STEEL BUILDINGS IN EUROPE

Multi-Storey Steel Buildings
Part 10: Technical Software Specification for Composite Beams

SUMMARY

This guide provides guidance to developers of software for the design of composite beams used in multi-storey buildings, according to the Eurocodes. It covers simply supported beams connected to the concrete slab using shear studs and gives technical requirements. The ULS verifications are to be based on plastic design.

1 SCOPE

This document is aimed at software developers to enable them to develop a simple software tool for the design of composite beams in multi-storey buildings, according to EN 1994-1-1[1]. This document can also been regarded as a guide to understand the functioning of existing software in the same field of application.

This guide does not contain programming code; it only contains detailed technical requirements.

This document covers simply supported composite beams comprising a rolled profile connected by welded shear studs to a concrete slab. Several options are considered:

- Primary or secondary beams
- Plain slab or slab with profiled steel sheeting
- Fully propped or unpropped beams during construction.

These technical requirements include:

- The calculation of internal forces and moments
- The verifications of the beam for ULS
- The calculations for SLS
- The calculation of the composite beam is based on the plastic resistance using full or partial connection.

The design procedure is summarized in the flowcharts given in Appendix A.

2 BASIC DATA

2.1 General parameters of the beam

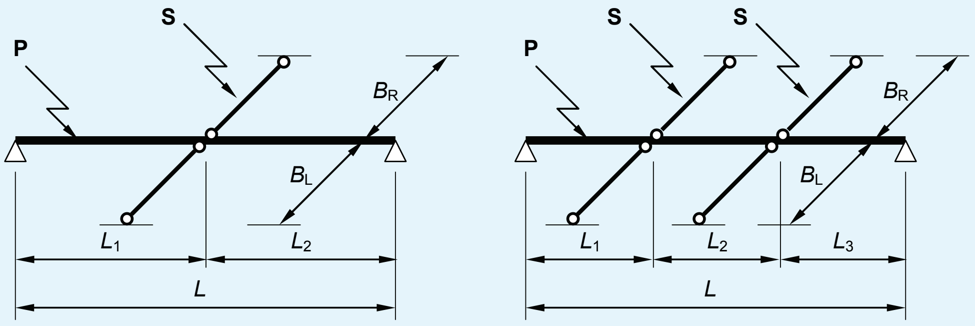
2.1.1 Dimensions

The general dimensions include:

L	is the span length
B_L, B_R	are the distances between beam axes
L_i	defines the positions of the secondary beams

The user can choose either a primary beam or a secondary beam. For a secondary beam, the loads are assumed to be uniformly distributed.

For a primary beam, the loads are transferred by one or two secondary beams to the primary beam under consideration.



P Primary beam
S Secondary beam

Figure 2.1 Primary beam and secondary beams

The following condition must be satisfied:

$L_i > L/5$

2.1.2 Propping and lateral restraint

Propping of the beam at the construction stage: fully propped or unpropped.

If the beam is fully propped, no calculation is performed at the construction stage.

If the beam is not propped at the construction stage, the user has to choose between a full lateral restraint against LTB at the construction stage and lateral restraints at the end supports only.

2.2 Steel section

The structural steel section is a hot rolled I-section defined by its geometry:

- h is the depth of the structural steel section
- b is the flange width
- t_f is the flange thickness
- t_w is the web thickness
- r is the root radius.

The following section properties can be obtained from an appropriate database:

- A is the section area
- $A_{v,z}$ is the shear area, according to EN 1993-1-1 § 6.2.6(3)
- I_y is the second moment of area about the strong axis
- I_z is the second moment of area about the weak axis
- I_t is the torsion constant
- I_w is the warping constant
- $W_{el,y}$ is the elastic modulus about the strong axis
- $W_{pl,y}$ is the plastic modulus about the strong axis.

The steel grade can be selected from the following list:

S235, S275, S355, S420, S460

2.3 Concrete slab

The concrete slab is defined by:

The type of slab: either plain slab or slab with profiled steel sheeting

- h_f is the slab thickness
- U is the concrete density

The concrete class can be chosen from:

- C20/25
- C25/30
- C30/37
- C35/45
- C40/50
- C45/55
- C50/60
- C55/67
- C60/75

The profiled steel sheeting, if used, is defined by its section geometry (Figure 2.2):

h_p is the overall depth of the profiled steel sheeting

t_p is the sheeting thickness

b_s is the distance between centres of adjacent ribs

b_r is the width of rib

b_b is the width of the bottom of the rib.

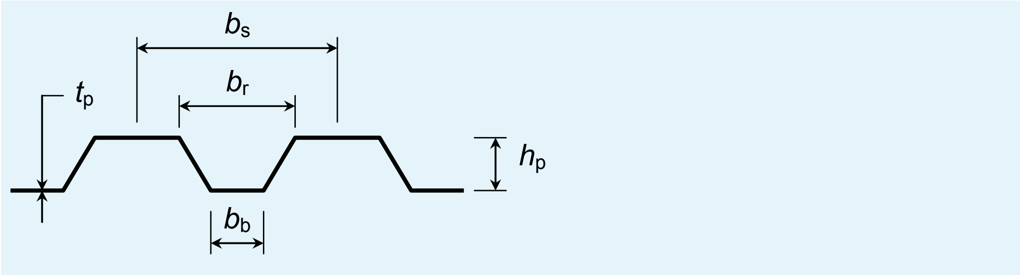


Figure 2.2 Cross-section of a profiled steel sheeting

One of the following options has to be selected:

- Ribs perpendicular to the beam
- Ribs parallel to the beam axis.

When the profiled steel sheeting is perpendicular to the beam axis, one of the following options has to be selected:

- Studs welded through the profiled steel sheeting
- Profiled steel sheeting with holes for studs
- Profiled steel sheeting interrupted on the beam (free positioning of the studs along the beam axis).

2.4 Shear connection

2.4.1 Description of a stud

The connectors are headed studs welded on the upper flange of the steel profile. For a given beam, all the studs are identical.

A stud is defined by:

h_{sc} is the overall nominal height.

d is the shank diameter that can be selected from the following list:
– 16 mm
– 19 mm
– 22 mm

$f_{u,sc}$ is the ultimate limit strength of the stud material.

2.4.2 Positioning of the connectors

The position of the connectors can be defined over 1, 2 or 3 segments of the beam. For more than one segment, the length of each segment has to be given. The sum of these lengths should be equal to the length of the beam.

For each segment, the following parameters have to be defined:

- The number of rows: 1 or 2
- The distance between two consecutive connectors along the beam.

When a profiled steel sheeting is perpendicular to the beam, the distance between studs is $n \times b_s$, where n can be equal to 1, 2 or 3.

