(4) Plane sections may be assumed to remain plane. Full composite action up to failure may be assumed between the steel and concrete components of the member.

(5) The tensile strength of concrete shall be neglected. The influence of tension stiffening of concrete between cracks on the flexural stiffness may be taken into account.

(6) Shrinkage and creep effects shall be considered if they are likely to reduce the structural stability significantly.

(7) For simplification, creep and shrinkage effects may be ignored if the increase in the first-order bending moments due to creep deformations and longitudinal force resulting from permanent loads is not greater than 10%.

(8) The following stress-strain relationships should be used in the non-linear analysis:
   - for concrete in compression as given in EN 1992-1-1, 3.1.5;
   - for reinforcing steel as given in EN 1992-1-1, 3.2.7;
   - for structural steel as given in EN 1993-1-1, 5.4.3(4).

(9) For simplification, instead of the effect of residual stresses and geometrical imperfections, equivalent initial bow imperfections (member imperfections) may be used in accordance with Table 6.5.

6.7.3 Simplified method of design

6.7.3.1 General and scope

(1) The scope of this simplified method is limited to members of doubly symmetrical and uniform cross-section over the member length with rolled, cold-formed or welded steel sections. The simplified method is not applicable if the structural steel component consists of two or more unconnected sections. The relative slenderness \( \lambda \) defined in 6.7.3.3 should fulfill the following condition:

\[
\lambda \leq 2.0
\]  

(6.28)

(2) For a fully encased steel section, see Figure 6.17a, limits to the maximum thickness of concrete cover that may be used in calculation are:

\[
\max c_z = 0.3h, \quad \max c_y = 0.4b
\]  

(6.29)

(3) The longitudinal reinforcement that may be used in calculation should not exceed 6% of the concrete area.

(4) The ratio of the cross-section’s depth \( h_c \) to width \( b_c \), see Figure 6.17a, should be within the limits \( 0.2 \leq h_c / b_c \leq 5.0 \).

6.7.3.2 Resistance of cross sections
(1) The plastic resistance to compression \( N_{pl,Rd} \) of a composite cross-section should be calculated by adding the plastic resistances of its components:

\[
N_{pl,Rd} = A_y f_{yd} + 0.85 A_c f_{cd} + A_s f_{sd}
\]  

(6.30)

Expression (6.30) applies for concrete encased and partially concrete encased steel sections. For concrete filled sections the coefficient 0.85 may be replaced by 1.0.

(2) The resistance of a cross-section to combined compression and bending and the corresponding interaction curve may be calculated assuming rectangular stress blocks as shown in Figure 6.18, taking account of the design shear force \( V_{Ed} \) in accordance with (3). The tensile strength of the concrete should be neglected.

![Interaction curve for combined compression and uniaxial bending](image)

(3) The influence of transverse shear forces on the resistance to bending and normal force should be considered when determining the interaction curve, if the shear force \( V_{a,Ed} \) on the steel section exceeds 50% of the design shear resistance \( V_{pl,a,Rd} \) of the steel section, see 6.2.2.2.

Where \( V_{a,Ed} > 0.5 V_{pl,a,Rd} \), the influence of the transverse shear on the resistance in combined bending and compression should be taken into account by a reduced design steel strength \( (1 - \rho) f_{yd} \) in the shear area \( A_v \) in accordance with 6.2.2.4(2) and Figure 6.18.

The shear force \( V_{a,Ed} \) should not exceed the resistance to shear of the steel section determined according to 6.2.2. The resistance to shear \( V_{c,Ed} \) of the reinforced concrete part should be verified in accordance with EN 1992-1-1, 6.2.

(4) Unless a more accurate analysis is used, \( V_{Ed} \) may be distributed into \( V_{a,Ed} \) acting on the structural steel and \( V_{c,Ed} \) acting on the reinforced concrete section by:

\[
V_{a,Ed} = V_{Ed} \frac{M_{pl,a,Rd}}{M_{pl,Rd}} \quad (6.31)
\]

\[
V_{c,Ed} = V_{Ed} - V_{a,Ed} \quad (6.32)
\]
where:

- $M_{pl,a,Rd}$ is the plastic resistance moment of the steel section and
- $M_{pl,Rd}$ is the plastic resistance moment of the composite section.

For simplification $V_{Ed}$ may be assumed to act on the structural steel section alone.

(5) As a simplification, the interaction curve may be replaced by a polygonal diagram (the dashed line in Figure 6.19). Figure 6.19 shows as an example the plastic stress distribution of a fully encased cross section for the points A to D. $N_{pm,Rd}$ should be taken as $0.85 f_{cd} A_c$ for concrete encased and partially concrete encased sections, see Figures 6.17a – c, and as $f_{cd} A_c$ for concrete filled sections, see Figures 6.17d - f.

![Figure 6.19: Simplified interaction curve and corresponding stress distributions](image)

(6) For concrete filled tubes of circular cross-section, account may be taken of increase in strength of concrete caused by confinement provided that the relative slenderness $\lambda$ defined in 6.7.3.3 does not exceed 0.5 and $e/d < 0.1$, where $e$ is the eccentricity of loading given by $M_{Ed}/N_{Ed}$ and $d$ is the external diameter of the column. The plastic resistance to compression may then be calculated from the following expression:

$$N_{pl,Rd} = \eta_s A_y f_{yd} + A_c f_{cd} \left(1 + \eta_s \frac{t}{d} \frac{f_y}{f_{ck}}\right) + A_s f_{sd}$$  \hspace{1cm} (6.33)

where:

- $t$ is the wall thickness of the steel tube.
For members with \( e = 0 \) the values \( \eta_a = \eta_{ao} \) and \( \eta_c = \eta_{co} \) are given by the following expressions:

\[
\eta_{ao} = 0.25 \left( 3 + 2 \bar{\lambda} \right) \quad \text{but} \leq 1.0 \tag{6.34}
\]

\[
\eta_{co} = 4.9 - 18.5 \bar{\lambda} + 17 \bar{\lambda}^2 \quad \text{but} \geq 0 \tag{6.35}
\]

For members in combined compression and bending with \( 0 < e/d \leq 0.1 \), the values \( \eta_a \) and \( \eta_c \) should be determined from (6.36) and (6.37), where \( \eta_{ao} \) and \( \eta_{co} \) are given by (6.34) and (6.35):

\[
\eta_a = \eta_{ao} + (1 - \eta_{ao}) \left( 10 \frac{e}{d} \right) \quad \text{(6.36)}
\]

\[
\eta_c = \eta_{co} \left( 1 - 10 \frac{e}{d} \right) \quad \text{(6.37)}
\]

For \( e/d > 0.1 \), \( \eta_a = 1.0 \) and \( \eta_c = 0 \).

6.7.3.3 Effective flexural stiffness, steel contribution ratio and relative slenderness

(1) The steel contribution ratio \( \delta \) is defined as:

\[
\delta = \frac{A_a f_{yd}}{N_{pl,Rd}} \quad \text{(6.38)}
\]

where:

\( N_{pl,Rd} \) is the plastic resistance to compression defined in 6.7.3.2(1).

(2) The relative slenderness \( \bar{\lambda} \) for the plane of bending being considered is given by:

\[
\bar{\lambda} = \sqrt{\frac{N_{pl,Rk}}{N_{cr}}} \quad \text{(6.39)}
\]

where:

\( N_{pl,Rk} \) is the characteristic value of the plastic resistance to compression given by (6.30) if, instead of the design strengths, the characteristic values are used;

\( N_{cr} \) is the elastic critical normal force for the relevant buckling mode, calculated with the effective flexural stiffness \((EI)_{eff}\) determined in accordance with (3) and (4).
(3) For the determination of the relative slenderness $\lambda$ and the elastic critical force $N_{cr}$, the characteristic value of the effective flexural stiffness $(EI)_{eff}$ of a cross section of a composite column should be calculated from:

$$(EI)_{eff} = E_a I_a + E_s I_s + K_e E_{cm} I_c$$

where:

$K_e$ is a correction factor that should be taken as 0.6.

$I_a$, $I_c$, and $I_s$ are the second moments of area of the structural steel section, the uncracked concrete section and the reinforcement for the bending plane being considered.

(4) Account should be taken to the influence of long-term effects on the effective elastic flexural stiffness. The modulus of elasticity of concrete $E_{cm}$ should be reduced to the value $E_{c,eff}$ in accordance with the following expression:

$$E_{c,eff} = \frac{E_{cm}}{1 + (N_{G,Ed} / N_{Ed}) \varphi_t}$$

where:

$\varphi_t$ is the creep coefficient according to 5.4.2.2(2);

$N_{Ed}$ is the total design normal force;

$N_{G,Ed}$ is the part of this normal force that is permanent.

### 6.7.3.4 Methods of analysis and member imperfections

(1) For member verification, analysis should be based on second-order linear elastic analysis.

(2) For the determination of the internal forces the design value of effective flexural stiffness $(EI)_{eff,II}$ should be determined from the following expression:

$$(EI)_{eff,II} = K_o (E_a I_a + E_s I_s + K_{e,II} E_{cm} I_c)$$

where:

$K_{e,II}$ is a correction factor which should be taken as 0.5;

$K_o$ is a calibration factor which should be taken as 0.9.

Long-term effects should be taken into account in accordance with 6.7.3.3 (4).
(3) Second-order effects need not to be considered where 5.2.1(3) applies and the elastic critical load is determined with the flexural stiffness $(EI)_{\text{eff,II}}$ in accordance with (2).

(4) The influence of geometrical and structural imperfections may be taken into account by equivalent geometrical imperfections. Equivalent member imperfections for composite columns are given in Table 6.5, where $L$ is the column length.

(5) Within the column length, second-order effects may be allowed for by multiplying the greatest first-order design bending moment $M_{\text{Ed}}$ by a factor $k$ given by:

$$k = \frac{\beta}{1 - N_{\text{Ed}} / N_{\text{cr,eff}}} \geq 1.0$$

(6.43)

where:

- $N_{\text{cr,eff}}$ is the critical normal force for the relevant axis and corresponding to the effective flexural stiffness given in 6.7.3.4(2), with the effective length taken as the column length;
- $\beta$ is an equivalent moment factor given in Table 6.4.

**Table 6.4 Factors $\beta$ for the determination of moments to second order theory**

<table>
<thead>
<tr>
<th>Moment distribution</th>
<th>Moment factors $\beta$</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1" alt="Moment distribution 1" /> $M_{\text{Ed}}$</td>
<td>First-order bending moments from member imperfection or lateral load: $\beta = 1.0$</td>
<td>$M_{\text{Ed}}$ is the maximum bending moment within the column length ignoring second-order effects</td>
</tr>
<tr>
<td><img src="image2" alt="Moment distribution 2" /> $M_{\text{Ed}}$</td>
<td>End moments: $\beta = 0.66 + 0.44r$ but $\beta \geq 0.44$</td>
<td>$M_{\text{Ed}}$ and $r M_{\text{Ed}}$ are the end moments from first-order or second-order global analysis</td>
</tr>
</tbody>
</table>

6.7.3.5 Resistance of members in axial compression
(1) Members may be verified using second order analysis according to 6.7.3.6 taking into account member imperfections.

(2) For simplification for members in axial compression, the design value of the normal force $N_{Ed}$ should satisfy:

$$\frac{N_{Ed}}{\chi N_{pl,Rd}} \leq 1.0$$

(6.44)

where:

$N_{pl,Rd}$ is the plastic resistance of the composite section according to 6.7.3.2(1), but with $f_{yd}$ determined using the partial factor $\gamma_M$ given by EN 1993-1-1, 6.1(1);

$\chi$ is the reduction factor for the relevant buckling mode given in EN 1993-1-1, 6.3.1.2 in terms of the relevant relative slenderness $\lambda$.

The relevant buckling curves for cross-sections of composite columns are given in Table 6.5, where $\rho_s$ is the reinforcement ratio $A_s/A_c$.

### 6.7.3.6 Resistance of members in combined compression and uniaxial bending

(1) The following expression based on the interaction curve determined according to 6.7.3.2 (2) - (5) should be satisfied:

$$\frac{M_{Ed}}{M_{pl,N,Rd}} = \frac{M_{Ed}}{\mu_d M_{pl,Rd}} \leq \alpha_M$$

(6.45)

where:

$M_{Ed}$ is the greatest of the end moments and the maximum bending moment within the column length, calculated according to 6.7.3.4, including imperfections and second order effects if necessary;

$M_{pl,N,Rd}$ is the plastic bending resistance taking into account the normal force $N_{Ed}$, given by $\mu_d M_{pl,Rd}$, see Figure 6.18;

$M_{pl,Rd}$ is the plastic bending resistance, given by point B in Figure 6.19.

