]. Some American profiles which have the flanges thickness smaller than 0,236 in [6 mm] are also covered.
- For slabs with formed steel decks, the minimum thickness of flanges is 0,315 in [8 mm], i.e. $t_f \geq 0,315$ in [8 mm]. From the version 2019-05, the minimum thickness of flanges can be reduced to 0,236 in [6 mm] when the relevant software option is activated (see § 4.6.6). This option extended the scope of the ETA report [1].

Error Code (see Annex D):

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

4.2.2. Steel grade

The steel grade to be chosen corresponds to the American Standards ASTM and to the scope of application for X-HVB connectors. Available grades are given in Table 2.

Standard	Grade	Label	F_y (ksi [MPa])	max. thickness (in [mm])
A36 [3]	A36	A36	36 [250]	no requirement
A572 [4]	Grade 42	A572-Grade 42	42 [290]	
A572 [4]	Grade 50	A572-Grade 50	50 [345]	

Table 2: American steel grades

Custom steel will also be available: the yield strength, defined by the user, should verify:

- US Customary unit system: $25 \text{ ksi} \leq F_y \leq 50 \text{ ksi}$
- metric unit system: $170 \text{ MPa} \leq F_y \leq 355 \text{ MPa}$

By default, the steel grade is A36.

The steel properties are calculated according to AISC-360 [2] and ASCE/SEI 7 [5]:

E is the elasticity modulus: $E = 29\,000 \text{ ksi}$ [200 000 MPa];

G is the shear modulus: $G = 11\,200 \text{ ksi}$ [77 200 MPa];

ρ_{steel} is the steel density: $\rho_{\text{steel}} = 490 \text{ lb/ft}^3$ [7850 kg/m³].

F_y is the specified minimum yield stress of the steel. For predefined grades, this value is obtained according to Table 2. For a custom steel, the yield stress is equal to the value defined by the User.

Error Code (see Annex D):

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

4.3. Concrete slab

4.3.1. Geometry

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

- Solid slab (default);
- Slab with formed steel deck (by default).

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

h is the slab thickness. The minimum slab thickness is given in Table 3, depending on the connector type and the concrete coverage (also in ETA-15/0876 [1])

In case of slab with formed steel deck, the slab thickness must fulfil the following condition (AISC Specifications I3.2c.1):

$$h - h_p \geq 2 \text{ in. [50 mm]}$$

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

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 weight of concrete per unit volume is not met.

According to AISC [2], there shall be at least ½ in. (13mm) of specified concrete cover above the top of the steel anchors (see AISC 360-16 I3.2c.1 (b)). Using American Standards, concrete coverage should thus always be required in the software. Nevertheless, the option without the effect of corrosion is available in the software and a message is displayed to warn the User that it does not fulfill the AISC requirements.

Error Codes (see Annex D):

Error code **31** is returned when the concrete coverage of connector is less than ½ in. or 13 mm.

X-HVB	with concrete coverage		without concrete coverage	
	in	mm	in	mm
40	2,362	60	1,969	50
50	2,756	70	2,362	60
80	3,937	100	3,150	80
95	4,528	115	3,740	95
110	5,118	130	4,331	110
125	5,709	145	4,921	125
140	6,299	160	5,512	140

Table 3: Minimum slab thickness from ETA agreement [1]



X-HVB 40 and 50 may be not available in some markets.

4.3.2. Concrete

The mechanical properties for concrete are defined according to AISC Specifications I1.2d [2] by:

w_c weight of concrete per unit volume (default value 125 lb/ft³ [2000 kg/m³] for normal concrete and 112 lb/ft³ [1800 kg/m³] for light concrete). This parameter must fulfil the following condition:

$$90 \leq w_c \leq 155 \text{ lb/ft}^3 \quad \text{in US customary units}$$

$$1500 \leq w_c \leq 2500 \text{ kg/m}^3 \quad \text{in metric units}$$

f'_c specified compressive strength of concrete. This value must fulfil the following conditions:

$$3000 \text{ psi [21 MPa]} \leq f'_c \leq 10\,000 \text{ psi [69 MPa]} \quad \text{for normal weight concrete}$$

$$3000 \text{ psi [21 MPa]} \leq f'_c \leq 6\,000 \text{ psi [41 MPa]} \quad \text{for lightweight concrete}$$

E_c is the modulus of elasticity of concrete, calculated by:

$$\begin{aligned} E_c &= w_c^{1.5} \sqrt{f'_c} \quad \text{in ksi} \\ E_c &= 0.043 w_c^{1.5} \sqrt{f'_c} \quad \text{in MPa} \end{aligned} \tag{1}$$

The concrete type is to be chosen between:

- Normal weight concrete;
- Light weight concrete.

4.4. Formed steel deck

4.4.1. Geometry

The geometry of the formed steel deck is defined by:

- h_r deck depth
- t_s deck thickness
- b_s trough spacing
- b_t top width of the rib
- b_b bottom width of the rib
- n_{rib} number of rib stiffeners, that can be equal to 0, 1 or 2. The value 0 is used for decks without rib stiffeners or for decks with stiffeners that can be bent down when the connectors are nailed
- s_{un} stiffener width (used only if $n_{rib} \geq 1$)
- s_{av} spacing between rib stiffeners (used only if $n_{rib} = 2$).

The average width of concrete rib w_r is defined as follows:

- for trapezoidal ribs (i.e. $b_b > b_t$): $w_r = b_t$
- for open ribs (i.e. $b_b \leq b_t$): $w_r = (b_t + b_b) / 2$

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

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

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.

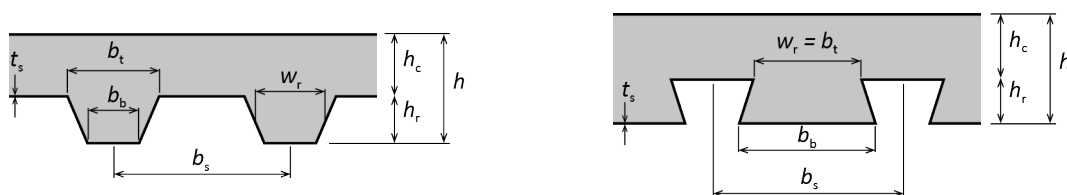


Figure 4: Dimensions of a formed steel deck



a) Rib with one stiffener

b) Rib with two stiffeners

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

Error Codes (see Annex D):

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

- $0,0197 \leq t_s \leq 0,0787$ in. $[0,5 \leq t_s \leq 2,0$ mm]
- $3,94 \leq b_s \leq 23,6$ in. $[100 \leq b_s \leq 600$ mm]
- $1,18 \leq b_t \leq 15,75$ in. $[30 \leq b_t \leq 400$ mm]
- $1,18 \leq b_b \leq 15,75$ in. $[30 \leq b_b \leq 400$ mm]
- $b_s \geq \max\{b_b, b_t\}$
- $0 \leq n_{rib} \leq 2$
- $0 \leq s_{un} \leq 1,97$ in. $[0 \leq s_{un} \leq 50$ mm]
- $0 \leq s_{av} \leq 7,87$ in. $[0 \leq s_{av} \leq 200$ mm]
- $w_b > 2 s_{un} + s_{av}$
- $h_r \leq 3$ in. (75 mm)
- $w_r = \min\{b_t, (b_t + b_b)/2\} \geq 2$ in. [50 mm]

4.4.2. Mechanical properties

The other properties of the formed steel deck are:

G_{deck} deck surface weight

F_{yp} yield strength of the steel

Error Codes (see Annex D):

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

- $0 \leq G_{deck} \leq 10,24$ lb/ft² $[0 \leq G_{deck} \leq 50$ kg/m²]
- $24,65 \leq F_{yp} \leq 87,02$ ksi $[170 \leq F_{yp} \leq 600$ MPa]

4.5. Reinforcement steel bars

The steel grade of reinforcement bars is required for the calculations. It can be chosen from:

- Grade 40 : according to ASTM-A615
- Grade 60 : according to ASTM-A615 or ASTM-A706
- Grade 75 : according to ASTM-A615
- Grade 80 : according to ASTM-A615 or ASTM-A706

The characteristic value of the yield strength of the reinforcement steel, denoted F_{ysr} , is given in Table 4.

Steel grade	Grade 40	Grade 60	Grade 75	Grade 80
F_{ysr} [ksi]	40	60	75	80
F_{ysr} [MPa]	280	420	520	550

Table 4 : Yield strength of the reinforcement steel

Error Code (see Annex D):

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

4.6. Shear connectors

4.6.1. General parameters

The user can choose the type of X-HVB connector 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 formed steel deck, connectors X-HVB 40 and X-HVB 50 are not applicable.



X-HVB 40 and 50 may be not available in some markets. Please contact your local Hilti retailer.

The dimensions of X-HVB connectors, used in calculations and for drawings, are (see Table 5 for values):

- h_{sc} total height
 w_b bottom length
 w transverse width

X-HVB	Connector height		Connectors dimensions	
	(in)	(mm)	w (mm)	w_b (mm)
40	1,69	43	24,3	51
50	2	52	24,3	50
80	3,15	80	24,3	50
95	3,74	95	24,3	50
110	4,43	112,5	31	51
125	5,02	127,5	31	51
140	5,61	142,5	31	51

Table 5 : Main dimensions of X-HVB connectors – According to ETA [1]

For composite slab (with formed steel deck), the height of the deck should not be greater than the values given in Table 6.

For formed steel deck with ribs perpendicular to the beam axis and with connectors parallel with the beam axis, the following additional condition must be fulfilled:

$$w_r / h_r \geq 1,0$$

(2)

Additionally, according to AISC 360-16 I3. 2c.1 (b), the following condition should be fulfilled in presence of formed steel deck:

$$h_{sc} \geq h_r + 1.5 \text{ in } [38 \text{ mm}] \quad (3)$$

X-HVB	Maximum value of h_p (in)			Maximum value of h_p (mm)		
	$\alpha \geq 1,8$	$1,0 < \alpha < 1,8$	$\alpha \leq 1,0$	$\alpha \geq 1,8$	$1,0 < \alpha < 1,8$	$\alpha \leq 1,0$
80	1,77	1,77	1,18	45	45	30
95	2,36	2,24	1,77	60	57	45
110	2,95	2,60	2,36	75	66	60
125	3,15	2,95	2,87	80	75	73
140	3,15	3,15	3,15	80	80	80

where $\alpha = w_r / h_r$

Table 6 : Maximum height of the profiled steel sheeting

Error Code (see Annex D):

The consistency of the selected connector with the previously defined parameters is also be checked as follows:

- first check: control of the minimum slab thickness, according to the requirement of concrete coverage – see Table 3. If the slab thickness is smaller than the minimum one, a warning message on the interface should alert the user about the inconsistency of the values. If the user does not correct the value and launches the calculations, the calculation module will send back an error index= 24.
- second check: for slabs with formed steel deck, a second check is performed regarding the maximum height of the composite decking, see Table 6. If the height of the profiled steel sheeting is greater than the maximum one, a warning message on the interface should alert the user about the inconsistency of the values. If the user does not correct the value and launches the calculation, the calculation module will send back an error index = 25.

If the condition of AISC 360-16 I3. 2c.1 (b) is not fulfilled (minimum height of connector above profiled deck), Error index = 30. If the condition of AISC 360-16 I3. 2c.1 (b) is not fulfilled (minimum concrete coverage of connectors), Error index = 31.

4.6.2. Shear resistance

The values of the nominal shear resistance Q_n of X-HVB connectors are given in Table 7 (according to Table 3 of ETA 15-0876 [1]).

