2. PRELIMINARY DESIGN OF STRUCTURAL ELEMENTS’ DIMENSIONS

2.1. USED MATERIAL

Concrete class C30/37: \( f_{ck} = 30 \text{ MPa} \)
\( f_{cd} = 20 \text{ MPa} \)

Reinforcing bars’ diameter: \( \varphi = 10 \text{ mm} \)

Steel class B500 B: \( f_{yk} = 500 \text{ MPa} \)
\( f_{yd} = 435 \text{ MPa} \)

2.2. DESIGN OF HORIZONTAL LOAD BEARING STRUCTURES - SLAB

Empirical estimation:
\[ h_s \geq \frac{1.1}{3.3} \cdot l_{n,\text{max}} = \frac{1.1}{3.3} \cdot 6300 \text{ mm} \]
\[ h_s \geq 210 \text{ mm} \to h_s = 240 \text{ mm} \] (estimation)

Cover depth:
\[ c = c_{\text{min}} + \Delta c_{\text{dev}} \]
\[ c_{\text{min}} = \max (c_{\text{min,b}}; c_{\text{min,dur}}; 10 \text{ mm}) \]
\[ c_{\text{min,dur}} = 10 \text{ mm} \] (S3, XC1) \( \to c_{\text{min}} = 10 \text{ mm} \)
\[ c = 10 \text{ mm} + 10 \text{ mm} = 20 \text{ mm} \]

Effective depth:
\[ d = h_s - c - \frac{\varphi}{2} = 240 - 20 - \frac{10}{2} = 215 \text{ mm} \]

Span/depth ratio:
\[ \lambda = \frac{l}{d} \leq \lambda_{\text{lim}} = \kappa_{c1} \cdot \kappa_{c2} \cdot \kappa_{c3} \cdot \lambda_{d,\text{tab}} \]
\[ \kappa_{c1} = 1, \kappa_{c2} = 1, \kappa_{c3} = 1.25 \] (reinf. utilisation 80%)
\[ \lambda_{d,\text{tab}} = 24.6 (\rho = 0.5 \%) \]
\[ d \geq \frac{6.6}{1+1+1.25+24.6} = 0.21463 \text{ m} \]
\[ d \geq 214.63 \text{ mm} \] O.K.

\[ h_s \geq d + \frac{\varphi}{2} + c = 215 + 5 + 20 = 240 \] O.K.

**CONCLUSION:** The slab is designed with thickness 240 mm, cover depth 20 mm, reinforcing bars diameter 10 mm. Its effective depth is 215 mm.
2.3. DESIGN OF VERTICAL LOAD BEARING STRUCTURES - COLUMN

2.3.1. COLUMN’S LOADING AREA

\[ A_{CA} = 4.92 \text{ m} \times 5.9 \text{ m} = 29.028 \text{ m}^2 \]

2.3.2. LOADS ACTING ON THE VERTICAL STRUCTURE

2.3.2.1. FLOOR SLAB COMPOSITION

- Tiles: \( \rho_1 = 2300 \text{ kg/m}^3 \), \( t_1 = 10 \text{ mm} \)
- Anhydrite layer: \( \rho_2 = 2200 \text{ kg/m}^3 \), \( t_2 = 40 \text{ mm} \)
- Separation layer: \( \rho_3 = 500 \text{ kg/m}^3 \), \( t_3 = 1 \text{ mm} \)
- XPS insulation: \( \rho_4 = 40 \text{ kg/m}^3 \), \( t_4 = 50 \text{ mm} \)
- Vapour barrier PE: \( \rho_5 = 0.19 \text{ kg/m}^2 \)
- Gypsum board 12,5mm: \( \rho_6 = 12.5 \text{ kg/m}^2 \)
2.3.2.2. LOAD FROM A FLOOR SLAB – PLANAR LOAD

