Commented example of combined precast and in-situ reinforced concrete slab design

Educational material

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1. Assignment
The goal of this commented example is to make the students of Faculty of Civil Engineering, CTU in Prague familiar with the design procedure of combined precast and in-situ slabs. Design of load-bearing members is shown also with respect to assembling stages of particular precast elements of the slab. The aim of the commented example is to show the complexity of structural design.

![Fig. 1 Scheme of the structure](image)

2. Design of the structure for service life
At the first step, the structure will be design for the loads that affect it during its service life, because these loads are the dominant ones.

2.1. Loading of the slab

<table>
<thead>
<tr>
<th>Permanent</th>
<th>Thickness [mm]</th>
<th>Bulk density [kg/m³]</th>
<th>Char. load [kN/m²]</th>
<th>Partial factor</th>
<th>Design load [kN/m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ceramic tiles</td>
<td>8</td>
<td>2000</td>
<td>0,16</td>
<td>1,35</td>
<td>0,22</td>
</tr>
<tr>
<td>Flexible cement</td>
<td>2</td>
<td>2100</td>
<td>0,04</td>
<td>1,35</td>
<td>0,06</td>
</tr>
<tr>
<td>Concrete</td>
<td>100</td>
<td>2500</td>
<td>2,50</td>
<td>1,35</td>
<td>3,38</td>
</tr>
<tr>
<td>Self-weight</td>
<td>150</td>
<td>2500</td>
<td>3,75</td>
<td>1,35</td>
<td>5,06</td>
</tr>
<tr>
<td>Plaster</td>
<td>15</td>
<td>1800</td>
<td>0,27</td>
<td>1,35</td>
<td>0,36</td>
</tr>
<tr>
<td>Total of permanent load</td>
<td>Σ 6,72</td>
<td></td>
<td></td>
<td></td>
<td>Σ 9,07</td>
</tr>
</tbody>
</table>

| Variable                   |                |                      |                    |                |                    |
| Category C4: Areas for physical activities | 5,00 | 1,5 | 7,5 |
| Removable partitions, weight < 3,0 kN/m | 1,20 | 1,5 | 1,8 |
| Total of variable          | Σ 6,20         |                      |                    |                | Σ 9,3               |
| TOTAL                      | 12,92          |                      |                    |                | 18,4               |
2.2. Calculation of cover depth

Description of the structure
- The structure is inside the building, with low-humidity operations => XC1
- The surface is not exposed to corrosion caused by chlorides
- The surface is not in contact with sea water
- The structure is protected, therefore is not exposed to freeze-thaw cycles
- There is no risk of chemical attack of the structure
- Slab-like structure

Requirements: XC1 …. C 30/37; w/c < 0.6, c > 260 kg/m³
Serviceability life 50 years => structural class S4

\[ c_{\text{nom}} = c_{\text{min}} + \Delta c_{\text{dev}} \]
\[ c_{\text{min}} = \max \{ c_{\text{min,b}}; c_{\text{min,dur}} + \Delta c_{\text{dur,y}} - \Delta c_{\text{dur,st}} - \Delta c_{\text{dur,add}}; 10 \text{ mm} \} \]
\[ c_{\text{min,b}} = 10 \text{ mm} \] (estimated diameter of rebars)
\[ c_{\text{min,dur}} = 10 \text{ mm} \]

<table>
<thead>
<tr>
<th>Structural class</th>
<th>Exposure class related to environmental conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X0</td>
</tr>
<tr>
<td>S1</td>
<td>10</td>
</tr>
<tr>
<td>S2</td>
<td>10</td>
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<tr>
<td>S3</td>
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<td>S4</td>
<td>10</td>
</tr>
<tr>
<td>S5</td>
<td>15</td>
</tr>
<tr>
<td>S6</td>
<td>20</td>
</tr>
</tbody>
</table>

\[ \Delta c_{\text{dur,y}}; \Delta c_{\text{dur,st}}; \Delta c_{\text{dur,add}} \] considered equal to 0
\[ c_{\text{min,stat}} = \max \{ 10; 100 + 0 - 0 - 0; 10 \} = 10 \text{ mm} \]
\[ \Delta c_{\text{dev}} = 10 \text{ mm} \] (precast structure)
\[ c_{\text{nom,stat}} = c_{\text{min,stat}} + \Delta c_{\text{dev}} = 10 + 10 = 20 \text{ mm} \]

2.3. Material characteristics

CONCRETE C 30/37 =>
\[ f_{ck} = 30 \text{ MPa} \]
\[ f_{cd} = \frac{30}{1,5} = 20,0 \text{ MPa} \]
\[ f_{cm} = 2,9 \text{ MPa} \]
\[ f_{ck,0.05} = 2,0 \text{ MPa} \]
\[ \varepsilon_{\text{cu3}} = 3,5\% \]

STEEL B500B =>
\[ f_{sk} = 500 \text{ MPa} \]
\[ f_{yd} = \frac{500}{1,15} = 434 \text{ MPa} \]
\[ E_s = 200\,000\,\text{MPa} \]
\[ \varepsilon_s = \frac{f_{yd}}{E_s} = \frac{434}{200\,000} = 2,174\% \]
\[ \varepsilon_{\text{bal}} = \frac{\varepsilon_{\text{cu3}}}{\varepsilon_{\text{cu3}} + \varepsilon_s} = \frac{3,5}{3,5 + 2,174} = 0,616 \]
Moment curve

In final stage, filigree panels act as continuous beam. Therefore it is necessary to place upper reinforcement above the supports (beams). This reinforcement will be placed into the slab duty in casting of in-situ part of the structure.