X-HVB	Nominal shear strength Q_n	
	(lb)	(kN)
40	6 520	29
50	6 520	29
80	7 306	32,5
95	7 868	35
110	7 868	35
125	8 430	37,5
140	8 430	37,5

Table 7 : Nominal shear resistance – According to ETA [1]

4.6.3. 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 always duckwalk;
- for solid slabs with other connectors, the orientation is always longitudinal;
- for slabs with formed steel deck with ribs parallel to the beams axis, the orientation is always longitudinal.

For slabs with formed steel deck with ribs transverse to the beam axis, the orientation of connectors must be chosen between:

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

4.6.4. Degree of connection

When a plastic design of the beam is performed (see § 3.2), the User has to choose the degree of connection for the calculation. The three possible choices are:

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

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

For partial connection, the minimum degree of shear connection is defined by AISC Commentary I3.2.2d.1 [2], as explained in § 8.5.4. If the User directly imposes the degree of shear connection (third choice), the software does not necessarily fulfil the latter condition. In this case, the relevant warning message is then given in the calculation sheet.

4.6.5. 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 formed steel deck, 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 Standards requirements.

For composite slabs with transverse sheeting, the software tries by default to optimise the connection. In order to minimise the number of connectors that are necessary to fulfil all the Standards 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.



According to AISC 360-16 I8.2d. (a), the steel anchors shall be uniformly distributed along the beam between the point of maximal bending and the adjacent points of zero moment, unless specified otherwise on the contract document. The optimisation of the connection is nevertheless be available in the software with American Standards, as default option. The User should then pay attention to its contract document. A warning message is given to the User.

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 formed steel deck, 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.

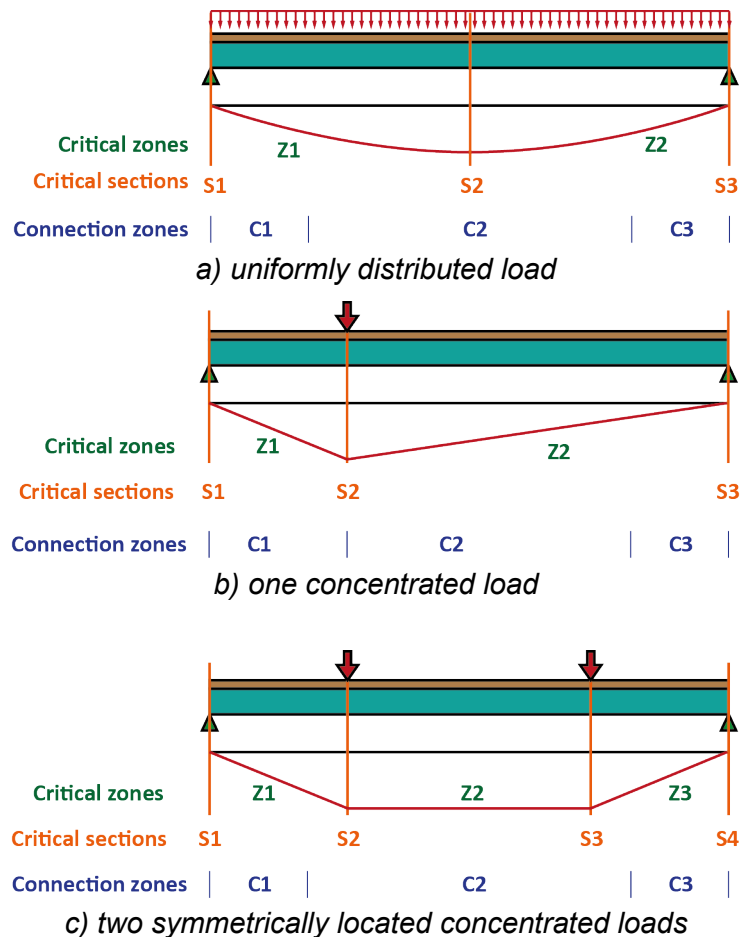


Figure 6 : Configurations of critical sections and connection zones.

4.6.6. 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 0,236 and 0,315 in. [6 and 8 mm], even for composite slabs (see § 4.2.1);
- the possibility to have only one row of connectors in formed steel deck with narrow ribs (see § 4.7.4).

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

- Composite slab with formed steel deck;
- Formed steel deck is perpendicular to the beam;
- The selected formed steel deck 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.

4.7. Spacing and positioning of connectors along the beam

4.7.1. General

At the end of the calculation, the software proposes a connector arrangement that is compatible with the User parameters and that fulfils the rules of ETA [1].

4.7.2. Maximum spacing connectors

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 23,6 in [600 mm]:

$$d \leq \min \{4 h; 23,6 \text{ in. [600 mm]}\} \quad (4)$$



The rule of AISC 360-16 I8. 2d.(e) leads to a less stringent requirement: $d \leq \min (8 h; 36 \text{ in. (900 mm)})$. ETA rule is then applied in the software.

4.7.3. Solid slab with multiple rows of connectors

The flange width, denoted b , of the steel section must fulfil the following condition (cf. Figure 7):

$$b \geq 50 (n_r - 1) + w \quad (5)$$

Where w is the transverse width of the connector and n_r the number of connectors in a row. This condition may limit the maximum number of connectors in a row – see § 8.5.1.

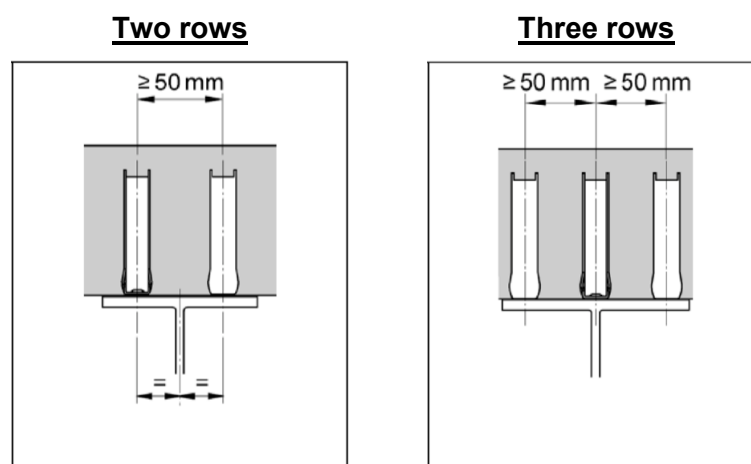


Figure 7 : Spacing of connectors for solid slabs

4.7.4. Slabs with transverse steel deck and connectors parallel to the beam axis

▪ Single row, without rib stiffeners

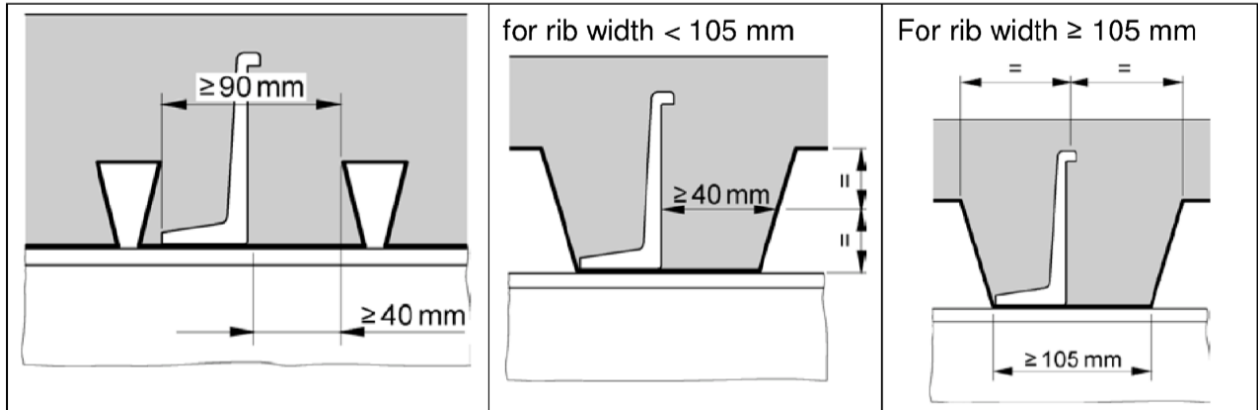


Figure 8 : Spacing of connectors for slabs with transverse decking and connectors parallel with the beam (single row).

For the rib width smaller than $b_b < 4,13$ in. [105 mm], the width at mid-height of the rib must fulfil the following condition:

$$w_r \geq \max \{w_b + 1,57 \text{ in. [40 mm]}; 3,54 \text{ in. [90 mm]}\} \quad (6)$$

Where w_b is the bottom width of the connector.

Error Code (see Annex D):

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

▪ Single row or multiple row, with rib stiffeners

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

$$\min \{b_b; b_s\} \geq 2 w_b + s_{un} \quad (7)$$

Error Code (see Annex D):

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

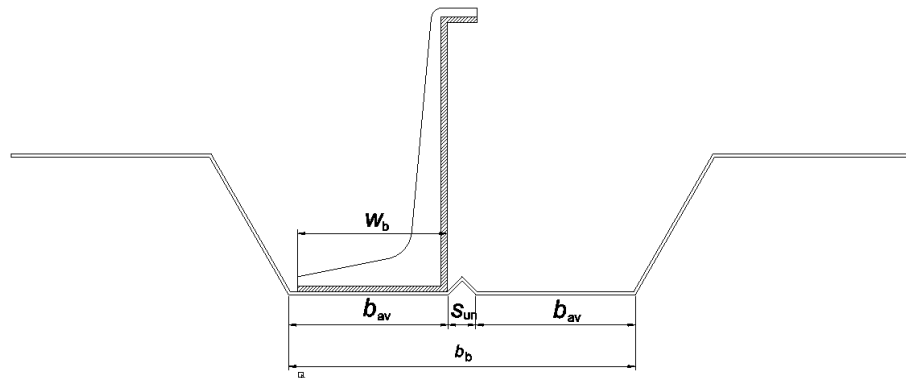


Figure 9 : Transverse sheeting with one rib stiffener and parallel connectors.

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

- if the spacing of stiffeners is 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:

$$\min \{b_b; b_t\} \geq 2 w_b + 2 s_{un} + 2 s_{av} \quad (8)$$

Error Code (see Annex D):

Error code **28** is returned when the two locations of the connectors shown on Figures 10 and 11 are not possible.

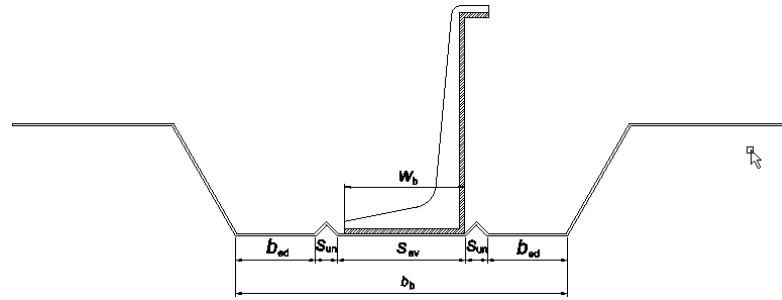


Figure 10 : Transverse sheeting with two rib stiffeners and parallel connectors.

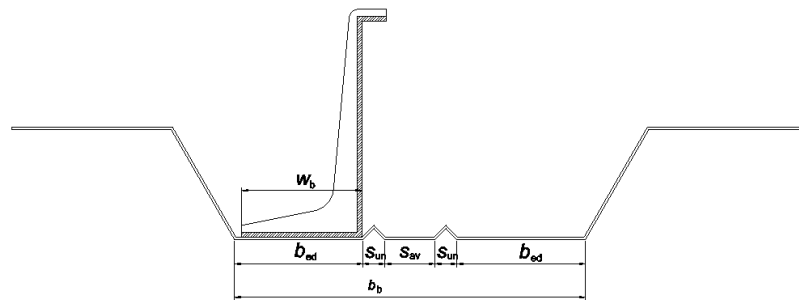
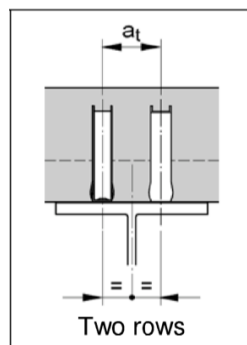


Figure 11 : Transverse sheeting with two rib stiffeners and parallel connectors.