<table>
<thead>
<tr>
<th></th>
<th>k-value [kN/m²]</th>
<th>γ</th>
<th>d-value [kN/m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Self-weight=</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>25 kN/m³*0,24 m</td>
<td>6</td>
<td>1,35</td>
<td>8,1</td>
</tr>
<tr>
<td>Tiles=23 kN/m³*0,01m</td>
<td>0,23</td>
<td>1,35</td>
<td>0,3105</td>
</tr>
<tr>
<td>Anhydrite=</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>22 kN/m³*0,04m</td>
<td>0,88</td>
<td>1,35</td>
<td>1,188</td>
</tr>
<tr>
<td>Separation=5 kN/m³*</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0,001m</td>
<td>0,005</td>
<td>1,35</td>
<td>0,00675</td>
</tr>
<tr>
<td>XPS=0,4 kN/m³*0,05m</td>
<td>0,02</td>
<td>1,35</td>
<td>0,027</td>
</tr>
<tr>
<td>PE foil=</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0,0019</td>
<td>1,35</td>
<td>0,00257</td>
</tr>
<tr>
<td>Gypsum board=</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0,125</td>
<td>1,35</td>
<td>0,16875</td>
</tr>
<tr>
<td>Variable: Live load=</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1,5</td>
<td>1,5</td>
<td>2,25</td>
</tr>
</tbody>
</table>

\[f_s = 8,7619 - 12,0536\]

category A (residential objects)

CONCLUSION: The total value of load of a floor structure is 8,762 kN/m². For further calculations the design value of 12,054 kN/m² will be used.

2.3.2.3. ROOF SLAB COMPOSITION

<table>
<thead>
<tr>
<th></th>
<th>(\rho_1) =</th>
<th>(\rho_2) =</th>
<th>(\rho_3) =</th>
<th>(\rho_4) =</th>
<th>(\rho_5) =</th>
<th>(\rho_6) =</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof tiles:</td>
<td>13,4 kg/m²</td>
<td>50 kg/m²</td>
<td>0,06 kg/m²</td>
<td>150 kg/m³</td>
<td>0,14 kg/m²</td>
<td>12,5 kg/m²</td>
</tr>
<tr>
<td>Wooden laths, est.:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Safety waterproofing:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MW insulation:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vapour barrier:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gypsum board 12,5 mm:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
2.3.2.4. **LOAD FROM A ROOF SLAB – PLANAR LOAD**

<table>
<thead>
<tr>
<th></th>
<th>k-value [kN/m²]</th>
<th>γ</th>
<th>d-value [kN/m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Permanent:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Self-weight=</td>
<td>0,5</td>
<td>1,35</td>
<td>0,675</td>
</tr>
<tr>
<td>Roofing=</td>
<td>0,134</td>
<td>1,35</td>
<td>0,1809</td>
</tr>
<tr>
<td>Waterproofing=</td>
<td>0,0006</td>
<td>1,35</td>
<td>0,00081</td>
</tr>
<tr>
<td>MW=1,5 kN/m³*0,22m</td>
<td>0,33</td>
<td>1,35</td>
<td>0,4455</td>
</tr>
<tr>
<td>Vapour barrier=</td>
<td>0,0014</td>
<td>1,35</td>
<td>0,00189</td>
</tr>
<tr>
<td>Gypsum board=</td>
<td>0,125</td>
<td>1,35</td>
<td>0,16875</td>
</tr>
<tr>
<td><strong>Variable:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Live load=</td>
<td>1</td>
<td>1,5</td>
<td>1,5</td>
</tr>
<tr>
<td>Snow load=0,8<em>0,8</em>1*0,7 kN/m²</td>
<td>0,448</td>
<td>1,5</td>
<td>0,672</td>
</tr>
</tbody>
</table>

\[ f_r = 2,539 \quad - \quad 3,64485 \]

**CONCLUSION:** The total value of load of a roof structure is 2,539 kN/m². For further calculations the design value of 3,645 kN/m² will be used.

2.3.2.5. **LOAD FROM PARTITIONS – LINEAR LOAD**

\[ \rho_p = 500 \text{ kg/m}^3 \text{ lightweight concrete YTONG} \]
\[ t_p = 150 \text{ mm} \]
\[ h_p = 3260 \text{ mm} \]
\[ h_s = 3500 \text{ mm} \]
\[ f_{p,k} = (\rho_p /100) * t_p * h_p = (500/100) * 0,15 * 3,26 = 2,445 \text{ kN/m} \]
\[ f_{p,d} = f_{p,k} * 1,35 = 2,445 * 1,35 = 3,301 \text{ kN/m} \]

\[ l_{p,x} = 8260 \text{ mm} \text{ length of partitions in x-direction on a column load area} \]
\[ l_{p,y} = 9020 \text{ mm} \text{ length of partitions in y-direction on a column load area} \]