2.4. ULS design of reinforcement – final design load

- Design of midspan reinforcement (positive moment)

Estimated diameter of rebars Ø 8

\[ d = h - c - \frac{\Phi}{2} = 150 - 20 - \frac{8}{2} = 126 \text{ mm} \]

\[ \mu = \frac{m_{Ed}}{b.d^2 \cdot \eta \cdot f_{cd}} = 9.2 \]

\[ 1.0.0.126^2.1.0.20.0.10^7 = 0.0289 \Rightarrow \zeta = 0.980 \]

\[ 0.038 \leq \zeta_{\lim} \Rightarrow \sigma_s = f_{yd} \]

\[ A_{s,req} = \frac{m_{Ed}}{f_{yd} \cdot \zeta \cdot d} = \frac{9.2 \cdot 10^6}{434.7.10^3.0.98.0.126} = 171.4 \text{ mm}^2 \]

Design: Ø8 per 150 mm (\( A_{s,prov} = 335 \text{ mm}^2 \))

Check of spacing (using detailing rules)

\[ s_{prov} = 150 \text{ mm} \]

\[ s_{max,1} = 2.h = 2.150 = 300 \text{ mm} \]

\[ s_{max,2} = 300 \text{ mm} \]

\[ s_{max} = \min(s_{max,1}; s_{max,2}) = 300 \text{ mm} \]

\[ s_{min} = \max(1.2; \frac{d_g}{5}; 20\text{mm}) = \max(1.2; 22; 20) \Rightarrow s_{min} = 27 \text{ mm} \]

\( d_g \) maximum diameter of aggregate grains
Check of minimum cross-sectional area of reinforcement

\[ A_{s,\text{prov}} = 335 \, \text{mm}^2 \]

\[ A_{s,\text{max}} = 0,04 \cdot A_c = 0,04 \cdot 150.1000 = 6 \, 000 \, \text{mm}^2 \]

\[ A_{s,\text{min}} = \frac{0,26 \cdot f_{\text{cm}} \cdot b \cdot d}{f_{\text{yk}}} = \frac{0,26 \cdot 2,9 \cdot 1000.126}{500} = 190,0 \, \text{mm}^2 \]

\[ A_{s,\text{min}} \geq 0,0013 \cdot b \cdot d = 0,0013 \cdot 1000.126 = 163,8 \, \text{mm}^2 \]

\[ A_{s,\text{min}} \leq A_{s,\text{prov}} \leq A_{s,\text{max}} \]

\[ 190,0 \leq 335 \leq 6 \, 000 \, \text{CHECKED} \]

Check of reinforcement for ultimate limit state (ULS)

\[ x = \frac{A_{s,\text{prov}} \cdot f_{\text{yd}}}{\lambda \cdot b \cdot \eta \cdot f_{\text{cd}}} = \frac{335.434,7}{0,8 \cdot 1000.1,0.20,0} = 9,1 \, \text{mm} \]

\[ \xi = \frac{x}{d} = \frac{9,1}{126} = 0,072 \leq 0,45 = \xi_{\text{lim}} \]

\[ z = d - 0,5 \cdot \lambda \cdot x = 0,126 - 0,5 \cdot 0,8 \cdot 0,091 = 0,121 \, \text{m} \]

\[ m_{\text{Ed}} = a_{s,\text{prov}} \cdot f_{\text{yd}} \cdot z = 335.434,7 \cdot 10^{-2} \cdot 0,121 = 17,69 \, \text{kNm} \]

\[ m_{\text{Ed}} = 17,69 \, \text{kNm} \geq 9,2 \, \text{kNm} = m_{\text{Ed}} \, \text{CHECKED} \]

\[ \text{reserve 92%} \]

• Reinforcement in support (negative moment)

