D1 Two-span composite slab unpropped at the construction stage

1. Purpose of example

This example demonstrates the design of a continuous composite slab over two spans. The composite slab consists of a cold-formed profiled steel sheeting covered with a concrete slab containing reinforcement. The composite slab is supported by steel beams, which act compositely with the concrete slab.

In composite slab design, we need to consider the construction stage and the composite stage. At the construction stage, the profiled steel sheeting acts as shuttering. The profiled sheeting has to carry its own weight, the wet concrete and the construction loads. In the composite stage, the slab is loaded with its own weight, the floor finishes and the variable load.

The composite slab is almost always continuous, because the profiled sheeting is provided in two-span lengths and the concrete is cast on the sheeting without joints. However, very often it is assumed that it is simply supported. According to clause 9.4.2(5), EN 1994-1-1, the continuous slab may be designed as a series of simply supported spans. In such cases, according to clause 9.8.1, EN 1994-1-1, the reinforcement for crack control is provided above internal supports.

The design resistance of the composite slab against longitudinal shear is carried out by the semi-empirical method called the $m$-$k$ method. The method is based on two empirical factors, $m$ and $k$. The design values of empirical factors $m$ and $k$ are based on slab tests and are provided by the manufacturer of the sheeting.

The partial connection method is an alternative to $m$-$k$ method. This method also relies on tests on the composite slab to estimate the shear connection. Both of these methods can be applied in cases where the longitudinal shear behaviour is ductile. However, if the longitudinal behaviour is non-ductile, only the $m$-$k$ method is permitted. According to clause B.3.5(1), EN 1994-1-1, in such cases the $m$-$k$ method can be used but with an additional partial factor of 1.25, expressed by the reduction factor 0.8.
2. **Static system, cross-section and actions**

![Figure D1.1 Static system](image)

The profiled steel sheeting is continuous over two spans. For simplicity, it is assumed as simply supported span. This assumption is adopted only for the construction stage. The static system, adopted for verifications of profiled sheeting for ultimate limit state and serviceability limit state, is shown in Figure D1.2.

![Figure D1.2 Static system for the construction stage](image)

The cross-section of the composite slab and the cross-section of selected profiled sheeting with dimensions are shown in Figure D1.3.
**Figure D1.3** Cross-sections: a) composite slab, b) profiled steel sheeting

Actions

a) Permanent action

**Remark:**

According to EN 1991-1-1 the density of the normal weight concrete is 24 kN/m³, increased by 1 kN/m³ for normal percentage reinforcement, and increased for the wet concrete by another 1 kN/m³.

Concrete slab area per m width:

\[ A_c = 1000 \cdot h - \left( \frac{1000}{b_s} \cdot \frac{b_i + b_r \cdot h_p}{2} \right) \]

\[ A_c = 1000 \cdot 130 - \left( \frac{1000}{152,5} \cdot \frac{15 + 40 \cdot 51}{2} \right) = 120800 \text{ mm}^2 = 1208 \text{ cm}^2 \]

- **Concrete slab and reinforcement (wet concrete):**
  \[ A_c \cdot 26 = 0,1208 \cdot 26 = 3,14 \text{ kN/m}^2 \]

- **Concrete slab and reinforcement (dry concrete):**
  \[ A_c \cdot 25 = 0,1208 \cdot 25 = 3,02 \text{ kN/m}^2 \]
**Construction stage**

- **concrete slab**
  
  \[ g_{k,1} = 3,30 \text{ kN/m}^2 \]

- **profiled steel sheeting**
  
  \[ g_{k,2} = 0,16 \text{ kN/m}^2 \]

**Total**

\[ g_{k,1} + g_{k,2} = 3,46 \text{ kN/m}^2 \]

**Composite stage**

- **concrete slab**
  
  \[ g_{k,3} = 3,02 \text{ kN/m}^2 \]

- **profiled steel sheeting**
  
  \[ g_{k,4} = 0,16 \text{ kN/m}^2 \]

**Total**

\[ g_{k,3} + g_{k,4} = 3,18 \text{ kN/m}^2 \]

Floor finishes

\[ g_{k,5} = 1,20 \text{ kN/m}^2 \]

b) Variable action

**Construction stage**

- **construction loads**
  
  \[ q_{k,1} = 1,50 \text{ kN/m}^2 \]

**Composite stage**

- **imposed floor load**
  
  \[ q_{k,2} = 7,0 \text{ kN/m}^2 \]