For steel grades between S235 and S355 inclusive, the coefficient $\alpha_M$ should be taken as 0.9 and for steel grades S420 and S460 as 0.8.

(2) The value $\mu_d = \mu_{dy}$ or $\mu_{dz}$ , see Figure 6.20, refers to the design plastic resistance moment $M_{pl,Rd}$ for the plane of bending being considered. Values $\mu_d$ greater than 1.0 should only be used where the bending moment $M_{Ed}$ depends directly on the action of the normal force $N_{Ed}$, for example where the moment $M_{Ed}$ results from an eccentricity of the normal force $N_{Ed}$. Otherwise an additional verification is necessary in accordance with clause 6.7.1 (7).
6.7.3.7 Combined compression and biaxial bending

(1) For composite columns and compression members with biaxial bending the values \( \mu_{dy} \) and \( \mu_{dz} \) in Figure 6.20 may be calculated according to 6.7.3.6 separately for each axis. Imperfections should be considered only in the plane in which failure is expected to occur. If it is not evident which plane is the more critical, checks should be made for both planes.

(2) For combined compression and biaxial bending the following conditions should be satisfied for the stability check within the column length and for the check at the end:

\[
\frac{M_{y,Ed}}{\mu_{dy} M_{pl,y,Rd}} \leq \alpha_{M,y} \quad \frac{M_{z,Ed}}{\mu_{dz} M_{pl,z,Rd}} \leq \alpha_{M,z}
\]  

(6.46)

Table 6.5 : Buckling curves and member imperfections for composite columns

<table>
<thead>
<tr>
<th>Cross-section</th>
<th>Limits</th>
<th>Axis of buckling</th>
<th>Buckling curve</th>
<th>Member imperfection</th>
</tr>
</thead>
<tbody>
<tr>
<td>concrete encased section</td>
<td>(y-y)</td>
<td>b</td>
<td>L/200</td>
<td></td>
</tr>
<tr>
<td>z-z</td>
<td>c</td>
<td>L/150</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Section Type</td>
<td>Moment Location</td>
<td>Moment Value</td>
<td>Limitation</td>
<td></td>
</tr>
<tr>
<td>--------------</td>
<td>-----------------</td>
<td>--------------</td>
<td>------------</td>
<td></td>
</tr>
<tr>
<td>Partially concrete encased section</td>
<td>y-y b</td>
<td>$L/200$</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>z-z c</td>
<td>$L/150$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Circular and rectangular hollow steel section</td>
<td>$\rho_s \leq 3%$ any a</td>
<td>$L/300$</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$3% &lt; \rho_s \leq 6%$ any b</td>
<td>$L/200$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Circular hollow steel sections with additional I-section</td>
<td>y-y b</td>
<td>$L/200$</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>z-z b</td>
<td>$L/200$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Partially concrete encased section with crossed I-sections</td>
<td>any b</td>
<td>$L/200$</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

$$\frac{M_{y,Ed}}{\mu_{dy} M_{pl,y,Rd}} + \frac{M_{z,Ed}}{\mu_{dz} M_{pl,z,Rd}} \leq 1,0 \quad (6.47)$$

where:

- $M_{pl,y,Rd}$ and $M_{pl,z,Rd}$ are the plastic bending resistances of the relevant plane of bending;
- $M_{y,Ed}$ and $M_{z,Ed}$ are the design bending moments including second-order effects and imperfections according to 6.7.3.4;
- $\mu_{dy}$ and $\mu_{dz}$ are defined in 6.7.3.6;
\[ \alpha_M = \alpha_{M,y} \] and \( \alpha_M = \alpha_{M,z} \) are given in 6.7.3.6(1).

### 6.7.4 Shear connection and load introduction

#### 6.7.4.1 General

(1)P Provision shall be made in regions of load introduction for internal forces and moments applied from members connected to the ends and for loads applied within the length to be distributed between the steel and concrete components, considering the shear resistance at the interface between steel and concrete. A clearly defined load path shall be provided that does not involve an amount of slip at this interface that would invalidate the assumptions made in design.

(2)P Where composite columns and compression members are subjected to significant transverse shear, as for example by local transverse loads and by end moments, provision shall be made for the transfer of the corresponding longitudinal shear stress at the interface between steel and concrete.

(3) For axially loaded columns and compression members, longitudinal shear outside the areas of load introduction need not be considered.

#### 6.7.4.2 Load introduction

(1) Shear connectors should be provided in the load introduction area and in areas with change of cross section, if the design shear strength \( \tau_{rd} \), see 6.7.4.3, is exceeded at the interface between steel and concrete. The shear forces should be determined from the change of sectional forces of the steel or reinforced concrete section within the introduction length. If the loads are introduced into the concrete cross section only, the values resulting from an elastic analysis considering creep and shrinkage should be taken into account. Otherwise, the forces at the interface should be determined by elastic theory or plastic theory, to determine the more severe case.

(2) In absence of a more accurate method, the introduction length should not exceed \( 2d \) or \( L/3 \), where \( d \) is the minimum transverse dimension of the column and \( L \) is the column length.

(3) For composite columns and compression members no shear connection need be provided for load introduction by endplates if the full interface between the concrete section and endplate is permanently in compression, taking account of creep and shrinkage. Otherwise the load introduction should be verified according to (5). For concrete filled tubes of circular cross-section the effect caused by the confinement may be taken into account if the conditions given in 6.7.3.2(6) are satisfied using the values \( \eta_a \) and \( \eta_c \) for \( \lambda \) equal to zero.
(4) Where stud connectors are attached to the web of a fully or partially concrete encased steel I-section or a similar section, account may be taken of the frictional forces that develop from the prevention of lateral expansion of the concrete by the adjacent steel flanges. This resistance may be added to the calculated resistance of the shear connectors. The additional resistance may be assumed to be $\mu \frac{P_{Rd}}{2}$ on each flange and each horizontal row of studs, as shown in Figure 6.21, where $\mu$ is the relevant coefficient of friction that may be assumed. For steel sections without painting, $\mu$ may be taken as 0.5. $P_{Rd}$ is the resistance of a single stud in accordance with 6.6.3.1. In absence of better information from tests, the clear distance between the flanges should not exceed the values given in Figure 6.21.

![Figure 6.21: Additional frictional forces in composite columns by use of headed studs](image)

(5) If the cross-section is partially loaded (as, for example, Figure 6.22A), the loads may be distributed with a ratio of 1:2.5 over the thickness $t_e$ of the end plate. The concrete stresses should then be limited in the area of the effective load introduction, for concrete filled hollow sections in accordance with (6) and for all other types of cross-sections in accordance with EN 1992-1-1, 6.7.

(6) If the concrete in a filled circular hollow section or a square hollow section is only partially loaded, for example by gusset plates through the profile or by stiffeners as shown in Figure 6.22, the local design strength of concrete, $\sigma_{c,Rd}$ under the gusset plate or stiffener resulting from the sectional forces of the concrete section should be determined by:

$$\sigma_{c,Rd} = f_{cd} \left(1 + \eta_{cl} \frac{t}{a} \frac{f_y}{f_{ck}} \right) \sqrt{\frac{A_c}{A_1}} \leq \frac{A_c f_{cd}}{A_1}, \leq f_{yd}$$ (6.48)

where:
- $t$ is the wall thickness of the steel tube;
- $a$ is the diameter of the tube or the width of the square section;
- $A_c$ is the cross sectional area of the concrete section of the column;
- $A_1$ is the loaded area under the gusset plate, see Figure 6.22;
\( \eta_{el} = 4.9 \) for circular steel tubes and 3.5 for square sections.

The ratio \( A_c/A_1 \) should not exceed the value 20. Welds between the gusset plate and the steel hollow sections should be designed according to EN1993-1-8, Section 4.

![Diagram of partially loaded circular concrete filled hollow section](image)

Figure 6.22: Partially loaded circular concrete filled hollow section

(7) For concrete filled circular hollow sections, longitudinal reinforcement may be taken into account for the resistance of the column, even where the reinforcement is not welded to the end plates or in direct contact with the endplates provided that the gap \( e_g \) between the reinforcement and the end plate does not exceed 30 mm, see Figure 6.22A.

(8) Transverse reinforcement should be in accordance with EN 1992-1-1, 9.5.3. In case of partially encased steel sections, concrete should be held in place by transverse reinforcement arranged in accordance with Figure 6.10.
(9) In the case of load introduction through only the steel section or the concrete section, for fully encased steel sections the transverse reinforcement should be designed for the longitudinal shear that results from the transmission of normal force \( N_{ct} \) in Figure 6.23 from the parts of concrete directly connected by shear connectors into the parts of the concrete without direct shear connection (see Figure 6.23, section A-A; the hatched area outside the flanges of Figure 6.23 should be considered as not directly connected). The design and arrangement of transverse reinforcement should be based on a truss model assuming an angle of 45° between concrete compression struts and the member axis.

6.7.4.3 Longitudinal shear outside the areas of load introduction

(1) Outside the area of load introduction, longitudinal shear at the interface between concrete and steel should be verified where it is caused by transverse loads and/or end moments. Shear connectors should be provided, based on the distribution of the design value of longitudinal shear, where this exceeds the design shear strength \( \tau_{Rd} \).

(2) In absence of a more accurate method, elastic analysis, considering long term effects and cracking of concrete, may be used to determine the longitudinal shear at the interface.

(3) Provided that the surface of the steel section in contact with the concrete is unpainted and free from oil, grease and loose scale or rust, the values given in Table 6.6 may be assumed for \( \tau_{Rd} \).

**Table 6.6 : Design shear strength \( \tau_{Rd} \)**

<table>
<thead>
<tr>
<th>Type of cross section</th>
<th>( \tau_{Rd} ) (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Completely concrete encased steel sections</td>
<td>0,30</td>
</tr>
</tbody>
</table>
Concrete filled circular hollow sections 0,55
Concrete filled rectangular hollow sections 0,40
Flanges of partially encased sections 0,20
Webs of partially encased sections 0,00

(4) The value of $\tau_{Rd}$ given in Table 6.6 for completely concrete encased steel sections applies to sections with a minimum concrete cover of 40mm and transverse and longitudinal reinforcement in accordance with 6.7.5.2. For greater concrete cover and adequate reinforcement, higher values of $\tau_{Rd}$ may be used. Unless verified by tests, for completely encased sections the increased value $\beta_c\tau_{Rd}$ may be used, with $\beta_c$ given by:

$$\beta_c = 1 + 0,02 \left( c_z - \frac{c_{z,\text{min}}}{c_z} \right) \leq 2,5$$  (6.49)

where:

- $c_z$ is the nominal value of concrete cover in mm, see Figure 6.17a;
- $c_{z,\text{min}} = 40$ mm is the minimum concrete cover.

(5) Unless otherwise verified, for partially encased I-sections with transverse shear due to bending about the weak axis due to lateral loading or end moments, shear connectors should always be provided. If the resistance to transverse shear is not be taken as only the resistance of the structural steel, then the required transverse reinforcement for the shear force $V_{c,Ed}$ according to 6.7.3.2(4) should be welded to the web of the steel section or should pass through the web of the steel section.

6.7.5 Detailing Provisions

6.7.5.1 Concrete cover of steel profiles and reinforcement

(1) For fully encased steel sections at least a minimum cover of reinforced concrete shall be provided to ensure the safe transmission of bond forces, the protection of the steel against corrosion and spalling of concrete.

(2) The concrete cover to a flange of a fully encased steel section should be not less than 40mm, nor less than one-sixth of the breadth $b$ of the flange.

(3) The cover to reinforcement should be in accordance with EN 1992-1-1, Section 4.

6.7.5.2 Longitudinal and transverse reinforcement

(1) The longitudinal reinforcement in concrete-encased columns which is allowed for in the resistance of the cross-section should be not less than 0,3% of the cross-section of the concrete. In concrete filled hollow sections normally no longitudinal reinforcement is necessary, if design for fire resistance is not required.
(2) The transverse and longitudinal reinforcement in fully or partially concrete encased columns should be designed and detailed in accordance with EN 1992-1-1, 9.5.