We choose \( \varnothing 10 \) diameter of upper reinforcement is usually one or two levels higher than for lower reinforcement

\[ d = h - c - \varnothing / 2 = 150 - 20 - 10/2 = 125 \, \text{mm} \]

\[ \mu = \frac{m_{\text{Ed}}}{b \cdot d^2 \cdot \eta \cdot f_{\text{cd}}} = \frac{11,48}{1,0.0.125^2.1.0.20.0.10^3} = 0,037 \Rightarrow \xi = 0,982 \]

\[ \xi = 0,045 \leq 0,45 = \xi_{\text{lim}} \Rightarrow \sigma_s = f_{\text{yd}} \]

\[ \xi = 0,045 \leq 0,45 = \xi_{\text{lim}} \Rightarrow \sigma_s = f_{\text{yd}} \]

\[ A_{s,\text{req}} = \frac{m_{\text{Ed}}}{f_{\text{yd}} \cdot \xi \cdot d} = \frac{11,48 \cdot 10^6}{434,7 \cdot 10^1 \cdot 0,982 \cdot 0,125} = 215,2 \, \text{mm}^2 \]

Design: \( \varnothing 10 \) per 300 mm \( (A_{s,\text{prov}} = 262 \, \text{mm}^2) \)

\[ \sim 5 \sim \]
Check of spacing

\[ s_{\text{prov}} = 300 \text{ mm} \]
\[ s_{\text{max,1}} = 2h = 2.150 = 300 \text{ mm} \]
\[ s_{\text{max,2}} = 300 \text{ mm} \]

\[ s_{\max} = \min(s_{\text{max,1}}, s_{\text{max,2}}) = 300 \text{ mm} \]

\[ s_{\min} = \max(1,2 \phi; d_g + 5; 20 \text{ mm}) = \max(1,2.10; 22 + 5; 20) \Rightarrow s_{\min} = 27 \text{ mm} \]

\[ (d_g \text{ maximum diameter of aggregate grains}) \]

\[ s_{\min} \leq s_{\text{prov}} \leq s_{\max} \]

\[ 27 \text{ mm} \leq 300 \text{ mm} \leq 300 \text{ mm} \]

**CHECKED**

Check of minimum cross-sectional area of reinforcement

\[ A_{r,\text{prov}} = 262 \text{ mm}^2 \]
\[ A_{r,\text{max}} = 0.04A_c = 0.04 \cdot 150 \cdot 1000 = 6000 \text{ mm}^2 \]
\[ A_{r,\text{min}} = \frac{0.26f_{cm}b_hd}{f_{yk}} = \frac{0.26 \cdot 29 \cdot 1000 \cdot 1.25}{500} = 188 \text{ mm}^2 \]

*(brittle failure)*

\[ A_{r,\text{min}} \geq 0.0013b_hd = 0.0013 \cdot 1000 \cdot 1.25 = 163 \text{ mm}^2 \]

*(reinforcement ratio)*

\[ A_{r,\text{min}} \leq A_{r,\text{prov}} \leq A_{r,\text{max}} \]

188 \leq 262 \leq 6000 **CHECKED**

Check of reinforcement in ultimate limit state (ULS)

\[ x = \frac{A_{r,\text{prov}}f_{yd}}{\lambda b_h f_{cd}} = \frac{262.434.7}{0.81000.10.20.0} = 7.1 \text{ mm} \]
\[ \xi = \frac{x}{d} = \frac{7.1}{125} = 0.056 \leq 0.45 = \xi_{\text{lim}} \]
\[ z = d - 0.5 \xi \lambda = 0.125 - 0.5 \cdot 0.056 = 0.121 \text{ m} \]
\[ m_{Rd} = a_{s,\text{prov}}f_{yd} \cdot z = 262.434.7 \cdot 10^{-3} \cdot 0.121 = 13.83 \text{ kNm} \]
\[ m_{Rd} = 13.83 \text{ kNm} \geq 11.48 \text{ kNm} = m_{Ed} \]

**CHECKED**

reserve 20%
2.5. SLS check of panel

For the purpose of this example, serviceability limit state will be checked using simplified method based on deflection control criterion.

Concrete class: C 30/37
Type of structure: continuous beam
\( A_{s,prov} = 335 \text{ mm}^2 \) (lower reinforcement is crucial for deflections)
\( A_{s,req} = 171 \text{ mm}^2 \) (required cross-sectional area of reinforcement from ULS)
Span: \( l = 2,5 \text{ m} \)

Reinforcement ratio \( \rho = A_{s,prov} / A_c = 335 / (1000.150) = 0,22 \% \)

As the reinforcement ratio of our panel is less than 0.5 \%, \( \lambda_{d,tab} \) can be taken as 30,8 and no interpolation is required.

\[ \lambda_{d,tab} = 30,8 \quad \text{pro} \quad \rho = 0,5\% \]
\[ \kappa_{c1} = 1,0 \quad (\text{rectangular cross-section}) \]
\[ \kappa_{c2} = 1,0 \quad (\text{span is less than 7,0 m}) \]
\[ \kappa_{c2} = \frac{500}{A_{s,prov}} \cdot \frac{A_{s,prov}}{A_{s,req}} = \frac{500 \cdot 335}{500 \cdot 171} = 1,96 \]
\[ \lambda_d = \kappa_{c1} \cdot \kappa_{c2} \cdot \kappa_{c3} \cdot \lambda_{d,tab} = 1,0 \cdot 1,0 \cdot 1,96 \cdot 21,00 = 41,16 \]
\[ l / d = 2,5 / 0,126 = 19,84 \]
\[ 19,84 < 41,16 \Rightarrow \text{Checked} \]

