Design of AAC roof slab according to EN 12602

Example 2: Roof slab with uniform load

1.1 Issue

Design of a roof slab

Materials

AAC with a compressive strength AAC 3,5 , density class 500, welded steel reinforcement with tensile yield strength 500 MPa and ultimate tensile strength 550 MPa.

EN 12602, table 1 and 2
EN 10080

1.2 Material properties

Dry Density

EN 12602, 4.2.2.3

Table 1: Density classes, dry densities in kg/m³

Density class	400	450	500	550	600	650	700
Mean dry density ρ_m	> 350 ≤ 400	> 400 ≤ 450	> 450 ≤ 500	> 500 ≤ 550	> 550 ≤ 600	> 600 ≤ 650	> 650 ≤ 700

Compressive strength

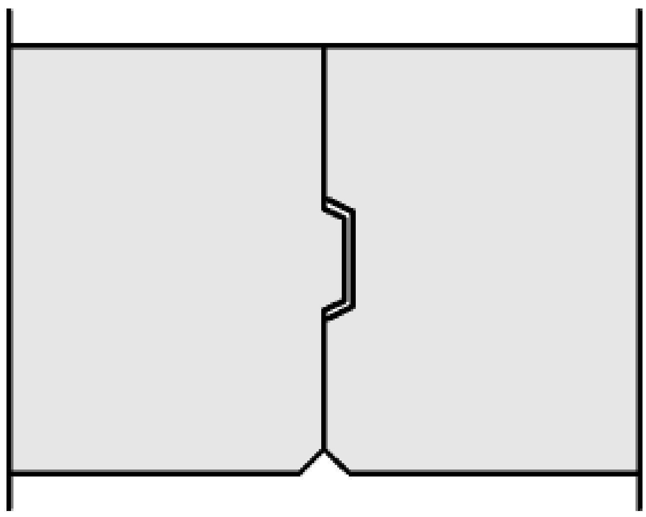
EN 12602, 4.2.4

Table 2: Compressive strength classes for AAC in MPa

Strength class	AAC 2	AAC 2,5	AAC 3	AAC 3,5	AAC 4	AAC 4,5	AAC 5
f_{ck}	2,0	2,5	3,0	3,5	4,0	4,5	5,0

1.3 Type of element

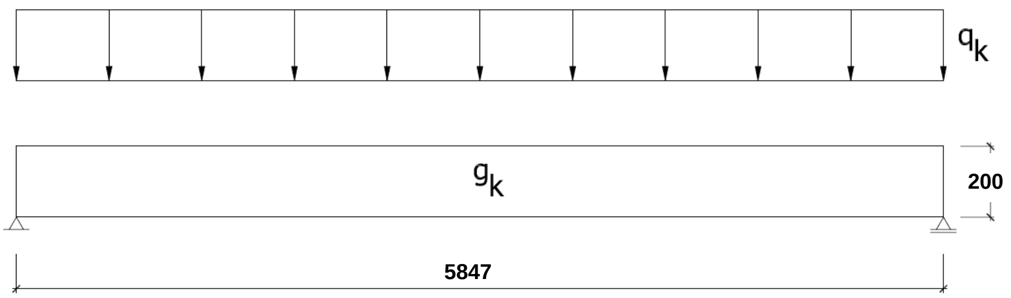
Profile

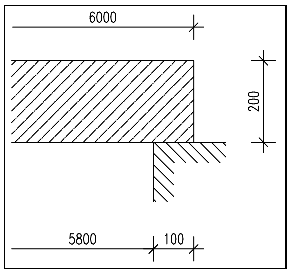


2 System and dimensions

2.1 System

Longitudinal section





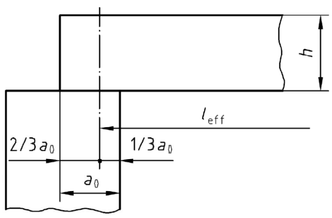
Minimum value for support length

EN 12602, A.11

AAC component	minimum requirement
beams	60 mm
floor elements	40 mm
roof elements	35 mm

Recommended values

AAC component	support material	minimum requirement
beams	masonry	100 mm
floor elements	masonry steel concrete	70 mm 50 mm 50 mm
roof elements	masonry steel concrete wood	70 mm 50 mm 50 mm 50 mm
wall elements	steel concrete	50 mm 50 mm



$$L_{eff} = l_w + \frac{1}{3}\, a_{1,min} + \frac{1}{3}\, a_{2,min} = 5{,}80 + \frac{2}{3} \cdot 0{,}07 = 5{,}847 \text{ m}$$

The component has to be designed for all load cases also for impacts resulting from transport.
The relevant load case for that is the transport with a fork lifter and for the weak axis.

