Design of AAC wall panel according to EN 12602

Example 3: Wall panel with wind load

1.1 Issue

Design of a wall panel at an industrial building

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

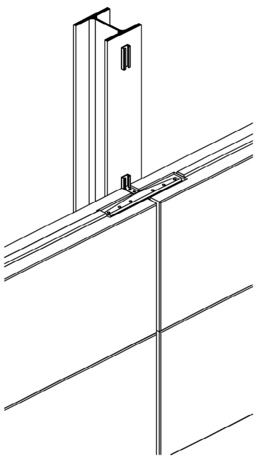
Compressive strength

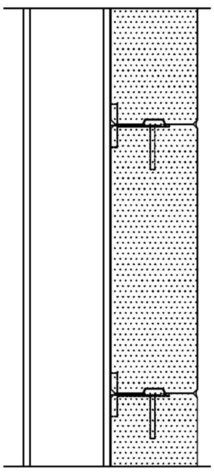
EN 12602, 4.2.4

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

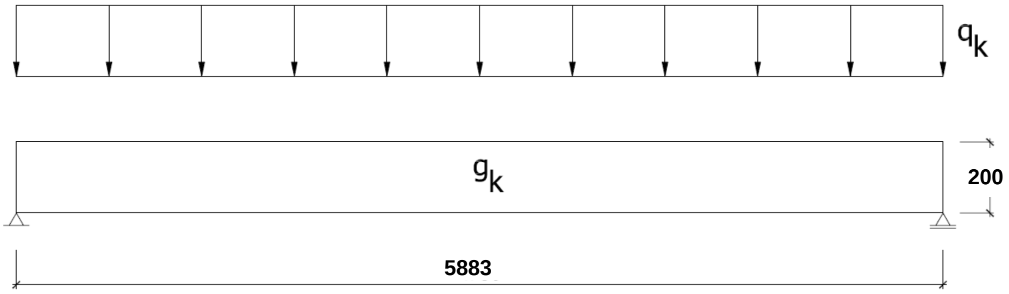


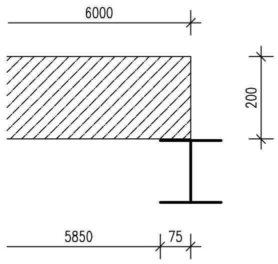


2 System and dimensions

2.1 System

Longitudinal section





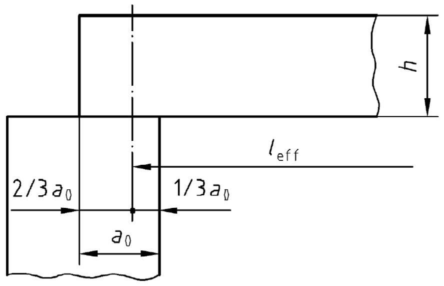
Minimum value for support length

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

EN 12602, A.11

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



$$L_{eff} = l_w + \frac{1}{3} a_{1,min} + \frac{1}{3} a_{2,min} = 5{,}85 + \frac{2}{3} \cdot 0{,}05 = 5{,}883 \text{ m}$$

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.