**CONCLUSION:** The total value of load of partitions is 2,445 kN/m. For further calculations the design value of 3,301 kN/m will be used.
2.3.2.6. LOAD FROM A COLUMN – FORCE LOAD

\[
\begin{align*}
\rho_c &= 2500 \text{ kg/m}^3 \\
d_{c,\text{est}} &= 300 \text{ mm} \quad \text{estimated dimension of column} \\
h_c &= 3260 \text{ mm} \quad 3500 \text{ mm-}h_s
\end{align*}
\]

\[
F_{c,k} = (\rho_c/100) \times d_{c,\text{est}}^2 \times h_c = (2500/100) \times 0,30^2 \times 3,26 = 7,335 \text{ kN}
\]

\[
F_{c,d} = 1,35 \times F_{c,k} = 1,35 \times 7,335 = 9,9 \text{ kN}
\]

**CONCLUSION:** The total estimated value of load of a column is 7,335 kN. For further calculations the design value of 9,9 kN will be used.

2.3.2.7. LOAD ACTING ON THE MOST LOADED COLUMN

<table>
<thead>
<tr>
<th>Load</th>
<th>type of load</th>
<th>Live/dead load</th>
<th>K-value</th>
<th>unit</th>
<th>( \gamma )</th>
<th>D-value</th>
<th>unit</th>
<th>n of floors of action</th>
<th>calculation</th>
<th>total load for all floors</th>
<th>unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor slab</td>
<td>planar</td>
<td>live + dead</td>
<td>8,7619</td>
<td>kN/m²</td>
<td>1,35; 1,5</td>
<td>12,054</td>
<td>kN/m²</td>
<td>4</td>
<td>4<em>12,054</em>A_{cc} = 4<em>12,054</em>29,028 =</td>
<td>1399,6</td>
<td>kN</td>
</tr>
<tr>
<td>Roof slab</td>
<td>planar</td>
<td>live + dead</td>
<td>2,539</td>
<td>kN/m²</td>
<td>1,35; 1,5</td>
<td>3,645</td>
<td>kN/m²</td>
<td>1</td>
<td>1<em>3,645</em>A_{cc} = 1<em>3,645</em>29,028 =</td>
<td>105,8</td>
<td>kN</td>
</tr>
<tr>
<td>Partitions</td>
<td>linear</td>
<td>dead</td>
<td>2,445</td>
<td>kN/m</td>
<td>1,35</td>
<td>3,301</td>
<td>kN/m</td>
<td>4</td>
<td>4<em>3,301</em>(l_{p,x} + l_{p,y}) = 4<em>3,301</em>(8,26 + 9,02) =</td>
<td>228,15</td>
<td>kN</td>
</tr>
<tr>
<td>Column</td>
<td>force</td>
<td>dead</td>
<td>7,335</td>
<td>kN</td>
<td>1,35</td>
<td>9,9</td>
<td>kN</td>
<td>4</td>
<td>4*9,9 =</td>
<td>39,6</td>
<td>kN</td>
</tr>
</tbody>
</table>

\( N_{Ed,\text{col.}} = 1399,6 + 105,8 + 228,5 + 39,6 = 1773,15 \text{ kN} \)

**CONCLUSION:** The total value of all loads acting on the most loaded column is 1773,15 kN.

2.3.3. COLUMN’S DIMENSIONS

\[
N_{Rd} \geq N_{Ed}
\]

\[
N_{Rd} = 0,8 \times A_c \times f_{cd} + A_s \times \sigma_s \geq N_{Ed}
\]
\[ A_c = 0.02 \cdot A_c \]
\[ \sigma_s = 400 \text{ MPa} \]

\[ A_c \geq \frac{N_{ed}}{0.8 \cdot f_{cd} + 0.02 \cdot \sigma_s} = \frac{1773,15 \cdot 10^3}{0.8 \cdot 20 + 0.02 \cdot 400} \]

\[ A_c \geq 73881 \text{ mm}^2 \]

\[ d_c \geq \sqrt{A_c} = \sqrt{73881} = 272 \text{ mm} \]

\[ d_c = 300 \text{ mm} \]

\[ N_{Rd} = 0.8 \cdot 300^2 \cdot 20 + 0.02 \cdot 300^2 \cdot 400 = 2160000 \text{ N} = 2160 \text{ kN} \geq 1773,15 \text{ kN} \quad \text{O.K.} \]

**CONCLUSION:** The designed dimensions of a column are 300 x 300 mm.

### 2.4. PRELIMINARY CHECK OF PUNCHING OF A COLUMN

\[ u_0 = 4 \cdot a = 4 \cdot 300 = 1200 \text{ mm} \]

\[ u_1 = 4 \cdot a + 2 \cdot \pi \cdot 2 \cdot d = 1200 + 2 \cdot \pi \cdot 2 \cdot 215 \text{ (page 1)} \]

\[ u_1 = 3901,77 \text{ mm} \]