(3) The clear distance between longitudinal reinforcing bars and the structural steel section may be smaller that required by (2), even zero. In this case, for bond the effective perimeter $c$ of the reinforcing bar should be taken as half or one quarter of its perimeter, as shown in Figure 6.24 at (a) and (b) respectively.

(4) For fully or partially encased members, where environmental conditions are class X0 according to EN 1992-1-1, Table 4.1, and longitudinal reinforcement is neglected in design, a minimum longitudinal reinforcement of diameter 8 mm and 250 mm spacing and a transverse reinforcement of diameter 6 mm and 200 mm spacing should be provided. Alternatively welded mesh reinforcement of diameter 4 mm may be used.

![Figure 6.24: Effective perimeter $c$ of a reinforcing bar](image)

6.8 Fatigue

6.8.1 General

(1) The resistance of composite structures to fatigue shall be verified where the structures are subjected to repeated fluctuations of stresses.

(2) Design for the limit state of fatigue shall ensure, with an acceptable level of probability, that during its entire design life, the structure is unlikely to fail by fatigue or to require repair of damage caused by fatigue.

(3) For headed stud shear connectors, under the characteristic combination of actions the maximum longitudinal shear force per connector should not exceed $0.75P_{Rd}$, where $P_{Rd}$ is determined according to 6.6.3.1.

(4) In buildings no fatigue assessment for structural steel, reinforcement, concrete and shear connection is required where, for structural steel, EN 1993-1-1, 4(4) applies and, for concrete, EN 1992-1-1, 6.8.1, does not apply.

6.8.2 Partial factors for fatigue assessment

(1) Partial factors $\gamma_{Mf}$ for fatigue strength are given in EN 1993-1-9, 3 for steel elements and in EN 1992-1-1, 2.4.2.4 for concrete and reinforcement. For headed studs in shear, a partial factor $\gamma_{Mf,s}$ should be applied.
Note: The value for $\gamma_{Mf,s}$ may be given in the National Annex. The recommended value for $\gamma_{Mf,s}$ is 1.0.

(2) Partial factors for fatigue loading $\gamma_{Ff}$ should be applied.

Note: Partial factors $\gamma_{Ff}$ for different kinds of fatigue loading may be given in the National Annex.

### 6.8.3 Fatigue strength

(1) The fatigue strength for structural steel and for welds should be taken from EN 1993-1-9, 7.

(2) The fatigue strength of reinforcing steel and pre-stressing steel should be taken from EN 1992-1-1. For concrete EN 1992-1-1, 6.8.5 applies.

(3) The fatigue strength curve of an automatically welded headed stud in accordance with 6.6.3.1 is shown in Fig. 6.25 and given for normal weight concrete by:

$$
(\Delta \tau_{R})^m N_R = (\Delta \tau_{c})^m N_c
$$

(6.50)

where:
- $\Delta \tau_{R}$ is the fatigue strength;
- $\Delta \tau_{c}$ is the reference value at 2 million cycles with $\Delta \tau_{c}$ equal to 90 N/mm$^2$;
- $m$ is the slope of the fatigue strength curve with the value $m = 8$;
- $N_R$ is the number of stress-range cycles.

![Figure 6.25 : Fatigue strength curve for headed studs in solid slabs](image)

(4) For studs in lightweight concrete with a density class according to EN 1992-1-1, 11, the fatigue strength should be determined in accordance with (3) but with $\Delta \tau_{R}$ replaced by $\eta_E \Delta \tau_{R}$ and $\Delta \tau_{c}$ replaced by $\eta_E \Delta \tau_{c}$, where $\eta_E$ is given in EN 1992-1-1, 11.3.2.

### 6.8.4 Internal forces and fatigue loadings

(1) Internal forces and moments should be determined by elastic global analysis of the structure in accordance with 5.4.1 and 5.4.2 and for the combination of actions given in EN 1992-1-1, 6.8.3.

(2) The maximum and minimum internal bending moments and/or internal forces resulting from the load combination according to (1) are defined as $M_{Ed,max,f}$ and $M_{Ed,min,f}$. 

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(3) For buildings fatigue loading should be obtained from the relevant Parts of EN 1991. Where no fatigue loading is specified, EN 1993-1-9, Annex A.1 may be used. Dynamic response of the structure or impact effects should be considered when appropriate.

6.8.5 Stresses

6.8.5.1 General

(1) The calculation of stresses should be based on 7.2.1.

(2) For the determination of stresses in cracked regions the effect of tension stiffening of concrete on the stresses in reinforcement shall be taken into account.

(3) Unless verified by a more accurate method, the effect of tension stiffening on the stresses in reinforcement may be taken into account according to 6.8.5.4.

(4) Unless a more accurate method is used, for the determination of stresses in structural steel the effect of tension stiffening may be neglected.

6.8.5.2 Concrete

(1) For the determination of stresses in concrete elements EN 1992-1-1, 6.8 applies.

6.8.5.3 Structural steel

(1) Where the bending moments $M_{Ed,max,f}$ and $M_{Ed,min,f}$ cause tensile stresses in the concrete slab, the stresses in structural steel for these bending moments may be determined based on the second moment of area $I_2$ according to 1.5.2.12

(2) Where $M_{Ed,min,f}$ and $M_{Ed,max,f}$, or only $M_{Ed,min,f}$, cause compression in the concrete slab, the stresses in structural steel for these bending moments should be determined with the cross-section properties of the un-cracked section.

6.8.5.4 Reinforcement

(1) Where the bending moments $M_{Ed,max,f}$ and $M_{Ed,min,f}$ cause tensile stresses in the concrete slab and where no more accurate method is used, the effects of tension stiffening of concrete on the stress $\sigma_{s,max,f}$ in reinforcement due to $M_{Ed,max,f}$ should be determined from the equations (7.4) to (7.6) in 7.4.3 (3).

(2) The tensile stress in the reinforcement due to $M_{Ed,min,f}$ is given by Figure 6.26 and can be determined from:

$$\sigma_{s,min,f} = \sigma_{s,max,f} \frac{M_{Ed,min,f}}{M_{Ed,max,f}}$$

(6.51)

(3) Where $M_{Ed,min,f}$ and $M_{Ed,max,f}$, or only $M_{Ed,min,f}$, cause compression in the concrete slab, the stresses in reinforcement for these bending moments should be determined with the cross-section properties of the un-cracked section.
Figure 6.26: Determination of the stresses $\sigma_{s,\text{max},f}$ and $\sigma_{s,\text{min},f}$ in cracked regions

6.8.5.5 Shear Connection

(1) The longitudinal shear per unit length shall be calculated by elastic analysis.

(2) In members where cracking of concrete occurs the effects of tension stiffening should be taken into account by an appropriate model. For simplification, the longitudinal shear forces at the interface between structural steel and concrete may be determined by using the properties of the un-cracked section.

6.8.6 Stress ranges

6.8.6.1 Structural steel and reinforcement

(1) The stress ranges should be determined from the stresses determined in accordance with 6.8.5.

(2) Where the verification for fatigue is based on damage equivalent stress ranges, in general a range $\Delta\sigma_E$ should be determined from:

$$\Delta\sigma_E = \lambda \phi |\sigma_{\max,f} - \sigma_{\min,f}|$$  \hspace{1cm} (6.52)

where:

$\sigma_{\max,f}$ and $\sigma_{\min,f}$ are the maximum and minimum stresses due to 6.8.4 and 6.8.5;

$\lambda$ is a damage equivalent factor;

$\phi$ is a damage equivalent impact factor.

(3) Where a member is subjected to combined global and local effects the separate effects should be considered. Unless a more precise method is used the equivalent constant amplitude stress due to global effects and local effects should be combined using:

$$\Delta\sigma_E = \lambda_{\text{glob}} \phi_{\text{glob}} \Delta\sigma_{E,\text{glob}} + \lambda_{\text{loc}} \phi_{\text{loc}} \Delta\sigma_{E,\text{loc}}$$  \hspace{1cm} (6.53)

in which subscripts “glob” and “loc” refer to global and local effects, respectively.
(4) For buildings, \( \Delta \sigma \) for structural steel may be taken as the stress range \( \Delta \sigma_{E,2} \) defined in EN 1993-1-9, 1.3 and for reinforcement as the stress range \( \Delta \sigma_{s,equ} \) given by EN 1992-1-1, 6.8.5.

(5) For buildings the damage equivalent factor \( \lambda \) is defined in EN 1993-1-9, 6.2 and in the relevant parts of EN 1993 for steel elements and for reinforcing steel in the relevant Parts of EN 1992.

(6) Where for buildings no value for \( \lambda \) is specified, the damage equivalent factor should be determined according to EN 1993-1-9, Annex A, using the slope of the relevant fatigue strength curve.

### 6.8.6.2 Shear connection

(1) For verification of stud shear connectors based on nominal stress ranges the equivalent constant stress range \( \Delta \tau_{E,2} \) for 2 million cycles is given by:

\[
\Delta \tau_{E,2} = \lambda_v \Delta \tau
\]

where:

- \( \lambda_v \) is the damage equivalent factor depending on the spectra and the slope \( m \) of the fatigue strength curve;
- \( \Delta \tau \) is the stress range due to fatigue loading.

(2) The equivalent constant amplitude shear stress range in welds of other types of shear connection should be calculated in accordance with EN 1993-1-9, 6.

(3) Where for stud connectors in buildings no value for \( \lambda_v \) is specified, the damage equivalent factor should be determined in accordance with EN 1993-1-9, Annex A, using the relevant slope of the fatigue strength curve of the stud connector, given in 6.8.3.

### 6.8.7 Fatigue assessment based on nominal stress ranges

#### 6.8.7.1 Structural steel, reinforcement and concrete

(1) The fatigue assessment for reinforcement should follow EN 1992-1-1, 6.8.5 or 6.8.6

(2) The verification for concrete in compression should follow EN 1992-1-1, 6.8.7.

(3) For buildings the fatigue assessment for structural steel should follow EN 1993-1-9, 8.

#### 6.8.7.2 Shear connection

(1) For stud connectors welded to a steel flange that is always in compression under the relevant combination of actions (see 6.8.4 (1)), the fatigue assessment should be made by checking the criterion:

\[
\gamma_{Ff} \Delta \tau_{E,2} \leq \Delta \tau_c / \gamma_{Mf,s}
\]

where:
Δτ_{E,2} \text{ is defined in 6.8.6.2(1);}

Δτ_{c} \text{ is the reference value of fatigue strength at 2 million cycles determined in accordance with 6.8.3.}

The stress range Δτ in the stud should be determined with the cross-sectional area of the shank of the stud using the nominal diameter \( d \) of the shank.

(2) Where the maximum stress in the steel flange to which stud connectors are welded is tensile under the relevant combination, the interaction at any cross-section between shear stress range Δτ_{E} in the weld of stud connectors and the normal stress range Δσ_{E} in the steel flange should be verified using the following interaction expressions.

\[
\frac{\gamma_{Ff} \Delta\sigma_{E,2}}{\Delta\sigma_{c} / \gamma_{Mf}} + \frac{\gamma_{Ff} \Delta\tau_{E,2}}{\Delta\tau_{c} / \gamma_{Mf,s}} \leq 1,3
\]  

(6.56)

\[
\frac{\gamma_{Ff} \Delta\sigma_{E,2}}{\Delta\sigma_{c} / \gamma_{Mf}} \leq 1,0 \quad \frac{\gamma_{Ff} \Delta\tau_{E,2}}{\Delta\tau_{c} / \gamma_{Mf,s}} \leq 1,0
\]  

(6.57)

where:

Δσ_{E,2} \text{ is the stress range in the flange determined in accordance with 6.8.6.1;}

Δσ_{c} \text{ is the reference value of fatigue strength given in EN1993-1-9, 7, by applying category 80, and the stress ranges Δτ_{E,2} and Δτ_{c} are defined in (1).}

Expression (6.56) should be checked for the maximum value of Δσ_{E,2} and the corresponding value Δτ_{E,2}, as well as for the combination of the maximum value of Δτ_{E,2} and the corresponding value of Δσ_{E,2}. Unless taking into account the effect of tension stiffening of concrete by more accurate methods, the interaction criterion should be verified with the corresponding stress ranges determined with both cracked and un-cracked cross-sectional properties.

Section 7  Serviceability limit states

7.1 General

(1) A structure with composite members shall be designed and constructed such that all relevant serviceability limit states are satisfied according to the Principles of 3.4 of EN 1990.