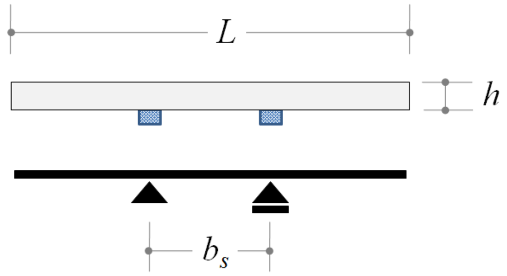


Figure 1: Transport situation fork lift truck

Assumption for distance forks: b_s = 1,00 m

$$L_{cantilever} = \frac{(L - bs)}{2} = \frac{(6{,}00 - 1{,}00)}{2} = 2{,}50 \text{ m}$$

2.2 Cross section

h = 200 mm
b = 625 mm

2.3 Concrete cover and effective depth

c_1 = 35 mm
c_2 = 35 mm

Assumption for fire resistance class: REI 90

With a granted diameter of 6 mm the effective depth is:

$$d = 200 \text{ mm} - 35 - \frac{6}{2} = 162 \text{ mm}$$

3 Loads

Self-weight of AAC element:
with 35 kg / m³ steel and 6 M-% moisture content of AAC

EN 12602, 4.2.2.4 (1)

Load	
Permanent loads • water proofing • AAC ($g = 5{,}7$ kN/m^3) permanent load, g_k =	 0,20 kN/m² <u>1,14 kN/m²</u> 1,34 kN/m²
Variable loads, q_K =	0,75 kN/m²

Thickness of slab = 0,20 m

Transport weight of AAC element:
$\rho_{trans} = 7{,}05$ kN/m³

EN 12602, 4.2.2.4 (3)

4 Internal forces

Internal forces are determined for a single component with a width of 625 mm.

4.1 *Internal forces for characteristic combinations*

Load combinations acc. to EN 1990

$G_{d1} = \gamma_G \cdot b \cdot g_k = 1{,}35 \cdot 0{,}625 \cdot 1{,}34 = 1{,}13$ kN/m

$Q_{d1} = \gamma_Q \cdot b \cdot q_k = 1{,}50 \cdot 0{,}625 \cdot 0{,}75 = 0{,}70$ kN/m

$$V_{Sd1} = \frac{(G_{d1}+Q_{d1})\cdot l_{eff}}{2} = \frac{(1{,}13+0{,}70)}{2} \cdot 5{,}847 = 5{,}35 \text{ kN}$$

$$M_{Sd1} = \frac{(G_{d1}+Q_{d1})\cdot l_{eff}^{2}}{8} = \frac{1{,}83\cdot 5{,}847^2}{8} = 7{,}82 \text{ kNm}$$

4.2 *Internal forces for frequent combinations*

$G_{d2} = b \cdot g_k = 0{,}625 \cdot 1{,}34 = 0{,}84$ kN/m

$Q_{d2} = \psi_1 \cdot b \cdot q_k = 0{,}2 \cdot 0{,}625 \cdot 0{,}75 = 0{,}09$ kN/m

$\psi_1 = 0{,}2$

$$V_{Sd2} = \frac{(G_{d2}+Q_{d2})}{2} \cdot l_{eff} = \frac{0{,}84+0{,}09}{2} \cdot 5{,}847 = 2{,}72 \text{ kN}$$

$M_{Sd2} = \frac{(G_{d2}+Q_{d2})\cdot l_{eff}^{2}}{8} = \frac{0{,}93\cdot 5{,}847^2}{8} = 3{,}97$ kNm ### 4.3 Internal forces for quasi-permanent combinations $G_{d3} = b \cdot g_k = 0{,}625 \cdot 1{,}34 = 0{,}84$ kN/m $Q_{d3} = \psi_2 \cdot b \cdot q_k = 0 \cdot 0{,}625 \cdot 2{,}00 = 0$ kN/m $V_{Sd3} = \frac{(G_{d3}+Q_{d3})}{2}\cdot l_{eff} = \frac{0{,}84+0}{2}\cdot 5{,}847 = 2{,}46$ kN $M_{Sd3} = \frac{(G_{d3}+Q_{d3})\cdot l_{eff}^{2}}{8} = \frac{0{,}84\cdot 5{,}847^2}{8} = 3{,}59$ kNm ### 4.4 Internal forces for transport situations $G_T = \gamma_G \cdot b \cdot g_T = 1{,}35 \cdot 0{,}625 \cdot 7{,}05 \cdot 0{,}20 = 1{,}19$ kN/m $V_T = \gamma_T \cdot \frac{G_T}{2} \cdot L = 1{,}3 \cdot 1{,}19 \cdot 2{,}50 = 3{,}87$ kN $M_T = \frac{\gamma_T \cdot G_T \cdot L^2_{cantilever}}{2} = \frac{1{,}3\cdot 1{,}19\cdot 2{,}5^2}{2} = 4{,}83$ kNm where $\gamma_T = 1{,}3$ (assumption for dynamic coefficient due to manipulation of components, when indicated consideration of national regulations)	$\psi_2=0$

5 Design

5.1 Material properties

Characteristic compressive strength,
f_{ck} = 3,5 MPa = 3500 kN/m²

EN 12602, 4.2.4

Basic Shear Strength,
$\tau_{Rd} = \frac{0{,}063 \cdot f_{ck}^{\;0{,}5}}{\gamma_c}$ = 0,063 · $3{,}5^{0{,}5}$ / 1,73 = 0,0681 MPa

EN12602, A.4.1.2 (A.6)

Mean Modulus of Elasticity of AAC slab,
E_{cm} = 5 · (ρ_m – 150) = 1750 N/mm²

EN 12602, 4.2.7

Characteristic yield strength of steel,
f_{yk} = 500 MPa = 500 N/mm²

EN 12602, 4.3.1

5.2 Design for bending

Finding equilibrium of stress / strain:

$$1000 \cdot m_d = \frac{1000 \cdot M_{Sd1} \cdot \gamma_c}{\alpha \cdot f_{ck} \cdot A_c \cdot d} = \frac{7{,}82 \cdot 1{,}44}{0{,}85 \cdot 3{,}5 \cdot 0{,}625 \cdot 0{,}162 \cdot 0{,}162} = 230{,}8$$