3. Check of the beam

3.1. Calculation of cover depth

Description of the structure:
- The structure is inside the building, with low-humidity operations \( \Rightarrow \) XC1
- The surface is not exposed to corrosion caused by chlorides
- The surface is not in contact with sea water
- The structure is protected, therefore is not exposed to freeze-thaw cycles
- There is no risk of chemical attack of the structure

Requirements: XC1 …. C 30/37; w/c < 0,6, c > 260 kg/m\(^3\)
Serviceability life 50 years \( \Rightarrow \) structural class S4
- Cover depth for stirrups

\[ c_{nom} = c_{min} + \Delta c_{dev} \]
\[ c_{min} = \max \{ c_{min,b}, c_{min,dur} + \Delta c_{dur,y} - \Delta c_{dur,st} - \Delta c_{dur,add}; 10 \text{ mm} \} \]
\[ c_{min,b} = 8 \text{ mm (estimated diameter of stirrups)} \]
\[ c_{min,dur} = 10 \text{ mm} \]
### Values of $c_{\text{min,dur}}$ [mm]

<table>
<thead>
<tr>
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</tr>
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<tbody>
<tr>
<td></td>
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<tr>
<td>S3</td>
<td>10</td>
</tr>
<tr>
<td>S4</td>
<td>10</td>
</tr>
<tr>
<td>S5</td>
<td>15</td>
</tr>
<tr>
<td>S6</td>
<td>20</td>
</tr>
</tbody>
</table>

- $\Delta c_{\text{dur,}\gamma}; \Delta c_{\text{dur,st}}; \Delta c_{\text{dur,add}}$ considered equal to 0
- $c_{\text{min,st}} = \max\{8; 10 + 0 - 0; 10\} = 20$ mm
- $\Delta c_{\text{dev}} = 10$ mm (precast structure)
- $c_{\text{nom,st}} = c_{\text{min,st}} + \Delta c_{\text{dev}} = 10 + 10 = 20$ mm

- **Cover depth for main bending reinforcement**
  
  $c_{\text{nom}} = c_{\text{min}} + \Delta c_{\text{dev}}$
  
  $c_{\text{min}} = \max\{c_{\text{min,b}}; c_{\text{min,dur}} + \Delta c_{\text{dur,}\gamma} - \Delta c_{\text{dur,st}} - \Delta c_{\text{dur,add}}; 10\}$ mm
  
  $c_{\text{min,b}} = 22$ mm (estimated diameter of rebars)
  
  $c_{\text{min,dur}} = 10$ mm
  
  $\Delta c_{\text{dur,}\gamma}; \Delta c_{\text{dur,st}}; \Delta c_{\text{dur,add}}$ considered equal to 0
  
  $c_{\text{min,st}} = \max\{22; 10 + 0 - 0; 10\} = 22$ mm
  
  $\Delta c_{\text{dev}} = 5$ mm (precast structure)
  
  $c_{\text{nom,st}} = c_{\text{min,st}} + \Delta c_{\text{dev}} = 22 + 5 = 27 \approx 30$ mm

### 3.2. Design of main bending reinforcement in midspan

- **Calculation of loads**
  
  \((18,4.2.5+0.25.0,35.25.1,35) = 48.9\text{ kN/m'}\)

  Slab is supported by masonry walls, therefore it acts as a simple beam.

  The beam is loaded by the load from the slab multiplied by loading width of the beam (axial span in our case) + the self weight of part of the beam under the slab.

  $M_{\text{Ed,midspan}} = 184.86$ kNm
We choose Ø22

\[ d = h - c - \frac{R}{2} = 500 - 30 - \frac{22}{2} = 459 \text{ mm} \]  (In final design state, beam collaborates with slab)

\[ \xi = 0.245 \leq 0.45 = \xi_{lim} \] CHECKED

\[ A_{s,req} = \frac{M_{Ed}}{f_y d \xi d} = \frac{184.86 \times 10^3}{434 \times 10^3 \times 0.902 \times 0.459} = 1028 \text{mm}^2 \]

Check of reinforcement ratio

\[ A_{s,prov} = 1140 \text{ mm}^2 \]

\[ A_{s,max} = 0.04A_c = 0.04 \times 500.250 = 5000 \text{ mm}^2 \]

\[ A_{s,min} \geq 0.0013b_n d = 0.0013 \times 250.459 = 149 \text{ mm}^2 \]

\[ A_{s,min} \leq A_{s,prov} \leq A_{s,max} \]