- **Multiple rows, without rib stiffener**

Two rows



Three rows

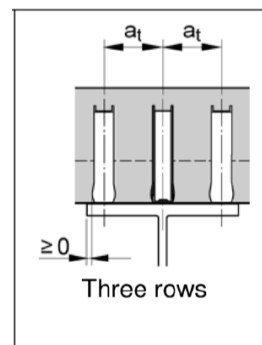


Figure 12 : Spacing of connectors for slabs with transverse decking and connectors parallel with the beam.

The following conditions must be fulfilled:

- For the flange width of the steel section:

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

Where: $a_t = 2 \text{ in. [50 mm]}$ for profiled decking with $w_r/h_r \geq 1,8$

(9)

$a_t = 4 \text{ in. [100 mm]}$ for other decking

This condition may limit the maximum number of connectors in a row – see § 8.5.1.

- For the bottom width of the rib:

$$b_b \geq 2,36 \text{ in. [60 mm]} \quad (10)$$

Error Code (see Annex D):

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

4.7.5. Slabs with transverse steel deck and connectors transverse to the beam axis

- **Single row, without rib stiffener**

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

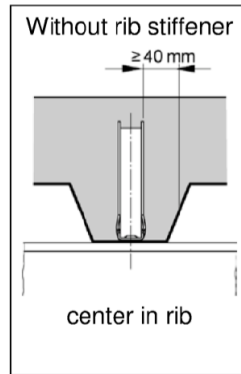


Figure 13 : Spacing of connectors for slabs with transverse decking and connectors transverse to the beam (single row).

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

$$w_r \geq w + 3,15 \text{ in. [80 mm]} \quad (11)$$

Error Code (see Annex D):

Error code **26** is returned when this condition is not met and if $b_b < 1,57 \text{ in. [40 mm]}$ (see conditions for multiple rows).

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

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

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

$$\begin{aligned} b_0 &< w + 3,15 \text{ in. [80 mm]} \\ b_b &\geq 1,57 \text{ in. [40 mm]} \end{aligned} \quad (12)$$

In this case, the calculation module starts the design process by considering 2 connectors in a row.

The minimum width of ribs for one connector can be reduced when the relevant software option is activated (see § 4.6.6). This option extended the scope of the ETA report [1]. In this case, following conditions are checked: $b_b \geq 1,18 \text{ in. [30 mm]}$ and $b_t \geq 1,18 \text{ in. [30 mm]}$.

▪ **Single rows and multiple rows, with rib stiffeners**

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

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

Error Code (see Annex D):

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

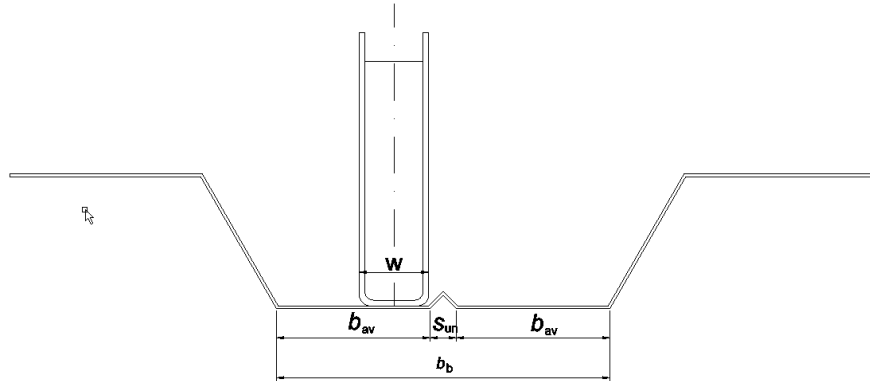


Figure 14 : Transverse sheeting with one rib stiffener and transverse connectors.

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

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

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

$$\min (b_b; b_t) \geq 2 w + 2 s_{un} + 2 s_{av} \quad (14)$$

Error Code (see Annex D):

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

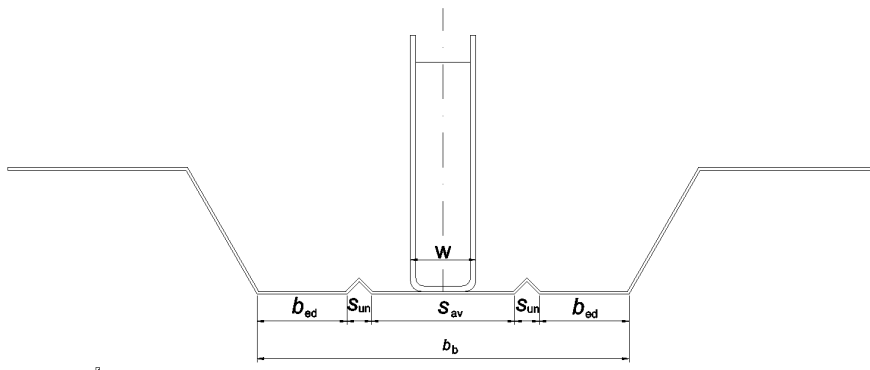


Figure 15 : Transverse sheeting with two rib stiffeners and transverse connectors.

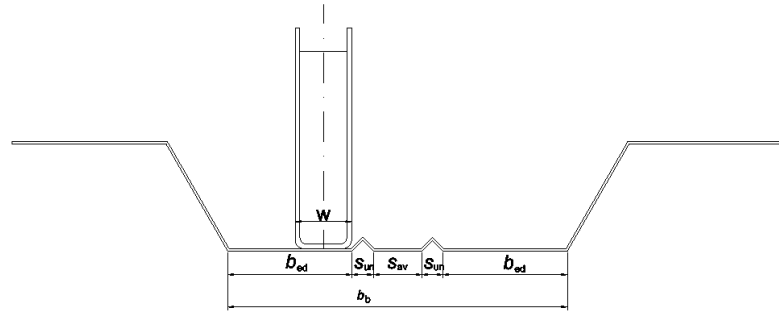


Figure 16 : Transverse sheeting with two rib stiffeners and transverse connectors.

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

▪ **Multiple rows, without rib stiffener**

For formed steel deck without rib stiffener, the following conditions must be fulfilled:

- For the flange width of the steel section:

$$b \geq (n_r - 1) a_t \quad (15)$$

Where: $a_t = 2$ in. [50 mm] for formed deck with $b_0/h_p \geq 1,8$ and with two rows of connectors

$a_t = 4$ in. [100 mm] for other deck or with three rows of connectors

This condition may limit the maximum number of connectors in a row – see § 8.5.1.

- For the bottom width of the rib:

$$b_b \geq 1,57 \text{ in. [40 mm]} \quad (16)$$

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

- For the bottom width of connectors:

$$a_t \geq 2 w_b \quad (17)$$

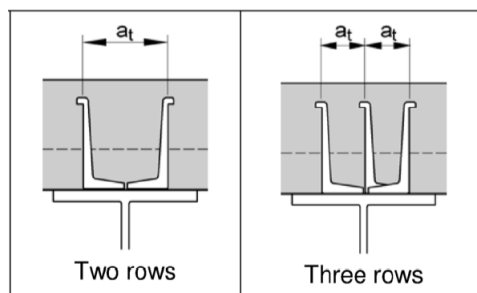


Figure 17 : Spacing of connectors for slabs with transverse decking and connectors transverse to the beam (multiple rows).

4.7.6. Slabs with parallel formed steel deck

For composite beams with parallel formed steel deck, the connectors are always located parallel to the beam axis.

▪ **Single rows, without rib stiffener**

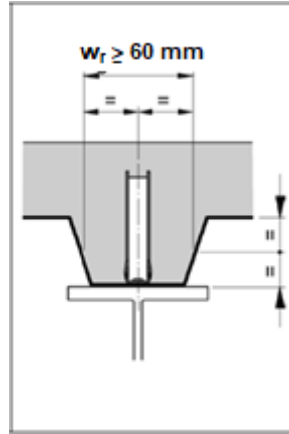


Figure 18 : Spacing of connectors for slabs with parallel decking (single rows).

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

$$w_r \geq 2,36 \text{ in. [60 mm]} \quad (18)$$

Error Code (see Annex D):

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

▪ **Single rows, with rib stiffeners**

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

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

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

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

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

$$b_f / 2 \geq s + w/2, \text{ where } s = (b_b + s_{un}) / 2 \quad (19)$$

$$b_{0,split} \geq 2,36 \text{ in. [60 mm]}, \text{ where } b_{0,split} = w_r + (b_f - b_b) \quad (20)$$

Error Code (see Annex D):

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

If this splitting 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:

$$s_{av} \geq w \quad (21)$$

Error Code (see Annex D):

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

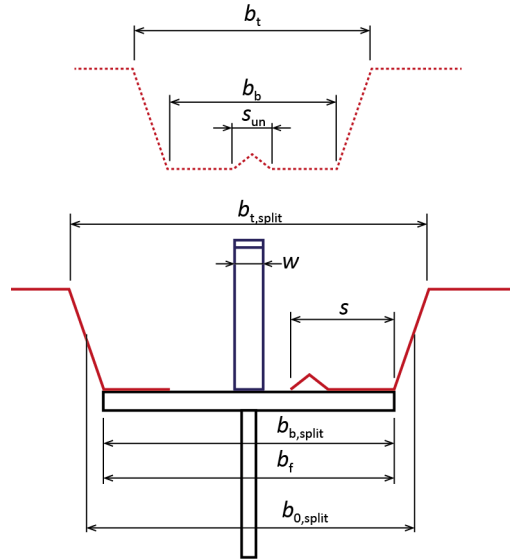


Figure 19 : Splitting of a rib with one rib stiffener (one row of parallel connectors).

▪ **Multiple rows, without rib stiffener**

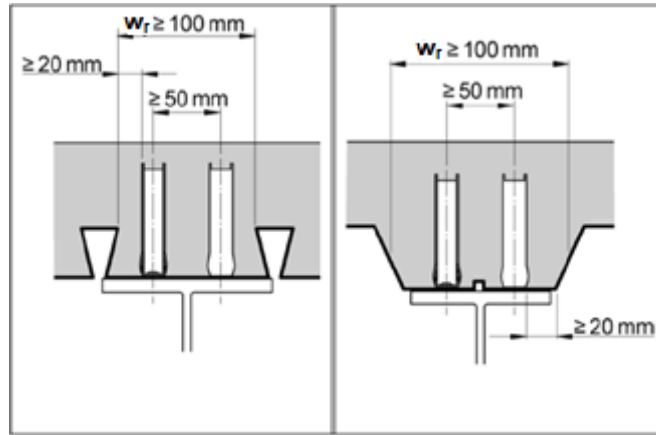


Figure 20 : Spacing of connectors for slabs with decking parallel with the beam (multiple rows).

The following conditions must be fulfilled:

- For the flange width of the steel section:

$$b \geq w_b + 2 \text{ in. [50 mm]} \quad (22)$$

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

$$\min (b_b; b_t) \geq w + 3,54 \text{ in. [90 mm]} \quad (23)$$

$$w_r \geq 4 \text{ in. [100 mm]} \quad (24)$$

▪ **Multiple rows, with rib stiffeners**

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

- For the flange width of the steel section:

$$b \geq w + \max\{2 \text{ in. [50 mm]}; s_{un} + w\} \quad (25)$$

- For the width of the rib:

$$\min(b_b; b_t) \geq w + 1,57 \text{ in. [40 mm]} + \max\{2 \text{ in. [50 mm]}; s_{un} + w\} \quad (26)$$

$$w_r \geq 4 \text{ in. [100 mm]} \quad (27)$$

Error Code (see Annex D):

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

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

- if $s_{av} \geq w + 2 \text{ in. [50 mm]}$ (see Figure 21), the connectors are located between the stiffeners. The same conditions as the case of sheetings without rib stiffener must be fulfilled.