2.5 Loads

The software allows the user to define elementary load cases that are used in the combinations of actions for ULS and SLS according to EN 1990[2].

Only gravity loads are considered (downwards).

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

- 1 permanent load case, denoted G
- 2 variable load cases, denoted Q_1 and Q_2

For each load case, it is possible to define a uniformly distributed surface load q_{surf}. For a beam defined as "secondary beam", a linear distributed load is derived:

$$q_{lin} = q_{surf} (B_L + B_R)/2$$

where:

B_L and B_R are the distances between beams (left and right).

For a beam defined as "primary beam", one or two point loads are derived from the distributed surface load.

The self weight of the rolled profile and the weight of the concrete slab are automatically calculated.

For each variable load case, the combination factors ψ_0, ψ_1 and ψ_2 have to be defined.

When the beam is unpropped at the construction stage, a construction load should be defined by the user. The default value is 0,75 kN/m^2.

2.6 Partial factors

2.6.1 Partial factors on actions

Within the field of application of the software, the partial factors on actions for the ULS combinations are:

- γ_G applied on the permanent actions
- γ_Q applied on the variable actions

2.6.2 Partial factors on resistances

Expressions for design resistance refer to the following partial factors:

- γ_{M0} is used for the resistance of the structural steel
- γ_{M1} is used for the resistance of the structural steel, for an ultimate limit state related to a buckling phenomenon
- γ_C is used for the compression resistance of the concrete
- γ_V is used for the resistance of headed studs
- γ_S is used for the resistance of the reinforcement steel bars

The values of the partial factors are given in the National Annexes. Recommended values are given in Table 2.1.

Table 2.1 Recommended values for the partial factors

Partial factors	γ_G	γ_Q	γ_{M0}	γ_{M1}	γ_C	γ_S	γ_V
Eurocode	EN 1990		EN 1993-1-1		EN 1992-1-1		EN 1994-1-1
Recommended values	1.35	1.50	1.0	1.0	1.5	1.15	1.25

2.7 Other design parameters

The values of the following design parameters have to be given:

- η is a coefficient for the shear resistance as defined in EN 1993-1-5 § 5.1. The value should be taken from the National Annex. The recommended value is 1.2.

The percentage of imposed loads for the evaluation of the natural frequency (SLS) has to be given by the user.

3 MATERIAL PROPERTIES

3.1 Structural steel

The steel properties are defined by EN 1993-1-1[3]:

E is the modulus of elasticity (E = 210000 N/mm^2)

G is the shear modulus (G = 80770 N/mm^2)

f_y is the yield strength that is derived from Table 3.1 of EN 1993-1-1, depending on the steel grade and the material thickness. For simplicity, the yield strength may be derived from the flange thickness.

f_{yw} is the yield strength of the web, derived from the web thickness.

H is the material parameter defined as:

$$H \sqrt{235 / f_y}$$

f_y is the yield strength in N/mm^2.

3.2 Reinforcement steel bars

The properties of reinforcing steel are defined by EN 1992-1-1:

$f_{yr,k}$ is the yield strength of the transverse reinforcement bars.

3.3 Concrete

The concrete properties are defined by EN 1992-1-1[4]. They are derived from the concrete class.

f_{ck} is the characteristic compressive strength at 28 days, as given in Table 3.1 of EN 1992-1-1.

f_{cd} is the design compressive strength (EN 1994-1-1 § 2.4.1.2(2)):

$$f_{cd} = f_{ck} / J_c$$

E_{cm} is the secant modulus of elasticity, as given in Table 3.1 of EN 1992-1-1.

4 CALCULATION OF INTERNAL FORCES AND MOMENTS

4.1 General

The section resistance of the composite beam has to be checked by taking into account the variation of the shear force and the bending moment, the variation of the bending resistance due to the effective width of the slab, the degree of connection and the influence of the shear force. Therefore the shear force and the bending moment should be calculated at several design points along the beam, for each elementary load case (i.e. G, Q_1, Q_2). Then the design internal forces and moments will be obtained for each combination of actions.

The design points are the supports and both sides of a point load. Additional design points are determined between the previous ones in order to get the critical section with sufficient accuracy. To this purpose, it is suggested that the distance between two consecutive design points is less than $L/20$.

4.2 Effects of a point load

Vertical reaction at the left support:

$R_{VL} = -F\,(L - x_F)\,/\,L$

Vertical reaction at the right support:

$R_{VR} = F - R_{VL}$

Shear force at the abscissa x from the left support:

If x dx_F: $V(x) = R_{VL}$

Else: $V(x) = R_{VL} + F$

Bending moment at the abscissa x from the left support:

If x dx_F: $M(x) = R_{VL}\,x$

Else: $M(x) = R_{VL}\,x + F\,(x - x_F)$

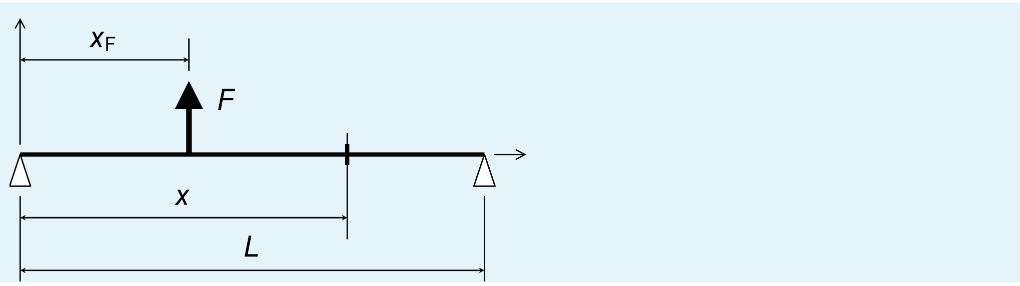


Figure 4.1 Point load applied to the beam

4.3 Effects of a uniformly distributed surface load

Vertical reaction at supports:

$R_{VL} = R_{VR} = -Q\ (B_L + B_R)\ L\ /\ 4$

Shear force at the abscissa x from the left support:

$V(x) = R_{VL} + Q\ (B_L + B_R)\ x\ /\ 2$

Bending moment at the abscissa x from the left support:

$M(x) = R_{VL}\ x + Q\ (B_L + B_R)\ x^2\ /4$

4.4 Combinations of actions

4.4.1 Ultimate Limit States (ULS)

The combinations of actions for the ULS verifications are the fundamental combinations as defined in EN 1990 § 6.4.3.2:

$\gamma_G\, G + \gamma_Q\, Q_1 + \gamma_Q\, \psi_{0,2}\, Q_2$
$\gamma_G\, G + \gamma_Q\, Q_2 + \gamma_Q\, \psi_{0,1}\, Q_1$

4.4.2 Serviceability Limit States (SLS)

The combinations of actions for the SLS verifications (deflection, vibration) can be either the characteristic or the frequent combinations, depending on the National Annex:

Characteristic combinations (EN 1990 § 6.5.3 a):

$G + Q_1 + \psi_{0,2}\, Q_2$
$G + Q_2 + \psi_{0,1}\, Q_1$

Frequent combinations (EN 1990 § 6.5.3 b):

$G + \psi_{1,1}\, Q_1 + \psi_{2,2}\, Q_2$
$G + \psi_{1,2}\, Q_2 + \psi_{2,1}\, Q_1$

5 CONSTRUCTION STAGE

5.1 General

When the beam is unpropped at the construction stage, ULS verifications have to be carried out. The following actions are considered at this stage:

- Self-weight of the steel profile (G)
- Weight of the concrete (Q_{cf})
- A construction load considered as variable action (Q_{ca})

The internal forces and moments are calculated according to Section 4 of this guide, for the following ULS combination of actions:

$$\gamma_G G + \gamma_Q (Q_{cf} + Q_{ca})$$

The ULS verifications include:

- Bending resistance
- Shear resistance
- Shear buckling resistance
- Bending moment and shear force interaction
- Lateral torsional buckling

Regarding Lateral Torsional Buckling (LTB), it is up to the user to select the design assumption, either the beam is fully laterally restrained to prevent LTB, or the beam is laterally restrained at the supports only. The LTB verification is performed accordingly.

5.2 ULS verifications

5.2.1 General

Different criteria are calculated at each design point along the beam. A criterion is the ratio of a design force to the relevant design resistance. Therefore the verification is satisfactory when the criterion, denoted Γ, does not exceed the unity:

$$\Gamma \le 1{,}0$$

5.2.2 Classification of the cross-section

The bending resistance of the cross-section depends on the class of the cross-section.

If	$0{,}5\,(b - t_w - 2\,r)/t_f$	$\le 9\,\varepsilon$	then the flange is Class 1,
If	$0{,}5\,(b - t_w - 2\,r)/t_f$	$\le 10\,\varepsilon$	then the flange is Class 2,
If	$0{,}5\,(b - t_w - 2\,r)/t_f$	$\le 14\,\varepsilon$	then the flange is Class 3,

Otherwise the flange is Class 4.

If $(h - 2\,(t_f + r))/t_w$ $\leq 72\,\varepsilon$ then the web is Class 1,
If $(h - 2\,(t_f + r))/t_w$ $\leq 83\,\varepsilon$ then the web is Class 2,
If $(h - 2\,(t_f + r))/t_w$ $\leq 124\,\varepsilon$ then the web is Class 3,

Otherwise the web is Class 4.

The class of the cross-section is the highest class of the compressed flange and the web.

5.2.3 Vertical shear resistance

The criterion for the vertical shear resistance is calculated according to 6.3.3 of this guide. For shear buckling, refer to Section 6.3.4 of this guide.

5.2.4 Bending resistance

The criterion for the bending resistance is calculated from:

$$\Gamma_M = M_{Ed} / M_{c,Rd}$$

where:

M_{Ed} is the maximum design moment along the beam

$M_{c,Rd}$ is the design bending resistance depending on the class of the cross-section:

$M_{c,Rd} = W_{pl,y}\, f_y / \gamma_{M0}$ for Class 1 or 2

$M_{c,Rd} = W_{el,y}\, f_y / \gamma_{M0}$ for Class 3

$M_{c,Rd} = W_{eff,y}\, f_y / \gamma_{M0}$ for Class 4

5.2.5 M-V interaction

When the web slenderness h_w/t_w exceeds $72\,\varepsilon/\eta$, the shear buckling criterion Γ_{bw} is calculated according to Section 6.3.4 as above mentioned in Section 5.2.3. When this criterion is higher than 0,5 and when the bending moment exceeds the bending resistance of the flanges, M-V interaction must be considered. The interaction criterion is (EN 1993-1-5 § 7.1(1)):

$$\Gamma_{MV} = \bar{\eta}_1 + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right)\left(2\bar{\eta}_3 - 1\right)^2 \qquad \text{if } M_{Ed} > M_{f,Rd}$$

where:

$\bar{\eta}_1 = M_{Ed} / M_{pl,Rd}$

$\bar{\eta}_3 = \Gamma_{bw}$

$M_{pl,Rd} = W_{pl,y}\, f_y / \gamma_{M0}$

$M_{f,Rd} = b\, t_f\, (h - t_f)\, f_y / \gamma_{M0}$

When shear buckling does not need to be considered and the shear criterion η_V is higher than 0,5, M-V interaction must be checked using the following criterion (EN 1993-1-1 § 6.2.8):

$$\eta_{MV} = \frac{M_{Ed}}{M_{V,Rd}}$$

where:

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

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

$$A_w = (h - 2\, t_f)\, t_w$$

5.2.6 Resistance to Lateral Torsional Buckling (LTB)

Design criterion

If the beam is assumed to be fully laterally restrained, no LTB verification is performed. If the beam is restrained at the supports only, the LTB criterion is calculated as follows:

$$\eta_{LT} = M_{Ed} / M_{b,Rd}$$

where:

M_{Ed} is the maximum design moment along the beam

$M_{b,Rd}$ is the design LTB resistance that is determined according to the appropriate LTB curve and the LTB slenderness as described below.

Elastic critical moment

The elastic critical moment is determined from the following equation:

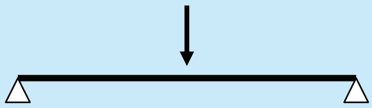
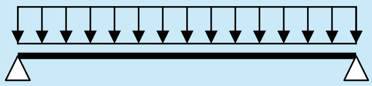
$$M_{cr} = C_1 \frac{\pi^2 E I_z}{L^2} \left\{ \sqrt{\frac{I_w}{I_z} + \frac{G I_t L^2}{\pi^2 E I_z} + \left(C_2 z_g \right)^2} - C_2 z_g \right\}$$

where:

z_g = $+h/2$ (the transverse loading is assumed to be applied above the upper flange)

The C_1 and C_2 factors can be taken from Table 5.1.