Reading from design table (see Annex A):

ε_c = 3,00 ‰	ε_s = 4,29 ‰
k_x = 0,406	1000·ϖ = 270,9

$$A_s = A_c \cdot \varpi \cdot \frac{\alpha \cdot f_{ck} \cdot \gamma_S}{\gamma_c \cdot f_{yk}} = \frac{0{,}625 \cdot 0{,}162 \cdot 270{,}9}{1000} \cdot \frac{0{,}85 \cdot 3{,}5 \cdot 1{,}15}{1{,}44 \cdot 500} = 1{,}30 \text{ cm}^2$$

chosen: 9 Ø 6,0 mm (A_{sl} = 2,54 cm²)

Upper reinforcement:

$$1000 \cdot m_d = \frac{1000 \cdot M_T \cdot \gamma_c}{\alpha \cdot f_{ck} \cdot A_c \cdot d} = \frac{4{,}83 \cdot 1{,}44}{0{,}85 \cdot 3{,}5 \cdot 0{,}625 \cdot 0{,}162 \cdot 0{,}162} = 142{,}5$$

Reading from design table (see Annex A):

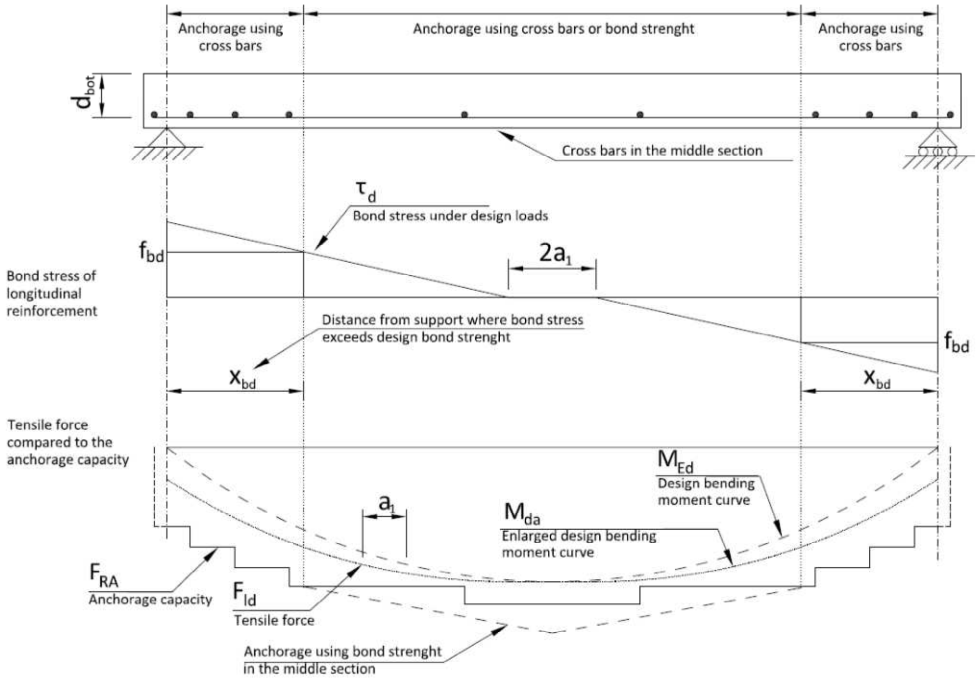
ε_c = 3,00 ‰	ε_s = 9,85 ‰
k_x = 0,233	1000·ϖ = 155,7

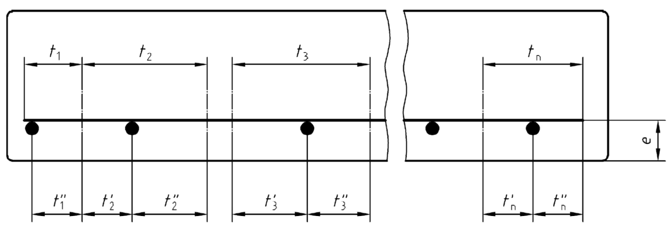
$A_S = A_c \cdot \varpi \cdot \frac{\alpha \cdot f_{ck} \cdot \gamma_S}{\gamma_c \cdot f_{yk}} = \frac{0{,}625 \cdot 0{,}162 \cdot 155{,}7}{1000} \cdot \frac{0{,}85 \cdot 3{,}5 \cdot 1{,}15}{1{,}44 \cdot 500} = 0{,}749\ cm^2$ chosen: 5 Ø 6,0 mm (A_{sl} = 1,41 cm²)	
5.3 Minimum reinforcement	
$f_{cflm} = 0{,}27 \cdot 3{,}5 = 0{,}945$ MPa $A_{ct} = b \cdot \frac{h}{2} = 62{,}5 \cdot 10{,}0 = 625\ cm^2$ $A_{s,min} = k \cdot A_{ct} \cdot f_{cflm} / f_{yk} = 0{,}4 \cdot 625 \cdot 0{,}945 / 500 = 0{,}47\ cm^2$ $A_{s,min} = 0{,}47\ cm^2 < 2{,}54\ cm^2$	EN 12602, A.3.4 (A.3)
5.4 Design for shear force Determination of reinforcement ratio: $\rho_l = \frac{A_{s,exis}}{(b \cdot d)} = \frac{2{,}54}{62{,}5 \cdot 16{,}2} = 0{,}0025 < 0{,}005$	EN 12602, A.4
Minimum design value of shear force $V_{Rd1} \geq 0{,}5 \cdot \frac{f_{ctk;0,05}}{\gamma_c} \cdot b_w \cdot d = 0{,}5 \cdot 0{,}10 \cdot 3500 / 1{,}73 \cdot 0{,}625 \cdot 0{,}162$ $= 10{,}24$ kN	EN 12602, (A.6)
Design value of shear force: $V_{Rd1} = \tau_{Rd} \cdot (1 - 0{,}83 \cdot d) \cdot (1 + 240 \cdot \rho_l) \cdot b_w \cdot d$ $= 68{,}1 \cdot (1 - 0{,}83 \cdot 0{,}162) \cdot (1 + 240 \cdot 0{,}0025) \cdot 0{,}625 \cdot 0{,}162$ $= 9{,}55$ kN Higher value is determinant (critical) : V_{Rd1} = 12,29 kN $V_{Rd1} = 10{,}24\ kN > 5{,}35 = V_{Sd1}$ Therefore, no shear reinforcement is required.	