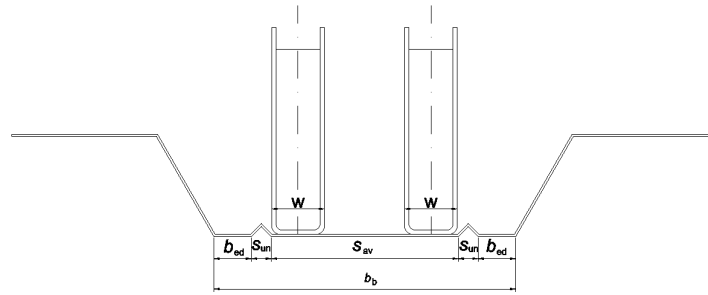


Figure 21 : Sheeting with two rib stiffeners and connectors parallel with the beam (multiple rows).

- if $s_{av} < w + 2 \text{ in. [50 mm]}$, the connectors are located outside the stiffeners, if the following conditions are fulfilled:

- For the flange width of the steel section:

$$b \geq w + \max\{2 \text{ in. [50 mm]} - w; s_{av} + 2 s_{un}\} \quad (28)$$

- For the width of the rib:

$$\min(b_b; b_t) \geq 2 w + 1,57 \text{ in.} + \max\{2 \text{ in.} - w; s_{av} + 2 s_{un}\} \text{ US customary units} \quad (29)$$

$$\min(b_b; b_t) \geq 2 w + 40 \text{ mm} + \max\{50 \text{ mm} - w; s_{av} + 2 s_{un}\} \text{ metric units}$$

$$w_r \geq 4 \text{ in. [100 mm]} \quad (30)$$

Error Code (see Annex D):

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

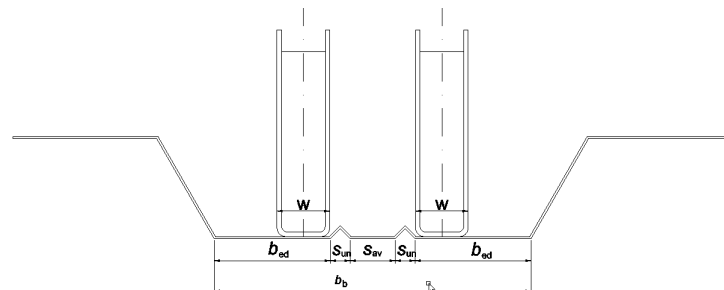


Figure 22 : Sheeting with two rib stiffeners and connectors parallel with the beam (multiple rows).

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

The minimum spacing between 2 connectors is 4 in. [100 mm].

This condition may limit the maximum number of connection rows – see § 8.5.1.

4.8. Loads

4.8.1. General definitions

The calculation module allows the user to define elementary variable load cases that will be used in the combinations of actions for ULS or SLS according to ASCE 7-16.

Only gravity loads are considered (downwards).

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

- One permanent load case, denoted **D**
- One live load during construction stage, denoted **L_c**
- Up to two live load cases during the final stage, denoted **L₁** and **L₂**

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

For each load case, it 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 **D**, 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 Interface.

For the final stage live load **L₁**, only the uniformly distributed surface load can be defined in the Interface.

4.8.2. Specific treatment of surface loads

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

For a “secondary interior beam”, a linear uniformly distributed load along the beam is derived:

$$q_{lin} = q_{surf} (b_1 + b_2)/2 \quad (31)$$

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

$$q_{lin} = q_{surf} (b_1 + b_2/2) \quad (32)$$

4.8.3. Automatic dead load assessment

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

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

$$\begin{aligned} \text{for interior beams : } q_{\text{surf,d}} &= 2 \frac{g \rho_{\text{steel}} A}{b_1 + b_2} \\ \text{for edge beams : } q_{\text{surf,d}} &= 2 \frac{g \rho_{\text{steel}} A}{2 b_1 + b_2} \end{aligned} \quad (33)$$

where: g is the gravity constant: $g = 32,2 \text{ ft/s}^2$ [$9,81 \text{ m/s}^2$]

ρ_{steel} : see § 4.2.2

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:

$$q_{\text{slab}} = g \rho_c h_{\text{eq}} \quad (34)$$

where: ρ_c : see § 4.3

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

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

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

h_p , b_r , b_b , b_s : see § 4.4

h : see § 4.3

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

$$q_{\text{deck}} = g G_{\text{deck}} \quad (35)$$

where: G_{deck} : see § 4.4.

4.8.4. Default surface live loads

The default surface live load (for live load case L_1) is defined according to the category of loaded surface and the National Annex – see Table 8. Values are given in EN 1991-1-1 and the associated NA.

Categories	Uniform load		Live load reduction ?
	customary (psf)	metric (kN/m ²)	
Assembly areas	100	4,79	No
Dining rooms and restaurant	100	4,79	No
Hospitals	60	2,87	Yes
Libraries	60	2,87	Yes
Manufacturing	125	6,00	No
Office building	50	2,40	Yes
Recreational uses	100	4,79	No
Residential	40	1,92	Yes
Roofs	20	0,96	Yes
Schools	40	1,92	Yes
Storage warehouse	125	6,00	No

Table 8 : Default surface live load

4.8.5. Live load reduction

When the option will be activated for the relevant live loads (see last column of Table 8 to check if the live load reduction is allowed), the value of the Live load considered in the calculation will be reduced as follows, according to ASCE 7-16 § 4.7.2:

$$\begin{aligned} \text{US Customary system unit: } L &= L_0 \left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}} \right) \\ \text{metric system unit: } L &= L_0 \left(0.25 + \frac{4.57}{\sqrt{K_{LL} A_T}} \right) \end{aligned} \quad (36)$$

where: L is the reduced live load;

L_0 is the default live load as given in Table 8;

K_{LL} is the live load element factor;

A_T is the tributary area (in m^2 or ft^2);

The product $K_{LL} A_T$ is defined as follows:

for interior beams: $K_{LL} A_T = L (b_1 + b_2)/2$

for edge beams: $K_{LL} A_T = L (b_1 + b_2)/2$

5. COMBINATIONS OF LOADS

5.1. Ultimate Limit States (ULS)

The ULS combinations are automatically generated according to ASCE7-16 §2.3.1:

$$\begin{aligned} 1,4 \mathbf{D} \\ 1,2 \mathbf{D} + 1,6 \mathbf{L} \end{aligned} \quad (37)$$

When 2 live loads $L1$ and $L2$ are defined, the latter equation will apply as follows:

$$1,2 \mathbf{D} + 1,6 (\mathbf{L1} + \mathbf{L2}) \quad (38)$$

It will be possible for the User to modify the load factors in the combinations.

5.2. Serviceability Limit States (SLS)

The SLS combination for the verification of deflections is automatically generated according to ASCE7-16 Appendix CC:

$$\mathbf{D} + \mathbf{L} \quad (39)$$

The SLS combination for the calculation of natural frequencies is:

$$\mathbf{D} + p_L \mathbf{L} \quad (40)$$

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

When 2 live loads $L1$ and $L2$ are defined, the latter equations will apply as follows:

$$\begin{aligned} \mathbf{D} + (\mathbf{L1} + \mathbf{L2}) \\ \mathbf{D} + p_L (\mathbf{L1} + \mathbf{L2}) \end{aligned} \quad (41)$$

Error Code (see Annex D):

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

- $0 \leq p_L \leq 50 \%$

6. GLOBAL ANALYSIS

6.1. 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.

6.2. Critical sections

ULS verifications are carried out at critical sections (AISC Specifications [2] I.8 2c) where:

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

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

The calculation of internal forces and moments is described hereafter for each individual point load and patch load. Any surface load will be considered with these 2 methods according to § 4.8.2.

6.3.1. Point load

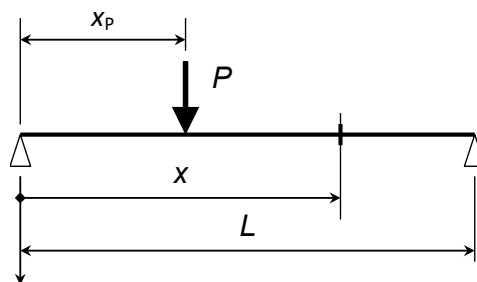


Figure 23 : Point load.

The reactions at supports are calculated by:

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

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

where: P is the applied point load;

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

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

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

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

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

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

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

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

6.3.2. Patch load

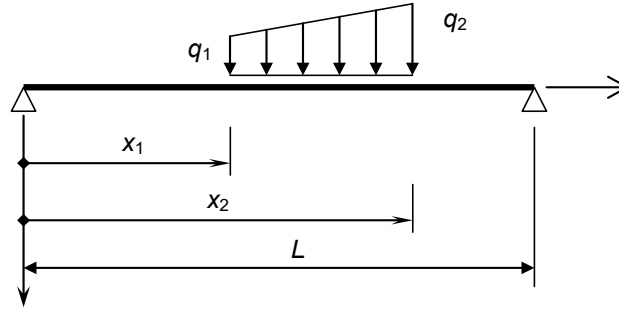


Figure 24 : Patch load.

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 24.

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:

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

6.4. Precambering

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

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

$$w_{0,x} = -4 w_0 \left(1 - \frac{x}{L} \right) \frac{x}{L} \quad (42)$$

Where w_0 is the preambering.

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

6.5. Influence of the connectors slip

The influence of the connector slip on the beam deflection will be treated according to AISC Commentary I3.2 [2]. For beams with partial connection, elastic deflections are calculated by using the equivalent moment of inertia:

$$I_{equiv} = I_a + \sqrt{\eta} (I_c - I_a) \quad (43)$$

where: η is the degree of connection of the beam (see § 8.5.5). Its value must fulfil the following condition:

$$\eta \geq 0,25$$

I_a is the maximum deflection of the beam considering only the steel beam flexural stiffness;

I_c is the maximum deflection of the beam considering the composite action with a full connection.

7. VERIFICATIONS AT THE CONSTRUCTION STAGE

7.1. General

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

The LFRD verifications for the steel beam (without concrete slab) include:

- Limit state of yielding for compact sections (compact web and compact flanges);
- Limit state of Lateral Torsional Buckling;
- Limit state of compression flange yielding for sections with non-compact web;
- Limit state of compression flange local buckling for sections with non-compact flanges;
- Web resistance to shear buckling.

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.