#### 2.4.1. MAXIMUM PUNCHING SHEAR RESISTANCE

\[ \nu_{Ed,0} = \frac{\beta \cdot V_{Ed}}{u_0 \cdot d} \leq V_{Rd,\text{max}} = 0.4 \cdot \nu \cdot f_{cd} \]

\[ \beta = 1,15 \text{ (inner column)} \]

\[ V_{Ed} = F_{s,d}/4 = 1399,6/4 = 349,89 \text{ kN} \text{ (normal force in the column from one floor)} \]

\[ \nu = 0.6 \cdot (1 - \frac{f_{ck}}{250}) = 0.6 \cdot (1 - \frac{30}{250}) = 0.528 \]

\[ \nu_{Ed,0} = \frac{1.15 \cdot 349,89 \cdot 10^3}{1200 \cdot 215} = 1,56 \text{ MPa} \]

\[ V_{Rd,\text{max}} = 0.4 \cdot 0.528 \cdot 20 = 4,224 \text{ MPa} \geq V_{Ed,0} \quad \text{O.K.} \]
2.4.2. MAXIMUM RESISTANCE WITH REINFORCEMENT

\[ v_{Ed,1} = \frac{\beta V_{Ed}}{u_1 d} \leq k_{max} \cdot v_{Rd,c} \]

\[ v_{Rd,c} = C_{Rd,c} \cdot k \cdot \sqrt[3]{100 \rho_t \cdot f_{ck}} \]

\[ k_{max} = 1,4575 \quad \text{coefficient of maximum resistance, interpolated value} \]

\[ C_{Rd,c} = 0,12 \quad \text{reduction factor} \]

\[ k = 1 + \sqrt{\frac{200}{d}} \leq 2,0 \quad \text{effect of depth} \]

\[ k = 1 + \sqrt{\frac{200}{215}} = 1,96 < 2,0 \]

\[ \rho_t = 0,005 \quad \text{ratio of tensile reinforcement, estimated value} \]

\[ v_{Rd,c} = 0,12 \cdot 1,96 \cdot \sqrt[3]{100 \cdot 0,005 \cdot 30} = 0,581 \text{ MPa} \]

\[ v_{Ed,1} = \frac{1,15 \cdot 349,89 \cdot 10^3}{3901,77 \cdot 215} = 0,48 \text{ MPa} \]

\[ k_{max} \cdot v_{Rd,c} = 1,4575 \cdot 0,581 = 0,847 \text{ MPa} > v_{Ed,1} \]

O.K.

CONCLUSION: The structure is suitable for punching reinforcement design.

2.5. CHECK OF LOAD-BEARING CAPACITY OF MASONRY WALL

2.5.1. STRENGTH OF MASONRY

The type of bricks used: HELUZ FAMILY 44, dimensions 440 x 247 x 249 mm

\[ f_u = 8 \text{ MPa} \quad \text{compressive strength} \]

\[ f_m = 5 \text{ MPa} \quad \text{strength of mortar} \]

\[ f_b = \delta \cdot f_u \]

\[ \delta = 1,155 \quad \text{effect of bricks’ dimensions, value taken from table} \]

\[ f_b = 1,155 \cdot 8 = 9,24 \text{ MPa} \quad \text{compressive strength of the units} \]
\[ f_k = K \cdot f_{b}^{0.7} \cdot f_{m}^{0.3} \]

\[ K = 0.65 \quad \text{coefficient, value taken from table} \]

\[ f_k = 0.65 \cdot 9.24^{0.7} \cdot 5^{0.3} = 4.995 \text{ MPa} \quad \text{characteristic compressive strength of masonry} \]

\[ f_d = f_k/\gamma_m \]

\[ \gamma_m = 2 \quad \text{partial factor for material, designed mortar} \]

\[ f_d = 4.995/2 = 2.498 \text{ MPa} \quad \text{design compressive strength of masonry} \]

### 2.5.2. LOADS ACTING ON MASONRY WALL

![Area of load acting on the wall](image)

\[ A_{WA} = 5.9 \cdot 2.74 = 16.166 \text{ m}^2 \quad \text{area of load acting on the wall} \]