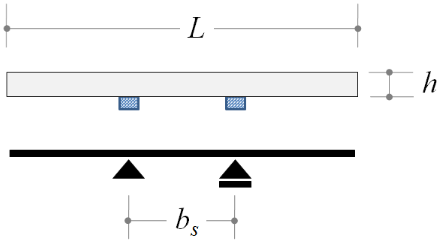


Figure 1: Transport situation fork lift truck

Assumption for distance forks: b_s = 1,00 m

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

2.2 Cross section

h = 200 mm
b = 625 mm

2.3 Concrete cover and effective depth

c_1 = 25 mm
c_2 = 25 mm

Assumption for fire resistance class: REI 60

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

$$d = 200 \text{ mm} - 25 - \frac{6}{2} = 172 \text{ mm}$$

## 3 Loads	
Self-weight of AAC element: with 35 kg / m³ steel and 6 M-% moisture content of AAC, g = 5,7 KN/m³	EN 12602, 4.2.2.4 (1)
wind load, w_e = 0,5 kN/m² (assumption)	EN 1991-1-4
Transport weight of AAC element: g_t = 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	
$Q_{d1} = \gamma_Q \cdot b \cdot q_k = 1{,}50 \cdot 0{,}625 \cdot 0{,}50 = 0{,}47$ kN/m	Load combinations acc. to EN 1990
$V_{Sd1} = \frac{Q_{d1} \cdot l_{eff}}{2} = \frac{0{,}47}{2} \cdot 5{,}883 = 1{,}38$ kN	
$M_{Sd1} = \frac{Q_{d1} \cdot l_{eff}^2}{8} = \frac{0{,}47 \cdot 5{,}883^2}{8} = 2{,}03$ kNm	
### 4.2 Internal forces for frequent combinations	
$G_{d2} = 0$ kN/m	
$Q_{d2} = \psi_1 \cdot b \cdot q_k = 0{,}2 \cdot 0{,}625 \cdot 0{,}50 = 0{,}06$ kN/m	ψ_1=0,2 (see EN 1990, Table A.1.1)
$V_{Sd2} = \frac{Q_{d2}}{2} \cdot l_{eff} = \frac{0{,}06}{2} \cdot 5{,}883 = 0{,}18$ kN	
$M_{Sd2} = \frac{Q_{d2} \cdot l_{eff}^2}{8} = \frac{0{,}06 \cdot 5{,}883^2}{8} = 0{,}26$ kNm	
### 4.3 Internal forces for quasi-permanent combinations	
$G_{d3} = 0$ kN/m $Q_{d3} = \psi_2 \cdot b \cdot q_k = 0 \cdot 0{,}625 \cdot 2{,}00 = 0$ kN/m	ψ_2=0 (see EN 1990, Table A.1.1)

V_{Sd3} = 0 kN M_{Sd3} = 0 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 $M_T = \frac{\gamma_T \cdot G_T \cdot L^2{}_{cantilever}}{2} = \frac{1{,}3 \cdot 1{,}19 \cdot 2{,}50^2}{2} = 4{,}83$ kNm where γ_T = 1,3 (assumption for dynamic coefficient due to manipulation of components, when indicated consideration of national regulations)	

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,

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

EN12602, A.4.1.2 (A.6)

Mean Modulus of Elasticity of AAC slab,
$E_{cm} = 5 \cdot (\rho_m - 150) = 1750$ N/mm²

EN 12602, 4.2.7

Characteristic 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:

$$1000 \cdot m_d = \frac{1000 \cdot M_T \cdot \gamma_c}{\alpha \cdot f_{ck} \cdot A_c \cdot d} = \frac{4{,}83 \cdot 1{,}44}{0{,}85 \cdot 3{,}5 \cdot 0{,}625 \cdot 0{,}172 \cdot 0{,}172} = 126{,}4$$

Reading from design table (see Annex A):

ε_c = 2,74 ‰	ε_s = 10,00 ‰
k_x = 0,216	$1000 \cdot \varpi$ = 136,8

$$A_s = A_c \cdot \varpi \cdot \frac{\alpha \cdot f_{ck} \cdot \gamma_S}{\gamma_c \cdot f_{yk}} = \frac{0{,}625 \cdot 0{,}172 \cdot 136{,}8}{1000} \cdot \frac{0{,}85 \cdot 3{,}5 \cdot 1{,}15}{1{,}44 \cdot 500} = 0{,}699 \text{ cm}^2$$

chosen: 2 layers with 4 Ø 6,0 mm each (A_{sl} = 1,13 cm²)

5.3 Minimum reinforcement

EN 12602 A.3.4 (A.3)

$f_{cflm} = 0{,}27 \cdot 3{,}5 = 0{,}945$ MPa

$$A_{ct} = b \cdot \frac{h}{2} = 62{,}5 \cdot 10{,}0 = 625 \text{ cm}^2$$

$A_{s,min} = k \cdot A_{ct} \cdot f_{cflm} / f_{yk} = 0{,}4 \cdot 625 \cdot 0{,}945 / 500 = 0{,}47$ cm²

$A_{s,min}$ = 0,47 cm² < 1,13 cm²

5.4 Design for shear force

EN 12602, A.4

Determination of reinforcement ratio:

$$\rho_l = \frac{A_{s,exis}}{(b \cdot d)} = \frac{1{,}13}{62{,}5 \cdot 17{,}2} = 0{,}0011 < 0{,}005$$

Minimum design value of shear force

EN 12602, (A.6)

$$V_{Rd1} \geq 0{,}5 \cdot \frac{f_{ctk;0,05}}{\gamma_c} \cdot b_w \cdot d = 0{,}5 \cdot 0{,}10 \cdot 3500 / 1{,}73 \cdot 0{,}625 \cdot 0{,}172$$

$$= 10{,}87 \text{ kN}$$

Design value of shear force:

$$V_{Rd1} = \tau_{Rd} \cdot (1 - 0{,}83 \cdot d) \cdot (1 + 240 \cdot \rho_l) \cdot b_w \cdot d$$

$$= 68{,}1 \cdot (1 - 0{,}83 \cdot 0{,}172) \cdot (1 + 240 \cdot 0{,}0011) \cdot 0{,}625 \cdot 0{,}172$$

$$= 7{,}93 \text{ kN}$$

Higher value is determinant (critical) : $V_{Rd1} = 13{,}05$ kN

$V_{Rd1} = 10{,}87$ kN $> 3{,}87 = V_T$

Therefore, no shear reinforcement is required.