(2) The verification of serviceability limit states should be based on the criteria given in EN 1990, 3.4(3).

(3) Serviceability limit states for composite slabs with profiled steel sheeting should be verified in accordance with Section 9.

7.2 Stresses

7.2.1 General
(1)P Calculation of stresses for beams at the serviceability limit state shall take into account the following effects, where relevant:

- shear lag;
- creep and shrinkage of concrete;
- cracking of concrete and tension stiffening of concrete;
- sequence of construction;
- increased flexibility resulting from significant incomplete interaction due to slip of shear connection;
- inelastic behaviour of steel and reinforcement, if any;
- torsional and distortional warping, if any.

(2) Shear lag may be taken into account according to 5.4.1.2.

(3) Unless a more accurate method is used, effects of creep and shrinkage may be taken into account by use of modular ratios according to 5.4.2.2.

(4) In cracked sections the primary effects of shrinkage may be neglected when verifying stresses.

(5)P In section analysis the tensile strength of concrete shall be neglected.

(6) The influence of tension stiffening of concrete between cracks on stresses in reinforcement and pre-stressing steel should be taken into account. Unless more accurate methods are used, the stresses in reinforcement should be determined according to 7.4.3.

(7) The influences of tension stiffening on stresses in structural steel may be neglected.

(8) The effects of incomplete interaction may be ignored, where full shear connection is provided and where, in case of partial shear connection in buildings, 7.3.1(4) applies.

7.2.2 Stress limitation for buildings

(1) Stress limitation is not required for beams if, in the ultimate limit state, no verification of fatigue is required and no pre-stressing by tendons and/or by controlled imposed deformations (e.g. jacking of supports) is provided.

(2) For composite columns in buildings normally no stress limitation is required.

(3) If required, the stress limitations for concrete and reinforcement given in EN 1992-1-1, 7.2 apply.
7.3 Deformations in buildings

7.3.1 Deflections

(1) Deflections due to loading applied to the steel member alone should be calculated in accordance with EN 1993-1-1.

(2) Deflections due to loading applied to the composite member should be calculated using elastic analysis in accordance with Section 5.

(3) The reference level for the sagging vertical deflection $\delta_{\text{max}}$ of un-propped beams is the upper-side of the composite beam. Only where the deflection can impair the appearance of the building should the underside of the beam be taken as reference level.

(4) The effects of incomplete interaction may be ignored provided that:
   a) the design of the shear connection is in accordance with 6.6,
   b) either not less shear connectors are used than half the number for full shear connection, or the forces resulting from an elastic behaviour and which act on the shear connectors in the serviceability limit state do not exceed $P_{\text{Rd}}$ and
   c) in case of a ribbed slab with ribs transverse to the beam, the height of the ribs does not exceed 80 mm.

(5) The effect of cracking of concrete in hogging moment regions on the deflection should be taken into account by adopting the methods of analysis given in 5.4.2.3.

(6) For beams with critical sections in Classes 1, 2 or 3 the following simplified method may be used. At every internal support where $\sigma_t$ exceeds $1.5 f_{\text{ctm}}$ or $1.5 f_{\text{lctm}}$ as appropriate, the bending moment determined by un-cracked analysis defined in 5.4.2.3(2) is multiplied by the reduction factor $f_1$ given in Figure 7.1, and corresponding increases are made to the bending moments in adjacent spans. Curve A may be used for internal spans only, when the loadings per unit length on all spans are equal and the lengths of all spans do not differ by more than 25%. Otherwise the approximate lower bound value $f_1 = 0.6$ (line B) should be used.

(7) For the calculation of deflection of un-propped beams, account may be taken of the influence of local yielding of structural steel over a support by multiplying the bending moment at the support, determined according to the methods given in this clause, with an additional reduction factor as follows:
   - $f_2 = 0.5$ if $f_y$ is reached before the concrete slab has hardened;
   - $f_2 = 0.7$ if $f_y$ is reached after concrete has hardened.

This applies for the determination of the maximum deflection but not for pre-camber.
(8) Unless specifically required by the client, the effect of curvature due to shrinkage of normal weight concrete need not be included when the ratio of span to overall depth of the beam is not greater than 20.

7.3.2 Vibration

(1) The dynamic properties of floor beams should satisfy the criteria in EN1990, A1.4.4.

7.4 Cracking of concrete

7.4.1 General

(1) For the limitation of crack width, the general considerations of EN 1992-1-1, 7.3.1(1) - (9) apply to composite structures. The limitation of crack width depends on the exposure classes according to EN 1992-1-1, 4.

(2) An estimation of crack width can be obtained from EN 1992-1-1, 7.3.4, where the stress $\sigma$ should be calculated by taking into account the effects of tension stiffening. Unless a more precise method is used, $\sigma$ may be determined according to 7.4.3(3).

(3) As a simplified and conservative alternative, crack width limitation to acceptable width can be achieved by ensuring a minimum reinforcement defined in 7.4.2, and bar spacing or diameters not exceeding the limits defined in 7.4.3.

(4) In cases where beams in buildings are designed as simply supported although the slab is continuous and the control of crack width is of no interest, the longitudinal reinforcement provided within the effective width of the concrete slab according to 6.1.2 should be not less than:

- 0.4 % of the area of the concrete, for propped construction;
- 0.2 % of the area of concrete, for un-propped construction.

The reinforcement in the beam designed as simply-supported should extend over a length of $0.25L$ each side of an internal support, or $0.5L$ adjacent to a cantilever, where $L$ is the length of the relevant span or the length of the cantilever respectively. No account should be taken of any profiled steel sheeting. The maximum spacing of the
bars should be in accordance with 9.2.1(5) for a composite slab, or with EN 1992-1-1, 9.3.1.1(3) for a solid concrete flange.

### 7.4.2 Minimum reinforcement

(1) Unless a more accurate method is used in accordance with EN 1992-1-1, 7.3.2(1), in all sections without pre-stressing by tendons and subjected to significant tension due to restraint of imposed deformations (e.g. primary and secondary effects of shrinkage), in combination or not with effects of direct loading the required minimum reinforcement area \( A_s \) for the slabs of composite beams is given by:

\[
A_s = k_s k_c f_{ct,\text{eff}} A_{ct} / \sigma_s
\]  

(7.1)

where:

- \( f_{ct,\text{eff}} \) is the mean value of the tensile strength of the concrete effective at the time when cracks may first be expected to occur. Values of \( f_{ct,\text{eff}} \) may be taken as those for \( f_{ctm} \), see EN 1992-1-1, Table 3.1, or as \( f_{ctm} \), see Table 11.3.1, as appropriate, taking as the class the strength at the time cracking is expected to occur. When the age of the concrete at cracking cannot be established with confidence as being less than 28 days, a minimum tensile strength of 3 N/mm\(^2\) may be adopted;

- \( k \) is a coefficient which allows for the effect of non-uniform self-equilibrating stresses which may be taken as 0.8;

- \( k_s \) is a coefficient which allows for the effect of the reduction of the normal force of the concrete slab due to initial cracking and local slip of the shear connection, which may be taken as 0.9;

- \( k_c \) is a coefficient which takes account of the stress distribution within the section immediately prior to cracking and is given by:

\[
k_c = \frac{1}{1 + h_c / (2 z_o)} + 0.3 \leq 1.0
\]  

(7.2)

- \( h_c \) is the thickness of the concrete flange, excluding any haunch or ribs;

- \( z_o \) is the vertical distance between the centroids of the un-cracked concrete flange and the un-cracked composite section, calculated using the modular ratio \( n_0 \) for short-term loading;

- \( \sigma_s \) is the maximum stress permitted in the reinforcement immediately after cracking. This may be taken as its characteristic yield strength \( f_{yk} \). A lower value, depending on the bar size, may however be needed to satisfy the required crack width limits. This value is given in Table 7.1;

- \( A_{ct} \) is the area of the tensile zone (caused by direct loading and primary effects of shrinkage) immediately prior to cracking of the cross section. For simplicity the area of the concrete section within the effective width may be used.

*Table 7.1 : Maximum bar diameters for high bond bars*

<table>
<thead>
<tr>
<th>Steel stress ( \sigma_s )</th>
<th>Maximum bar diameter ( \phi^* ) (mm) for design crack width ( w_k )</th>
</tr>
</thead>
</table>
(2) The maximum bar diameter for the minimum reinforcement may be modified to a value \( \phi \) given by:

\[
\phi = \phi^* \frac{f_{ct,eff}}{f_{ct,o}}
\]

(7.3)

where:

- \( \phi^* \) is the maximum bar size given in Table 7.1;
- \( f_{ct,o} \) is a reference strength of 2.9 N/mm\(^2\).

(3) At least half of the required minimum reinforcement should be placed between mid-depth of the slab and the face subjected to the greater tensile strain.

(4) For the determination of the minimum reinforcement in concrete flanges with variable depth transverse to the direction of the beam the local depth should be used.

(5) For buildings the minimum reinforcement according to (1) and (2) should be placed where, under the characteristic combination of actions, stresses are tensile.

(6) In buildings minimum lower longitudinal reinforcement for the concrete encasement of the web of a steel I-section should be determined from expression (7.1) with \( k_c \) taken as 0.6 and \( k \) taken as 0.8.

### 7.4.3 Control of cracking due to direct loading

(1) Where at least the minimum reinforcement given by 7.4.2 is provided, the limitation of crack widths to acceptable values may generally be achieved by limiting bar spacing or bar diameters. Maximum bar diameter and maximum bar spacing depend on the stress \( \sigma_s \) in the reinforcement and the design crack width. Maximum bar diameters are given in Table 7.1 and maximum bar spacing in Table 7.2.

(2) The internal forces should be determined by elastic analysis in accordance with Section 5 taking into account the effects of cracking of concrete. The stresses in the reinforcement should be determined taking into account effects of tension stiffening of concrete between cracks. Unless a more precise method is used, the stresses may be calculated according to (3).

### Table 7.2 Maximum bar spacing for high bond bars

<table>
<thead>
<tr>
<th>(N/mm(^2))</th>
<th>( w_k = 0.4)mm</th>
<th>( w_k = 0.3)mm</th>
<th>( w_k = 0.2)mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>160</td>
<td>40</td>
<td>32</td>
<td>25</td>
</tr>
<tr>
<td>200</td>
<td>32</td>
<td>25</td>
<td>16</td>
</tr>
<tr>
<td>240</td>
<td>20</td>
<td>16</td>
<td>12</td>
</tr>
<tr>
<td>280</td>
<td>16</td>
<td>12</td>
<td>8</td>
</tr>
<tr>
<td>320</td>
<td>12</td>
<td>10</td>
<td>6</td>
</tr>
<tr>
<td>360</td>
<td>10</td>
<td>8</td>
<td>5</td>
</tr>
<tr>
<td>400</td>
<td>8</td>
<td>6</td>
<td>4</td>
</tr>
<tr>
<td>450</td>
<td>6</td>
<td>5</td>
<td>-</td>
</tr>
</tbody>
</table>
Maximum bar spacing (mm) for design crack width $w_k$

<table>
<thead>
<tr>
<th>Steel stress $\sigma_s$ (N/mm$^2$)</th>
<th>$w_k=0.4\text{mm}$</th>
<th>$w_k=0.3\text{mm}$</th>
<th>$w_k=0.2\text{mm}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>160</td>
<td>300</td>
<td>300</td>
<td>200</td>
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<tr>
<td>200</td>
<td>300</td>
<td>250</td>
<td>150</td>
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<td>240</td>
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<td>100</td>
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<td>280</td>
<td>200</td>
<td>150</td>
<td>50</td>
</tr>
<tr>
<td>320</td>
<td>150</td>
<td>100</td>
<td>-</td>
</tr>
<tr>
<td>360</td>
<td>100</td>
<td>50</td>
<td>-</td>
</tr>
</tbody>
</table>

(3) In composite beams where the concrete slab is assumed to be cracked and not pre-stressed by tendons, stresses in reinforcement increase due to the effects of tension stiffening of concrete between cracks compared with the stresses based on a composite section neglecting concrete. The tensile stress in reinforcement $\sigma_s$ due to direct loading may be calculated from:

$$\sigma_s = \sigma_{s,o} + \Delta\sigma_s$$

(7.4)

with:

$$\Delta\sigma_s = \frac{0.4 \ f_{ctm}}{\alpha_s \ \rho_s}$$

(7.5)

$$\alpha_s = \frac{A I}{A_s I_s}$$

(7.6)

where:

- $\sigma_{s,o}$ is the stress in the reinforcement caused by the internal forces acting on the composite section, calculated neglecting concrete in tension;
- $f_{ctm}$ is the mean tensile strength of the concrete, for normal concrete taken as $f_{ctm}$ from EN 1992-1-1, Table 3.1 or for lightweight concrete as $f_{lcm}$ from Table 11.3.1;
- $\rho_s$ is the reinforcement ratio, given by $\rho_s = (A_s / A_{ct})$;
- $A_{ct}$ is the effective area of the concrete flange within the tensile zone; for simplicity the area of the concrete section within the effective width should be used;
- $A_s$ is the total area of all layers of longitudinal reinforcement within the effective area $A_{ct}$;
- $A, I$ are area and second moment of area, respectively, of the effective composite section neglecting concrete in tension and profiled sheeting, if any;
- $A_a, I_a$ are the corresponding properties of the structural steel section.