#### 2.5.2.1. LOAD FROM SLABS – ROOF AND TYPICAL FLOORS – PLANAR LOAD

**NOTE:** The values are the same as in 2.3.2. LOADS ACTING ON THE VERTICAL STRUCTURE.

#### 2.5.2.2. LOAD FROM PARTITIONS – LINEAR LOAD

\[ \rho_p = 500 \text{ kg/m}^3 \quad \text{lightweight concrete YTONG} \]

\[ t_p = 150 \text{ mm} \]

\[ h_p = 3260 \text{ mm} \quad 3500 \text{ mm-}h_s \]
f_{p,k} = (\rho_p /100) * t_p * h_p = (500/100) * 0,15 * 3,26 = 2,445 kN/m

f_{p,d} = f_{p,k} * 1,35 = 2,455 * 1,35 = 3,301 kN/m

l_{p,x} = 0 mm  \text{ length of partitions in } x \text{-direction on a wall load area}
l_{p,y} = 2300 \text{ mm length of partitions in } y \text{-direction on a wall load area}

NOTE: The values of characteristic and design partition linear load are the same as in 2.3.2. LOADS ACTING ON THE VERTICAL STRUCTURE.

2.5.2.3. LOAD FROM MASONRY – LINEAR LOAD

\rho_w = 650 \text{ kg/m}^3 \text{ density of the material}
t_w = 440 \text{ mm}
h_w = 3260 \text{ mm 3500 mm-} h_s

f_{w,k} = (\rho_w /100) * t_w * h_w = (650/100) * 0,44 * 3,26 = 9,324 kN/m

f_{w,d} = f_{w,k} * 1,35 = 9,324 * 1,35 = 12,587 kN/m

l_{w,x} = 5900 \text{ mm length of walls in } x \text{-direction on a wall load area}
l_{w,y} = 0 mm \text{ length of wall in } y \text{-direction on a wall load area}

CONCLUSION: The total value of load of masonry walls is 9,324 kN/m. For further calculations the design value of 12,587 kN/m will be used.

2.5.2.4. LOAD ACTING ON THE MOST LOADED PART OF MASONRY WALL

<table>
<thead>
<tr>
<th>Load</th>
<th>type of load</th>
<th>Live/dead load</th>
<th>K-value</th>
<th>unit</th>
<th>n of floors</th>
<th>calculation</th>
<th>total load for all floors</th>
<th>unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor slab</td>
<td>planar</td>
<td>live + dead</td>
<td>8,7619</td>
<td>kN/m²</td>
<td>1,35; 1,5</td>
<td>3 * 12,054 * A_w = 3 * 12,054 * 16,166 =</td>
<td>584,6</td>
<td>kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roof slab</td>
<td>planar</td>
<td>live + dead</td>
<td>2,539</td>
<td>kN/m²</td>
<td>1,35; 1,5</td>
<td>1 * 3,645 * A_w = 1 * 3,645 * 16,166 =</td>
<td>58,9</td>
<td>kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Partitions</td>
<td>linear</td>
<td>dead</td>
<td>2,445</td>
<td>kN/m</td>
<td>1,35</td>
<td>2 * 3,301 * (l_{p,x} + l_{p,y}) = 2 * 3,301 * (0 + 2,3) =</td>
<td>15,2</td>
<td>kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Masonry</td>
<td>linear</td>
<td>dead</td>
<td>9,324</td>
<td>kN/m</td>
<td>1,35</td>
<td>2 * 12,587 * (l_{w,x} + l_{w,y}) = 2 * 12,587 * (5,9 + 0) =</td>
<td>148,5</td>
<td>kN</td>
</tr>
</tbody>
</table>
N_{Ed,wall} = 584,6 + 58,9 + 15,2 + 148,5 = 807,2 kN

**CONCLUSION:** The total value of all loads acting on the most loaded part of masonry wall is 807,2 kN.

### 2.5.3. CHECK OF RESISTANCE OF THE MASONRY WALL

**2.5.3.1. SLENDERNESS RATIO**

\[
\frac{h_{ef}}{t} \leq 27
\]

\[
h_{ef} = \rho_n \times h_w
\]

\[
\rho_n = 0,75 \quad \text{factor of supporting the wall at its top and bottom}
\]

\[
h_w = 3260 \text{ mm} \quad 3500 - h_s
\]

\[
h_{ef} = 0,75 \times 3260 = 2445 \text{ mm} = 2,445 \text{ m}
\]

\[
\frac{h_{ef}}{t} = \frac{2,445}{0,44} = 5,557 < 27 \quad \text{O.K.}
\]