3. **Properties of materials**

Concrete strength class: C 25/30

\[ f_{ck} = 25 \text{ N/mm}^2 \]

\[ f_{cd} = \frac{f_{ck}}{\gamma_c} = \frac{25}{1,5} = 16,7 \text{ N/mm}^2 \]

\[ 0,85f_{cd} = 0,85 \cdot 16,7 = 14,17 \text{ N/mm}^2 \]

\[ E_{cm} = 31000 \text{ N/mm}^2 \]

Reinforcement:

\[ f_{sk} = 500 \text{ N/mm}^2 \]

\[ f_{sd} = \frac{f_{sk}}{\gamma_s} = \frac{500}{1,15} = 435 \text{ N/mm}^2 \]
Profiled steel sheeting:

\[ t = 1,1 \text{ mm} \]
\[ h_p = 51 \text{ mm} \]
\[ A_p = A_{pe} = 1938 \text{ mm}^2/m \]
\[ I_p = 68,5 \cdot 10^4 \text{ mm}^4/m \]
\[ E_p = E_a = 210000 \text{ N/mm}^2 \]
\[ f_{yp,k} = 350 \text{ N/mm}^2 \]
\[ f_{yp,d} = \frac{f_{yp,k}}{Y_M} = \frac{350}{1,0} = 350 \text{ N/mm}^2 \]

Resistance moment (provided by manufacturer):
\[ M_{Rd} = 7,0 \text{ kNm/m (sagging)} \]
\[ M_{Rd} = 8,88 \text{ kNm/m (hogging)} \]

Empirical factors (provided by manufacturer):
\[ m = 128,5 \text{ N/mm}^2 \]
\[ k = 0 \text{ N/mm}^2 \]

4. Structural details of composite slab

4.1 Slab thickness and reinforcement

The composite slab should satisfy the conditions given in clause 9.2, EN 1994-1-1.

a) The slab acts compositely with a beam, and the following conditions should be satisfied:

- the overall depth of slab \( h \geq 90 \text{ mm}, \rightarrow h = 130 \text{ mm} \) (satisfied),

- the thickness of concrete above the main flat surface of the top of the ribs of sheeting \( h_c \geq 50 \text{ mm}, \rightarrow h_c = 79 \text{ mm} \) (satisfied),

- the ratio of the width of the sheet rib to the rib spacing \( \frac{b_r}{b_s} \leq 0,6, \rightarrow \frac{b_r}{b_s} = \frac{40}{152,5} = 0,26 \) (satisfied).

b) The minimum amount of reinforcement in both directions should not be less than 80 mm²/m. For the unpropped construction, the area of reinforcement, according to clause 9.8.1(2), EN 1994-1-1, is:

\[ A_{s,\min} = 0,002 \cdot h_c \cdot b = 0,002 \cdot 79 \cdot 1000 = 158 \text{ mm}^2/m \rightarrow A_s = 80 \text{ mm}^2/m \]
The reinforcement bars are assumed to be 6 Φ6/1000 mm. The cross-sectional area of reinforcement is:

\[ A_s = 6 \cdot \frac{6^2 \cdot \pi}{4} = 170 \text{ mm}^2/\text{m} \]

c) Spacing of reinforcement bars

\[ e < 2 \cdot h = 2 \cdot 130 = 260 \text{ or } < 350 \text{ mm} \]

4.2 Largest nominal aggregate size

\[ d_g \leq 0,4 \cdot h_c \quad 0,4 \cdot 79 = 31,6 \text{ mm} \]
\[ d_g \leq b_g/3 \quad 112,5/3=37,5 \text{ mm} \]
\[ d_g \leq 31,5 \text{ mm} \quad = 31,5 \text{ mm} \]

The minimum adopted value is \( d_g = 31,5 \text{ mm} \).

4.3 Minimum value for nominal thickness of steel sheet

In accordance with clause 3.5(2), EN 1994-1-1, the recommended value for the nominal thickness of steel sheet is 0,70 mm. The thickness of the selected profiled steel sheeting is 1,10 mm. The condition is satisfied.

4.4 Composite slab bearing requirements

According to clause 9.2.3(2), EN 1994-1-1, the recommended bearing lengths and support details differ depending upon the support material and they are different for internal supports and end supports, see Figure D1.4.