(4) For buildings without pre-stressing by tendons the quasi-permanent combination of actions normally should be used for the determination of $\sigma_s$.

Section 8  Composite joints in frames for buildings

8.1 Scope
(1) A composite joint is defined in 1.5.2.8. Some examples are shown in Figure 8.1. Other joints in composite frames should be designed in accordance with EN 1992-1-1 or EN 1993-1-8, as appropriate.

![Figure 8.1: Examples of composite joints](image)

(2) Section 8 concerns joints subject to predominantly static loading. It supplements or modifies EN 1993-1-8.

### 8.2 Analysis, modelling and classification

#### 8.2.1 General

(1) The provisions in EN 1993-1-8, 5 for joints connecting H or I sections are applicable with the modifications given in 8.2.2 and 8.2.3 below.

#### 8.2.2 Elastic global analysis

(1) Where the rotational stiffness $S_j$ is taken as $S_{j,ini} / \eta$ in accordance with EN 1993-1-8, 5.1.2, the value of the stiffness modification coefficient $\eta$ for a contact-plate connection should be taken as 1.5.

#### 8.2.3 Classification of joints

(1) Joints should be classified in accordance with EN 1993-1-8, 5.2, taking account of composite action.

(2) For the classification, the directions of the internal forces and moment should be considered.

(3) Cracking and creep in connected members may be neglected.
8.3 Design methods

8.3.1 Basis and scope

(1) EN 1993-1-8, 6 may be used as a basis for the design of composite beam-to-column joints and splices provided that the steelwork part of the joint is within the scope of that section.

(2) The structural properties of components assumed in design should be based on tests or on analytical or numerical methods supported by tests.

   Note: Properties of components are given in 8.4 and Annex A herein and in EN 1993-1-8, 6.

(3) In determining the structural properties of a composite joint, a row of reinforcing bars in tension may be treated in a manner similar to a bolt-row in tension in a steel joint, provided that the structural properties are those of the reinforcement.

8.3.2 Resistance

(1) Composite joints should be designed to resist vertical shear in accordance with relevant provisions of EN 1993-1-8.

(2) The design resistance moment of a composite joint with full shear connection should be determined by analogy to provisions for steel joints given in EN 1993-1-8, 6.2.7, taking account of the contribution of reinforcement.

(3) The resistance of components should be determined from 8.4 below and EN 1993-1-8, 6.2.6, where relevant.

8.3.3 Rotational stiffness

(1) The rotational stiffness of a joint should be determined by analogy to provisions for steel joints given in EN 1993-1-8, 6.3.1, taking account of the contribution of reinforcement.

(2) The value of the coefficient $\psi$, see EN 1993-1-8, 6.3.1(6), should be taken as 1.7 for a contact-plate joint.

8.3.4 Rotation capacity

(1) The influence of cracking of concrete, tension stiffening and deformation of the shear connection should be considered in determining the rotation capacity.

(2) The rotation capacity of a composite joint may be demonstrated by experimental evidence. Account should be taken of possible variations of the properties of materials from specified characteristic values. Experimental demonstration is not required when using details which experience has proved have adequate properties.

(3) Alternatively, calculation methods may be used, provided that they are supported by tests.
8.4 Resistance of components

8.4.1 Scope

(1) The resistance of the following basic joint components should be determined in accordance with 8.4.2 below:
   - longitudinal steel reinforcement in tension;
   - steel contact plate in compression.

(2) The resistance of components identified in EN 1993-1-8 should be taken as given therein, except as given in 8.4.3 below.

(3) The resistance of concrete encased webs in steel column sections should be determined in accordance with 8.4.4 below.

8.4.2 Basic joint components

8.4.2.1 Longitudinal steel reinforcement in tension

(1) The effective width of the concrete flange should be determined for the cross-section at the connection according to 5.4.1.2.

(2) It should be assumed that the effective area of longitudinal reinforcement in tension is stressed to its design yield strength $f_{yd}$.

(3) Where unbalanced loading occurs, a strut-tie model may be used to verify the introduction of the forces in the concrete slab into the column, see Figure 8.2.

(4) For a single-sided configuration designed as a composite joint, the effective longitudinal slab reinforcement in tension should be anchored sufficiently well beyond the span of the beam to enable the design tension resistance to be developed.

8.4.2.2 Steel contact plate in compression
(1) Where a height or breadth of the contact plate exceeds the corresponding dimension of the compression flange of the steel section, the effective dimension should be determined assuming dispersion at 45° through the contact plate.
(2) It should be assumed that the effective area of the contact plate in compression may be stressed to its design yield strength $f_{yd}$.

8.4.3 Column web in transverse compression

(1) For a contact plate connection, the effective width of the column web in compression $b_{eff,w,c}$ should be determined assuming dispersion at 45° through the contact plate.

8.4.4 Reinforced components

8.4.4.1 Column web panel in shear

(1) Where the steel column web is encased in concrete, see Figure 6.17b, the design shear resistance of the panel, determined in accordance with EN 1993-1-8, 6.2.6.1 may be increased to allow for the encasement.

(2) For a single-sided joint, or a double-sided joint in which the beam depths are similar, the design shear resistance of concrete encasement to the column web panel $V_{wp,c,Rd}$ should be determined using:

$$V_{wp,c,Rd} = 0,85 \nu A_c f_{cd} \sin \theta$$

(8.1)

with:

$$A_c = 0,8 (b_c - t_w) (h - 2t_f) \cos \theta$$

(8.2)

$$\theta = \arctan \left(\frac{(h - 2t_f)}{z}\right)$$

(8.3)

where:

- $b_c$ is the breadth of the concrete encasement;
- $h$ is the depth of the column section;
- $t_f$ is the column flange thickness;
- $t_w$ is the column web thickness;
- $z$ is the lever arm, see EN 1993-1-8, 6.2.7.1 and Figure 6.15.

(3) The reduction factor $\nu$ to allow for the effect of longitudinal compression in the column on the design resistance of the column web panel in shear should be determined using:

$$\nu = 0.55 \left[1 + 2 \left(\frac{N_{Ed}}{N_{pl,Rd}}\right)\right] \leq 1,1$$

(8.4)

where:

- $N_{Ed}$ is the design compressive normal force in the column;
$N_{p1,Rd}$ is the design plastic resistance of the column’s cross-section including the encasement, see 6.7.3.2.

### 8.4.4.2 Column web in transverse compression

(1) Where the steel column web is encased in concrete the design resistance of the column web in compression, determined in accordance with EN 1993-1-8, 6.2.6.2 may be increased to allow for the encasement.

(2) The design resistance of the concrete encasement to the column web in transverse compression $F_{c,wc,c,Rd}$ should be determined using:

$$F_{c,wc,c,Rd} = 0.85 k_{wc,c} t\text{eff,c} (b_c - t_w) f_{cd}$$

(8.5)

where:

- $t\text{eff,c}$ is the effective length of concrete, determined in a similar manner to the effective width $b_{\text{eff,wc}}$ defined in EN 1993-1-8, 6.2.6.2.

(3) Where the concrete encasement is subject to a longitudinal compressive stress, its effect on the resistance of the concrete encasement in transverse compression may be allowed for by multiplying the value of $F_{c,wc,c,Rd}$ by a factor $k_{wc,c}$ given by:

$$k_{wc,c} = 1.3 + 3.3 \frac{\sigma_{\text{com,c,Ed}}}{f_{cd}} \leq 2.0$$

(8.6)

where:

- $\sigma_{\text{com,c,Ed}}$ is the longitudinal compressive stress in the encasement due to the design normal force $N_{Ed}$.

In the absence of a more accurate method, $\sigma_{\text{com,c,Ed}}$ may be determined from the relative contribution of the concrete encasement to the plastic resistance of the column section in compression $N_{p1,Rd}$, see 6.7.3.2.

### Section 9  Composite slabs with profiled steel sheeting for buildings

#### 9.1 General

#### 9.1.1 Scope

(1) This Section deals with composite floor slabs spanning only in the direction of the ribs. Cantilever slabs are included. It applies to designs for building structures where the imposed loads are predominantly static, including industrial buildings where floors may be subject to moving loads.

(2) The scope is limited to sheets with narrowly spaced webs.

Note: Narrowly spaced webs are defined by an upper limit on the ratio $b_r / b_s$, see Figure 9.2. The value for the limit may be given in the National Annex. The recommended value is 0.6.
(3)P For structures where the imposed load is largely repetitive or applied abruptly in such a manner as to produce dynamic effects, composite slabs are permitted, but special care shall be taken over the detailed design to ensure that the composite action does not deteriorate in time.

(4)P Slabs subject to seismic loading are not excluded, provided an appropriate design method for the seismic conditions is defined for the particular project or is given in another Eurocode.

(5) Composite slabs may be used to provide lateral restraint to the steel beams and to act as a diaphragm to resist horizontal actions, but no specific rules are given in this Standard. For diaphragm action of the profiled steel sheeting while it is acting as formwork the rules given in EN1993-1-3, 10 apply.

9.1.2 Definitions

9.1.2.1 Types of shear connection

The profiled steel sheet shall be capable of transmitting horizontal shear at the interface between the sheet and the concrete; pure bond between steel sheeting and concrete is not considered effective for composite action. Composite behaviour between profiled sheeting and concrete shall be ensured by one or more of the following means, see Figure 9.1:

a) mechanical interlock provided by deformations in the profile (indentations or embossments);
b) frictional interlock for profiles shaped in a re-entrant form;
c) end anchorage provided by welded studs or another type of local connection between the concrete and the steel sheet, only in combination with (a) or (b);
d) end anchorage by deformation of the ribs at the end of the sheeting, only in combination with (b).

Other means are not excluded but are not within the scope of this Standard.

![Figure 9.1: Typical forms of interlock in composite slabs](image-url)
9.1.2.2 Full shear connection and partial shear connection

A span of a slab has full shear connection when increase in the resistance of the longitudinal shear connection would not increase the design bending resistance of the member. Otherwise, the shear connection is partial.

9.2 Detailing provisions

9.2.1 Slab thickness and reinforcement

1.1 Figure 9.2: Sheet and slab dimensions

(1)P The overall depth of the composite slab \( h \) shall be not less than 80 mm. The thickness of concrete \( h_c \) above the main flat surface of the top of the ribs of the sheeting shall be not less than 40 mm.

(2)P If the slab is acting compositely with the beam or is used as a diaphragm, the total depth shall be not less than 90 mm and \( h_c \) shall be not less than 50 mm.

(3)P Transverse and longitudinal reinforcement shall be provided within the depth \( h_c \) of the concrete.

(4) The amount of reinforcement in both directions should be not less than 80 mm²/m.

(5) The spacing of the reinforcement bars should not exceed 2\( h \) and 350 mm, whichever is the lesser.

9.2.2 Aggregate

(1)P The nominal size of the aggregate depends on the smallest dimension in the structural element within which concrete is poured, and shall not exceed the least of:

- 0,40 \( h_c \), see Figure 9.2;
- \( b_0/3 \), where \( b_0 \) is the mean width of the ribs (minimum width for re-entrant profiles), see Figure 9.2;
- 31,5 mm (sieve C 31,5).
9.2.3 Bearing requirements

(1)P The bearing length shall be such that damage to the slab and the bearing is avoided; that fastening of the sheet to the bearing can be achieved without damage to the bearing and that collapse cannot occur as a result of accidental displacement during erection.