Table 5.1 C_1 **and** C_2 **factors**

Loading	C_1	C_2
	1,35	0,59
	$1 + 2{,}92\,\gamma^3$	$\gamma(2{,}44 - 3{,}24\,\gamma)C_1$
	1,13	0,45

LTB slenderness

The LTB slenderness is calculated as:

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}}$$

where:

$W_y = W_{pl,y}$ for a class 1 or 2 cross-section

$W_y = W_{el,y}$ for a class 3 cross-section

$W_y = W_{eff,y}$ for a class 4 cross-section

Reduction factor

The reduction factor is calculated according to EN 1993-1-1 § 6.3.2.3 for rolled profiles:

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \beta\bar{\lambda}_{LT}^2}}$$

but: $\chi_{LT} \le 1$

and: $\chi_{LT} \le \dfrac{1}{\bar{\lambda}_{LT}^2}$

where:

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

The parameters $\bar{\lambda}_{LT,0}$ and β may be given in the National Annex to EN 1993-1-1. The recommended values are:

$\bar{\lambda}_{LT,0} = 0{,}4$

$\beta = 0{,}75$

α_{LT} is the imperfection factor depending on the LTB curve to be considered for hot rolled profiles, according to EN 1993-1-1 Table 6.5:

If $h/b_f \le 2$ Curve b $\alpha_{LT} = 0{,}34$

If $h/b_f > 2$ Curve c $\alpha_{LT} = 0{,}49$

LTB Resistance

The LTB Resistance is given by:

$$M_{b,Rd} = \chi_{LT,mod}\, W_y\, f_y / \gamma_{M1}$$

where:

$\chi_{LT,mod}$ is the modified reduction factor calculated according to EN 1993-1-1 § 6.3.2.3 (2). As simplification, it can be taken equal to χ_{LT}.

$$\chi_{LT,mod} = \chi_{LT} / f \qquad \text{but: } \chi_{LT,mod} \le 1$$

$$\text{and: } \chi_{LT,mod} \le \frac{1}{\bar{\lambda}_{LT}^2}$$

$$f = 1 - 0,5\left(1 - k_c\right)\left[1 - 2\left(\bar{\lambda}_{LT} - 0,8\right)^2\right] \qquad \text{but: } f \le 1$$

k_c is a correction factor that can be determined by the following expression:

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

5.3 SLS Calculations

At the construction stage, the deflection can be calculated using the formula given in Section 6.5.3 of this guide, where the second moment of area is the one of the steel section.

6 FINAL STAGE

6.1 Effective width of the slab

The effective width of the concrete slab is determined according to EN 1994-1-1 § 5.4.1.2. The following expressions are limited to the field of application of these specifications.

$b_e = \mathrm{Min}(L/8; B/2)$ where: $B = (B_L + B_R)/2$

$\beta = (0{,}55 + 0{,}025\ L/b_e)$

If $\beta > 1{,}0$ then $\beta = 1{,}0$

For a given design section, located at an abscissa x, the effective width $b_{eff}(x)$ is obtained from:

If $x \leq 0{,}25\ L$: $b_{eff}(x) = 2\ b_e\ [\ \beta + 4(1 - \beta)\ x/L]$

If $x \geq 0{,}75\ L$: $b_{eff}(x) = 2\ b_e\ [\ \beta + 4(1 - \beta)\ (L - x)/L]$

Otherwise: $b_{eff}(x) = 2\ b_e$

Note that, by simplification, as stated in EN 1994-1-1 § 5.4.1.2(9), the distance b_0 between 2 rows of studs is taken equal to 0 for the determination of the effective width of the slab.

6.2 Shear connection

6.2.1 Resistance of a headed stud

Design resistance

According to EN 1994-1-1 § 6.6.3.1, the design resistance of a headed stud is the minimum value of the two following expressions:

$$P_{Rd} = \frac{0{,}8\, f_{u,sc}\, \pi d^2 / 4}{\gamma_V}$$

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

where:

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

$\alpha = 1{,}0$ for $h_{sc}/d > 4$

$f_{u,sc}$ is the ultimate tensile strength of the stud material. The maximum value is 500 N/mm^2.

Steel sheeting with ribs parallel to the beam axis

When the ribs are parallel to the beam axis, a factor k_ℓ applies to the design resistance of a headed stud. It depends on the distance b_0 determined as follows:

If $b_r > b_b$: $b_0 = (b_r + b_b)/2$
Otherwise: $b_0 = b_r$

$$k_\ell = 0.6\frac{b_0}{h_p}\left(\frac{h_{sc}}{h_p} - 1\right)$$

The maximum value of h_{sc} is $h_p + 75$ mm.

The maximum value of k_ℓ is 1,0.

Steel sheeting with ribs perpendicular to the beam axis

When the ribs are perpendicular to the beam axis, a factor k_t applies to the design resistance of a headed stud:

$$k_t = \frac{0.7}{\sqrt{n_r}}\frac{b_0}{h_p}\left(\frac{h_{sc}}{h_p} - 1\right)$$

where:

b_0 is defined in Section 6.1.3

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

The reduction factor k_t should not exceed the maximum values given in Table 6.1 (EN 1994-1-1 Table 6.2).

The values of the reduction factor k_t are valid when:

$h_p \leq 85$ mm

$b_0 \geq h_p$

Table 6.1 Maximum values of the reduction factor k_t

		Studs welded through profiled steel sheeting			Profiled steel sheeting with holes		
	Diameter	16	19	22	16	19	22
$n_r = 1$	$t_p \leq 1$ mm	0,85		Not accepted in EN 1994-1-1	Not covered by EN 1994	0,75	
	$t_p > 1$ mm	1,00				0,75	
$n_r = 2$	$t_p \leq 1$ mm	0,70				0,60	
	$t_p > 1$ mm	0,80				0,60	

6.2.2 Degree of connection

At a given design point along the beam, the degree of connection K can be calculated as follows:

$$K = \frac{F_{sc}}{\text{Min}\left(N_{pl,Rd}\,;N_{c,Rd}\right)}$$

where:

F_{sc} is the design resistance of the shear connection at the design point

$N_{c,Rd}$ is the design compression resistance of the concrete slab at the design point

$N_{pl,Rd}$ is the design axial resistance of the structural steel.

Resistance of the connection

At a given design point of the beam, the resistance of the connection, F_{sc}, is:

$$F_{sc} = \text{Min}(n_{sc,left}\,;n_{sc,right})\,k\,P_{Rd}$$

where:

$n_{sc,left}$ is the number of connectors between the left support and the design point

$n_{sc,right}$ is the number of connectors between the right support and the design point

k = 1 for a plain slab

= k_{ℓ} for a slab made of a profiled steel sheeting with ribs parallel to the beam axis

= k_t for a slab made of a profiled steel sheeting with ribs perpendicular to the beam axis.

Resistance of the concrete slab

At a design point along the beam, defined by the abscissa x, the design resistance of the concrete slab is given by:

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

For a plain slab, h_p is taken equal to 0.

Resistance of the structural steel

The design axial resistance of the steel section is given by:

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

6.2.3 Minimum degree of connection

The minimum degree of connection, K_{min}, is calculated according to EN 1994-1-1 § 6.6.1.2, as follows:

If L d25 m: $K_{min} = 1 - (355/f_y)\ (0,75 - 0,03\ L)$

But K_{min} t 0,4

Otherwise: $K_{min} = 1$

where:

L is the span length in meters

f_y is the yield strength in N/mm^2

6.2.4 Verification of the degree of connection

At the point of maximum bending moment, if the degree of connection is lower than the minimum degree of connection ($K < K_{min}$), the plastic theory does not apply (EN 1994-1-1 § 6.1.1(7)). In this case, the following message should display: "Insufficient degree of connection: you should increase the resistance of the shear connection".