<table>
<thead>
<tr>
<th>Bearing on</th>
<th>( l_{bs} (\text{mm}) )</th>
<th>( l_{bc} (\text{mm}) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>steel or concrete</td>
<td>50</td>
<td>75</td>
</tr>
<tr>
<td>other materials</td>
<td>70</td>
<td>100</td>
</tr>
</tbody>
</table>

**Figure D1.4 Minimum bearing lengths**
For composite slabs bearing on steel or concrete, the minimum bearing lengths are: 
\( l_{bc} = 75 \text{ mm} \) and \( l_{bs} = 50 \text{ mm} \). The composite slab is supported by steel beams with the section IPE 500, and the top-flange width is 200 mm. Therefore, the condition for the bearing length is satisfied.

5. **Ultimate limit state**

5.1 **Construction stage**

At the construction stage, it is necessary to carry out verifications of profiled steel sheeting for the ultimate and serviceability limit states in accordance with EN 1993-1-3.

Profiled steel sheetings are normally continuous over two or more spans. However, in some cases, a single span is unavoidable due to the floor geometry.

Usually the manufacturer gives information on the properties of profiled steel sheeting. These properties are usually based on test results performed in accordance with EN 1993-1-3, Annex A. Characteristic and design values of resistance moment, crushing resistance, second moment of area etc. may be estimated using methods of reliability analysis in accordance with EN 1990. Properties of profiled steel sheeting estimated by calculation are more conservative than equivalent properties based on testing.

For simplicity, the profiled steel sheeting is considered as a simply supported span. The static system and the design load for the construction stage are shown in Figure D1.5.

\[ e_d = Y_G \cdot g_{k,1} + Y_Q \cdot q_{k,1} \]

\[ e_d = 1,35 \cdot 3,30 + 1,5 \cdot 1,5 = 6,71 \text{ kN/m}^2 \]

Therefore, the design values of bending moment and shear force are:
\[ M_{Ed} = \frac{e_d \cdot L^2}{8} = \frac{6.71 \cdot 2.5^2}{8} = 5.24 \text{ kNm/m} \]

\[ V_{Ed} = \frac{e_d \cdot L}{2} = \frac{6.71 \cdot 2.5}{2} = 8.39 \text{ kN/m} \]

Check for bending:

\[ \frac{M_{Ed}}{M_{Rd}} \leq 1.0 \]

\[ \frac{5.24}{7.0} = 0.75 < 1.0, \text{ the condition is satisfied} \]

In accordance with EN 1993-1-3, the following checks should be carried out:

- shear resistance of the cross-section according to clause 6.1.5, EN 1993-1-3,
- local resistance according to clause 6.1.7.3, EN 1993-1-3,
- combined bending and shear according to clause 6.1.10, EN 1993-1-3,
- combined web crushing and bending moment according to clause 6.1.11, EN 1993-1-3.

5.2 Composite stage

The continuous composite slab is designed as a series of simply supported spans, in accordance with clause 9.4.2(5), EN 1994-1-1, provided that the criterion for minimum reinforcement above internal supports of composite slab is satisfied, clause 9.8.1, EN 1994-1-1.

The static system and the design load for the composite stage are shown in Figure D1.6.

![Static system and design load for the composite stage](image)

**Figure D1.6 Static system and design load for the composite stage**

Design load for ultimate limit state:

\[ e_d = b \cdot (y_G \cdot (g_{k,2} + g_{k,3}) + y_Q \cdot q_{k,2}) \]
Therefore, the design values of bending moment and shear force are:

\[ M_{Ed} = \frac{e_d \cdot L^2}{8} = \frac{16,4 \cdot 2,5^2}{8} = 12,8 \text{ kNm/m} \]

\[ V_{Ed} = \frac{e_d \cdot L}{2} = \frac{16,4 \cdot 2,5}{2} = 20,5 \text{ kN/m} \]

5.2.1 **Plastic resistance moment in sagging region**

It is assumed that the neutral axis lies above the sheeting. The assumed distribution of longitudinal bending stresses is shown in Figure D1.7. The design compressive force in concrete, \( N_{c,f} \), is:

\[ N_{c,f} = 0,85 \cdot f_{cd} \cdot h_c \cdot b, \quad b = 1000 \text{ mm} \]

\[ N_{c,f} = 0,85 \cdot 16,7 \cdot 79 \cdot 1000 \cdot 10^{-3} = 1121 \text{ kN/m} \]

The design tensile force in the steel sheeting for a width of sheeting \( b \) is calculated with the characteristic of the effective steel section \( A_{pe} \):

\[ N_p = f_{yp,d} \cdot A_{pe} \]

\[ N_p = 350 \cdot 1938 \cdot 10^{-3} = 678 \text{ kN/m} \]

Since \( N_p < N_{c,f} \), the plastic neutral axis lies within the concrete. The design resistance moment in sagging region is calculated according to the distribution of stresses shown in Figure D1.7.