(2) The bearing lengths $l_{bc}$ and $l_{bs}$ as indicated in Figure 9.3 should not be less than the following limiting values:
- for composite slabs bearing on steel or concrete: $l_{bc} = 75$ mm and $l_{bs} = 50$ mm;
- for composite slabs bearing on other materials: $l_{bc} = 100$ mm and $l_{bs} = 70$ mm.

Figure 9.3 : Minimum bearing lengths

9.3 Actions and action effects

9.3.1 Design situations

(1)P All relevant design situations and limit states shall be considered in design so as to ensure an adequate degree of safety and serviceability.

(2)P The following situations shall be considered:

a) Profiled steel sheeting as shuttering: Verification is required for the behaviour of the profiled steel sheeting while it is acting as formwork for the wet concrete. Account shall be taken of the effect of props, if any.

b) Composite slab: Verification is required for the floor slab after composite behaviour has commenced and any props have been removed.

9.3.2 Actions for profiled steel sheeting as shuttering

(1) The following loads should be taken into account in calculations for the steel deck as shuttering:
- weight of concrete and steel deck;
- construction loads including local heaping of concrete during construction, in accordance with EN 1991-1-6, 4.11.2;
– storage load, if any;
– “ponding” effect (increased depth of concrete due to deflection of the sheeting).

(2) If the central deflection $\delta$ of the sheeting under its own weight plus that of the wet concrete, calculated for serviceability, is less than 1/10 of the slab depth, the ponding effect may be ignored in the design of the steel sheeting. If this limit is exceeded, this effect should be allowed for. It may be assumed in design that the nominal thickness of the concrete is increased over the whole span by $0.7\delta$.

9.3.3 Actions for composite slab

(1) Loads and load arrangements should be in accordance with EN 1991-1-1.

(2) In design checks for the ultimate limit state, it may be assumed that the whole of the loading acts on the composite slab, provided this assumption is also made in design for longitudinal shear.

9.4 Analysis for internal forces and moments

9.4.1 Profiled steel sheeting as shuttering

(1) The design of the profiled steel sheeting as shuttering should be in accordance with EN1993-1-3.

(2) Plastic redistribution of moments should not be allowed when temporary supports are used.

9.4.2 Analysis of composite slab

(1) The following methods of analysis may be used for ultimate limit states:
   a) Linear elastic analysis with or without redistribution;
   b) Rigid plastic global analysis provided that it is shown that sections where plastic rotations are required have sufficient rotation capacity;
   c) Elastic-plastic analysis, taking into account the non-linear material properties.

(2) Linear methods of analysis should be used for serviceability limit states.

(3) If the effects of cracking of concrete are neglected in the analysis for ultimate limit states, the bending moments at internal supports may optionally be reduced by up to 30%, and corresponding increases made to the sagging bending moments in the adjacent spans.

(4) Plastic analysis without any direct check on rotation capacity may be used for the ultimate limit state if reinforcing steel of class C in accordance with EN 1992-1-1, Annex C is used and the span is not greater than 3,0 m.

(5) A continuous slab may be designed as a series of simply supported spans. Nominal reinforcement in accordance with 9.8.1 should be provided over intermediate supports.
9.4.3 Effective width of composite slab for concentrated point and line loads

(1) Where concentrated point or line loads are to be supported by the slab, they may be considered to be distributed over an effective width, unless a more exact analysis is carried out.

(2) Concentrated point or line loads parallel to the span of the slab should be considered to be distributed over a width $b_m$, measured immediately above the ribs of the sheeting, see Figure 9.4, and given by:

$$b_m = b_p + 2(h_c + h_f)$$  \hspace{1cm} (9.1)

![Figure 9.4: Distribution of concentrated load](image)

(3) For concentrated line loads perpendicular to the span of the slab, expression (9.1) should be used for $b_m$, with $b_p$ taken as the length of the concentrated line load.

(4) If $h_p / h$ does not exceed 0.6 the width of the slab considered to be effective for global analysis and for resistance may for simplification be determined with expressions (9.2) to (9.4):

(a) for bending and longitudinal shear:

- for simple spans and exterior spans of continuous slabs

$$b_{em} = b_m + 2L_p \left( 1 - \frac{L_p}{L} \right) \leq \text{slab width}$$  \hspace{1cm} (9.2)

- for interior spans of continuous slabs

$$b_{em} = b_m + 1.33L_p \left( 1 - \frac{L_p}{L} \right) \leq \text{slab width}$$  \hspace{1cm} (9.3)

(b) for vertical shear:

$$b_{ev} = b_m + L_p \left( 1 - \frac{L_p}{L} \right) \leq \text{slab width}$$  \hspace{1cm} (9.4)

where:

$L_p$ is the distance from the centre of the load to the nearest support;
is the span length.

(5) If the characteristic imposed loads do not exceed the following values, a nominal transverse reinforcement may be used without calculation:

- concentrated load: 7.5 kN;
- distributed load: 5.0 kN/m².

This nominal transverse reinforcement should have a cross-sectional area of not less than 0.2% of the area of structural concrete above the ribs, and should extend over a width of not less than $b_{ew}$ as calculated in this clause. Minimum anchorage lengths should be provided beyond this width in accordance with EN 1992-1-1. Reinforcement provided for other purposes may fulfil all or part of this rule.

(6) Where the conditions in (5) are not satisfied, the distribution of bending moments caused by line or point loads should be determined and adequate transverse reinforcement determined using EN 1992-1-1.

9.5 Verification of profiled steel sheeting as shuttering for ultimate limit states

(1) Verification of the profiled steel sheeting for ultimate limit states should be in accordance with EN 1993-1-3. Due consideration should be given to the effect of embossments or indentations on the design resistances.

9.6 Verification of profiled steel sheeting as shuttering for serviceability limit states

(1) Section properties should be determined in accordance with EN 1993-1-3.

(2) The deflection $\delta_s$ of the sheeting under its own weight plus the weight of wet concrete, excluding the construction load, should not exceed $\delta_{s,\text{max}}$.

Note: Values for $\delta_{s,\text{max}}$ may be given in the National Annex. The recommended value is $L/180$ where $L$ is the effective span between supports (props being supports in this context).

9.7 Verification of composite slabs for the ultimate limit states

9.7.1 Design criterium

(1) The design values of internal forces shall not exceed the design values of resistance for the relevant ultimate limit states.

9.7.2 Flexure

(1) In case of full shear connection the bending resistance $M_{Rd}$ of any cross section should be determined by plastic theory in accordance with 6.2.1.2(1) but with the design yield strength of the steel member (sheeting) taken as that for the sheeting, $f_{y,\text{pd}}$.

(2) In hogging bending the contribution of the steel sheeting shall only be taken into account where the sheet is continuous and when for the construction phase
redistribution of moments by plastification of cross-sections over supports has not been used.

(3) For the effective area $A_{pe}$ of the steel sheeting, the width of embossments and indentations in the sheet should be neglected, unless it is shown by tests that a larger area is effective.

(4) The effect of local buckling of compressed parts of the sheeting should be taken into account by using effective widths not exceeding twice the limiting values given in EN 1993-1-1, Table 5.2 for Class 1 steel webs.

(5) The sagging bending resistance of a cross-section with the neutral axis above the sheeting should be calculated from the stress distribution in Figure 9.5.

![Figure 9.5: Stress distribution for sagging bending if the neutral axis is above the steel sheeting](image)

(6) The sagging bending resistance of a cross-section with the neutral axis in the sheeting should be calculated from the stress distribution in Figure 9.6.

For simplification $z$ and $M_{pr}$ may be determined with the following expressions respectively:

$$z = h - 0,5 h_c - e_p + (e_p - e) \frac{N_{cf}}{A_{pe} f_{yp,d}} \quad (9.5)$$

$$M_{pr} = 1,25 M_{pa} \left( 1 - \frac{N_{cf}}{A_{pe} f_{yp,d}} \right) \leq M_{pa} \quad (9.6)$$
Figure 9.6: Stress distribution for sagging bending if neutral axis is in the steel sheeting

(7) If the contribution of the steel sheeting is neglected the hogging bending resistance of a cross-section should be calculated from the stress distribution in Figure 9.7.

Figure 9.7: Stress distribution for hogging bending

9.7.3 Longitudinal shear for slabs without end anchorage

(1) The provisions in this clause 9.7.3 apply to composite slabs with mechanical or frictional interlock (types (a) and (b) as defined in 9.1.2.1).

(2) The design resistance against longitudinal shear should be determined by the \( m-k \) method, see (4) and (5) below, or by the partial connection method as given in (7) – (10). The partial connection method should be used only for composite slabs with a ductile longitudinal shear behaviour.

(3) The longitudinal shear behaviour may be considered as ductile if the failure load exceeds the load causing a recorded end slip of 0.1 mm by more than 10%. If the maximum load is reached at a midspan deflection exceeding \( L/50 \), the failure load should be taken as the load at the midspan deflection of \( L/50 \).

(4) If the \( m-k \) method is used it should be shown that the maximum design vertical shear \( V_{Ed} \) for a width of slab \( b \) does not exceed the design shear resistance \( V_{l,Rd} \) determined from the following expression:

\[
V_{l,Rd} = \frac{b d p}{\gamma_{V_s}} \left( \frac{m A_p}{b L_s} + k \right) \tag{9.7}
\]

where:
\( b, d_p \) are in mm;
\( A_p \) is the nominal cross-section of the sheeting in \( \text{mm}^2 \);
\( m, k \) are design values for the empirical factors in \( \text{N/mm}^2 \) obtained from slab tests meeting the basic requirements of the \( m-k \) method;
\( L_s \) is the shear span in mm and defined in (5) below;
\( \gamma_{Vs} \) is the partial safety factor for the ultimate limit state.

Note 1: The value for \( \gamma_{Vs} \) may be given in the National Annex. The recommended value for \( \gamma_{Vs} \) is 1,25.

Note 2: The test method as given in Annex B may be assumed to meet the basic requirements of the \( m-k \) method.

Note 3: In expression (9.7) the nominal cross-section \( A_p \) is used because this value is normally used in the test evaluation to determine \( m \) and \( k \).

(5) For design, \( L_s \) should be taken as:

- \( L/4 \) for a uniform load applied to the entire span length;
- the distance between the applied load and the nearest support for two equal and symmetrically placed loads;
- for other loading arrangements, including a combination of distributed and asymmetrical point loads, an assessment should be made based upon test results or by the following approximate calculation. The shear span should be taken as the maximum moment divided by the greater vertical shear force adjacent to the supports for the span considered.

(6) Where the composite slab is designed as continuous, it is permitted to use an equivalent isostatic span for the determination of the resistance. The span length should be taken as:

- 0,8\( L \) for internal spans;
- 0,9\( L \) for external spans.

(7) If the partial connection method is used it should be shown that at any cross-section the design bending moment \( M_{Ed} \) does not exceed the design resistance \( M_{Rd} \).

(8) The design resistance \( M_{Rd} \) should be determined as given in 9.7.2(6) but with \( N_{cf} \) replaced by:

\[
N_c = \frac{\tau_{u,Rd} b L_x}{A_{pe} f_{yp,d}} \leq N_{cf}
\]

(9.8)

and:

\[
z = h - 0,5 x_{pl} - e_p + \left( e_p - e \right) \frac{N_c}{A_{pe} f_{yp,d}}
\]

(9.9)

where:

\( \tau_{u,Rd} \) is the design shear strength \((\tau_{u,Rd}/\gamma_{Vs})\) obtained from slab tests meeting the basic requirements of the partial interaction method;

\( L_x \) is the distance of the cross-section being considered to the nearest support.
Note 1: The value for $\gamma_{Vs}$ may be given in the National Annex. The recommended value for $\gamma_{Vs}$ is 1,25.

Note 2: The test method as given in Annex B may be assumed to meet the basic requirements for the determination of $\tau_{u,Rd}$

(9) In expression (9.8) $N_c$ may be increased by $\mu R_{Ed}$ provided that $\tau_{u,Rd}$ is determined taking into account the additional longitudinal shear resistance caused by the support reaction, where:

$R_{Ed}$ is the support reaction,

$\mu$ is a nominal factor.

Note: The value for $\mu$ may be given in the National Annex. The recommended value for $\mu$ is 0,5.

(10) In the partial connection method additional bottom reinforcement may be taken into account.

9.7.4 Longitudinal shear for slabs with end anchorage

(1) Unless a contribution to longitudinal shear resistance by other shear devices is shown by testing, the end anchorage of type (c), as defined in 9.1.2.1, should be designed for the tensile force in the steel sheet at the ultimate limit state.