173 \leq 1140 \leq 5000 CHECKED

Check of spacing

\[ s_{prov} = \frac{b_n - 2c - n\phi}{n - 1} = \frac{250 - 2.30 - 3.22}{2} = 62 \text{ mm} \]

\[ s_{min} = \text{max} \left( 1.2\phi; d_g + 5; 20 \text{mm} \right) = \text{max} \left( 1.2 \times 22; 16 + 5; 20 \right) \Rightarrow s_{min} = 26.4 \text{mm} \]

\[ (d_g \text{ maximum diameter of aggregate grains}) \]

\[ s_{min} \leq s_{prov} \leq s_{max} \]

26.4 \text{ mm} \leq 62 \text{ mm} CHECKED
Check of reinforcement in ultimate limit state (ULS)

**Design:** $3\Omega 22$ ($A_{s,prov} = 1140 \text{ mm}^2$)

$$ x = \frac{A_{sl} \cdot f_{sl}}{0.8 \cdot b \cdot f_{cd}} = \frac{1140 \cdot 10^{-6} \cdot 434}{0.8 \cdot 0.25 \cdot 20.0} = 124 \text{ mm} $$

$$ \zeta = \frac{x}{d} = \frac{0.124}{0.459} = 0.270 \leq 0.45 = \zeta_{\text{lim}} \quad \text{CHECKED} $$

$$ z = d - 0.4 \cdot x = 459 - 0.4 \cdot 124 = 409 \text{ mm} $$

$$ M_{rd} = A_{s} \cdot f_{cd} \cdot z = 1140 \cdot 10^{-6} \cdot 434 \cdot 0.409 = 194,32 \text{ kNm} $$

$$ M_{rd} = 194,32 \text{ kNm} \geq 184,86 \text{ kNm} = M_{Ed} \quad \text{CHECKED} $$

**Reserve 5%**

### 3.3. Design of shear reinforcement

$V_{Ed} = 134.4 \text{ kNm} \quad \text{(maximum value of shear force)}$

Capacity of compressed concrete strut

$$ \cot \theta = 1.5 \quad \text{(recommended value for optimum slope of shear cracks)} $$

$$ v = 0.6 \left(1 - \frac{f_{ck}}{250}\right) = 0.6 \left(1 - \frac{30}{250}\right) = 0.528 $$

$$ V_{rd,\text{max}} = v \cdot f_{cd} \cdot b \cdot z \cdot \frac{\cot \theta}{1 + \cot^2 \theta} = 0.528 \cdot 20.0 \cdot 0.25 \cdot 0.409 \cdot \frac{1.5}{1 + 1.5^2} = 356.9 \text{ kN} $$

**Design of stirrups**

<table>
<thead>
<tr>
<th>Ø</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>number of legs</td>
<td>2</td>
</tr>
<tr>
<td>$s$</td>
<td>150 mm</td>
</tr>
<tr>
<td>$f_{y,wk}$</td>
<td>500 MPa</td>
</tr>
</tbody>
</table>

$$ \rho_{w,\text{min}} = 0.08 \sqrt{\frac{f_{ck}}{250}} = 0.08 \sqrt{\frac{30}{500}} = 0.00088 $$

$$ s_{1,\text{max}} = \frac{A_{w}}{b \cdot \rho_{w,\text{min}}} = \frac{2.50.6}{250 \cdot 0.00088} = 460 \text{ mm} $$

$$ s_{2,\text{max}} = \min \left(0.75 \cdot d; 400\right) = 0.75 \cdot 459 = 344 \text{ mm} $$

$$ s_{\text{max}} = \min \left(460; 344\right) = 344 \text{ mm} \quad \text{(maximum spacing of stirrups according to detailing rules)} $$

$$ s_{prov} = 150 \text{ mm} \leq 344 \text{ mm} = s_{\text{max}} \quad \text{CHECKED} $$
3.4. Connection

In case of full connection between beam and slab, the strain in critical cross-section is continuous up to the stage when limit elastic state of the cross-section is reached. This situation can be best characterized as a stage when yield limit of tensile reinforcement is reached (the strain of reinforcement is $\varepsilon_y$). Load-bearing capacity of such cross-section is given by force couple $N_c$ and $N_s$ with lever arm $z$.

![Fig. 6 Behavior of two fully connected members subjected to bending](image)

![Fig. 7 Distribution of internal forces in precast and in-situ part of the cross-section.](image)

$$V_{Rd,s} = \frac{n \cdot A_s \cdot f_{yed} \cdot \zeta \cdot \cot \theta}{s} \cdot 150 = 4.50 \cdot 26.500 / 1.15 = 459.15 = 171.4 \, kN$$

$$V_{Ed} = 134.4 \, kN \leq 171.4 \, kN = V_{Rd} \leq 356.9 \, kN = V_{Rd,\text{max}}$$

CHECKED
reseve 27%

Minimum bedding length of filigree panel on the beam is 35 mm. From technological point of view, even smaller values are possible, but temporary linear support right next to the beam is required in such a case. This option requires a way of support different from the one shown in this example.