![Figure D1.7 Cross-section of composite slab and stress blocks for sagging bending](image)

The position of the plastic neutral axis of the composite section \( x_{pl} \) is:
\[ x_{pl} = \frac{A_{pe} \cdot f_{sp,d}}{0,85 \cdot f_{cd} \cdot b}, \quad b = 1000 \text{ mm slab width} \]

\[ x_{pl} = \frac{1938 \cdot 350}{0,85 \cdot 16,7 \cdot 1000} = 47,8 \text{ mm} < h_c = 79 \text{ mm} \]

For full shear connection, the design plastic resistance moment in sagging region \( M_{pl,Rd} \) is calculated as:

\[ M_{pl,Rd} = \min(N_{c,f} , N_p) \cdot z \]

\[ M_{pl,Rd} = N_p \cdot (d_p - \frac{x_{pl}}{2}) \]

\[ M_{pl,Rd} = 678 \cdot (113,3 - \frac{47,8}{2}) \cdot 10^{-3} = 60,6 \text{ kNm/m} \]

Check:

\[ \frac{M_{Ed}}{M_{pl,Rd}} \leq 1,0 \]

\[ \frac{12,8}{60,6} = 0,21 < 1,0 , \text{ the condition is satisfied} \]

The design plastic resistance moment in sagging region for full shear connection is adequate.

5.2.2  Longitudinal shear resistance

It is assumed that there is no end anchorage. Therefore, the longitudinal shear resistance is calculated according to clause 9.7.3, EN 1994-1-1. The design resistance of the composite slab against longitudinal shear is carried out by the semi-empirical method called the \( m-k \) method. According to clause 9.7.3(4), EN 1994-1-1, the maximum design vertical shear \( V_{Ed} \) for a width of slab \( b \) is limited due to the design longitudinal shear resistance \( V_{l,Rds} \) given as:

\[ V_{l,Rd} = \frac{b \cdot d_p}{\gamma_{vs}} \cdot \left( \frac{m \cdot A_p}{b \cdot L_s} + k \right) \]
where:

- $b, d_p$ are in mm,
- $A_p$ is the nominal cross-section of the sheeting in mm$^2$,
- $m, k$ are design values for the empirical factors in N/mm$^2$ obtained from slab tests meeting the basic requirements of the $m$-$k$ method,
- $L_s$ is the shear span in mm, defined in clause 9.7.3(5), EN 1994-1-1,
- $\gamma_{vs}$ is the partial factor for ultimate limit state; the recommended value is 1,25.

If the $m$-$k$ method is used, it should be verified that the maximum design vertical shear $V_{Ed}$ does not exceed the design shear resistance $V_{l,Rd}$:

$$\frac{V_{Ed}}{V_{l,Rd}} \leq 1.0$$

Design values of empirical factors $m$ and $k$ are based on slab tests and are provided by the manufacturer of the sheeting:

- $m = 128.5$ N/mm$^2$
- $k = 0$ N/mm$^2$

According to clause 9.7.3(5), EN 1994-1-1, the shear span $L_s$ for the uniform load applied to the entire span length is:

$$L_s = \frac{L}{4} = \frac{2500}{4} = 625 \text{ mm}$$

The design longitudinal shear resistance $V_{l,Rd}$ is:

$$V_{l,Rd} = \frac{b \cdot d_p}{\gamma_{vs}} \cdot \left( \frac{m \cdot A_p}{b \cdot L_s} + k \right)$$

$$V_{l,Rd} = \left[ \frac{1000 \cdot 113.3}{1.25} \cdot \left( \frac{128.5 \cdot 1938}{1000 \cdot 625} + 0 \right) \right] \cdot 10^{-3} = 36.1 \text{ kN/m}$$

This value, $V_{l,Rd} = 36.1$ kN/m, must not be exceeded by the vertical shear in the slab.

Check:
\[ \frac{V_{Ed}}{V_{v,Rd}} \leq 1,0 \]

\[ \frac{20,5}{36,1} = 0,57 < 1,0, \text{ the condition is satisfied} \]

### 5.2.3 Check for vertical shear resistance

According to 9.7.5, EN 1994-1-1, the vertical shear resistance, \( V_{v,Rd} \), should be determined according to the method given in EN 1992-1-1. According to clause 6.2.2., EN 1992-1-1, the design shear resistance \( V_{v,Rd} \) is calculated as:

\[
V_{v,Rd} = V_{Rd,c} = [C_{Rd,c} \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d_p \geq V_{v,Rd,\text{min}}
\]

The minimum value of \( V_{v,Rd,\text{min}} \) is:

\[
V_{v,Rd,\text{min}} = (V_{\text{min}} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d_p
\]

The minimum requirement for \( V_{v,Rd} \) is related to the fact that the member without reinforcement still has some shear resistance.