5.5 Spacing of Longitudinal Bars Centre distance between bars : 50 mm ≤ s_{l1} ≤ 2 d Therefore, we consider the longitudinal bars at a distance of 60 mm centre to centre as per the limits. And the distance of longitudinal bars from the panel surface is supposed to be 35 mm.	EN 12602, 5.2.7.2.2

6 Anchoring of longitudinal reinforcement

EN 12602, A.10.3





Cross-Sectional View

Description reinforcement layout:

diameter cross bars: $Ø_t$ = 5,0 mm

distance between longitudinal bars: s_t = 60 mm

distance to bottom side of panel:

$e = c + Ø_{sl} + Ø_t / 2 = 35 + 6,0 + 5,0 / 2 = 43,5$ mm

Effective length of transverse anchorage bars

$t_2 = t_3 = t_4 = t_5 = t_6 = t_7 = t_8 = 2 \cdot s_t/2 = 60 \text{ mm} < 14 \cdot Ø_t = 70$ mm

$\Rightarrow t_2 = t_3 = t_4 = t_5 = t_6 = t_7 = t_8 = 60$ mm

$t_1 = t_9 = 30 + 15 \text{ mm} = 45 \text{ mm} < 14 \cdot Ø_t = 70$ mm

$\Rightarrow t_1 = t_9 = 45$ mm

$t_i' < 8 \cdot Ø_t = 40$ mm

$t_t = t_1 + t_2 + t_3 + t_4 + t_5 + t_6 + t_7 + t_8 + t_9 = 510$ mm

Maximum tensile force:

$F_{ld,max} = M_{d1,max} / z = 7,82 / (0,9 \cdot 0,162) = 53,6$ kN

$F_{ld,support} = M_{d1,support} / z = 0,96 / (0,9 \cdot 0,162) = 6,58$ kN

Assume 9 transverse cross bars with diameter 5,0 mm for half of the panel and the arrangement is shown below:

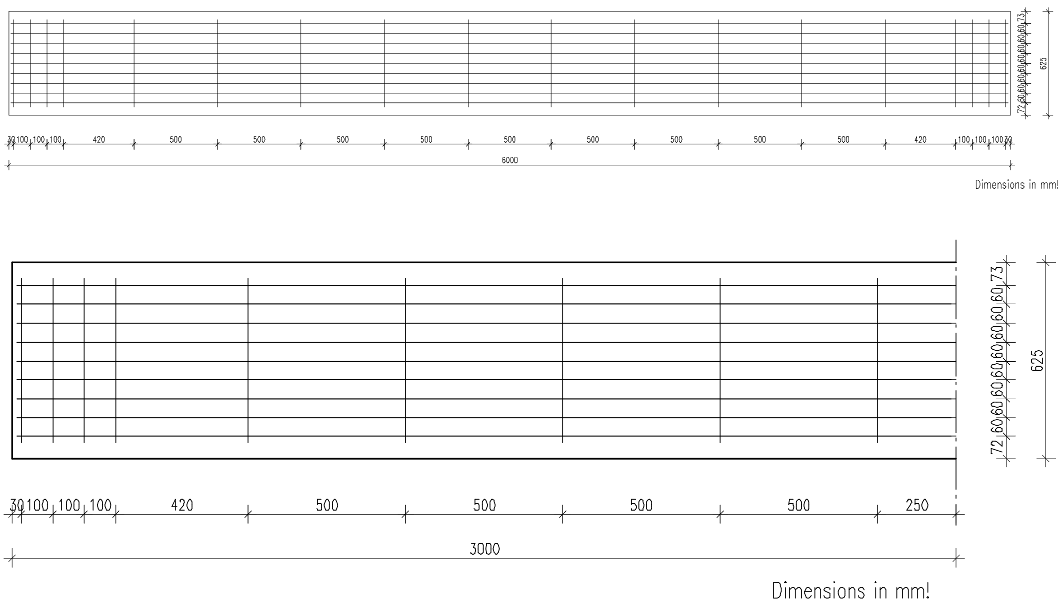


Figure 1: Reinforcement layout

Design value for bearing strength at support ($m = 1,3$; $n_p = 1$, transverse compression at support):

$$f_{ld,support} = \frac{1,35 \cdot m \cdot (e / Øt)^{1/3} \cdot \alpha \cdot f_{ck}}{\gamma_c} \leq 2,2 \cdot \frac{f_{ck}}{\gamma_c}$$

$$= \frac{1,35 \cdot 1,3 \cdot (43,5 / 5,0)^{1/3} \cdot 0,85 \cdot 3,5}{1,44} \leq 2,2 \cdot \frac{3,5}{1,44}$$