**2.5.3.2. ECCENTRICITY DUE TO LOADS**
F₁  \textit{force from the structure above (excl. the connected slab)}

\[ F₁ = 1 \times \text{roof load} + 2 \times \text{slab load} + 2 \times \text{masonry wall load} + 2 \times \text{partition load} \]

\[ F₁ = 58,9 \text{ kN} + 2 \times (584,6 \text{ kN} / 3) + 148,5 \text{ kN} + 15,2 \text{ kN} = 612,3 \text{ kN} \]

\[ e₁ = 0 \text{ mm} \]

F₂  \textit{force from the connected slab}

\[ F₂ = 1 \times \text{slab load} = 584,6 \text{ kN} / 3 = 194,87 \text{ kN} \]

\[ e₂ = 80 \text{ mm} = 0,08 \text{ m} \]

F₃  \textit{force from concrete beam}

\[ F₃ = (0,28 \text{ m} \times 0,25 \text{ m} \times 5,9 \text{ m}) \times 25 \text{ kN/m}^3 = 10,325 \text{ kN} \]

\[ e₃ = 80 \text{ mm} = 0,08 \text{ m} \]

\textbf{NOTE:} The design values of force loads are taken from 2.5.2. \textit{LOADS ACTING ON MASONRY WALL}.

\[ e_f = \Sigma F_i \times e_i = \frac{F₁ \times e₁ + F₂ \times e₂ + F₃ \times e₃}{F₁ + F₂ + F₃} \]

\[ e_f = \frac{612,3 \times 0 + 194,87 \times 0,08 + 10,325 \times 0,08}{612,3 + 194,87 + 10,325} = \frac{16,4156}{817,495} = 0,0201 \]

\[ e_a = \frac{h_{ef}}{450} = 2,445/450 = 0,00543 \] \textit{initial eccentricity}

\[ e_t = e_f + e_a = 0,0201 + 0,00543 = 0,02553 \] \textit{total eccentricity}

\[ \phi = 1 - \frac{2 \times e_f}{t} = 1 - \frac{2 \times 0,02553}{0,44} = 0,0884 \] \textit{capacity reduction factor}

\textbf{2.5.3.3. RESISTANCE AT THE TOP OF THE WALL}

\[ N_{Ed,A} = N_{Ed,wall} = 807,2 \text{ kN} \]

\textbf{NOTE:} The design value of force load acting on the top of the wall is taken from 2.5.2. \textit{LOADS ACTING ON MASONRY WALL}. 
\[ N_{Rd,i} = \phi_i \cdot A \cdot f_d \]

\[ A = t \cdot l_{w,x} = 0,44 \cdot 5,9 = 2,596 \text{ m}^2 \]

\[ f_d = 2,498 \text{ MPa} \]

**NOTE:** The value of design compressive strength of masonry is taken from 1.5.1. STRENGTH OF MASONRY.

\[ N_{Rd,i} = 0,884 \cdot 2,596 \cdot 2,498 \cdot 10^3 = 5726,1 \text{ kN} > N_{Ed,A} \text{ O.K.} \]

### 2.5.3.4. RESISTANCE AT THE MIDDLE OF THE WALL

\[ N_{Ed,B} = N_{Ed,wall} + \frac{1}{2} \text{ wall} = 807,2 \text{ kN} + (148,5 \text{kN}/2)/2 = 844,3 \text{ kN} \]

**NOTE:** The design value of force load of a masonry wall is taken from 2.5.2. LOADS ACTING ON MASONRY WALL.

\[ N_{Rd,m} = \phi_m \cdot A \cdot f_d \]

\[ e_{mf} = e_f \cdot 0,5 = 0,0201 \cdot 0,5 = 0,01005 \]

\[ \frac{e_{mk}}{t} = \frac{e_{mf} + e_a + e_k}{t} \quad \text{relative eccentricity} \]

\[ e_k = 0 \quad \text{eccentricity due to creep} \]

\[ \frac{e_{mk}}{t} = \frac{0,01005 + 0,00543 + 0}{0,44} = 0,0352 \]

\[ \phi_m = 0,885 \quad \text{capacity reduction factor, value taken from table, estimated eccentricity 0,05} \]

\[ N_{Rd,m} = 0,885 \cdot 2,596 \cdot 2,498 \cdot 10^3 = 5739 \text{ kN} > N_{Ed,B} \text{ O.K.} \]