7.2. LFRD verifications

7.2.1. Classification of the cross-section for local buckling

The classification of the cross-section is carried out according to the Table B4.1b of AISI Specifications [2]:

- For the compressed flange of rolled sections:

$$\text{Compact:} \quad \text{if } 0,5b/t_f \leq 0,38\sqrt{E/F_y} \quad (44)$$

$$\text{Non-compact,} \quad \text{if } 0,5b/t_f \leq 1,0\sqrt{E/F_y} \quad (45)$$

$$\text{Slender,} \quad \text{otherwise}$$

- For the compressed flange of built-up sections:

$$\text{Compact:} \quad \text{if } 0,5b/t_f \leq 0,38\sqrt{E/F_y} \quad (46)$$

$$\text{Non-compact,} \quad \text{if } 0,5b/t_f \leq 0,95\sqrt{k_c E/F_L} \quad (47)$$

$$\text{Slender,} \quad \text{otherwise}$$

Where:

$$k_c = 4/\sqrt{(h_t - 2t_f - 2r_1)/t_w}; \quad 0,35 \leq k_c \leq 0,76$$

$$F_L = 0,7F_y \quad \text{for slender web, compact and non-compact web with } W_{el,y,t}/W_{el,y,c} \geq 0,7$$

$$F_L = F_y \quad W_{el,y,t}/W_{el,y,c} \geq 0,5F_y \quad \text{for compact and non-compact web with } W_{el,y,t}/W_{el,y,c} < 0,7$$

$$W_{el,y,c}, W_{el,y,t} = \text{elastic section modulus referred to compression and tension flanges, respectively}$$

- For the web in bending:

$$\text{Compact:} \quad \text{if } (h_t - 2t_f - 2r_1)/t_w \leq 3.76\sqrt{E/F_y} \quad (48)$$

$$\text{Non-compact,} \quad \text{if } (h_t - 2t_f - 2r_1)/t_w \leq 5.70\sqrt{E/F_y} \quad (49)$$

Slender, otherwise

As slender cross-sections are not covered by AISC code, they are excluded from the scope of the X-HVB software.

Error code (see Annex D):

If a cross-section is classified as slender, other calculations are not performed and an error message is returned (error code = 11).

7.2.2. Limit state of yielding for compact sections (compact web and compact flanges)

The limit state of yielding (Y) applies only for compact sections (where web and flanges are both compact). The criterion for this limit state is calculated as follows (see AISC 360-16 – F2.1):

$$\Gamma_Y = M_u/(\phi M_n) \leq 1.0 \quad (50)$$

where: M_u maximum design bending moment along the beam using LRFD load combination

ϕ resistance factor, $\phi = 0.90$ (AISC Specifications F.1 [2])

M_n nominal flexural strength according to the limit states of yielding, given by:

$$M_n = M_p = W_{pl,y} F_y \quad (51)$$

$W_{pl,y}$ plastic section modulus (see Annex A)



It is not possible for the User to modify the value of resistance factors, except for the elastic resistance criteria.

7.2.3. Limit state of Lateral Torsional Buckling

The criterion for LTB resistance is calculated as follows:

$$\Gamma_{LT} = M_u/(\phi M_{n,LT}) \leq 1.0 \quad (52)$$

where: M_u maximum design bending moment along the beam using LRFD load combination

ϕ resistance factor, $\phi = 0.90$ (AISC Specifications F.1 [2])

$M_{n,LT}$ nominal flexural strength according to the limit states of LTB

$M_{n,LT} = M_p$ if $L \leq L_p$ for all types of section

▪ **Sections with compact web, with compact or non-compact flanges:**

(see AISC 360-16 F2.2 and F3.1)

$M_{n,LT}$ nominal flexural strength according to the limit states of LTB, given by:

When $L_p < L \leq L_r$:

$$M_{n,LT} = C_b \left[M_p - (M_p - 0.7F_y W_{el,y}) \left(\frac{L - L_p}{L_r - L_p} \right) \right] \leq M_p \quad (53)$$

When $L_r < L$:

$$M_{n,LT} = F_{cr} W_{el,y} \leq M_p \quad (54)$$

F_{cr} critical stress:

$$F_{cr} = C_b \frac{\pi^2 E}{\left(\frac{L}{r_{ts}} \right)^2} \sqrt{1 + 0.078 \frac{I_t}{W_{el,y} h_0} \left(\frac{L}{r_{ts}} \right)^2} \quad (55)$$

$$r_{ts}^2 = \frac{\sqrt{I_z I_w}}{W_{el,y}} \quad (56)$$

L_p limiting laterally unbraced length for the limit state of yielding:

$$L_p = 1.76 r_z \sqrt{\frac{E}{F_y}} \quad (57)$$

r_z radius of gyration about z-axis

L_r limiting unbraced length for the limit state of inelastic LTB:

$$L_r = 1.95 r_{ts} \frac{E}{0.7 F_y} \left[\sqrt{\frac{I_t}{W_{el,y} h_0} + \sqrt{\left(\frac{I_t}{W_{el,y} h_0} \right)^2 + 6.76 \left(\frac{0.7 F_y}{E} \right)^2}} \right] \quad (58)$$

▪ **Sections with non-compact web, with compact or non-compact flanges:**

(see AISC 360-16 F4.2)

$M_{n,LT}$ nominal flexural strength according to the limit states of LTB, given by:

When $L_p < L \leq L_r$:

$$M_{n,LT} = C_b \left[R_{pc} M_{yc} - (R_{pc} M_{yc} - F_L W_{el,y,c}) \left(\frac{L - L_p}{L_r - L_p} \right) \right] \leq R_{pc} M_{yc} \quad (59)$$

When $L_r < L$:

$$M_{n,LT} = F_{cr} W_{el,y,c} \leq R_{pc} M_{yc} \quad (60)$$

M_{yc} yield moment in the compression flange: $M_{yc} = F_y W_{el,y,c}$

F_L nominal compression flange stress above which the inelastic buckling limit states apply:

$$F_L = 0.7F_y \quad \text{if } W_{el,y,t}/W_{el,y,c} \geq 0.7$$

$$F_L = F_y W_{el,y,t}/W_{el,y,c} \geq 0.5F_y \quad \text{if } W_{el,y,t}/W_{el,y,c} < 0.7$$

$W_{el,y,c}$, $W_{el,y,t}$ = elastic section modulus referred to compression and tension flanges, respectively

F_{cr} critical stress:

$$F_{cr} = C_b \frac{\pi^2 E}{\left(\frac{L}{r_t}\right)^2} \sqrt{1 + 0.078 \frac{I_t}{W_{el,y,c} h_0} \left(\frac{L}{r_t}\right)^2} \quad (61)$$

$$I_t = 0 \text{ when } I_{zc}/I_z \leq 0.23$$

$$I_{zc} = \text{moment of inertia of the compression flange about z-axis: } I_{zc} = 2t_f b_f^3 / 12$$

r_t effective radius of gyration for LTB:

$$r_t = \frac{b_{fc}}{\sqrt{12 \left(1 + \frac{1}{6} a_w\right)}} \quad (62)$$

$$a_w = \frac{h_c t_w}{b_f t_f} \quad (63)$$

$$h_c = h_t - 2t_f - 2r_1 \quad \text{for rolled sections}$$

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

L_p limiting laterally unbraced length for the limit state of yielding:

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}} \quad (64)$$

L_r limiting unbraced length for the limit state of inelastic LTB:

$$L_r = 1.95 r_t \frac{E}{F_L} \left[\sqrt{\frac{I_t}{W_{el,y,c} h_0} + \sqrt{\left(\frac{I_t}{W_{el,y,c} h_0}\right)^2 + 6.76 \left(\frac{F_L}{E}\right)^2}} \right] \quad (65)$$

R_{pc} web plastification factor:

When $I_{zc}/I_z > 0.23$:

$$\text{When } h_c/t_w \leq \lambda_p: \quad R_{pc} = \frac{M_p}{M_{yc}} \quad (66)$$

$$\text{When } h_c/t_w > \lambda_p: \quad R_{pc} = \frac{M_p}{M_{yc}} - \left(\frac{M_p}{M_{yc}} - 1\right) \left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right) \leq \frac{M_p}{M_{yc}} \quad (67)$$

When $I_{zc}/I_z \leq 0.23$:

$$R_{pc} = 1.0 \quad (68)$$

$$M_p = W_{pl,y} F_y \leq 1.6 W_{el,y} F_y$$

$$\lambda = h_c / t_w$$

▪ **For all types of sections**

h_0 distance between the flange centroids: $h_0 = h_t - t_f$

C_b LTB modification factor for non-uniform moment diagrams when both ends of the beam are braced:

$$C_b = \frac{12.5 M_{\max}}{2.5 M_{\max} + 3 M_A + 4 M_B + 3 M_C} \quad (69)$$

M_{\max} absolute value of maximum moment along the beam

M_A absolute value of moment at quarter point of the beam

M_B absolute value of moment at centerline of the beam

M_C absolute value of moment at three-quarter point of the beam

7.2.4. Limit state of compression flange yielding for sections with non-compact web

The limit state of compression flange yielding (CFY) applies only for sections with noncompact web. The criterion for this limit state is calculated as follows (see AISC 360-16 – F4.1):

$$\Gamma_{CFY} = M_u / (\phi M_n) \leq 1.0 \quad (70)$$

where: M_u maximum design bending moment along the beam using LRFD load combination

ϕ resistance factor, $\phi = 0.90$ (AISC Specifications F.1 [2])

M_n nominal flexural strength according to the limit states of CFY, given by:

$$M_n = R_{pc} M_{yc} \quad (71)$$

$M_{yc} = F_y W_{el,y,c}$ = yield moment in the compression flange

$W_{el,y,c}$ = elastic section modulus referred to compression flange

R_{pc} = the web plastification factor (see § 7.2.3)

7.2.5. Limit state of compression flange local buckling for sections with non-compact flanges

The limit state of compression flange local buckling (FLB) applies only for sections with non-compact flanges. The criterion for this limit state is calculated as follows:

$$\Gamma_{FLB} = M_u / (\phi M_n) \leq 1.0 \quad (72)$$

where: M_u maximum design bending moment along the beam using LRFD load combination

ϕ resistance factor, $\phi = 0.90$ (AISC Specifications F.1 [2])

M_n nominal flexural strength according to the limit state of FLB, calculated by:
For sections with compact web (see AISC 360-16 – F3.2):

$$M_n = M_p - (M_p - 0.7F_y W_{el,y}) \left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right) \quad (73)$$

For sections with non-compact web (see AISC 360-16 – F4.3):

$$M_n = R_{pc} M_{yc} - (R_{pc} M_{yc} - F_L W_{el,y,c}) \left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right) \quad (74)$$

7.2.6. Web resistance to shear

Only unstiffened webs are considered in this document. The web resistance to shear must be calculated according to AISC Specifications G2.1 [2]. The verification criterion is:

$$\Gamma_{vb} = V_u / (\phi_v V_n) \leq 1.0 \quad (75)$$

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

$$V_n = 0.6F_y A_w C_{v1} \quad (76)$$

- **For webs of rolled sections with $h/t_w \leq 2.24 \sqrt{E/F_y}$**

$$\phi_v = 1.0$$

$$C_{v1} = 1.0 = \text{the web shear strength coefficient}$$

$$h = h_t - 2t_f - 2r_1$$

- **For all other sections**

$$\phi_v = 0.9$$

$$h = h_t - 2t_f - 2r_1 \quad \text{for rolled sections}$$

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

$$C_{v1} = 1.0 \quad \text{when } h/t_w \leq 1.1 \sqrt{k_v E/F_y}$$

$$C_{v1} = (1.1 \sqrt{k_v E/F_y}) / (h/t_w) \quad \text{when } h/t_w > 1.1 \sqrt{k_v E/F_y}$$

$k_v = 5.34$ the web plate shear buckling coefficient for webs without transverse stiffeners.

7.3. SLS verifications

No SLS checks at construction stage.

The software will provide the deflection under SLS combination.

8. VERIFICATION AT FINAL STAGE

8.1. General

The LRFD verifications for the composite beam include:

- Limit state of yielding;
- Web resistance to shear.

The SLS verifications include:

- Total deflection;
- Deflexion under variable load cases;
- Vibrations of the beam.