Bond Class B1

$$= 7,46 \text{ MPa} > 5,35 \text{ MPa}$$

therefore, $f_{ld,support} = 5,35$ MPa

Design value for bearing strength at middle of span ($m = 1,03$; $n_p = 1$, transverse compression at support):

$$f_{ld,field} = \frac{1,35 \cdot m \cdot (e / Øt)^{1/3} \cdot \alpha \cdot f_{ck}}{\gamma_c} \leq 2,2 \cdot \frac{f_{ck}}{\gamma_c}$$

$$= \frac{1,35 \cdot 1,03 \cdot (43,5 / 5,0)^{1/3} \cdot 0,85 \cdot 3,5}{1,73} \leq 2,2 \cdot \frac{3,5}{1,73}$$

$$= 4,92 \text{ MPa} > 4,45 \text{ MPa}$$

therefore, $f_{ld,field} = 4,45$ MPa

where, α is a reduction coefficient for long term effect on compressive strength of AAC (α = 0,85)

EN 12602, A.3.2

Anchorage force capacity (F_{RA}) per cross bar:

$$F_{RA,support} = 0{,}83 \cdot n_t \cdot Ø_t \cdot t_t \cdot f_{ld,support} \leq 0{,}6 \cdot n_l \cdot F_{wg} / \gamma_S$$
$$= 0{,}83 \cdot 1 \cdot 5{,}0 \cdot 510 \cdot 5{,}35 \leq 0{,}6 \cdot 9 \cdot 0{,}25 \cdot A_{sl} \cdot f_{yk} / \gamma_S$$
$$= 11{,}32 \text{ kN} < 16{,}60 \text{ kN}$$

Welding Strength Class S1

$$F_{RA,max} = \sum \min\left[0{,}83 \cdot \phi_{tot} \cdot t_t \cdot f_{ld}(n_t); \frac{0{,}60 \cdot n_l \cdot n_t \cdot F_{wg}}{\gamma_S}\right]$$
$$= 86{,}68 \text{ kN} < 149{,}36 \text{ kN}$$

As, $F_{RA,support} \geq F_{ld,support}$ and $F_{RA,max} \geq F_{ld,max}$

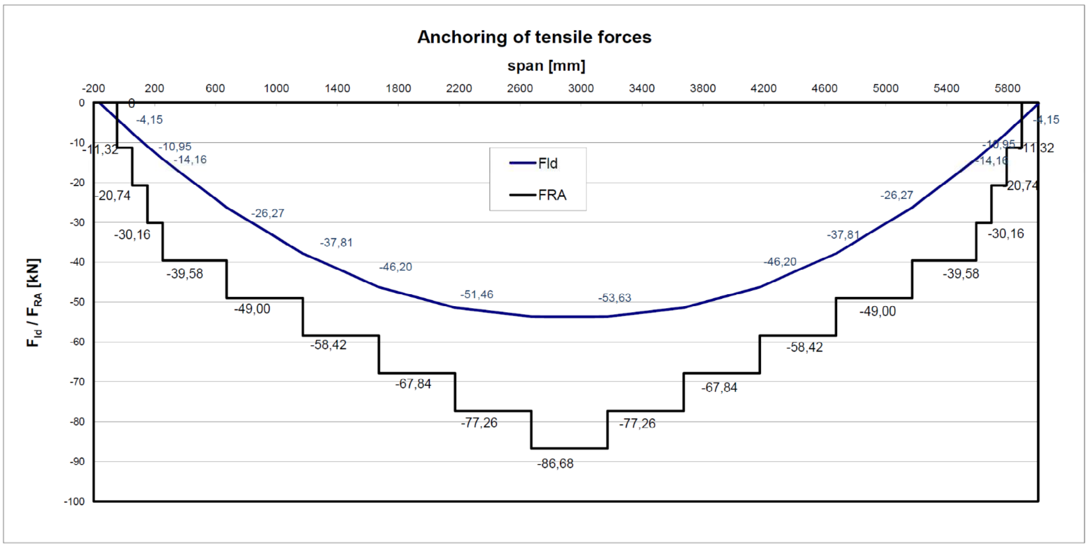


Figure 2: Anchorage of tensile forces

As we can see from fig. 2 that the anchorage capacity force does exceed the design tensile force at each section of the panel.

Therefore, the assumption is satisfied for the required conditions. So, we can use 18 cross bars with Ø 5,0 mm (for whole panel).

The sufficient anchorage has to be proven also for the upper reinforcement (according to the method above) which is not shown here.

7 Serviceability Limit States

EN 12602, A.9.4

Cracking moment, $M_{cr} = (b \cdot h^2 / 6) \cdot f_{cflm}$
$= (0{,}625 \cdot 0{,}20^2 / 6) \cdot (0{,}27 \cdot 0{,}8 \cdot 3{,}5)$
$= 3{,}15$ kNm

EN 12602, A.9.4.3 and 4.2.5

where, f_{cflm} is the flexural strength of AAC ($= 0{,}27 \cdot 0{,}8 \cdot f_{ck}$)

As, $M_f > M_{cr}$, therefore, the slab is considered to behave in a manner intermediate between uncracked and cracked condition.