**CONCLUSION:** The masonry wall is able to withstand the loading.
2.6. CHECK OF LOAD-BEARING CAPACITY OF LINTELS

2.6.1. LOAD ACTING ON THE LINTEL

\[ f_{\text{lintel,d}} = 1^{*} \text{slab} + 1^{*} \text{wall above the lintel} = f_{s,d} * l_s + \rho_{m,d} * t * h \]

\[ l_s = 2300 \text{ mm} \]

\[ \rho_{m,d} = (650 \text{ kg/m}^3 / 100) * 1,35 = 6,5 \text{ kN/m}^3 * 1,35 = 8,775 \text{ kN/m}^3 \]

\[ h = 1750 \text{ mm} \]

\[ f_{\text{lintel,d}} = 12,054 \text{ kN/m}^2 * 2,3 \text{ m} + 8,775 \text{ kN/m}^3 * 0,44 \text{ m} * 1,75 \text{ m} = 34,481 \text{ kN/m} \]

CONCLUSION: The design value of total load acting on the lintel subjected to check is 34,481 kN/m.

2.6.2. DESIGNED LINTEL PARAMETERS

The used lintels are Porotherm 7, length 2500 mm, bedding 250 mm on each side, clear span 2000 mm. In the given structure there is 5 of them used above the opening.
\( q_d = 40 \text{ kN/m} > f_{\text{lintel,d}} \) load-bearing capacity of lintels if 4 of them are used

O.K.

**CONCLUSION:** The lintels are able to withstand the loading.
3. DESIGN OF SLAB REINFORCEMENT

3.1. CALCULATION IN SCIA ENGINEER SOFTWARE

3.1.1. MODEL

Fig. 1 Ground plan of model with invisible structure structure mesh

Fig. 2 Bird’s-eye view of model
Fig. 3 Bird’s-eye view of model with invisible structure mesh

Fig. 4 Worm’s-eye view of model
3.1.2. LOADS

**Fig. 5 Variable load applied on the model**

**Fig. 6 Permanent load applied on the model**

**NOTE:** SCIA Engineering SW calculates the value of self-weight of the structure by itself.
3.1.3. RESULTS

**Fig. 7** Mesh dividing the structure into many separate elements, for which the calculation is then performed.

**Fig. 8** The envelope of bending moments appearing in the slab structure in the x-direction. These values are then used for further calculation of bending reinforcement.
Fig. 9 The envelope of bending moments appearing in the slab structure in the y-direction. These values are then used for further calculation of bending reinforcement.

Fig. 10 Scheme of total displacement appearing in the structure. The maximum displacement was calculated with value of 1.8 mm.

NOTE: The results of bending moments are depicted in the attached Annex file 1. Diagrams of bending moment envelopes.
### 3.2. DESIGN OF SLAB’S REINFORCEMENT

#### 3.2.1. CALCULATION OF MINIMUM REINFORCEMENT

Cross-sectional area of bending reinforcement:

\[ a_{s,prov} \geq a_{s,req} = \frac{m_{cd}}{z \cdot f_{yd}} \]

\( z = 0.9 \cdot d \)  

**lever arm of internal forces – estimation**

Brittle failure precaution:

\[ a_{s,prov} \geq a_{s,min,1} = \max \left( 0.26 \cdot \left( \frac{f_{ctm}}{f_{yk}} \right) \cdot b \cdot d ; 0.0013 \cdot b \cdot d \right) \]

\( f_{ctm} = 2.9 \text{ MPa} \)  

\( b = 1 \text{ m} \)  

**width of the slab, in this case equal to 1 m**

Excessive cracking precaution:

\[ a_{s,prov} \geq a_{s,min,2} = \frac{k_c \cdot k \cdot f_{ct,eff} \cdot A_{ct}}{\sigma_s} \]

\( k_c = 0.4 \) and \( k = 1.0 \)  

**coefficients describing stress distributions in the cross-section**

\( f_{ct,eff} = f_{ctm} = 2.9 \text{ MPa} \)  

**mean value of tensile strength in time when first cracks might appear**

\( \sigma_s = f_{yk} = 500 \text{ MPa} \)  

**maximum stress inside the reinforcement after cracks appear**

\( A_{ct} = 0.5 \cdot b \cdot d \)  

**area of concrete within tensile zone at the first crack**

**NOTE:** The values of \( a_{s,min,2} \) will not be specified in further calculations due to the fact that these values are negligibly small compared to other calculated limits of minimum reinforcement.