6.3 Cross-section resistance

6.3.1 General

Different criteria are calculated at each design point along the beam. A criterion is the ratio of a design force to the relevant design resistance. Therefore the verification is satisfactory when the criterion, denoted *, does not exceed the unity:

* d1,0 Verification OK

6.3.2 Classification of the cross-section

It is reminded that the field of application of these specifications is limited to the plastic design of the cross-section. So it shall be checked that each cross-section is class 2 (or class 1).

The class of the cross-section is the maximum of the class of the compressed flange (upper flange) and the class of the web.

The limit of slenderness depends on the material parameter H as defined in Section 3.1 of this guide.

The first step is to determine the position $y_{pl,a}$ of the Plastic Neutral Axis in the structural steel section, measured from the bottom of the section. For the calculation of $y_{pl,a}$, refer to Section 6.3.7 where no influence of the shear force is taken into account (i.e. $U = 0$ in the expressions of $y_{pl,a}$).

Class of the compressed upper flange

If $y_{pl,a} > h - t_f$ The upper steel flange is not fully in compression. So the flange has not to be classified.

For the classification, the flange slenderness is: $Q_f = 0{,}5\,(b - t_w - 2\,r)/t_f$

If $Q_f \leq 10\,\varepsilon$ The flange is class 2 (or 1) (EN 1993-1-1 Table 5.2).

When $Q_f > 10\,\varepsilon$, the following requirements shall be fulfilled to conclude that the flange is class 2 (EN 1994-1-1 § 5.5.2(1) and § 6.6.5.5):

- For plain slabs or slabs with profiled steel sheeting parallel to the beam axis, the longitudinal spacing between the connectors is lower than $22\,\varepsilon t_f$.
- For slabs with profiled steel sheeting perpendicular to the beam axis, the longitudinal spacing between the connectors is lower than $15\,\varepsilon t_f$.
- The longitudinal spacing between the connectors is lower than 6 times the slab depth ($6\,h_f$).
- The longitudinal spacing between the connectors is lower than 800 mm.
- The clear distance from the edge of the flange to the nearest line of connectors is not greater than $9\,\varepsilon t_f$.

Class of the web

If $y_{pl,a} > h - t_f - r$ The web is fully in tension. So the web has not to classified.

For the classification, the flange slenderness is: $Q_w = (h - 2t_f - 2r)/t_w$

The compression part of the web is estimated by the α ratio:

$$\alpha = \frac{h - t_f - r - y_{pl,a}}{h - 2t_f - 2r}$$

Here the α ratio is supposed to be lower than 0,5.

If $Q_w \leq \dfrac{456\,\varepsilon}{13\,\alpha - 1}$ The web is class 2 (or 1).

6.3.3 Vertical shear resistance

The vertical shear resistance of a cross-section is calculated according to EN 1993-1-1 § 6.2.6. The contribution of the concrete slab is neglected.

$$V_{pl,Rd} = \frac{A_{v,z}\, f_y}{\sqrt{3}\,\gamma_{M0}}$$

The criterion is calculated by:

$$\chi_V = \frac{|V_{Ed}|}{V_{pl,Rd}}$$

6.3.4 Shear buckling resistance

When the web slenderness h_w/t_w exceeds $72\,\varepsilon/\eta$, the shear buckling resistance $V_{bw,Rd}$ has to be calculated according to EN 1993-1-5 § 5.2, with the following assumptions:

- Only the contribution of the web is considered
- The end posts are non rigid

Therefore the design resistance for shear buckling is obtained from:

$$V_{bw,Rd} = \frac{\chi_w h_w t_w f_{yw}}{\sqrt{3}\,\gamma_{M1}}$$

where:

h_w is the height of the web: $h_w = h - 2\,t_f$

χ_w is the reduction factor for shear buckling that depends on the web slenderness $\bar{\lambda}_w$

The web slenderness is:

$$\bar{\lambda}_w = \frac{h_w}{37{,}4\,t_w\,\varepsilon_w\sqrt{k_\tau}}$$

where:

$\varepsilon_w = \sqrt{235/f_{yw}}$

$k_\tau = 5{,}34$

The reduction factor χ_w is calculated as follows:

If $\bar{\lambda}_w < 0{,}83/\eta$ $\quad \chi_w = \eta$

Otherwise $\quad \chi_w = 0{,}83/\bar{\lambda}_w$

Then the criterion is calculated by:

$$\Gamma_{Vb} = \frac{|V_{Ed}|}{V_{bw,Rd}}$$

6.3.5 Bending resistance

The bending resistance M_{Rd} of a cross-section is calculated according to 6.3.7, by taking the parameter ρ equal to 0 (i.e. no influence of the shear force). The criterion is obtained from:

$$\Gamma_M = \frac{|M_{Ed}|}{M_{Rd}}$$

6.3.6 M-V interaction

When the web slenderness h_w/t_w exceeds $72\,\varepsilon/\eta$, the shear buckling criterion η_{bw} is calculated according to 6.3.4. When this criterion is higher than 0,5, M-V interaction shall be considered. The interaction criterion is:

$$\eta_{MV} = \frac{|M_{Ed}|}{M_{V,Rd}}$$

The bending resistance $M_{V,Rd}$ is calculated according to 6.3.7 with the parameter ρ obtained from:

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

When shear buckling has not to be considered, $V_{bw,Rd}$ is replaced by $V_{pl,Rd}$. If the shear criterion η_V is higher than 0,5, interaction must be considered and $M_{V,Rd}$ is calculated according to 6.3.7 with the parameter ρ obtained from:

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

6.3.7 General expression of the bending resistance

The following procedure allows the user to calculate the design bending resistance, including the reduction due to the shear force. When the effect of the shear force can be neglected, the parameter ρ is taken equal to 0.
The plastic stress distribution is shown in Figure 6.1.