(2) The design resistance against longitudinal shear of slabs with end anchorage of types (c) and (d), as defined in 9.1.2.1, may be determined by the partial connection method as given in 9.7.3(7) with $N_c$ increased by the design resistance of the end anchorage.

(3) The design resistance $P_{pb,Rd}$ of a headed stud welded through the steel sheet used for end anchorage should be taken as the smaller of the design shear resistance of the stud in accordance with 6.6.3.1 or the bearing resistance of the sheet determined with the following expression:

$$P_{pb,Rd} = k_{\phi} d_{do} t f_{yp,d}$$  \hspace{1cm} (9.10)

with:

$$k_{\phi} = 1 + \frac{a}{d_{do}} \leq 6,0$$  \hspace{1cm} (9.11)

where:

$d_{do}$ is the diameter of the weld collar which may be taken as 1,1 times the diameter of the shank of the stud;

$a$ is the distance from the centre of the stud to the end of the sheeting, to be not less than 1,5 $d_{do}$;

$t$ is the thickness of the sheeting.

9.7.5 Vertical shear

(1) The vertical shear resistance $V_{v,Rd}$ of a composite slab over a width equal to the distance between centres of ribs, should be determined in accordance with EN 1992-1-1, 6.2.2.

9.7.6 Punching shear
(1) The punching shear resistance $V_{p,Rd}$ of a composite slab at a concentrated load should be determined in accordance with EN 1992-1-1, 6.4.4, where the critical perimeter should be determined as shown in Figure 9.8.

**Figure 9.8 : Critical perimeter for punching shear**

### 9.8 Verification of composite slabs for serviceability limit states

#### 9.8.1 Control of cracking of concrete

(1) The crack width in hogging moment regions of continuous slabs should be checked in accordance with EN 1992-1-1, 7.3.

(2) Where continuous slabs are designed as simply-supported in accordance with 9.4.2(5), the cross-sectional area of the anti-crack reinforcement above the ribs should be not less than 0,2% of the cross-sectional area of the concrete above the ribs for un-propped construction and 0,4% of this cross-sectional area for propped construction.

#### 9.8.2 Deflection

(1) EN 1990, 3.4.3, applies.

(2) Deflections due to loading applied to the steel sheeting alone should be calculated in accordance with EN 1993-1-3, Section 7.

(3) Deflections due to loading applied to the composite member should be calculated using elastic analysis in accordance with Section 5, neglecting the effects of shrinkage.

(4) Calculations of deflections may be omitted if both:
the span to depth ratio does not exceed the limits given in EN 1992-1-1, 7.4, for lightly stressed concrete, and
the condition of (6) below, for neglect of the effects of end slip, is satisfied.

(5) For an internal span of a continuous slab where the shear connection is as defined in 9.1.2.1(a), (b) or (c), the deflection may be determined using the following approximations:
- the second moment of area may be taken as the average of the values for the cracked and un-cracked section;
- for concrete, an average value of the modular ratio for both long- and short-term effects may be used.

(6) For external spans, no account need be taken of end slip if the initial slip load in tests (defined as the load causing an end slip of 0.5 mm) exceeds 1.2 times the design service load.

(7) Where end slip exceeding 0.5 mm occurs at a load below 1.2 times the design service load, then end anchors should be provided. Alternatively deflections should be calculated including the effect of end slip.

(8) If the influence of the shear connection between the sheeting and the concrete is not known from experimental verification for a composite floor with end anchorage, the design should be simplified to an arch with a tensile bar. From that model, the lengthening and shortening gives the deflection that should be taken into account.

Annex A (Informative) Stiffness of joint components in buildings

A.1 Scope

(1) The stiffness of the following basic joint components may be determined in accordance with A.2.1 below:
- longitudinal steel reinforcement in tension;
- steel contact plate in compression.

(2) Stiffness coefficients $k_i$ are defined by EN 1993-1-8, expression (6.27). The stiffness of components identified in that Standard may be taken as given therein, except as given in A.2.2 below.

(3) The stiffness of concrete encased webs in steel column sections may be determined in accordance with A.2.3 below.

(4) The influence of slip of the shear connection on joint stiffness may be determined in accordance with A.3.

A.2 Stiffness coefficients

A.2.1 Basic joint components
A.2.1.1 Longitudinal steel reinforcement in tension

(1) The stiffness coefficient $k_{s,r}$ for a row $r$ may be obtained from Table A.1.

A.2.1.2 Steel contact plate in compression

(1) The stiffness coefficient may be taken as equal to infinity.

A.2.2 Other components in composite joints

A.2.2.1 Column web panel in shear

(1) For an unstiffened panel in a joint with a steel contact plate connection, the stiffness coefficient $k_1$ may be taken as 0.87 times the value given in EN 1993-1-8, Table 6.11.

A.2.2.2 Column web in transverse compression

(1) For an un-stiffened web and a contact plate connection, the stiffness coefficient $k_2$ may be determined from:

$$k_2 = \frac{0.2 b_{\text{eff},wc} t_{wc}}{d_c} \quad \text{(A.1)}$$

where:

$b_{\text{eff},wc}$ is the effective width of the column web in compression, see 8.4.3.1.

Other terms are defined in EN 1993-1-8, 6.

Table A.1 : Stiffness coefficient $k_{s,r}$

<table>
<thead>
<tr>
<th>Configuration</th>
<th>Loading</th>
<th>Stiffness coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single-sided</td>
<td></td>
<td>$k_{s,r} = \frac{A_{s,r}}{3.6 h}$</td>
</tr>
<tr>
<td></td>
<td>$M_{\text{Ed},1} = M_{\text{Ed},2}$</td>
<td>$k_{s,r} = \frac{A_{s,r}}{(h/2)}$</td>
</tr>
</tbody>
</table>
|                | $M_{\text{Ed},1} > M_{\text{Ed},2}$ | For the joint with $M_{\text{Ed},1}$:  
|                |                           | $k_{s,r} = \frac{A_{s,r}}{h \left( \frac{1+\beta}{2} + K_\beta \right)}$  
|                |                           | with:  
|                |                           | $K_\beta = \beta \left( 4.3 \beta^2 - 8.9 \beta + 7.2 \right)$  
|                |                           | For the joint with $M_{\text{Ed},2}$:  
|                |                           | $k_{s,r} = \frac{A_{s,r}}{h \left( \frac{1-\beta}{2} \right)}$  |
$A_{sr}$ is the cross-sectional area of the longitudinal reinforcement in row $r$ within the effective width of the concrete flange determined for the cross-section at the connection according to 5.4.1.2;

$M_{Ed,i}$ is the design bending moment applied to a connection $i$ by a connected beam, see Figure A.1;

$h$ is the depth of the column's steel section, see Figure 6.17;

$\beta$ is the transformation parameter given in EN 1993-1-8, 5.3.

Note: The stiffness coefficient for $M_{Ed,1} = M_{Ed,2}$ is applicable to a double-sided beam-to-beam joint configuration under the same loading condition, provided that the breadth of the flange of the supporting primary beam replaces the depth $h$ of the column section.

Figure A.1: Joints with bending moments

A.2.3 Reinforced components

A.2.3.1 Column web panel in shear

(1) Where the steel column web is encased in concrete, see Figure 6.17b, the stiffness of the panel may be increased to allow for the encasement. The addition $k_{1,c}$ to the stiffness coefficient $k_1$ may be determined from:

$$k_{1,c} = 0,06 \frac{E_{cm}}{E_a} \frac{b_c h_c}{\beta z}$$  \hspace{1cm} (A.2)

where:

$E_{cm}$ is the modulus of elasticity for concrete;

$z$ is the lever arm, see EN 1993-1-8, Figure 6.15.

A.2.3.2 Column web in transverse compression

(1) Where the steel column web is encased in concrete, see Figure 6.17b, the stiffness of the column web in compression may be increased to allow for the encasement.
(2) For a contact plate connection, the addition $k_{2,c}$ to the stiffness coefficient $k_2$ may be determined from:

$$k_{2,c} = 0.13 \frac{E_{cm}}{E_a} \frac{t_{eff,c} b_c}{h_c}$$  \hspace{1cm} \text{(A.3)}$$

where:

$t_{eff,c}$ is the effective thickness of concrete, see 8.4.4.2(2).

(3) For an end plate connection, the addition $k_{2,c}$ may be determined from:

$$k_{2,c} = 0.5 \frac{E_{cm}}{E_a} \frac{t_{eff,c} b_c}{h_c}$$  \hspace{1cm} \text{(A.4)}$$

### A.3 Deformation of the shear connection

(1) Unless account is taken of deformation of the shear connection by a more exact method, the influence of slip on the stiffness of the joint may be determined by (2) - (5) below.

(2) The stiffness coefficient $k_{s,r}$, see A.2.1.1, may be multiplied by the reduction factor, $k_{slip}$:

$$k_{slip} = \frac{1}{1 + \frac{E_s k_{s,r}}{K_{sc}}}$$  \hspace{1cm} \text{(A.5)}$$

with:

$$K_{sc} = \frac{N k_{sc}}{v - \left(\frac{v - 1}{1 + \xi}\right) \frac{h_s}{d_s}}$$  \hspace{1cm} \text{(A.6)}$$

$$v = \sqrt{\frac{(1 + \xi) N k_{sc} \ell d_s^2}{E_a I_a}}$$  \hspace{1cm} \text{(A.7)}$$

$$\xi = \frac{E_s I_a}{d_s^2 E_s A_s}$$  \hspace{1cm} \text{(A.8)}$$

where:

$h_s$ is the distance between the longitudinal reinforcing bars in tension and the centre of compression; see EN 1993-1-8, Figure 6.15 for the centre of compression;

$d_s$ is the distance between the longitudinal reinforcing bars in tension and the centroid of the beam's steel section;

$I_a$ is the second moment of area of the beam's steel section;

$\ell$ is the length of the beam in hogging bending adjacent to the joint, which in a braced frame may be taken as 15% of the length of the span;

$N$ is the number of shear connectors distributed over the length $\ell$;
\( k_{sc} \) is the stiffness of one shear connector.

(3) The stiffness of the shear connector, \( k_{sc} \), may be taken as \( 0.7 P_{Rk} / s \), where:

\( P_{Rk} \) is the characteristic resistance of the shear connector;

\( s \) is the slip, determined from push tests in accordance with Annex B, at a load of \( 0.7 P_{Rk} \).

(4) Alternatively, for a solid slab or for a composite slab in which the reduction factor \( k_t \) is unity, see 6.6.4.2, the following approximate values may be assumed for \( k_{sc} \):

- for 19mm diameter headed studs:
  100kN/mm
- for cold-formed angles of 80mm to 100mm height:
  70kN/mm.

(5) For a composite joint with more than a single layer of reinforcement considered effective in tension, (2) above is applicable provided that the layers are represented by a single layer of equivalent cross-sectional area and equivalent distances from the centre of compression and the centroid of the beam’s steel section.

**Annex B (Informative) Standard tests**

**B.1 General**

(1) In this Standard rules are given for:

a) tests on shear connectors in B.2 and

b) testing of composite floor slabs in B.3.

Note: These standard testing procedures are included in the absence of Guidelines for ETA. When such Guidelines have been developed this Annex can be withdrawn.

**B.2 Tests on shear connectors**

**B.2.1 General**

(1) Where the design rules in 6.6 are not applicable, the design should be based on tests, carried out in a way that provides information on the properties of the shear connection required for design in accordance with this Standard.

(2) The variables to be investigated include the geometry and the mechanical properties of the concrete slab, the shear connectors and the reinforcement.

(3) The resistance to loading, other than fatigue, may be determined by push tests in accordance with the requirements in this Annex.
(4) For fatigue tests the specimen should also be prepared in accordance with this Annex.

B.2.2 Testing arrangements

(1) Where the shear connectors are used in T-beams with a concrete slab of uniform thickness, or with haunches complying with 6.6.5.4, standard push tests may be used. In other cases specific push tests should be used.

(2) For standard push tests the dimensions of the test specimen, the steel section and the reinforcement should be as given in Figure B.1. The recess in the concrete slabs is optional.