**Reduction of shear forces** (for optimization of results)

$$V_{Ed,1} = V_{Ed} - \left( \frac{b + d}{2} \cdot (g + q) \right) = 134.45 - \left( \frac{0.250}{2} + 0.459 \right) \cdot (48.9) = 105.9 \, kN$$

~ 11 ~
\[ v_{Ed} = \frac{\beta \cdot V_{Ed,i}}{z \cdot b_i} = \frac{1.0 \cdot 105.9 \cdot 10^3}{409 \cdot (250 - 2.35)} = 1.43 \, \text{MPa} \]

\( \beta \) is ratio between longitudinal force in in-situ part of cross-section and total longitudinal force in compressed part of cross-section, in our case \( \beta = 1.0 \), because neutral axis lays in in-situ part of cross-section (see check of beam),

\( z \) is lever arm of internal forces (see check of beam),

\( b_i \) is width of connection area between in-situ and precast concrete members,

\[ v_{rd} = c \cdot f_{cd} + \mu \cdot \sigma_n + \rho \cdot f_{rd} \cdot (\mu \cdot \sin \alpha + \cos \alpha) \leq 0.5 \cdot v \cdot f_{cd} \]

**Connection type: rough connection**

\( c = 0.45 \); \( \mu = 0.7 \)

where:

\( c \), \( \mu \) are coefficients depending of roughness of connection area,

\( f_{cd} \) is design tensile strength of concrete calculated from characteristic strength \( f_{ck,0.05} \)

\( \sigma_n \) is normal stress perpendicular to connection area, in our case we take \( \sigma_n = 0 \),

\( \rho \) is reinforcement ratio of connection area by connecting members,

\( \alpha \) is angle between connecting reinforcement and shear plane, in our case reinforcement will be perpendicular to concrete surface and therefore \( \alpha = 90^\circ \),

\( v \) is reduction factor for compressive strength of concrete in shear, \( v = 0.6 \left( 1 - \frac{f_{ck}}{250} \right) \)

\[ v = 0.6 \left( 1 - \frac{30}{250} \right) = 0.528 \]

\[ v_{rd} = 0.45 \cdot \frac{2.0}{1.5} + 0.7 \cdot 0 + \frac{100.5,6,66}{250 \cdot 1000} \cdot 434.7 \cdot (0.7 \cdot 1 + 0) \leq 0.5 \cdot 0.528 \cdot 20 \]

\[ v_{rd} = 1.45 \, \text{MPa} \leq 5.28 \, \text{MPa} \quad \text{CHECKED} \]

\[ v_{Ed} = 1.43 \, \text{MPa} \leq 1.45 \, \text{MPa} = v_{rd} \quad \text{CHECKED} \]
4. Temporary design situations

4.1. Design of lifting clutches of the beam

\[ F_p = V \cdot \rho = 0.25 \cdot 0.35 \cdot 5.5 \cdot 25 = 12.03 \text{ kN} \quad \text{(self-weight of the beam)} \]

\[ F_{\text{adh}} = A \cdot q = \left[ (0.35 + 0.25 + 0.35) \cdot 5.5 + (0.35 \cdot 0.25) \cdot 2 \right] \cdot 1 = 5.4 \text{ kN} \quad \text{(increase in self-weight due to adhesion to formwork)} \]

### Adhesion coefficient for formwork

<table>
<thead>
<tr>
<th>Formwork Type</th>
<th>Coefficient q</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smooth, oiled formwork</td>
<td>q = 1 \text{ kN/m}^2</td>
</tr>
<tr>
<td>Smooth, non-oiled formwork</td>
<td>q = 2 \text{ kN/m}^2</td>
</tr>
<tr>
<td>Rough formwork</td>
<td>q = 3 \text{ kN/m}^2</td>
</tr>
</tbody>
</table>

To minimize the labour consumption in production process, formwork on all sides of the panel is considered. This is also the worst case for calculation.

\[ N_{d,1} = \delta \cdot \frac{\gamma_{go}}{n \cdot \cos \alpha} \left( F_p + F_{\text{adh}} \right) = 1.3 \cdot \frac{1.35}{2 \cdot \cos 30} \cdot (12.03 + 5.4) = 11.26 \text{ kN} \quad \text{(force in one lifting clutch during formwork stripping)} \]

\[ N_{d,2} = \delta \cdot \frac{\gamma_{go}}{n \cdot \cos \alpha} F_p = 2 \cdot \frac{1.35}{2 \cdot \cos 30} \cdot 12.03 = 13.13 \text{ kN} \quad \text{(force in one lifting clutch during manipulation on construction site)} \]

\[ N_d = \max (N_{d,1}; N_{d,2}) = \max (11.26; 13.13) = 13.13 \text{ kN} \]

During manipulation, the member is subjected to dynamic effects as well. These effects are taken into account by dynamic coefficient \( \delta \), which increases static effects. The size of dynamic coefficient is given by lifting device and its rectification.