7.1 Deflection under uncracked condition

7.1.1 Short-term deflection

Ratio of the modulus of elasticity of reinforcing steel and AAC:

$$n = \frac{E_s}{E_{cm}} \approx \frac{200000 \quad N/mm^2}{1750 \quad N/mm^2} = 114{,}3$$

EN 12602, (A.42)

Moment of area of AAC and reinforcement:

$$I_{c,brutto} = \frac{b \cdot h^3}{12} + n \cdot (9 \cdot \pi \cdot (ø_1/2)^4 / 4 + 5 \cdot \pi \cdot (ø_2/2)^4 / 4) = 41676{,}85 \text{ cm}^4$$

The upper longitudinal reinforcement can be fully taken into account to determine the moment of inertia. The position of the centre of gravity of the reinforcement layer is supposed to be 3,8 cm from the panel surface.

Parts of moment of inertia from consideration of the reinforcement:

$A_{s1} = 2{,}54$ cm^2
$A_{s2} = 1{,}41$ cm^2

Centre of gravity,

$$y_S = \frac{b \cdot h \cdot h/2 + n \cdot (A_{s1} \cdot y_{s1} + A_{s2} \cdot y_{s2})}{b \cdot h + n \cdot (A_{s1} + A_{s2})} = \frac{16214{,}1}{1701{,}5} = 9{,}53 \text{ cm}$$

where, y_{s1} and y_{s2} are the distances from the centre of the reinforcement steel to the bottom surface of the slab.

$$I_{ST} = b \cdot h \cdot (\frac{h}{2} - y_s)^2 + n \cdot (A_{s1} \cdot (y_{s1} - y_s)^2 + A_{s2} \cdot (y_{s2} - y_s)^2)$$
$$= 1250 \cdot (10{,}0 - 9{,}53)^2 + 114{,}3 \cdot (2{,}54 \cdot (3{,}8 - 9{,}53)^2 + 1{,}41 \cdot (16{,}2 - 9{,}53)^2)$$
$$= 16978{,}20 cm^4$$
$$E_{cm} \cdot I_{ci} = E_{cm} \cdot (I_{C;BRUTTO} + I_{st}) = 1750 \cdot (41676{,}8 + 16978{,}2) \cdot 10^{-8}$$
$$= 1{,}026 MNm^2$$

Deflection due to load combination 2 (frequent action combinations):

$$y_{el} = \frac{5}{48} \cdot \frac{M_{Sd2} \cdot L_{eff}^{\ 2}}{E_{cm} \cdot I_{ci}} = \frac{5}{48} \cdot \frac{0{,}00397 \cdot 5{,}847^2}{1{,}026} = 0{,}0138m$$

$$y_{el} = 0{,}0138m = 1{,}38cm < 2{,}34cm = \frac{L_{eff}}{250}$$

General note:
The limit value for the maximum deflection may be found in a national application document. The recommended value for the calculated deflection of roof and floor components subjected to quasi-permanent loads is (according to EN 12602) span/250.

EN 12602, 9.4.1, Note 1

7.1.2 Long-term deflection

For long term deflection an effective modulus of elasticity,

$$E_{c,eff} = E_{cm} / (1 + \varphi)$$

EN 12602, (A.43)

is used.

Therefore, $E_{c,eff}$ = 875 N/mm^2

and

$$n = \frac{E_s}{E_{c,eff}} \approx \frac{200000 \quad N/mm^2}{875 \quad N/mm^2} = 228{,}6$$

Moment of area of AAC and reinforcement,

$$I_{c,brutto} = \frac{b \cdot h^3}{12} + n \cdot (9 \cdot \pi \cdot (ø_1/2)^4 / 4 + 5 \cdot \pi \cdot (ø_2/2)^4 / 4) = 41687{,}02 \text{ cm}^4$$

Centre of gravity,

$$y_S = \frac{b \cdot h \cdot h/2 + n \cdot (A_{s1} \cdot y_{s1} + A_{s2} \cdot y_{s2})}{b \cdot h + n \cdot (A_{s1} + A_{s2})} = \frac{19928{,}1}{2153{,}0} = 9{,}26 \text{ cm}$$

Moment of inertia for reinforcement,

$$I_{ST} = b \cdot h \cdot (\frac{h}{2} - y_s)^2 + n \cdot (A_{s1} \cdot (y_{s1} - y_s)^2 + A_{s2} \cdot (y_{s2} - y_s)^2)$$

$$= 1250 \cdot (10 - 9{,}26)^2 + 228{,}6 \cdot (2{,}54 \cdot (3{,}8 - 9{,}26)^2 + 1{,}41 \cdot (16{,}2 - 9{,}26)^2)$$