(3) Specific push tests should be carried out such that the slabs and the reinforcement are suitably dimensioned in comparison with the beams for which the test is designed. In particular:
   a) the length $l$ of each slab should be related to the longitudinal spacing of the connectors in the composite structure;
   b) the width $b$ of each slab should not exceed the effective width of the slab of the beam;
   c) the thickness $h$ of each slab should not exceed the minimum thickness of the slab in the beam;
   d) where a haunch in the beam does not comply with 6.6.5.4, the slabs of the push specimen should have the same haunch and reinforcement as the beam.

![Figure B.1: Test specimen for standard push test](image)

B.2.3 Preparation of specimens
(1) Each of both concrete slabs should be cast in the horizontal position, as is done for composite beams in practice.

(2) Bond at the interface between flanges of the steel beam and the concrete should be prevented by greasing the flange or by other suitable means.

(3) The push specimens should be air-cured.

(4) For each mix a minimum of four concrete specimens (cylinders or cubes) for the determination of the cylinder strength should be prepared at the time of casting the push specimens. These concrete specimens should be cured alongside the push specimens. The concrete strength $f_{cm}$ should be taken as the mean value.

(5) The compressive strength $f_{cm}$ of the concrete at the time of testing should be $70\% \pm 10\%$ of the specified strength of the concrete $f_{ck}$ of the beams for which the test is designed. This requirement can be met by using concrete of the specified grade, but testing earlier than 28 days after casting of the specimens.

(6) The yield strength, the tensile strength and the maximum elongation of a representative sample of the shear connector material should be determined.

(7) If profiled steel sheeting is used for the slabs, the tensile strength and the yield strength of the profiled steel sheet should be obtained from coupon tests on specimens cut from the sheets as used in the push tests.

**B.2.4 Testing procedure**

(1) The load should first be applied in increments up to 40\% of the expected failure load and then cycled 25 times between 5\% and 40\% of the expected failure load.

(2) Subsequent load increments should then be imposed such that failure does not occur in less than 15 minutes.

(3) The longitudinal slip between each concrete slab and the steel section should be measured continuously during loading or at each load increment. The slip should be measured at least until the load has dropped to 20\% below the maximum load.

(4) As close as possible to each group of connectors, the transverse separation between the steel section and each slab should be measured.

**B.2.5 Test evaluation**

(1) If three tests on nominally identical specimens are carried out and the deviation of any individual test result from the mean value obtained from all tests does not exceed 10\%, the design resistance may be determined as follows:

- the characteristic resistance $P_{Rk}$ should be taken as the minimum failure load (divided by the number of connectors) reduced by 10\%;
- the design resistance $P_{Rd}$ should be calculated from:
where:

\( f_u \) is the minimum specified ultimate strength of the connector material;

\( f_{ut} \) is the actual ultimate strength of the connector material in the test specimen;

and

\( \gamma_V \) is the partial safety factor for shear connection.

Note: The value for \( \gamma_V \) may be given in the National Annex. The recommended value for \( \gamma_V \) is 1.25.

(2) If the deviation from the mean exceeds 10%, at least three more tests of the same kind should be made. The test evaluation should then be carried out in accordance with EN 1990, Annex D.

(3) Where the connector is composed of two separate elements, one to resist longitudinal shear and the other to resist forces tending to separate the slab from the steel beam, the ties which resist separation shall be sufficiently stiff and strong so that separation in push tests, measured when the connectors are subjected to 80% of their ultimate load, is less than half of the longitudinal movement of the slab relative to the beam.

(4) The slip capacity of a specimen \( \delta_u \) should be taken as the maximum slip measured at the characteristic load level, as shown in Figure B.2. The characteristic slip capacity \( \delta_{uk} \) should be taken as the minimum test value of \( \delta_u \) reduced by 10% or determined by statistical evaluation from all the test results. In the latter case, the characteristic slip capacity should be determined in accordance with EN 1990, Annex D.

![Figure B.2: Determination of slip capacity \( \delta_u \)](image)

**B.3 Testing of composite floor slabs**

**B.3.1 General**
(1) Tests according to this section should be used for the determination of the factors $m$ and $k$ or the value of $\tau_{u,Rd}$ to be used for the verification of the resistance to longitudinal shear as given in Section 9.

(2) From the load-deflection curves the longitudinal shear behaviour is to be classified as brittle or ductile. The behaviour is deemed to be ductile if it is in accordance with 9.7.3(3). Otherwise the behaviour is classified as brittle.

(3) The variables to be investigated include the thickness and the type of steel sheeting, the steel grade, the coating of the steel sheet, the density and grade of concrete, the slab thickness and the shear span length $L_s$.

(4) To reduce the number of tests as required for a complete investigation, the results obtained from a test series may be used also for other values of variables as follows:
   - for thickness of the steel sheeting $t$ larger than tested;
   - for concrete with specified strength $f_{ck}$ not less than $0.8 f_{cm}$, where $f_{cm}$ is the mean value of the concrete strength in the tests;
   - for steel sheeting having a yield strength $f_{yp}$ not less than $0.8 f_{ypm}$, where $f_{ypm}$ is the mean value of the yield strength in the tests.

**B.3.2 Testing arrangement**

(1) Tests should be carried out on simply supported slabs.

(2) The test set-up should be as shown in Figure B.3 or equivalent.

(3) Two equal concentrated line loads, placed symmetrically at $L/4$ and $3L/4$ on the span, should be applied to the specimen.

![Figure B.3 : Test set-up](image)

(4) The distance between the centre line of the supports and the end of the slab should not exceed 100 mm.
(5) The width of the bearing plates and the line loads should not exceed 100 mm.

(6) When the tests are used to determine \( m \) and \( k \) factors, for each variable to be investigated two groups of three tests (indicated in Figure B.4 by regions A and B) or three groups of two tests should be performed. For specimens in region A, the shear span should be as long as possible while still providing failure in longitudinal shear and for specimens in region B as short as possible while still providing failure in longitudinal shear, but not less than \( 3h_t \) in length.

(7) When the tests are used to determine \( \tau_{u,Rd} \) for each type of steel sheet or coating not less than four tests should be carried out on specimens of same thickness \( h_t \) without additional reinforcement or end anchorage. In a group of three tests the shear span should be as long as possible while still providing failure in longitudinal shear and in the remaining one test as short as possible while still providing failure in longitudinal shear, but not less than \( 3h_t \) in length. The one test with short shear span is only used for classifying the behaviour in accordance with B.3.1(2).

**B.3.3 Preparation of specimens**

(1) The surface of the profiled steel sheet shall be in the 'as-rolled' condition, no attempt being made to improve the bond by degreasing the surface.

(2) The shape and embossment of the profiled sheet should accurately represent the sheets to be used in practice. The measured spacing and depth of the embossments shall not deviate from the nominal values by more than 5% and 10% respectively.

(3) In the tension zone of the slabs crack inducers should be placed across the full width of the test slab under the applied loads. The crack inducers should extend at least to the depth of the sheeting. Crack inducers are placed to better define the shear span length, \( L_s \) and to eliminate the tensile strength of concrete.

(4) It is permitted to restrain exterior webs of the deck so that they act as they would act in wider slabs.

(5) The width \( b \) of test slabs should not be less than three times the overall depth, 600mm and the cover width of the profiled sheet.

(6) Specimens should be cast in the fully supported condition. This is the most unfavourable situation for the shear bond mode of failure.

(7) Mesh reinforcement may be placed in the slab, for example to reinforce the slab during transportation, against shrinkage, etc. If placed it must be located such that it acts in compression under sagging moment.

(8) The concrete for all specimens in a series to investigate one variable should be of the same mix and cured under the same conditions.
(9) For each group of slabs that will be tested within 48 hours, a minimum of four concrete specimens, for the determination of the cylinder or cube strength, should be prepared at the time of casting the test slabs. The concrete strength $f_{cm}$ of each group should be taken as the mean value, when the deviation of each specimen from the mean value does not exceed 10%. When the deviation of the compressive strength from the mean value exceeds 10%, the concrete strength should be taken as the maximum observed value.

(10) The tensile strength and yield strength of the profiled steel sheet should be obtained from coupon tests on specimens cut from each of the sheets used to form the test slabs.

B.3.4 Test loading procedure

(1) The test loading procedure is intended to represent loading applied over a period of time. It is in two parts consisting of an initial test, where the slab is subjected to cyclic loading; this is followed by a subsequent test, where the slab is loaded to failure under an increasing load.

(2) If two groups of three tests are used, one of the three test specimen in each group may be subjected to just the static test without cyclic loading in order to determine the level of the cyclic load for the other two.

(3) Initial test: the slab should be subjected to an imposed cyclic load, which varies between a lower value not greater than 0,2$W_t$ and an upper value not less than 0,6$W_t$, where $W_t$ is the measured failure load of the preliminary static test according (2).

(4) The loading should be applied for 5000 cycles in a time not less than 3 hours.

(5) Subsequent test: on completion of the initial test, the slab should be subjected to a static test where the imposed load is increased progressively, such that failure does not occur in less than 1 hour. The failure load $W_t$ is the maximum load imposed on the slab at failure plus the weight of the composite slab and spreader beams.

(6) In the subsequent test the load may be applied either as force-controlled or deflection-controlled.

B.3.5 Determination of design values for $m$ and $k$
(1) If the behaviour is ductile, see 9.7.3(3), the representative experimental shear force $V_t$ should be taken as 0.5 times the value of the failure load $W_t$ as defined in B.3.4. If the behaviour is brittle this value shall be reduced, using a factor 0.8.

(2) From all the test values of $V_t$ the characteristic shear strength should be calculated as the 5% fractile by using an appropriate statistical model and drawn as a characteristic linear regression line, as shown in Figure B.4.

(3) If two groups of three tests are used and the deviation of any individual test result in a group from the mean of the group does not exceed 10%, the design relationship may be determined in accordance with Annex D of EN 1990 or as follows:

From each group the characteristic value is deemed to be the one obtained by taking the minimum value of the group reduced by 10%. The design relationship is formed by the straight line through these characteristic values for groups A and B.

**B.3.6 Determination of the design values for $\tau_{nd}$**

(1) The partial interaction diagram as shown in Figure B.5 should be determined using the measured dimensions and strengths of the concrete and the steel sheet. For the concrete strength the mean value $f_{cm}$ of a group as specified in B.3.3(9) may be used.
Figure B.5 : Determination of the degree of shear connection from $M_{\text{test}}$

(2) From the maximum applied loads, the bending moment $M$ at the cross-section under the point load due to the applied load, dead weight of the slab and spreader beams should be determined. The path A --> B --> C in Figure B.5 then gives a value $\eta$ for each test, and a value $\tau_u$ from:

$$\tau_u = \frac{\eta N_{\text{ct}}}{b (L_s + L_o)}$$  \hspace{1cm} (B.2)

where:

$L_o$ is the length of the overhang.

(3) If in design the additional longitudinal shear resistance caused by the support reaction is taken into account in accordance with 9.7.3(9), $\tau_u$ should be determined from:

$$\tau_u = \frac{\eta N_{\text{ct}} - \mu V_t}{b (L_s + L_o)}$$  \hspace{1cm} (B.3)

where:

$\mu$ is the default value of the friction coefficient to be taken as 0.5;

$V_t$ is the support reaction under the ultimate test load.

(4) The characteristic shear strength $\tau_{u,Rk}$ should be calculated from the test values as the 5% fractile using an appropriate statistical model in accordance with EN 1990, Annex D.

(5) The design shear strength $\tau_{u,Rd}$ is the characteristic strength $\tau_{u,Rk}$ divided by the partial safety coefficient $\gamma_{Vs}$.

Note: The value for $\gamma_{Vs}$ may be given in the National Annex. The recommended value for $\gamma_{Vs}$ is 1.25.
Annex C (Informative)
Shrinkage of concrete for composite structures for buildings

(1) Unless accurate control of the profile during execution is essential, or where shrinkage is expected to take exceptional values, the nominal value of the total final free shrinkage strain may be taken as follows in calculations for the effects of shrinkage:

- in dry environments (whether outside or within buildings but excluding concrete-filled members):
  - $325 \times 10^{-6}$ for normal concrete
  - $500 \times 10^{-6}$ for lightweight concrete;

- in other environments and in filled members:
  - $200 \times 10^{-6}$ for normal concrete
  - $300 \times 10^{-6}$ for lightweight concrete.
Bibliography

EN 1991-1-5: Actions on structures: Thermal actions-to be published.

EN 1991-1-6: Actions on structures: Actions during execution-to be published.

EN 13670: Requirements for the execution of concrete structures-to be published.