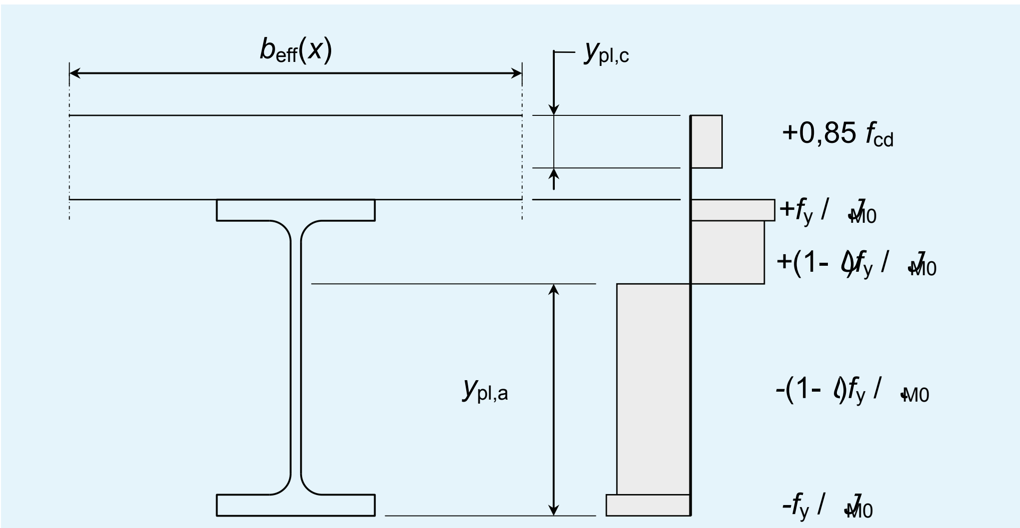


Figure 6.1 Plastic stress distribution with partial connection

Contribution of the concrete slab

At each design point located at abscissa x, the bending resistance depends on the shear resistance of the connection, F_{sc}, determined according to § 6.2.2. The position of the plastic neutral axis in the slab is obtained from the following expression (see Figure 6.1):

$$y_{pl,c} = \frac{\mathrm{Min}(N_{pl,V,Rd}\,;F_{sc})}{b_{eff}(x) \times 0{,}85\, f_{cd}}$$

But: $y_{pl,c} \leq h_f - h_p$

For a plain slab: $h_p = 0$

$N_{pl,V,Rd}$ is the plastic resistance to the axial force of the steel section, reduced by the effect of the shear force:

$N_{pl,V,Rd} = [A - \rho(h - 2\,t_f)\,t_w + (4 - \pi)r^2)]\,f_y / \gamma_{M0}$

Therefore, the resulting compression force in the concrete slab is:

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

It applies at $y_{pl,c}/2$ from the top of the slab.

Position of the plastic neutral axis in the steel section

The plastic neutral axis in the steel section has to be determined. It can be located in one of the three following parts of the cross-section:

1. In the web if: $N_c \leq N_{pl,1}$
   with: $N_{pl,1} = (h - 2\,t_f - 2\,c)\,t_w\,(1 - \rho)f_y / \gamma_{M0}$

$$y_{pl,a} = \frac{1}{2}\left(h - \frac{N_c}{t_w\,(1-\rho)f_y/\gamma_{M0}}\right)$$

2. In the fillets if: $N_{pl,1} < N_c \leq N_{pl,2}$
   with: $N_{pl,2} = (A - 2\,b\,t_f)\,(1 - \rho)f_y / \gamma_{M0}$

$$y_{pl,a} = h - t_f - c - \frac{1}{2}\left\{\sqrt{t_w^2 - 4\left[t_w\left(\frac{h}{2} - t_f - c\right) - \frac{N_c}{2\,(1-\rho)f_y/\gamma_{M0}}\right]} - t_w\right\}$$

3. In the upper flange if: $N_{pl,2} < N_c \leq N_{pl,V,Rd}$

$$y_{pl,a} = h - \frac{N_{pl,V,Rd} - N_c}{2\,b\,f_y/\gamma_{M0}}$$

where:

$$c = r \times \sqrt{2 - \pi/2}$$

Plastic moment resistance

Depending on the position of the plastic neutral axis, the expression of the design plastic moment resistance is given hereafter:

1. In the web:

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

2. In the fillets:

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

3. In the upper flange:

$$M_{Rd} = (h - y_{pl,a}) b y_{pl,a} \frac{f_y}{\gamma_{M0}} + M_{slab}$$

where:

$$M_{slab} = N_c \left(h_f + \frac{h - y_{pl,c}}{2} \right)$$

6.4 Longitudinal shear resistance

6.4.1 Minimum transverse reinforcement ratio

According to EN 1994-1-1 § 6.6.6.3, the minimum transverse reinforcement ratio can be obtained from EN 1992-1-1 § 9.2.2(5):

$$\rho_{w,min} = \frac{0{,}08\sqrt{f_{ck}}}{f_{yr,k}}$$

where:

f_{ck} is the characteristic value of the compression resistance in N/mm^2

$f_{yr,k}$ is the yield strength of the reinforcement bars in N/mm^2

6.4.2 Calculation of the transverse reinforcement ratio

The transverse reinforcement ratio is obtained from (EN 1992-1-1 § 6.2.4(4)):

$$\frac{A_{sf} f_{yd}}{s_f} \geq \frac{v_{Ed} h_f}{\cot \theta_f}$$

where:

A_{sf}/s_f is the transverse reinforcement ratio (in cm^2/m for example)

f_{yd} is the design value of the yield strength of the reinforcement bars:

$f_{yd} = f_{yr,k} / \gamma_s$

θ_f is the angle between concrete compression struts and tension chords. This can be defined by the National Annex. Here it is proposed to take: $\theta_f = 45°$

v_{Ed} is the longitudinal shear action defined by:

$$v_{Ed} = \frac{\Delta F_d}{h_f \Delta x}$$

ΔF_d is the variation of the compression axial force in the slab along a distance Δx between two given sections.

The calculation is performed along a segment close to each end of the beam. Then:

$\Delta F_d = (N_c - 0)/2 = N_c/2$

N_c is calculated according to 6.3.7.

For uniform distributed loads, the calculation is performed between the section located at mid span and the support ($\Delta x = L/2$).

For a beam with point loads, the calculation has to be performed along a segment between the section under the point load and the closest support.