Generally, the check is carried out as follows:

\[ \frac{V_{Ed}}{V_{v,Rd}} \leq 1,0 \]

According to clause 6.2.2(1), EN 1992-1-1, the values needed for calculation \( V_{v,Rd} \) are:

\[
\frac{C_{Rd,c}}{\gamma_c} = \frac{0,18}{1,5} = 0,12
\]

\[
k = 1 + \frac{200}{d_p} \leq 2,0
\]

\[
k = 1 + \frac{200}{113,3} = 2,32 \quad \rightarrow \text{ adopted } k = 2,0
\]
The resistance of the cross-section is dependent on the area of the tensile reinforcement, whose section has to be extended by an appropriate anchorage length, \((l_{bd} + d)\) see – Figure 6.3, EN 1992-1-1 – where \(l_{bd}\) is the design anchorage length and \(d\) is the effective depth of the section, taken as the depth from the top surface to the centroid of the profile for a composite slab. The anchorage of the profiled sheeting was confirmed by the check on longitudinal shear, and the sheeting can be treated as reinforcement, i.e. \(A_{sl} = A_{pe} = 1938\, \text{mm}^2\).

In accordance with Figure D1.8, the smallest width of the cross-section in the tensile area \(b_w\) is calculated per metre width as follows:

\[
b_w = b \cdot b_0 = \frac{b_s \cdot b_0}{152.5} = 738\, \text{mm/m}
\]

\[
\rho_l = \frac{A_d}{b_w \cdot d_p} \leq 0.02
\]

\[
\rho_l = \frac{1938}{738 \cdot 113.3} = 0.023 > 0.020
\]

The value \(\rho_l = 0.02\) is adopted.
The design axial force is $N_{Ed} = 0$ and therefore $\sigma_{cp} = \frac{N_{Ed}}{A_c} = 0$.

$k_1 = 0.15$, according to clause 6.2.2(1), EN 1992-1-1

The design shear resistance $V_{v,Rd}$ is:

$$V_{v,Rd} = V_{Rd,c} = [C_{Rd,c} \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d_p$$

$$V_{v,Rd} = [0.12 \cdot 2.0 \cdot (100 \cdot 0.02 \cdot 25)^{1/3} + 0.15 \cdot 0] \cdot 738 \cdot 113.3 \cdot 10^{-3}$$

$$V_{v,Rd} = 73.9 \text{ kN/m}$$

The minimum value is:

$$V_{v,Rd,\text{min}} = (v_{\text{min}} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d_p$$

$$v_{\text{min}} = 0.035 \cdot k^{3/2} \cdot f_{ck}^{1/2} = 0.035 \cdot 2.0^{3/2} \cdot 25^{1/2} = 0.49 \text{ N/mm}^2$$

$$V_{v,Rd,\text{min}} = (0.49 + 0.15 \cdot 0) \cdot 738 \cdot 113.3 \cdot 10^{-3} = 41 \text{ kN/m} < V_{v,Rd} = 73.9 \text{ kN/m}$$

Check:

$$\frac{V_{Ed}}{V_{v,Rd}} \leq 1.0$$

$$\frac{20.5}{73.9} = 0.28 < 1.0$$, the condition is satisfied

**Remark:**

Since it is unlikely that the profiled steel sheet can satisfy the requirement “full anchorage”, the design shear resistance is equal to the minimum value:

$$V_{v,Rd} = V_{v,Rd,\text{min}} = 41 \text{ kN/m}$$

Also, the required condition is satisfied since that is

$$V_{v,Rd} = V_{v,Rd,\text{min}} = 41 \text{ kN/m} > V_{Ed} = 20.5 \text{ kN/m}.$$
6. Serviceability limit state

6.1 Control of cracking of concrete

Since the slab is designed as simply supported, it only requires reinforcement for crack width limitation.