### Recommended values of dynamic coefficient

<table>
<thead>
<tr>
<th>Conditions</th>
<th>( \delta )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fixed crane, rail crane (&lt; 90 \text{m/min})</td>
<td>1.0 – 1.2</td>
</tr>
<tr>
<td>Fixed crane, rail crane (&gt; 90 \text{m/min})</td>
<td>1.3 – 1.4</td>
</tr>
<tr>
<td>Lifting and transport in flat terrain</td>
<td>1.5 – 1.65</td>
</tr>
<tr>
<td>Lifting and transport in rough terrain</td>
<td>(&gt; 2.0)</td>
</tr>
</tbody>
</table>

\( \gamma_{go} \) and \( \gamma_{\text{adh}} \) are the adhesion coefficients for formwork.

\( \gamma_{go} \) for smooth, oiled formwork is \( q = 1 \text{ kN/m}^2 \), for smooth, non-oiled formwork is \( q = 2 \text{ kN/m}^2 \), and for rough formwork is \( q = 3 \text{ kN/m}^2 \).
4.2. **Number of connecting elements for manipulation**

If the connecting reinforcement protrudes from the surface, they can be used for manipulation and no further lifting clutches are required.

Force in one leg of the connecting element

\[
N = 0.6 \cdot \frac{N_d}{\cos(\beta + 30)} = 0.6 \cdot \frac{13.13}{\cos(45 + 30)} = 21.56 \text{ kN}
\]

\[
\chi = 0.05 \cdot \phi + 0.3 = 0.05 \cdot 8 + 0.3 = 0.7
\]

\[
A_{s,req} = \frac{N_d}{\chi \cdot f_{yd}} = \frac{21.56}{0.7 \cdot 434.7} = 70.8 \text{ mm}^2
\]

Cross-sectional area of one leg of stirrup is Ø8=50.3 mm²

Required number of stirrups \( n = \frac{70.8}{50.3} \approx 1.4 \Rightarrow 2 \text{ kusy} \)

4.3. **Check of beam during transport**

For manipulation, lifting clutches placed 1 m from the end of the beam will be used.

\[ M_{Ed,v poli} = 4.52 \text{ kNm} \]

\[
d = h - c - \frac{\bar{R}}{2} = 350 - 30 - \frac{22}{2} = 309 \text{ mm} \quad \text{(only precast part is effective during transport)}
\]

- Check of reinforcement

Návrh 3Ø22 \( (A_{s,prov} = 1140 \text{ mm}^2) \) (already designed)

\[
x = \frac{A_{s,vec} \cdot f_{yd}}{0.8\cdot b \cdot f_{ed}} = \frac{1140 \cdot 10^{-6} \cdot 434}{0.8 \cdot 0.25 \cdot 20.0} = 124 \text{ mm}
\]

\[
\frac{\xi}{d} = \frac{0.124}{0.309} = 0.40 \leq 0.617 = \xi_{\text{max}} \quad \text{CHECKED}
\]

~ 14 ~
Design of support reinforcement

We choose 2Ø10 (minimum beam reinforcement required by detailing rules)

d = h - c - \frac{\bar{R}}{2} = 350 - 30 - \frac{10}{2} = 315 \text{ mm} \quad (\text{only precast part is effective during transport})

4.4. Design of lifting clutches of filigree panel

\( F_p = V \cdot p = 0.05 \cdot 1.2 \cdot 2.5 \cdot 25 = 3.75 \text{ kN} \) \quad (self-weight of the panel)

\( F_{adh} = A \cdot q = 1.2 \cdot 2.5 \cdot 1 = 3.25 \text{ kN} \) \quad (increase in self-weight due to adhesion to formwork)
\[ N_{d,1} = \frac{\alpha_{go}}{n \cos \alpha} \left( F_p + F_{tan} \right) = 1,3 \cdot \frac{1,35}{2 \cdot \cos 30} \cdot (3,75 + 3,25) = 6,55 \text{kN} \quad \text{(force in one lifting clutch during formwork stripping)} \]

\[ N_{d,2} = \frac{\alpha_{go}}{n \cos \alpha} \cdot F_p = 2 \cdot \frac{1,35}{2 \cdot \cos 30} \cdot 3,25 = 4,09 \text{kN} \quad \text{(force in one lifting clutch during manipulation on construction site)} \]

\[ N_d = \max(N_{d,1}, N_{d,2}) = \max(6,55; 4,09) = 6,55 \text{kN} \]

- Required number of stirrups

\[ N = 0,6 \cdot \frac{N_d}{\cos (\beta + 30)} = 0,6 \cdot \frac{6,55}{\cos (45 + 30)} = 9,25 \text{kN} \]

\[ \chi = 0,05 \cdot \phi + 0,3 = 0,05 \cdot 8 + 0,3 = 0,7 \]

\[ A_{x,req} = \frac{N_d}{\chi \cdot f_{yd}} = \frac{9,25}{0,7 \cdot 434,7} = 35,5 \text{mm}^2 \]