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

8.2. Effective width of the concrete slab

The effective width of the concrete slab is determined according to AISC Specifications I3.1a [2]:

$$\begin{aligned} b_e &= \min\{L/8 ; b_1/2\} + \min\{L/8 ; b_2/2\} && \text{for an interior beam} \\ b_e &= \min\{L/8 ; b_1\} + \min\{L/8 ; b_2/2\} && \text{for an edge beam} \end{aligned} \quad (77)$$

8.3. Design resistance of the connector

The nominal shear strength of a connector Q_n is obtained as follows:

- **For solid slabs:** Q_n is directly obtained from the Table 7 (including the appropriate resistance factor).
- **For slabs with decking transverse to the beam axis:**
 - **Connector longitudinal with the beam:** (see formulae given in Table 4 of [1])

$$\begin{aligned} Q_{n,t} &= k_{t,l} Q_n \\ \text{where: } k_{t,l} &= \frac{0,66}{\sqrt{n_r}} \frac{b_0}{h_p} \left(\frac{h_{sc}}{h_p} - 1 \right) \leq 1 \end{aligned} \quad (78)$$

- **Connector transverse with the beam:** (see formulae given in Table 4 of [1])

$$\begin{aligned} Q_{n,t} &= 0,89 k_{t,t} Q_n \\ \text{where: } k_{t,t} &= \frac{1,18}{\sqrt{n_r}} \frac{b_0}{h_p} \left(\frac{h_{sc}}{h_p} - 1 \right) \leq 1 \end{aligned} \quad (79)$$

- **For slabs with decking parallel to the beam axis:** (see formulae given in Table 5 of [1])

$$\begin{aligned} Q_{n,l} &= k_l Q_n \\ \text{where: } k_l &= 0,6 \frac{b_0}{h_p} \left(\frac{h_{sc}}{h_p} - 1 \right) \leq 1 \end{aligned} \quad (80)$$



AISC I8.2a gives other reduction factors for slabs with formed steel deck, which are relevant only for welded headed studs. ETA formulas are applied in the software for X-HVB connectors.

For formulas (78) to (80):

w_r : see § 4.4

h_{sc} : see Table 5

n_r : number of connectors in a row

For connectors X-HVB 80 to 140, an additional reduction factor is applied to the design resistance of the connector if the flange thickness is less than 0,315 in. [8 mm]:

$$\begin{aligned} Q_{n,red} &= \frac{t_f}{0,315} Q_n \geq 6520 \text{ lb in US customary units} \\ Q_{n,red} &= \frac{t_f}{8} Q_n \geq 29 \text{ kN in metric units} \end{aligned} \quad (81)$$

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 § 4.2.1). In the latter case, which is outside the scope of [1], a notification is given in the calculation report.

For custom steel (see § 4.2.2) with low yield strength, an additional reduction factor is applied to the resistance of the connector:

$$\alpha_{BM,red} = 0,95 \quad (82)$$

This reduction factor $\alpha_{BM,red}$ is applied for custom steels whose yield strength is lower than 36 ksi [250 MPa].

8.4. Participating depth of the concrete slab

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

$$\begin{aligned} \text{■ for plain slabs :} & \quad e_{part} = \eta h \\ \text{■ for slabs with steel sheeting :} & \quad e_{part} = \eta (h - h_p) \end{aligned} \quad (83)$$

Where: η is the degree of connection (see § 4.6.4); $\eta = 1$ for full connection.

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

8.5. Connection in plastic design

8.5.1. Principles

For plastic design, the connection between the slab and the steel profile is automatically designed by the software (see § 4.6.4 and § 4.6.5).

The number of connectors per each row of connection is denoted n_r . The final output of the module is 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 § 4.7). 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.

8.5.2. 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 LRFD criteria.

Table 9 gives the software strategies according to the value or state of these parameters.

LRFD 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 LRFD criteria are met

Table 9 : Software strategies for the assessment of the number of connectors

8.5.3. Number of connectors for full connection

For a critical cross-section (see § 6.2), 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 § 4.4

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

$$\eta = \frac{n_{\text{ribs}}(n_r Q_{n,t})}{\min \{N_{a,n}; N_{c,n}\}} \quad (85)$$

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

$$N_{a,n} = A F_y$$

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

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

The number of connectors, denoted n_f , is then determined according to the value of η :

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

b) Slab with decking parallel with the beam axis

$$n_f = \frac{\min \{N_{a,n}; N_{c,n}\}}{n_r Q_{n,l}} \quad (86)$$

c) Solid slab

$$n_f = \frac{\min \{N_{a,n}; N_{cs,n}\}}{n_r Q_n} \quad (87)$$

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

The number of connectors n_f must fulfil the requirements of connectors positioning as defined in § 4.7. If this control is negative, an error code is sent back to the interface.

8.5.4. Minimum number of connectors for partial connection

a) Minimum degree of connection

There are no direct requirements in the AISC Specifications [2] for determining a minimum degree of connection (denoted η_{\min}). However, according to the AISC Commentary I3. 2d. 1 [2], the following condition is checked:

$$\eta_{\min} \geq 0,5 \quad (88)$$

Additionally, if the length is greater than 30 ft [9,1 m], the rules of AISC Commentary I3. 2d. 1 are not strictly fulfilled and a warning message is displayed.

b) Slab with decking transverse to the beam axis

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

$$n_0 = \eta_{\min} n_f \quad (89)$$

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 is returned.
- If $n_0 \leq n_{\text{ribs}} < 2 n_0$: $n_{\min} = n_{\text{ribs}}$ (A connector is placed at each rib)
- If $2 n_0 \leq n_{\text{ribs}} < 3 n_0$: $n_{\min} = n_{\text{ribs}} / 2$ (A connector is placed at every two ribs)
- etc...

Error code (see Annex D):

If the partial connection is not possible, other calculations are not be performed and an error message is returned (error code = 27).

c) Other slabs

The minimum number of connectors is calculated by:

$$n_{\min} = \eta_{\min} n_f \quad (90)$$

8.5.5. Determination of the number of connectors for partial connection

Step 1: LRFD 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: LRFD 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:

$$n = n_{\min} \quad (91)$$

Otherwise, continue Step 3.

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

8.6. Connection in elastic design

8.6.1. Longitudinal shear flow

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

$$v_{Ed,j} = \frac{V_{Ed,j} \cdot \sum E_i S_i}{E I_{y,j}} \quad (92)$$

where:

$$\sum E_i S_i = \frac{E_{cm}}{n_{eq,j}} b_{eff} (h - h_p) \left(\frac{h + h_p}{2} + h_t - z_{ENA,j} \right) \quad (93)$$

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

$$v_{Ed} = v_{Ed,l} + v_{Ed,s} \quad (94)$$

8.6.2. Number of connectors

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

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

$$n_{f,min} = \frac{v_{u,max}}{Q_n} \quad (95)$$

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

$$\begin{aligned} \min\{4h ; 23,6 \text{ in}\} &\geq \frac{1}{n_{f,min}} \text{ and } \frac{1}{n_{f,min}} \geq 4 \text{ in} && \text{in US customary units} \\ \min\{4h ; 600 \text{ mm}\} &\geq \frac{1}{n_{f,min}} \text{ and } \frac{1}{n_{f,min}} \geq 100 \text{ mm} && \text{in metric units} \end{aligned} \quad (96)$$

where: Q_n is the design shear resistance of the connector (see §8.3);
 $v_{u,max}$ is the maximum longitudinal shear flow along the beam.

Error code (see Annex D):

If the second condition is not fulfilled, the connection between the concrete and the steel profile is not possible. An error message is returned (error code = 29).

In order to fulfil the first condition of formula (96), $n_{f,min}$ is calculated by:

$$\begin{aligned} n_{f,min} &= \max \left\{ \frac{v_{u,max}}{Q_n} ; \frac{1}{\min\{4h ; 23,6 \text{ in}\}} \right\} && \text{in US customary units} \\ n_{f,min} &= \max \left\{ \frac{v_{u,max}}{Q_n} ; \frac{1}{\min\{4h ; 600 \text{ mm}\}} \right\} && \text{in metric units} \end{aligned} \quad (97)$$

a) Slab with decking transverse to the beam axis

The number of ribs by length unit is calculated by:

$$n_{\text{ribs}} = 1 / b_s \quad (98)$$

where b_s is the spacing of two adjacent ribs.

The required geometrical condition is defined by:

$$\eta_{\text{req}} = n_{\text{ribs}} / n_{f,\text{min}} \geq 1 \quad (99)$$

Error code (see Annex D):

If the condition is not fulfilled, the connection between the concrete and the steel profile is not possible. An error message is returned (error code = 29).

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

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

b) Slab with decking parallel with the beam axis or Solid slab

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

8.7. LRFD verifications principles

According to AISC Specifications I3. 2a [2]:

- When $h/t_w \leq 3.76\sqrt{E/F_y}$ (compact web in bending), the flexural strength is determined from the plastic stress distribution (plastic design) for the limit state of yielding (plastic moment)
- When $h/t_w > 3.76\sqrt{E/F_y}$, the flexural strength is determined from the superposition of elastic stresses (elastic design) for the limit state of yielding (yield moment)

8.8. Plastic design

The criterion for the limit state of yielding is calculated as follows:

$$\Gamma_Y = M_u / (\phi M_n) \leq 1.0 \quad (100)$$

where: M_u maximum design bending moment along the beam using LRFD load combination

ϕ resistance factor, $\phi = 0.90$ (AISC Specifications I3.2a [2])

M_n nominal flexural strength according to the limit states of yielding, given by:

$$M_n = M_p = W_{pl,y} F_y \quad (101)$$

$W_{pl,y}$ plastic section modulus of the composite section (see Annex C)

8.9. Elastic design

8.9.1. Criterion

The criterion for the flexural strength of the steel profile is calculated as follows:

$$\Gamma_{\sigma,a} = \sigma_a / (\phi F_y) \leq 1.0 \quad (102)$$

where: σ_a elastic stress at the bottom of the steel profile;
 ϕ resistance factor, $\phi = 0,90$

The criterion for the flexural strength of the concrete slab is calculated as follows:

$$\Gamma_{\sigma,c} = \sigma_c / (\phi f'_c) \leq 1.0 \quad (103)$$

where: σ_c elastic stress at the top of the concrete slab;
 ϕ resistance factor, $\phi = 0,90$



The User can modify the value of the resistance factor for the elastic criteria (and only for these criteria). The default value is 0,9.

8.9.2. Calculation of elastic stresses

The elastic stresses are calculated for the following points (see Figure 25).

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

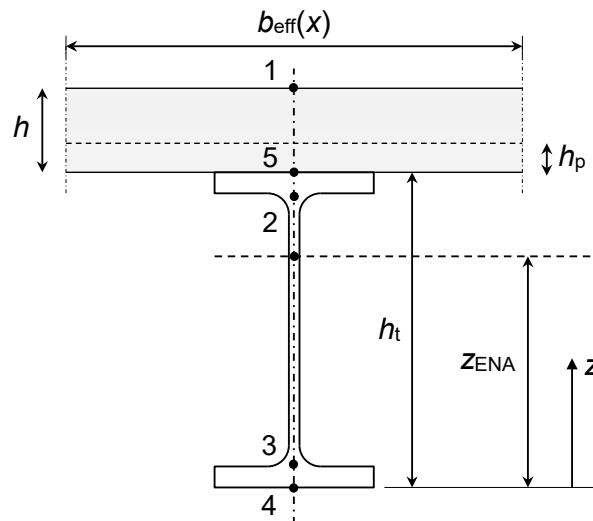


Figure 25: Points for the calculation of the elastic stresses

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

- under self-weight:
 - if the beam is shored during construction, the normal stresses are obtained considering a composite stage with the long-term modular ratio;
 - if the beam is unshored, the normal stresses are calculated considering the steel profile only.
- under other permanent loads, the normal stresses are obtained considering a composite stage with the long-term modular ratio;
- under live loads, the normal stresses are obtained considering a composite stage with the short-term modular ratio.