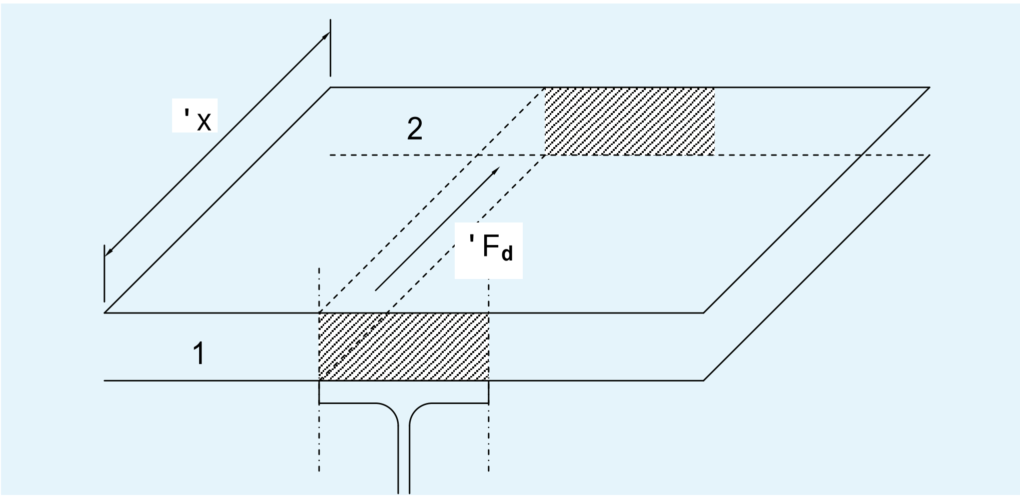


Figure 6.2 Determination of ΔF_d

6.4.3 Concrete strength in the compression struts

The criterion for the concrete strength of the compression struts is calculated by the following expression:

$$\Gamma_{Vh} = \frac{v_{Ed}}{\nu f_{cd} \sin \theta_f \cos \theta_f}$$

This criterion is calculated for each segment considered in 6.4.2 and then the maximum value is derived.

6.5 Serviceability limit states

6.5.1 General

For the serviceability limit states, there is no stress limitation in the buildings. The limit states are:

- The deflection of the beam
- The natural frequency of the beam that is derived from the deflection.

6.5.2 Inertia of the composite beam

The deflection is estimated from the combination of actions under consideration and from the stiffness of the composite beam. The stiffness depends on the second moment of area of the composite section that is determined using a modular ratio n between the structural steel and the concrete.

As stated by EN 1994-1-1 § 5.4.2.2(11), the deflection in buildings under both permanent actions and variable actions is calculated using:

$n = 2\ E_a / E_{cm}$

For the estimation of the natural frequency, the deflection has to be calculated using the short term modular ratio:

$n = E_a / E_{cm}$

The position of the elastic neutral axis is calculated from:

$$y_{el} = \frac{A h/2 + b_{eff}\left(h_f - h_p\right)\left(h + \left(h_f + h_p\right)/2\right)/n}{A + b_{eff}\left(h_f - h_p\right)/n}$$

The second moment of area of the composite cross-section is calculated from:

$$I_{eq} = I_y + \frac{b_{eff}\left(h_f - h_p\right)^3}{12\, n} + A\left(y_{el} - h/2\right)^2 + \frac{b_{eff}\left(h_f - h_p\right)}{n}\left(h + \left(h_f + h_p\right)/2 - y_{el}\right)^2$$

Note that:

b_{eff} is the effective width at mid-span.

For a plain slab, $h_p = 0$.

6.5.3 Deflections

General

The deflection can be calculated at the various key points along the beam for each combination of actions under consideration. Then the maximum value can be derived.

The deflection should be calculated for each variable load case, Q_1 and Q_2, and for each SLS combination of actions, either characteristic or frequent combination depending on the National Annex.

When the beam is fully propped at the construction stage, the deflection under the self-weight (steel profile and concrete) is calculated with composite action.

When the beam is unpropped, this deflection is calculated with no contribution of the concrete slab – the second moment of area of the steel profile is then considered: $I_{eq} = I_y$.

Deflection under a distributed load

The deflection w at the abscissa x, under a uniformly distributed load denoted Q, is calculated by:

$$w(x) = \frac{QL^3}{24EI_{eq}}\left[\frac{x}{L} - 2\left(\frac{x}{L}\right)^3 + \left(\frac{x}{L}\right)^4\right]$$

Deflection under a point load

The deflection w of a section located at the abscissa x, under a point load denoted F located at x_F, is calculated by (see Figure 4.1):

$$w(x) = \frac{F}{6EI_{eq}L}\left[L^2 - (L - x_F)^2 - x^2\right](L - x_F)\,x \quad \text{if } x < x_F$$

$$w(x) = \frac{F}{6EI_{eq}L}\left[L^2 - (L - x)^2 - x_F^2\right](L - x)\,x_F \quad \text{if } x > x_F$$

6.5.4 Vibrations

The natural frequency (in Hz) of the composite beam can be estimated from the following equations:

$f = \dfrac{18{,}07}{\sqrt{w}}$ for a uniformly distributed load

$f = \dfrac{15{,}81}{\sqrt{w}}$ for a concentrated load at mid span

where:

w is the deflection in millimetres calculated with the short term modular ratio for a combination of actions including only a percentage of the imposed loads. Depending on the National Annex, the combination can be either the characteristic or the frequent one.

7 LIST OF THE MAIN OUTPUTS

The following list is a summary of the main results of the calculations:

At the construction stage:

- The maximum bending moment and its location along the beam
- The maximum criterion for the bending resistance ($*_{M,max}$)
- The maximum vertical shear force and its location along the beam
- The maximum criterion for the vertical shear resistance ($*_{V,max}$)
- The maximum criterion for the shear buckling resistance, when necessary ($*_{Vb,max}$)
- The criterion for the LTB resistance ($*_{LT}$)
- The maximum deflection under self-weight of the beam and under the weight of the concrete
- The maximum deflection under the construction loads.

At the final stage:

- The effective width of the concrete slab
- The shear resistance of headed studs
- The maximum bending moment and its location along the beam
- The maximum vertical shear force and its location along the beam
- The degree of connection
- The minimum degree of connection
- The maximum criterion for vertical shear resistance ($*_{V,max}$)
- The maximum criterion for shear buckling resistance ($*_{Vb,max}$)
- The maximum criterion for bending resistance ($*_{M,max}$)
- The maximum criterion for bending resistance reduced by the influence of the vertical shear force ($*_{MV,max}$)
- The maximum criterion for the resistance to the horizontal shear force in the concrete slab ($*_{Vh,max}$)
- The transverse reinforcement ratio
- The maximum deflection under each variable load case Q_1 and Q_2
- The maximum deflection under each SLS combination
- The natural frequency under each SLS combination.