$$= 33518{,}81 cm^4$$

$$E_{c,eff} \cdot I_{ci} = E_{c,eff} \cdot (I_{C;BRUTTO} + I_{st}) = 875 \cdot (41687{,}0 + 33518{,}8) \cdot 10^{-8}$$ $$= 0{,}658 MNm^2$$ Deflection due to load combination 3 (quasi-permanent combinations): $$y_\infty = \frac{5}{48} \cdot \frac{M_{Sd3} \cdot L_{eff}{}^2}{E_{c,eff} \cdot I_{ci}} = \frac{5}{48} \cdot \frac{0{,}00359 \cdot 5{,}847^2}{0{,}658} = 0{,}0194 m$$ $$y_\infty = 0{,}0194 m = 1{,}94 cm < 2{,}34 cm = \frac{L_{eff}}{250}$$	
7.2 Deflection under cracked condition	
7.2.1 Short-term deflection The ratio of the modulus of elasticity of reinforcing steel and AAC: $$n = \frac{E_s}{E_{cm}} \approx \frac{200000 \quad N/mm^2}{1750 \quad N/mm^2} = 114{,}3$$	EN 12602, (A.42)
In this case, we consider only compression zone of AAC and reinforcement for the calculation of moment of inertia. Therefore, first we will find the x-equilibrium $$x = \frac{\sqrt{1 + 4 \cdot d \cdot A} - 1}{2 \cdot A} = 8{,}47 \text{ cm}$$ where, x is height of compression zone from top surface of panel d is effective height, $A = b \cdot E_{cm} / (2 \cdot A_{s1} \cdot E_S)$ The upper longitudinal reinforcement can be fully taken into account to determine the moment of inertia. Moment of area of compression zone AAC and reinforcements, $$I_{c,brutto} = \frac{b \cdot x^3}{12} + n \cdot (9 \cdot \pi \cdot (ø_1/2)^4 / 4 + 5 \cdot \pi \cdot (ø_2/2)^4 / 4) = 3175{,}00 \text{ cm}^4$$ The position of the centre of gravity of the reinforcement layer is supposed to be 3,8 cm from the panel surface. Parts of moment of inertia from consideration of the reinforcement: $A_{s1} = 2{,}54 \text{ cm}^2$ $A_{s2} = 1{,}41 \text{ cm}^2$	

Centre of gravity is:

$$y_S = \frac{b \cdot x \cdot (h - x/2) + n \cdot (A_{s1} \cdot y_{s1} + A_{s2} \cdot y_{s2})}{b \cdot x + n \cdot (A_{s1} + A_{s2})} = \frac{12059{,}7}{980{,}9} = 12{,}29 \text{ cm}$$

where, y_{S1} and y_{S2} are the distances from the centre of the reinforcement steel to the bottom surface of the slab

$$I_{ST} = b \cdot x \cdot (h - x/2 - y_s)^2 + n \cdot (A_{s1} \cdot (y_{s1} - y_s)^2 + A_{s2} \cdot (y_{s2} - y_s)^2)$$

$$= 29782{,}85 cm^4$$

$$E_{cm} \cdot I_{ci} = E_{cm} \cdot (I_{C;BRUTTO} + I_{st}) = 1750 \cdot (3175{,}0 + 29782{,}8) \cdot 10^{-8}$$

$$E_{cm} \cdot I_{ci} = 0{,}577 MNm^2$$

Deflection due to load combination 2 (frequent action combinations):

$$y_{el} = \frac{5}{48} \cdot \frac{M_{Sd2} \cdot L_{eff}^{\;2}}{E_{cm} \cdot I_{ci}} = \frac{5}{48} \cdot \frac{0{,}00397 \cdot 5{,}847^2}{0{,}577} = 0{,}0245 m$$

$$y_{el} = 0{,}0245 m = 2{,}45 cm > 2{,}34 cm = \frac{L_{eff}}{250}$$

7.2.2 Long term deflection

For long term deflection an effective modulus of elasticity is used,

$E_{c,eff} = E_{cm} / (1 + \varphi)$

therefore, $E_{c,eff} = 875 \text{ N / mm}^2$

and

$$n = \frac{E_s}{E_{c,eff}} \approx \frac{200000 \quad N/mm^2}{875 \quad N/mm^2} = 228{,}6$$

Moment of area of AAC and reinforcement,

$$I_{c,brutto} = \frac{b \cdot x^3}{12} + n \cdot (9 \cdot \pi \cdot (\text{ø}_1/2)^4 / 4 + 5 \cdot \pi \cdot (\text{ø}_2/2)^4 / 4) = 3185{,}18 \text{ cm}^4$$

Centre of gravity,

$$y_S = \frac{b \cdot x \cdot (h - x/2) + n \cdot (A_{s1} \cdot y_{s1} + A_{s2} \cdot y_{s2})}{b \cdot x + n \cdot (A_{s1} + A_{s2})} = \frac{15773{,}7}{1432{,}3} = 11{,}01 \text{ cm}$$

Moment of inertia for reinforcement,

$I_{ST} = b \cdot x \cdot (h - \frac{x}{2} - y_s)^2 + n \cdot (A_{s1} \cdot (y_{s1} - y_s)^2 + A_{s2} \cdot (y_{s2} - y_s)^2)$ $= 50835{,}64 cm^4$ $E_{c,eff} \cdot I_{ci} = E_{c,eff} \cdot (I_{C;BRUTTO} + I_{st}) = 875 \cdot (3185{,}2 + 50835{,}6) \cdot 10^{-8}$ $= 0{,}473 MNm^2$ Deflection due to load combination 3 (quasi-permanent combinations): $y_\infty = \frac{5}{48} \cdot \frac{M_{Sd3} \cdot L_{eff}^{\;2}}{E_{c,eff} \cdot I_{ci}} = \frac{5}{48} \cdot \frac{0{,}00359 \cdot 5{,}847^2}{0{,}473} = 0{,}0270 m$ $y_\infty = 0{,}0270 m = 2{,}70 cm > 2{,}34 cm = \frac{L_{eff}}{250}$ ***7.3 Combination of deflection uncracked / cracked*** **7.3.1 Short-term deflection** The short term deflection for the intermediate situation (cracked/uncracked) due to frequent loads is: $k \cdot p_{II} + (1-k) \cdot p_I = 0{,}496 \cdot 2{,}45 + (1 - 0{,}496) \cdot 1{,}38 = 1{,}91 cm$ where $k = 1 - 0{,}8 \cdot (M_{cr} / M_{sd2})^2 = 1 - 0{,}8 \cdot (3{,}15 / 3{,}97)^2 = 0{,}496$ M_{cr}: cracking moment M_{sd2}: bending moment for frequent combination of loading p_{II}: short-term deflection for cracked condition p_I: short-term deflection for uncracked condition $y_{el} = 1{,}91 cm < 2{,}34 cm = \frac{L_{eff}}{250}$	EN 12602 (A.44)