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

$$\begin{aligned} \text{short term actions:} \quad n_{eq} &= E_a / E_{cm} \\ \text{long term actions:} \quad n_{eq} &\approx 3 E_a / E_{cm} \end{aligned} \quad (104)$$

where: E : Young's modulus for steel (see § 4.2.2)

E_c : elasticity modulus for concrete (see § 4.3)

Following methods are considered for the calculation of the elastic stresses:

a) Calculation of elastic stresses under composite stages

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

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

Concrete:

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

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

$$\begin{aligned} \sigma_{c,j}(z) &= \frac{1}{n_{eq,j}} \times \frac{M_{c,j,u}}{I_{y,j}} \times (z - z_{ENA,j}) \quad \text{for} \quad z_{ENA,j} \leq z \\ \sigma_{c,j}(z) &= 0 \quad \text{for} \quad z_{ENA,j} > z \geq h_t + h_p \end{aligned} \quad (105)$$

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

$$\sigma_{c,j}(z) = \frac{1}{n_{eq,j}} \times \frac{M_{c,j,u}}{I_{y,j}} \times (z - z_{ENA,j}) \quad \text{for} \quad z \geq h_t + h_p \quad (106)$$

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

$$\sigma_c = \sigma_{c,l} + \sigma_{c,s} \quad (107)$$

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

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

Steel profile:

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

$$\sigma_{a,c,j}(z) = \frac{M_{c,j,u}}{I_{y,j}} \times (z_{ENA,j} - z) \quad \text{for} \quad h_t \geq z \quad (108)$$

b) Calculation of elastic normal stresses under self-weight

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

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

$$\sigma_{a,a}(z) = \frac{M_{a,u}}{I_{y,a}} \times \left(\frac{h_t}{2} - z \right) \quad \text{for} \quad h_t \geq z \quad (109)$$

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

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

The total stress is defined as the sum of the stress due to actions at the construction stage and the stress due to actions at the final stage:

$$\sigma_a(z) = \sigma_{a,a}(z) + \sigma_{a,c,l}(z) + \sigma_{a,c,s}(z) \quad (110)$$

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

8.10. Web resistance to shear

The available shear strength of composite beam is calculated using the properties of the steel section alone according to AISC Specifications I4.2 [2]. The same verifications as presented in § 7.2.6 are carried out.

8.11. SLS verifications

8.11.1. General

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

For fully shored 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 requires the calculation of the fundamental natural frequency of the composite beam. The later is evaluated by using the Rayleigh method in which the modal shape is considered equal to the deflection of the beam under dead loads considering the composite effect with the short term modular ratio.

AISC standards is not explicit about the treatment of the modular ratio for long-term actions. The same simplified rules are adopted as in Eurocodes. The modular ratio used for the calculations of composite stage deflections are given by Formula (104).

8.11.2. Deflection limits

According to [8], following limit criteria will be checked:

- deflection under live load (default limit $L/360$)
- deflection under dead and live load (default value $L/120$)

8.11.3. Vibrations

The floor vibration check is carried out according to AISC Design Guide 11 [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):

- Walking excitation;
- Rhythmic excitation.

a) Walking excitation

For the walking excitation, a peak acceleration criterion is checked:

$$\frac{a_p}{g} = \frac{P_0 \exp(-0.35f_n)}{\beta W} \leq \frac{a_0}{g} \quad (111)$$

Where: P_0 constant force representing the excitation
 f_n fundamental natural frequency of the composite beam
 β modal damping ratio defined by the user between 2 and 5 %; for Shopping malls, Dining and Dancing activities, $\beta = 2\%$ in calculations;
 W effective weight supported by the composite beam

Activity	Constant force P_0	Damping ratio β	Acceleration limit a_0/g (%)		
			1 Hz	4-8 Hz	40 Hz
Offices, Residences, Churches	65 lb (0.29 kN)	0.02-0.05	1.0	0.5	2.5
Shopping Malls	65 lb (0.29 kN)	0.02	3.0	1.5	7.0
<i>For frequency ranges 1-4 Hz and 8-40 Hz, acceleration limits is obtained by linear interpolation</i>					

Table 10 : Recommended values of parameters for walking excitation

The effective weight is estimated from:

$$W = wBL \quad (112)$$

Where: w supported weight per unit area
 L beam span
 B effective width

For the composite beam, the effective width is given by:

$$B = C(D_s/D)^{1/4}L \leq 2/3 \times \text{Floor Width} \quad (113)$$

Where: $C = 2.0$

D_s transformed slab moment of inertia per unit width = $12d_e^3/(12n)$ in⁴/ft
or = $d_e^3/(12n)$ mm³
 d_e effective width of the concrete slab, calculated by: $d_e = h_c + h_p/2$
 n dynamic modular ratio = $E / 1,35 E_c$
 E modulus of elasticity of steel
 E_c modulus of elasticity of concrete
 D transformed moment of inertia per unit width = I_t/S , mm³ (in⁴/ft)

I_t effective moment of inertia of the beam
 S beam spacing

The type of activity and the damping ratio is defined by the User in the interface.

For the calculation of w , the combination of load $\mathbf{D+p_L L}$ is considered, with $p_L = 0.2$ by default. The value of ψ can be modified by the User in the interface.

b) Rhythmic excitation

The peak acceleration a_i of the floor due to the i -th harmonic rhythmic force is calculated from the following expression:

$$\frac{a_i}{g} = \frac{k \alpha_i w_p / w_t}{\sqrt{\left[\left(\frac{f_n}{f} \right)^2 - 1 \right]^2 + \left(\frac{2 \beta f_n}{f} \right)^2}} \quad (114)$$

Where:

- f_n fundamental natural frequency of the beam
- f forcing frequency, $= i f_{step}$; $i = 1, 2, 3$
- f_{step} step frequency
- k constant (see Table 11)
- α_i dynamic coefficient
- w_p effective weight per unit area of participants distributed over the floor panel
- w_t effective total weight per unit area distributed over the floor panel (weight of participants plus weight of floor system)
- β modal damping ratio; $\beta = 6\%$ is used in calculations;

The values of all the above parameters are given in Table 11. The effective maximum peak acceleration that takes into account for all harmonics, is calculated and checked according to the following criterion:

$$\frac{a_{pr}}{g} = \left[\sum \left(\frac{a_i}{g} \right)^{1.5} \right]^{1/1.5} \leq \frac{a_0}{g} \quad (115)$$

Where: a_0/g ratio of peak acceleration limit to the acceleration due to gravity (see Table 11).

Activity	Forcing frequency f , Hz	Weight of participants, w_p		Dynamic coefficient α_i	Dynamic load, $\alpha_i w_p$		Constant k
		kPa	psf		kPa	psf	
Dancing: First harmonic	1.5 – 3 ⁽¹⁾	0.6	12.5	0.5	0.3	6.2	1.3
Lively concert or sport event: First harmonic	1.5 – 3 ⁽¹⁾	1.5	31.0	0.25	0.4	7.8	1.7
Second harmonic	3 – 5 ⁽¹⁾	1.5	31.0	0.05	0.075	1.6	
Jumping exercises: First harmonic	2 – 2.75 ⁽¹⁾	0.2	4.2	1.5	0.3	6.3	2.0
Second harmonic	4 – 5.5 ⁽¹⁾	0.2	4.2	0.6	0.12	2.5	
Third harmonic	6 – 8.25 ⁽¹⁾	0.2	4.2	0.1	0.020	0.42	

Note : (1) the value taken into account in HVB software

Table 11 : Estimated loadings during rhythmic events

Activity	Acceleration limit a_0/g (%)		
	1 Hz	4-8Hz	40 Hz
Dining and Dancing, Rhythmic activities	3.0	1.5	7.0
Lively concert or Sport event	10	5.0	25

Table 12 : Acceleration limit for Rhythmic excitation

8.11.4. 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:

$$f = \frac{1}{2\pi} \sqrt{g \frac{\sum P_i |w_i|}{\sum P_i w_i^2}} \quad (116)$$

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 .

$g = 9.81 \text{ m/s}^2$ in SI or $g = 32.2 \text{ ft/s}^2$ in US customary unit

8.12. Longitudinal shear resistance

8.12.1. Introduction

The horizontal shear strength on the connection of the slab is checked according to ACI 318-19 [7]. Following assumptions are considered:

- For beams with plain slabs, two layers of reinforcement (top and bottom layers) are assumed. The connectors go through the bottom layer but not through the top one;
- For slabs with formed steel deck, either longitudinal or perpendicular, only one layer is assumed;
- Transverse reinforcement is perpendicular to the shear plane that is parallel to the beam axis.

8.12.2. Minimum transverse reinforcement ratio

In order to resist horizontal shear, the minimum area of transverse reinforcement is calculated according to ACI 318-19 § 16.4.6:

$$\rho_{v,min} = \min \left\{ \frac{0.75\sqrt{f'_c}}{F_{ysr}}; \frac{50}{F_{ysr}} \right\} \text{ in US customary units} \quad (117)$$

$$\rho_{v,min} = \min \left\{ \frac{0.75\sqrt{f'_c}}{F_{ysr}}; \frac{0.345}{F_{ysr}} \right\} \text{ in metric units}$$

where: f'_c compressive strength of concrete (see § 4.3), in psi [MPa]

F_{ysr} yield strength of transverse reinforcement (see § 4.5), in psi [MPa]

8.12.3. Check of transverse reinforcement

The required area of shear-friction reinforcement is calculated using:

$$A_{vf} = \frac{V_u}{\phi F_{ysr} \mu} \quad (118)$$

Where: V_u ultimate longitudinal shear, calculated by Formula (121)

μ coefficient of friction: $\mu = 0,7\lambda$ (case *d* of ACI 318-19 Table 22.9.4.2)

λ factor depending on the concrete:

For normal weight concrete: $\lambda = 1,0$

For lightweight concrete: λ based on equilibrium density (taken from ACI 318-19 Table 19.2.4.1 (a) but shall not exceed 0.85)

Density w_c , in lb/ft ³	λ
≤ 100	0,75
$100 < w_c \leq 135$	$0,0075w_c \leq 0,85$
> 135	0,85

Table 13: Values of λ factor

The nominal shear strength across the assumed shear plan is calculated by:

$$V_n = \mu A_{vf} F_{ysr} \leq \min\{0,2 f'_c A_c ; 800 A_c\} \quad (119)$$

Where: A_c area of concrete section resisting shear transfer , given by:

$$A_c = \Delta x h_f \quad (120)$$

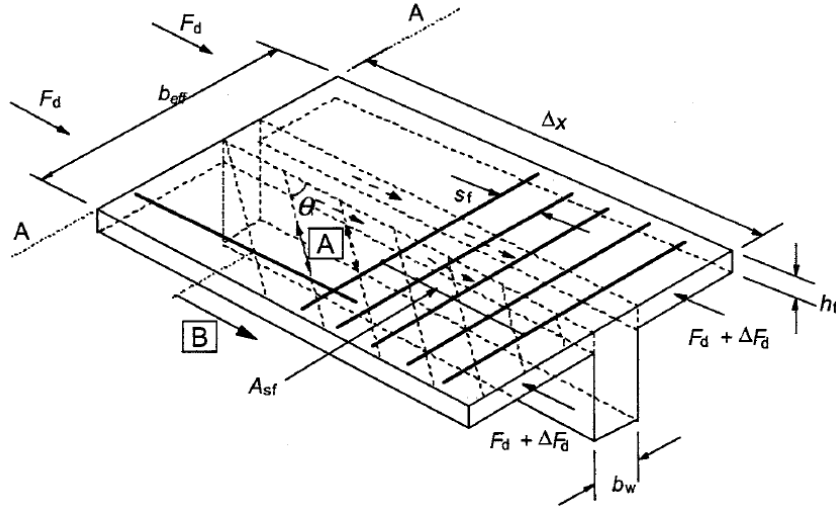


Figure 26 : Transverse reinforcement

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

The ultimate longitudinal shear V_u between the critical section and the closest support that should be transferred by the reinforcement through the shear plane is given by:

$$V_u = N_{n,slab} \frac{\max\{b_{eff,left} ; b_{eff,right}\}}{b_{eff}} \quad (121)$$

where: x_c is the location of the critical section;

$N_{n,slab}$ is the nominal plastic strength in compression of the slab at the critical section, given by:

$$N_{n,slab} = \min\{N_{c,n} ; N_{a,n}\}$$

$$N_{c,n} = 0.85 b_{eff} e_{part} f'_c \quad (122)$$

$$N_{a,n} = A f_y$$

b_{eff} is the effective width of concrete slab at the critical section;

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

$$b_{eff} = b_{eff,left} + b_{eff,right}$$

e_{part} see § 8.4

f'_c see § 4.3

b) Longitudinal shear stress

The longitudinal shear stress is given by:

$$v_u = \frac{V_u}{\Delta x h_f} \quad (123)$$

where: Δx is the shear length;

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

The shear length Δx is obtained as follows:

- for a critical section associated to a concentrated load, the shear length is the distance to the closest support,
- for a critical section associated to the maximum bending of the ULS combination, the shear length is the shortest distance between a support and the cross-section where the compression force in the slab is obtained. For partial connection, this distance is equal to the distance to the closest support. But in full connection, the shear length is lower than the distance to the relevant support (see Figure 27).