According to clause 9.8.1(2), EN 1994-1-1, for unpropped construction the required cross-sectional area of reinforcement $A_s$ is 0,2% of the area of concrete above the ribs:

$$A_s = \frac{0.2}{100} \cdot 1000 \cdot (h - h_p) \text{ mm}^2/\text{m}$$

$$A_s = \frac{0.2}{100} \cdot 1000 \cdot (130 - 51) = 158 \text{ mm}^2/\text{m}$$

The reinforcement bars are assumed to be 6 φ6/1000 mm. Therefore, the cross-sectional area of reinforcement is:

$$A_s = 6 \cdot \frac{6^2 \cdot \pi}{4} = 170 \text{ mm}^2/\text{m} > 158 \text{ mm}^2/\text{m}$$

The selected minimum amount of reinforcement may be insufficient to control cracking at the supports of continuous slabs for certain exposure classes. In such cases, the slab should be designed as continuous, and in hogging regions the crack widths should be estimated according to EN 1992-1-1.

6.2 Limit of span/depth ratio of slab

According to clause 9.8.2(4), EN 1994-1-1, calculation of the deflection of the composite slab can be omitted if the two conditions are satisfied. According to the first condition, the span/depth ratio of the slab should not exceed the limits given in EN 1992-1-1. These are:

- $\frac{L}{d} < 20$ for a simply supported span
- $\frac{L}{d} < 26$ for an external span of continuous slab
- $\frac{L}{d} < 30$ for an internal span of continuous slab

According to clause 9.8.2(6), EN 1994-1-1, the second condition is given as:
the load causing an end slip of 0.5 mm in the tests on composite slab exceeds 1.2 times the design service load.

If the second condition is not satisfied, i.e. the end slip exceeds 0.5 mm at a load 1.2 times the design service load, two options exist:

- end anchors should be provided, or
- deflections should be calculated including the effect of end slip.

According to clause 9.8.2(8), EN 1994-1-1, in cases where the behaviour of the shear connection between the profiled sheeting and the concrete are not known from tests, the tied-arch model may be used, see [34].

For the considered slab, with $L = 2500$ mm and $d_p = 113.3$ mm, the following span/depth ratio is obtained:

$$\frac{L}{d} = \frac{2500}{113.3} = 22 < 26$$ for the external span of continuous slab

Therefore it is not necessary to carry out the calculation of the deflection. However, the calculation is carried out for educational reasons.

6.3 Calculation of deflections

6.3.1 Construction stage deflection

According to clause 9.6(2), EN 1994-1-1, the deflection, $\delta_s$, of the profiled sheeting due to its own weight and the weight of wet concrete should not exceed the following limit:

$$\delta_{s,\text{max}} = \frac{L}{180} = \frac{2500}{180} = 14 \text{ mm}$$

For simplicity, the profiled steel sheeting is considered as a simply supported span. The static system and the design load for the construction stage are shown in Figure D1.9.

Figure D1.9 Static system and design load for the construction stage
The premature local buckling of the profiled sheeting under the weight of wet concrete and construction loading is checked to prevent irreversible deformation. This verification is important in regions of internal support.

Design load for the serviceability limit state is:

\[ e_d = b \cdot g_{k,1} \]

\[ e_d = 1,0 \cdot 3,30 = 3,30 \text{ kN/m} \]

Maximum sagging bending moment in the serviceability limit state is:

\[ M_{Ed} = \frac{e_d \cdot L^2}{8} = \frac{3,30 \cdot 2,5^2}{8} = 2,58 \text{ kNm/m} \]

Maximum compressive stress in the top flange of the profiled sheeting is:

\[ \sigma_{com} = \frac{M_{Ed} \cdot z}{I_p} = \frac{2,58 \cdot 10^6}{68,5 \cdot 10^4} \cdot (51 - 16,7) = 129 \text{ N/mm}^2 \]

In accordance with clause 4.4, EN 1993-1-5, the plate slenderness, \( \overline{\lambda_p} \), is calculated as:

\[ \overline{\lambda_p} = \frac{b}{\sqrt{\sigma_{cr}}} = \frac{b}{\frac{t}{28,4 \cdot \varepsilon \cdot \sqrt{k_a}}} \]

\[ \varepsilon = \sqrt{\frac{235}{\sigma_{com}}} = \sqrt{\frac{235}{129}} = 1,35 \]

According to Table 4.1, EN 1994-1-1, for the stress ratio \( \psi = 1 \), the buckling factor is \( k_a = 4 \).

Therefore, the plate slenderness, \( \overline{\lambda_p} \), with the design thickness of the sheet \( t = 1,06 \) mm (not including coatings) and \( b = b_r = 40 \) mm, is:

\[ \overline{\lambda_p} = \frac{40}{28,4 \cdot 1,35 \cdot \sqrt{4}} = 0,492 \]
Since that is $\overline{\lambda}_p = 0,492 < 0,673$, the reduction factor is $\rho = 1,0$ and the cross-section is fully effective.