Cross-sectional area of one leg of stirrup Ø6=28,3 mm²

Required number of stirrups \( n = 35,5/28,3=1,2 \Rightarrow 2 \text{ kusy} \)

### 5. Temporary supports during concreting

#### 5.1. Filigree panel supports

Assembling loading

<table>
<thead>
<tr>
<th>Permanent</th>
<th>Thickness [mm]</th>
<th>Bulk density [kg/m³]</th>
<th>Char. load [kN/m²]</th>
<th>Partial factor</th>
<th>Design load [kN/m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>In-situ concrete layer</td>
<td>100</td>
<td>2500</td>
<td>2,5</td>
<td>1,35</td>
<td>3,38</td>
</tr>
<tr>
<td>Filigree panel</td>
<td>50</td>
<td>2500</td>
<td>1,25</td>
<td>1,35</td>
<td>1,69</td>
</tr>
<tr>
<td>Total of permanent load</td>
<td>( \Sigma )</td>
<td></td>
<td>3,75</td>
<td>( \Sigma )</td>
<td>5,06</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Variable</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Variable assembling load</td>
<td>0,75</td>
<td>1,5</td>
<td>1,13</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total of variable load</td>
<td>( \Sigma )</td>
<td></td>
<td>0,75</td>
<td>( \Sigma )</td>
<td>1,13</td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td>( \Sigma )</td>
<td></td>
<td>4,05</td>
<td>( \Sigma )</td>
<td>6,19</td>
</tr>
</tbody>
</table>

The standard determines variable load 0,75 kN/m² during assembling

The calculation is based on an assumption of one temporary support in the middle of span of filigree panel.
Fig. 10 Moment curve with temporary support

\[ M_{Ed} = \frac{1}{8} \cdot (g + q) \cdot \left( \frac{l}{2} \right)^2 = \frac{1}{8} \cdot 6.19 \cdot \left( \frac{2.5}{2} \right)^2 = 1.20 \text{kNm} \]

Fig. 11 Distribution of internal forces in slab (negative moment)

- Check of reinforcement in ULS

Ø10 per 300 mm \( (A_{s,prov} = 262 \text{ mm}^2) \) (already designed)

\( d = h - c - \varnothing / 2 = 50 - 20 - 8/2 = 26 \text{ mm} \)

\[
x = \frac{A_{s,prov} \cdot f_{yd}}{\lambda \cdot b \cdot \eta \cdot f_{cd}} = \frac{262.434.7}{0.8.10.010.1.0.20.0} = 7.1 \text{ mm}
\]

\[
\xi = \frac{x}{d} = \frac{7.1}{26} = 0.27 \leq 0.617 = \xi_{\text{lim}}
\]

\[
z = d - 0.5 \cdot \lambda \cdot x = 26 - 0.5 \cdot 0.8 \cdot 7.1 = 23.2 \text{ mm}
\]

\[
m_{Ed} = a_{s,prov} \cdot f_{yd} \cdot z = 262.434.7 \cdot 10^{-3} \cdot 0.023 = 2.6 \text{ kNm}
\]

\[
m_{Ed} = 2.6 \text{kNm} = 1.20 \text{kNm} = m_{Ed} \quad \text{CHECKED}
\]

One support in the midspan is required. Assumption checked.

5.2. Beam supports

Loading area \( s = 2.5/2 = 1.25 \text{ m} \)

\[
M_{Ed} = \frac{1}{8} \cdot [(g + q) \cdot s] \cdot \left( \frac{l}{2} \right)^2 = \frac{1}{8} \cdot 6.19 \cdot 1.25 \cdot 5.5^2 = 29.25 \text{ kNm}
\]
• Design of midspan reinforcement

\[ d = h - c - \frac{\bar{R}}{2} = 350 - 30 - \frac{22}{2} = 309 \text{ mm} \text{ (only precast part is effective during transport)} \]

• Check of reinforcement

Design: 3 \( \varnothing 22 \) \( (A_{s,prov} = 1140 \text{ mm}^2) \) \( (already \ designed) \)

\[ x = \frac{A_{sd} \cdot f_{yd}}{0.8 \cdot b \cdot f_{cd}} = \frac{1140 \cdot 10^{-6} \cdot 434}{0.8 \cdot 0.25 \cdot 20.0} = 124 \text{ mm} \]

\[ \xi = \frac{x}{d} = \frac{0.124}{0.309} = 0.40 \leq 0.617 = \xi_{max} \text{ CHECKED} \]

\[ z = d - 0.4 \cdot x = 309 - 0.4 \cdot 124 = 256 \text{ mm} \]

\[ M_{rd} = A_{s} \cdot f_{yd} \cdot z = 1140 \cdot 10^{-6} \cdot 434 \cdot 0.256 = 128.34 \text{ kNm} \]

\[ M_{rd} = 128.34 \text{ kNm} \geq 29.25 \text{ kNm} = M_{Ed} \text{ CHECKED} \]

The beam does not require temporary supports.