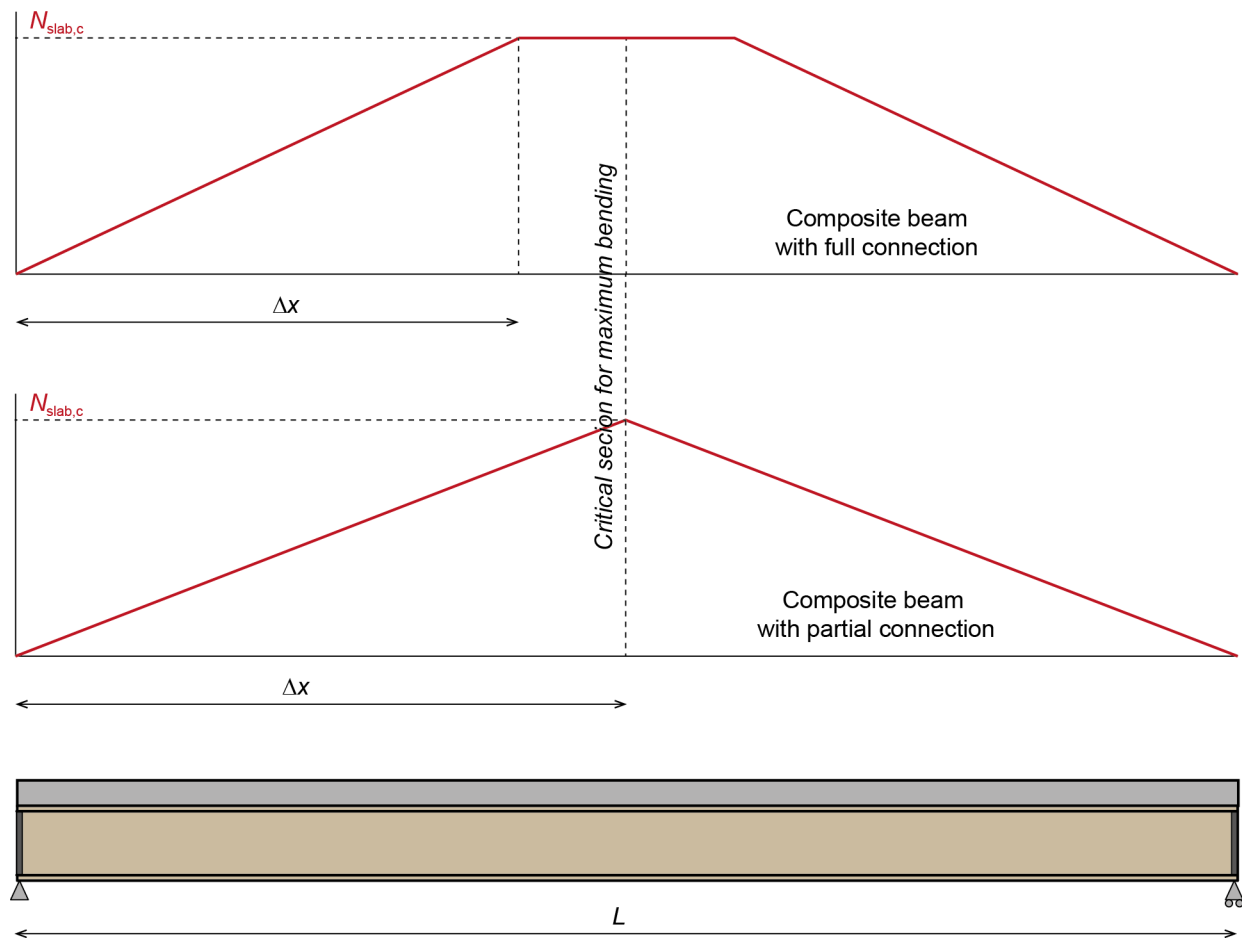


Figure 27 : Shear length

c) **Nominal longitudinal shear strength**

The nominal longitudinal shear strength is given by:

$$v_n = \frac{V_n}{\Delta x h_f} \quad (124)$$

The criterion for the transverse reinforcement is calculated as follows:

$$\Gamma_{\text{cstrut}} = v_u / (\phi v_n) \leq 1.0 \quad (125)$$

Annex A : PROPERTIES OF STEEL SECTIONS

The properties of a steel cross-section are calculated using following analytical formulae. In these formulae:

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

A Section area

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

I_y Second moment of area about the strong axis

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

with:

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

I_z Second moment of area about the weak axis

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

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

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

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

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

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

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

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

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

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

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

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

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

I_t Torsional constant

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

I_w Warping constant

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

Annex B : ELASTIC PROPERTIES OF A COMPOSITE BEAM

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

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

Location of the elastic neutral axis:

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

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

Second moment of area:

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

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

j is the index for the type of actions:

$j = l$ for long-term actions;

$j = s$ for short-term actions;

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

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

Location of the elastic neutral axis:

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

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

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

Second moment of area:

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

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

Location of the elastic neutral axis:

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

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

Second moment of area:

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

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

Location of the elastic neutral axis:

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

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

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

Second moment of area:

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

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

Location of the elastic neutral axis:

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

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

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

Second moment of area:

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

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

The bending resistance to bending of a cross section is assessed considering the participating depth of the slab.

Plastic Neutral Axis in the concrete slab

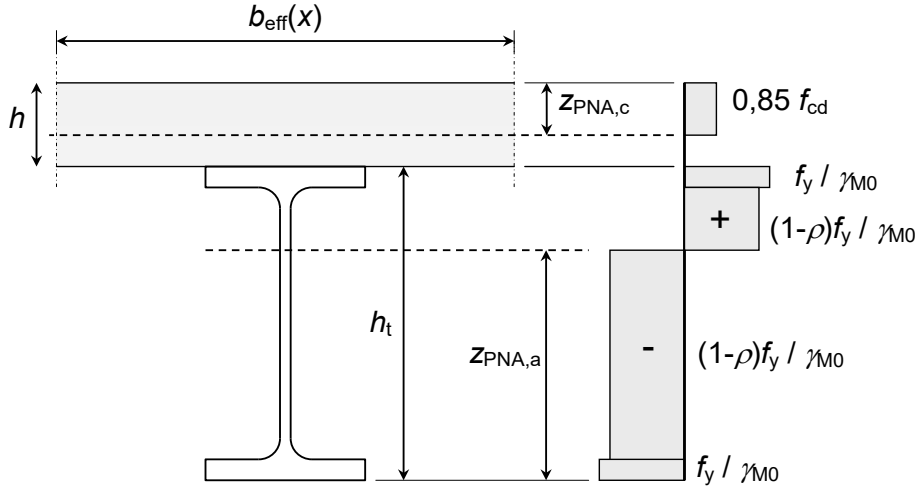


Figure 28: Plastic stresses at ultimate limit state

The position of the plastic neutral axis (PNA) in the concrete slab is calculated by (see Figure 28):

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

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

$h_p = 0$ for solid slabs

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

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

The bending resistance of the cross section is calculated by:

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

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

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

Plastic Neutral Axis in the steel section

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

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

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

The moment resistance of the section is calculated by:

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

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

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

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

The moment resistance of the section is calculated by:

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

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

Annex D : ERROR CODES MANAGEMENT

ErrorCode	Description	Reference
1	The National Annex Selected is not valid. Please check project settings to verify. [error code: 1]	
2	The value of Loaded Categories is not correct. [error code: 2]	
3	Not used for American standards	
4	Not used for American standards	
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]	§ 4.1
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]	§ 4.1
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]	§ 4.2.1
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]"	§ 4.2.2
10	Value of Steel Grade not correct. [error code: 10]	
11	The class of section is out of scope (Slender section). Please reduce the slenderness of the profile plate or reduce the steel grade in the "Materials" tab. [error code: 11]	§ 7.2.1

ErrorCode	Description	Reference
12	The value of Slab Thickness is not correct or Concrete Type is not correct or Concrete Class is not correct. Please increase the slab thickness (≥ 2 in. [50 mm]) in the "Slab" tab. [error code: 12]	§ 4.3
13	The value of concrete density should be between 90 and 155 lb/ft ³ (1500 and 2500 kg/m ³) . Please modify the concrete density in the "Materials" tab. [error code: 13]"	§ 4.3
14	Haunch height is only valid for Solid slab and $h_h \geq 0$. [error code: 14]	
15	Dimensions of the formed steel deck is not correct. Please check and correct the dimensions of the profiled sheeting in the "Slab" tab [error code: 15]	§ 4.4
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: $0 \leq G_{deck} \leq 10,24 \text{ lb/ft}^2$ [$0 \leq G_{deck} \leq 50 \text{ kg/m}^2$] $24,65 \leq F_{yp} \leq 87,02 \text{ ksi}$ [$170 \text{ MPa} \leq F_{yp} \leq 600 \text{ MPa}$] [error code: 16]	§ 4.4
17	The Value of Steel Grade for reinforcement steel bars is not correct. [error code: 17]	§ 4.5
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]	§ 4.2.1
23	The thickness of the formed steel deck 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: - $t \leq 0,0787$ in. [2,0 mm] for X-HVB 80, X-HVB 95 and X-HVB 110 - $t \leq 0,059$ [1,5 mm] for XHVB 125 and X-HVB 140 [error code: 23]	

ErrorCode	Description	Reference
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>§ 4.3</p> <p>§ 4.6.1</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>	§ 4.6.1
26	<p>The dimensions of the ribs are not satisfied according to Annexes B5-B8 of ETA 15/0876.</p> <p>Please change the dimensions of the profiled sheeting ribs in the "Slab" tab or the size of the connector in the "Shear connection" tab.</p> <p>[error code: 26]</p>	§ 4.7.4
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>	§ 8.5.4
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>	§ 4.7.4
29	<p>The connection between the concrete and the steel profile is not possible.</p> <p>Increase the connector and slab strength.</p> <p>[error code: 29]</p>	§ 8.6.2
30	<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 30]</p>	§ 4.6.1
31	<p>The concrete coverage of connectors (at least ½ in. or 13 mm) does not fulfill the requirement of AISC 360-16 I3. 2c.1.</p> <p>Increase the depth of the slab.</p> <p>[error code 31]</p>	§ 4.6.1

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