The deflection of the profiled steel sheeting for the simply supported span, Figure D1.9, is:

$$\delta_1 = \frac{5}{384} \cdot \frac{e_d \cdot L^4}{E_a \cdot I_p}$$

$$\delta_1 = \frac{5 \cdot 3,30 \cdot 2500^4}{384 \cdot 210000 \cdot 68,5 \cdot 10^4} = 11,7 \text{ mm}$$

$$\delta_1 = 11,7 \text{ mm } < \delta_{\text{max}} = \frac{L}{180} = \frac{2500}{180} = 14 \text{ mm}$$

The deflection due to the self-weight of profiled sheeting and the weight of the wet concrete meets the criterion $L/180$.

Since the deflection $\delta_1$ is less than 10% of the slab depth, $\delta_1 = 11,7 \text{ mm } < 0,10 \cdot h = 0,1 \cdot 130 = 13 \text{ mm}$, according to clause 9.3.2(2), EN 1994-1-1, the ponding effects can be neglected at the construction stage.

The conditions for the serviceability limit state are satisfied, and the profiled steel sheeting can be used at the construction stage.

6.3.2 Composite stage deflection

For the calculation of the deflection at the composite stage, the slab is considered as continuous over two spans. According to clause 9.8.2(5), EN 1994-1-1, the following approximations can be applied:

- The second moment of area can be taken as the average of the values for the cracked and uncracked section.
- An average value of the modular ratio, $n$, for both short-term and long-term effects can be used:

$$n = \frac{E_a}{E_{cm}} = \frac{E_a}{\frac{1}{2} \cdot (E_{cm} + \frac{E_{cm}}{3})} = \frac{210000}{\frac{2}{3} \cdot 31000} = 10,2$$

- Elastic analysis is used to calculate the deflection of the slab.
a) The second moment of area for the cracked section, \( I_{cc} \), for slab width \( b \) is calculated in accordance with Figure D1.10.

The second moment of area for the cracked section and the slab width \( b \) is calculated as:

\[
I_{cc} = \frac{b \cdot x_c^3}{3 \cdot n} + A_p \cdot (d_p - x_c)^2 + I_p
\]

The position of the elastic neutral axis relative to the upper side of the slab is obtained as:

\[
\chi_c = \frac{\Sigma A_i \cdot z_i}{\Sigma A_i} = \frac{n \cdot A_p}{b} \left( \sqrt{1 + \frac{2 \cdot b \cdot d_p}{n \cdot A_p}} - 1 \right)
\]

\[
\chi_c = \frac{10,2 \cdot 1938}{1000} \cdot \left( \sqrt{1 + \frac{2 \cdot 1000 \cdot 113,3}{10,2 \cdot 1938}} - 1 \right) = 50,0 \text{ mm}
\]

The second moment of area for the cracked section is:

\[
I_{cc} = \frac{1000 \cdot 50,0^3}{3 \cdot 10,2} + 1938 \cdot (113,3 - 50,0)^2 + 685000 = 12,54 \cdot 10^6 \text{ mm}^4/m
\]

![Diagram of ENA - elastic neutral axis](image)

**Figure D1.10 Second moment of area calculation for cracked cross-section, \( I_{cc} \)**

b) The second moment of area for the uncracked section, \( I_{cu} \), for slab width \( b \) is calculated in accordance with Figure D1.11.
Figure D1.11 Second moment of area calculation for uncracked cross-section, \(I_{cu}\)

The second moment of area for the uncracked section and the slab width \(b\) is calculated as:

\[
I_{cu} = \frac{b \cdot h_c^3}{12 \cdot n} + \frac{b \cdot h_c}{n} \left( x_u - \frac{h_c}{2} \right)^2 + \frac{b_m \cdot h_p^3}{12 \cdot n} + \frac{b_m \cdot h_p}{n} \left( h_t - x_u - \frac{h_p}{2} \right)^2 + A_p \cdot (d_p - x_u)^2 + I_p
\]

where:

\[
x_u = \frac{\frac{b \cdot h_c^2}{2} + b_m \cdot h_p \left( h_t - \frac{h_p}{2} \right) + n \cdot A_p \cdot d_p}{b \cdot h_c + b_m \cdot h_p + n \cdot A_p}
\]