#### 3.2.2. CHECK OF THE DESIGN

Height of compressed zone of concrete cross-section:

\[ x = \frac{a_{s,prov} \cdot f_{yd}}{0.8 \cdot b \cdot f_{cd}} \]
\[ z = d - 0.4x \]

*actual value of lever arm of internal forces*

\[ m_{rd} = a_{s,prov} \times f_{yd} \times z \]

\[ m_{rd} \geq m_{ed} \]

3.2.3. DETAILING RULES

Relative height of compressed zone:
\[ \zeta = \frac{x}{d} \leq 0.45 \]

Spacing of rebars:
\[ s \leq \min (2h_s; 250 \text{ mm}) \]
\[ 2h_s = 480 \text{ mm} \]
\[ s \leq 250 \text{ mm} \]

**NOTE:** The values calculated in 3.2.1. Calculation of minimum reinforcement, 3.2.2. Check of the design and 3.2.3. Detailing rules are specified in attached *Annex file 2. Table of bending reinforcement design.*

3.3. DETAILED CHECK OF PUNCHING

\[ v_{ed,1} = \frac{\beta \times V_{ed}}{u_1 \times d} \leq v_{rd,c} \]

\[ v_{rd,c} = \max \left[ C_{rd,c} \times k \times \frac{3}{\sqrt{100 \times \rho_t \times f_{ck}}} ; 0.035 \times \frac{2}{\sqrt{k_2 \times f_{ck}}} \right] \]

\[ d = \frac{d_x + d_y}{2} \]
\[ d_x = 240 \text{ mm} – 20 \text{ mm} – (16 \text{ mm/2}) = 212 \text{ mm} \quad \text{reinf. } \phi 16 \text{ á 105 mm} \]
\[ d_y = 240 \text{ mm} – 20 \text{ mm} – 16 \text{ mm} – (14 \text{ mm/2}) = 197 \text{ mm} \quad \text{reinf. } \phi 14 \text{ á 95 mm} \]
\[ d = \frac{212 + 197}{2} = 204.5 \text{ mm} \]
\[
\rho_l = \sqrt{\rho_{lx} + \rho_{ly}} \leq 0.02
\]

\[
\rho_{lx} = \frac{a_{s,x}}{1000*d_y} \quad \rho_{ly} = \frac{a_{s,y}}{1000*d_y}
\]

\[a_{s,x} = 1915 \text{ mm}^2 \] \textit{reinf. } \phi 16 \text{ á } 105 \text{ mm}

\[a_{s,y} = 1620 \text{ mm}^2 \] \textit{reinf. } \phi 14 \text{ á } 95 \text{ mm}

\[
\rho_{lx} = \frac{1915}{1000*212} = 0.00903
\]

\[
\rho_{ly} = \frac{1620}{1000*197} = 0.00822
\]

\[
\rho_l = \sqrt{\rho_{lx} + \rho_{ly}} = 0.00861 \leq 0.02
\]

\[C_{Rd,c} = 0.12 \quad \text{reduction factor}\]

\[k = 1 + \sqrt{\frac{200}{d}} \leq 2.0 \quad \text{effect of depth}\]

\[k = 1 + \sqrt{\frac{200}{204.5}} = 1.99 < 2.0\]

\[C_{Rd,c}*k*\sqrt{(100 * \rho_l * f_{ck})} = 0.12*1.99*\sqrt{(100 * 0.00861 * 30)} = 0.706 \text{ MPa}\]

\[0.035*\sqrt{(k^3 * f_{ck})} = 0.035 * 2\sqrt{(1.99^3 * 30)} = 0.538 \text{ MPa}\]

\[\max [C_{Rd,c}*k*\sqrt{(100 * \rho_l * f_{ck})} ; 0.035*\sqrt{(k^3 * f_{ck})}] = 0.706 \text{ MPa} = v_{Rd,c}\]

\[v_{ed,1} = \frac{1.15*349.89*10^3}{3901.77*204.5} = 0.5 \text{ MPa}\]

\[v_{Rd,c} = 0.706 \text{ MPa} > v_{ed,1} = 0.5 \text{ MPa} \quad \text{O.K.}\]

**CONCLUSION:** The detailed check of punching confirmed that no punching reinforcement design is needed.

**NOTE:** Some value used in this calculation are taken from 2.4. Preliminary check of punching of a column.