7.3.2 Long-term deflection

By considering an effective modulus of elasticity ($E_{c,eff}$) and quasi-permanent combination of loading is:

$$k \cdot p_{II} + (1-k) \cdot p_{I} = 0{,}496 \cdot 2{,}70 + (1-0{,}496) \cdot 1{,}94 = 2{,}32 cm$$

where $k = 1-0{,}8 \cdot (M_{cr} / M_{sd2})^2 = 1-0{,}8 \cdot (3{,}15/3{,}97)^2 = 0{,}496$

M_{cr}: cracking moment

M_{sd2}: bending moment for frequent combination of loading

p_{II}: long-term deflection for cracked condition

p_{I}: long-term deflection for uncracked condition

$$y_{\infty} = 2{,}32 cm \; < \; 2{,}34 cm = \frac{L_{eff}}{250}$$

$$1000 \cdot m_d = \frac{1000 \cdot M_{Sd1} \cdot \gamma_C}{\alpha \cdot f_{ck} \cdot A_C \cdot d}$$

$$A_s = A_C \cdot \varpi \cdot \frac{\alpha \cdot f_{ck} \cdot \gamma_S}{\gamma_C \cdot f_{yk}}$$

M_{sd1}	bending moment under characteristic combination of loading (respecting transport load situations)
d	effective depth of component
A_c	cross section of AAC, $A_c = b \cdot d$
A_s	cross sectional area of reinforcement
f_{ck}	characteristic compressive strength of AAC
f_{yk}	characteristic yield strength of reinforcing steel
$\gamma_{c,ductile}$	partial safety factor of AAC for ductile failure
γ_S	partial safety factor for reinforcing steel

ε_c [‰]	ε_s [‰]	k_x	k_z	$1000 \cdot m_d$	$1000 \cdot \varpi$ stainless steel, f_{yk} = 235 MPa	steel, f_{yk} = 500 MPa
0,25	10,00	0,024	0,992	1,512	1,524	
0,50	10,00	0,048	0,984	5,858	5,952	
0,75	10,00	0,070	0,977	12,78	13,08	
1,00	10,00	0,091	0,970	22,04	22,73	
1,25	10,00	0,111	0,963	33,44	34,72	
1,50	10,00	0,130	0,957	46,79	48,91	
1,75	10,00	0,149	0,950	61,92	65,16	
2,00	10,00	0,167	0,944	78,70	83,33	
2,25	10,00	0,184	0,938	95,72	102,0	
2,50	10,00	0,200	0,931	111,7	120,0	
2,75	10,00	0,216	0,924	126,8	137,3	
3,00	10,00	0,231	0,917	141,0	153,8	
3,00	9,75	0,235	0,915	143,5	156,9	
3,00	9,50	0,240	0,913	146,1	160,0	
3,00	9,25	0,245	0,912	148,8	163,3	
3,00	9,00	0,250	0,910	151,6	166,7	
3,00	8,75	0,255	0,908	154,5	170,2	
3,00	8,50	0,261	0,906	157,5	173,9	
3,00	8,25	0,267	0,904	160,7	177,8	
3,00	8,00	0,273	0,902	163,9	181,8	
3,00	7,75	0,279	0,899	167,3	186,0	
3,00	7,50	0,286	0,897	170,8	190,5	
3,00	7,25	0,293	0,894	174,5	195,1	
3,00	7,00	0,300	0,892	178,3	200,0	
3,00	6,75	0,308	0,889	182,3	205,1	
3,00	6,50	0,316	0,886	186,5	210,5	
3,00	6,25	0,324	0,883	190,9	216,2	
3,00	6,00	0,333	0,880	195,5	222,2	
3,00	5,75	0,343	0,876	200,3	228,6	
3,00	5,50	0,353	0,873	205,3	235,3	
3,00	5,25	0,364	0,869	210,6	242,4	
3,00	5,00	0,375	0,865	216,1	250,0	
3,00	4,75	0,387	0,860	222,0	258,1	
3,00	4,50	0,400	0,856	228,1	266,7	
3,00	4,25	0,414	0,851	234,6	275,9	
3,00	4,00	0,429	0,845	241,5	285,7	
3,00	3,75	0,444	0,840	248,7	296,3	
3,00	3,50	0,462	0,833	256,4	307,7	
3,00	3,25	0,480	0,827	264,5	320,0	
3,00	3,00	0,500	0,819	273,1	333,3	
3,00	2,75	0,522	0,812	282,3	347,8	
3,00	2,50	0,545	0,803	292,0	363,6	
3,00	2,25	0,571	0,794	302,3	381,0	
3,00	2,00	0,600	0,783	313,3	400,0	434,8
3,00	1,75	0,632	0,772	325,0	421,1	523,0
3,00	1,50	0,667	0,759	337,4	444,4	644,1
3,00	1,25	0,706	0,745	350,6	470,6	818,4
3,00	1,00	0,750	0,729	364,6	510,9	1.087,0