In accordance with Figure D1.11, the value of \(b_m\) is:

\[
b_m' = \frac{(b_b - 2 \cdot t) + b_0}{2} = \frac{(137,5 - 2 \cdot 1,10) + 112,5}{2} = 123,9 \text{ mm}
\]

\[
b_m = \frac{b}{b_m' \cdot b_m} = \frac{1000}{152,5} = 812 \text{ mm/m}
\]

The position of the elastic neutral axis relative to the upper side of the slab is:

\[
x_u = \frac{1000 \cdot \frac{79^2}{2} + 812 \cdot 51 \cdot (130 - \frac{51}{2}) + 10,2 \cdot 1938 \cdot 113,3}{1000 \cdot 79 + 812 \cdot 51 + 10,2 \cdot 1938} = 69,1 \text{ mm}
\]

The second moment of area for the uncracked section is:
\[ I_{\text{cu}} = \frac{1000 \cdot 79^3}{12 \cdot 10,2} + \frac{1000 \cdot 79}{10,2} \cdot (69,1 - \frac{79}{2})^2 + \frac{812 \cdot 51^3}{12 \cdot 10,2} + \frac{812 \cdot 51}{10,2} \cdot (130 - 69,1 - \frac{51}{2})^2 + 1938 \cdot (113,3 - 69,1)^2 + 685000 \]

\[ I_{\text{cu}} = 21,25 \cdot 10^6 \text{ mm}^4/\text{m} \]

The mean value of \( I_{\text{cc}} \) and \( I_{\text{cu}} \) is:

\[ I_c = \frac{I_{\text{cc}} + I_{\text{cu}}}{2} \]

\[ I_c = \frac{12,54 \cdot 10^6 + 21,25 \cdot 10^6}{2} = 16,90 \cdot 10^6 \text{ mm}^4/\text{m} \]

**Calculation of deflections**

- **Deflection due to permanent action**

The design load of the weight of dry concrete, the weight of the profiled sheeting and the floor finishes is:

\[ e_d = b \cdot (g_{k,2} + g_{k,3}) = 1,0 \cdot (3,18 + 1,20) = 4,38 \text{ kN/m} \]

![Figure D1.12 Static system and load for calculation of deflection at the composite stage](image)

The deflection is:

\[ \delta_1 = 0,0054 \cdot \frac{e_d \cdot L^4}{E \cdot I_c} \]

\[ \delta_1 = 0,0054 \cdot \frac{4,38 \cdot 2500^4}{210000 \cdot 16,90 \cdot 10^6} = 0,26 \text{ mm} = L/9615 \]
Deflection due to frequent value of variable action and the selected combination factor is $\psi_1 = 0.7$

The design load is calculated for the frequent combination:

$$ e_d = b \cdot \psi_1 \cdot q_{k,2} = 1.0 \cdot 0.7 \cdot 7.0 = 4.9 \text{ kN/m} $$

![Figure D1.13 Static system and load for calculation of deflection at the composite stage](image)

The deflection is:

$$ \delta_2 = 0.0099 \cdot \frac{e_d \cdot L^4}{E_a \cdot I_c} $$

$$ \delta_2 = 0.0099 \cdot \frac{4.9 \cdot 250^4}{210000 \cdot 16 \cdot 10^6} = 0.53 \text{ mm} = L/4716 $$

**Remark:**

The limit of the deflection is adopted according to clause 7.4.1(4), EN 1992-1-1. The recommended limitation is:

$$ \delta_{total} \leq \frac{L}{250} $$

The total deflection is:

$$ \delta_{total} = \delta_1 + \delta_2 = 0.26 + 0.53 = 0.79 \text{ mm} \leq \frac{L}{250} = \frac{2500}{250} = 10.0 \text{ mm} $$

The total deflection meets the criterion $L/250$.

7. **Commentary**

The design of composite slabs is mainly based on data provided by the supplier of the profiled sheeting. However, the reliability of the obtained data is very
important for structural reliability.

At the construction stage, profiled steel sheetings act as both a working platform and also permanent formwork and they can stabilise beams during execution. The requirements for the construction stage are governed for the design of profiled steel sheetings.

The area of cross-section of profiled steel sheeting that satisfies the criteria for the construction stage usually provides enough bottom reinforcement for the composite slab. In such cases, the composite slabs are considered as simply supported. The top longitudinal reinforcement at internal supports should be provided to control the widths of cracks. It is recommended that reinforcement for crack control is provided in the form of a mesh over the full area of the slab. In this way, the mesh reinforcement has very favourable structural effects under fire conditions.