REFERENCES

1 EN 1994-1-1:2004 Eurocode 4 Design of composite steel and concrete structures. General rules and rules for buildings.

2 EN 1990:2002 Eurocode Basis of structural design.

3 EN 1993-1-1:2005 Eurocode 3 Design of steel structures. General rules and rules for buildings

4 EN 1992-1-1:2004 Eurocode 2: Design of concrete structures. General rules and rules for buildings.

APPENDIX A Overall flowchart

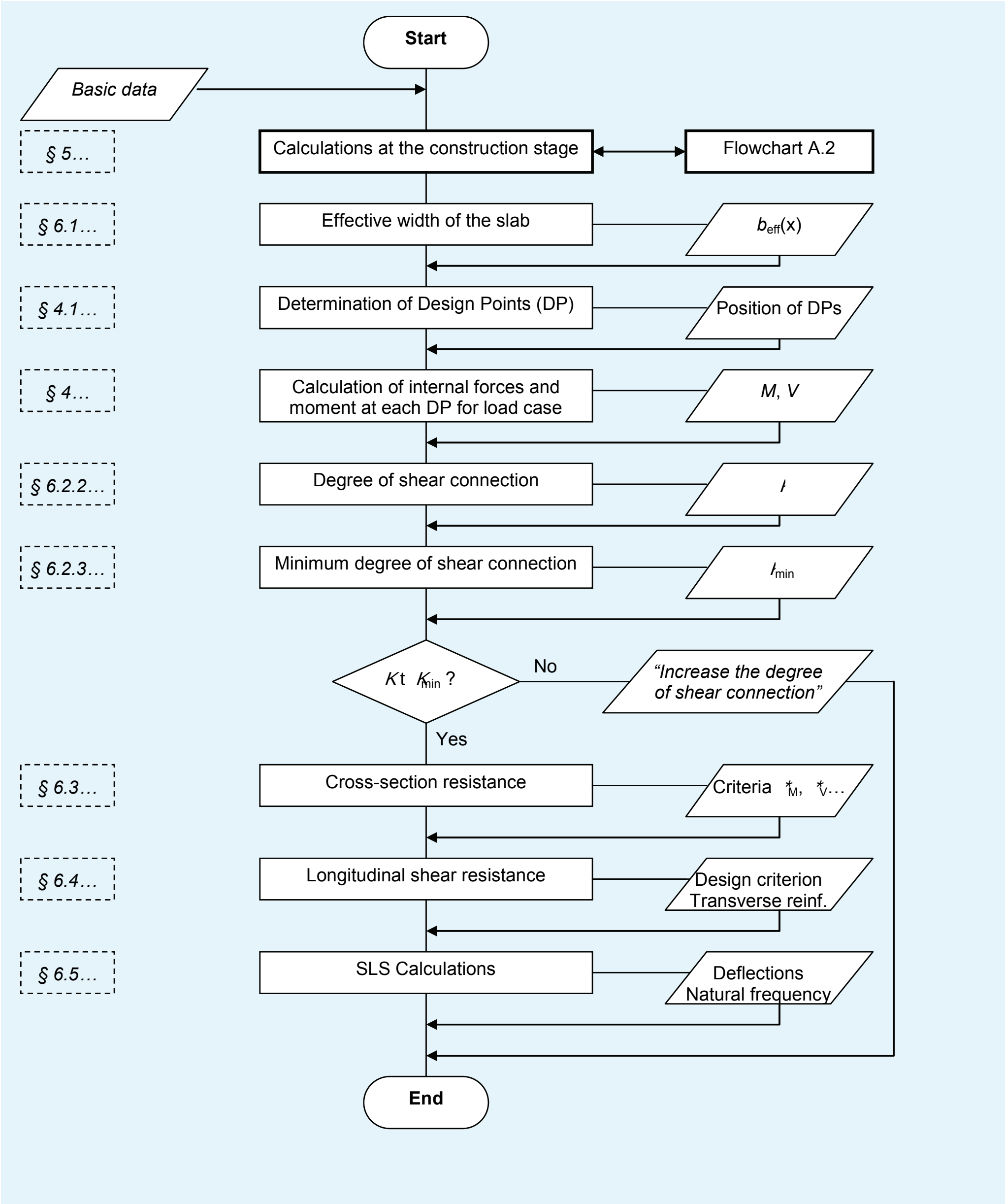


Figure A.1 Overall flowchart of the calculations

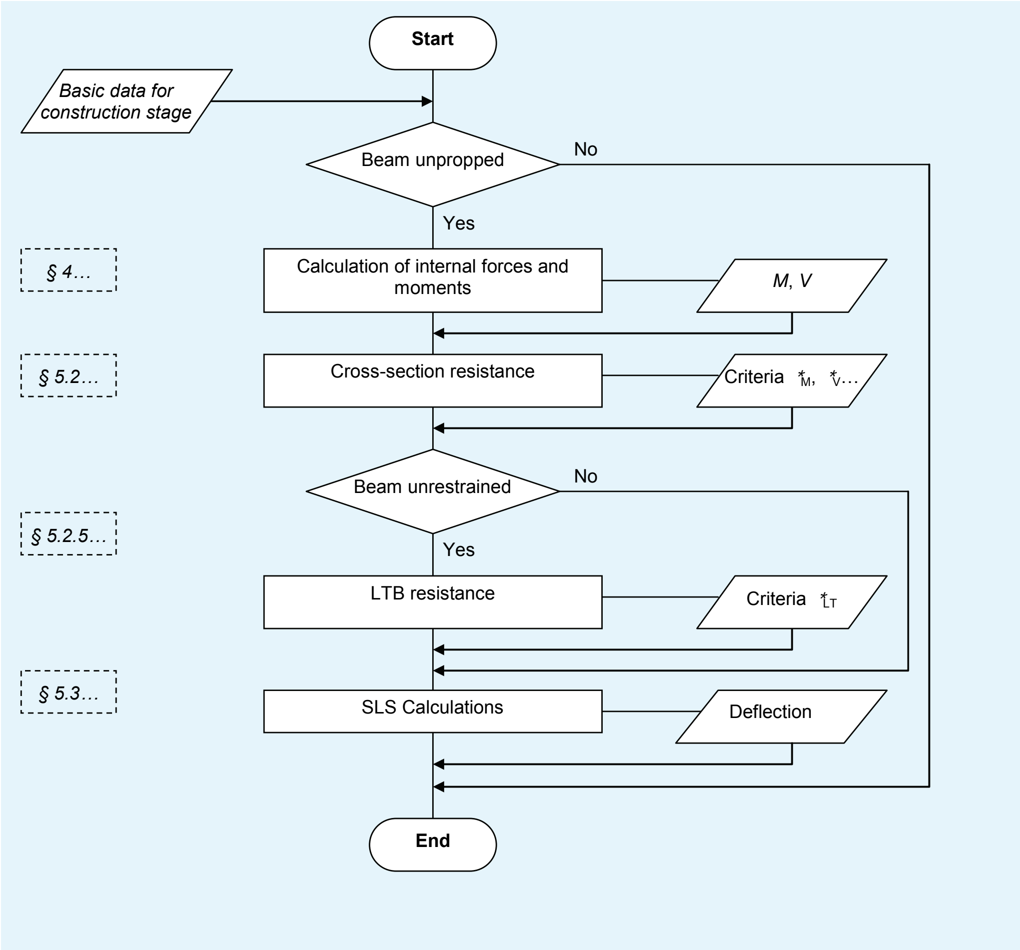


Figure A.2 Calculations at the construction stage