5.5 Spacing of Longitudinal Bars

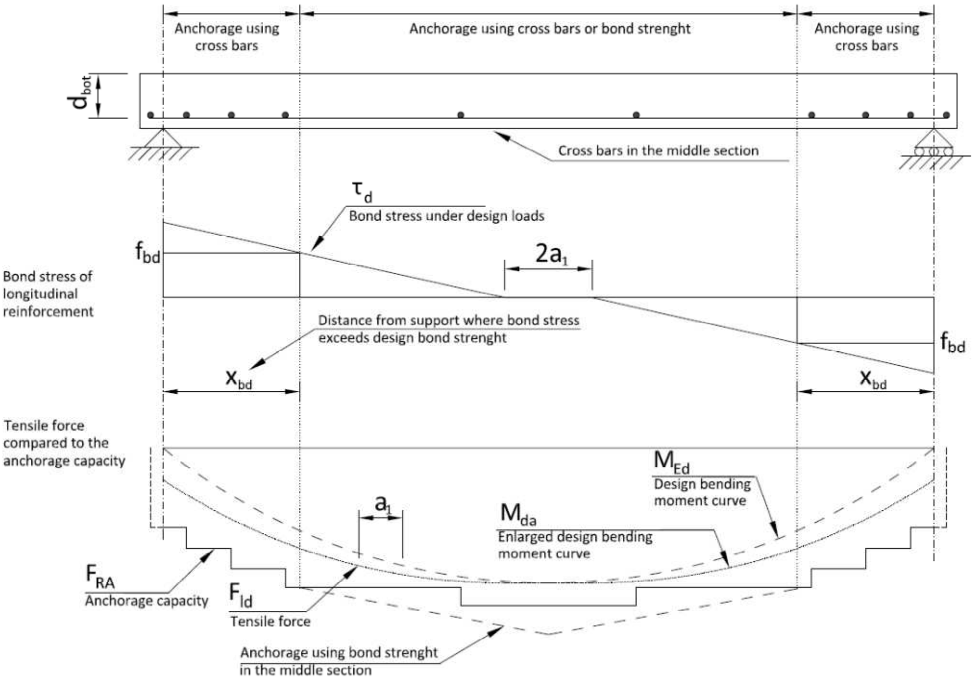
EN 12602, 5.2.7.2.2

Centre distance between bars : 50 Ø ≤ s_{l1} ≤ 700

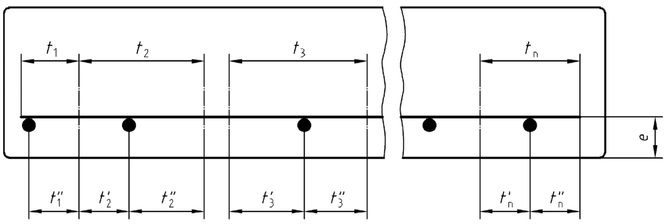
Therefore, we consider the longitudinal bars at a distance of 125 mm centre to centre as per the limits.
And the distance of longitudinal bars from the panel surface is supposed to be 125 mm.

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: $s_t = 125$ mm

distance to bottom side of panel:

$e = c + Ø_{sl} + Ø_t / 2 = 25 + 6{,}0 + 5{,}0 / 2 = 33{,}5$ mm

Effective length of transverse anchorage bars

$t_2 = t_3 = 2 \cdot s_t/2 = 125 \text{ mm} > 14 \cdot Ø_t = 70 \text{ mm} \Rightarrow t_2 = t_3 = 70$ mm

$t_1' = t_4'' = 15 \text{ mm} < 8 \cdot Ø_t = 40 \text{ mm} \Rightarrow t_1' = t_4'' = 15$ mm

$t_1'' = t_4' = s_t/2 = 62{,}5 \text{ mm} > 8 \cdot Ø_t = 40 \text{ mm} \Rightarrow t_1'' = t_4' = 40$ mm

$t_1 = t_4 = 40 + 15 \text{ mm} = 55 \text{ mm} > 8 \cdot Ø_t = 40 \text{ mm} \Rightarrow t_1 = t_4 = 40$ mm

$t_t = t_1 + t_2 + t_3 + t_4 = 180$ mm

Maximum tensile force:

$F_{ld,max} = M_{T,max} / z = 4{,}83 / (0{,}9 \cdot 0{,}172) = 31{,}20$ kN

$F_{ld,support} = M_{T,support} / z = 0{,}96 / (0{,}9 \cdot 0{,}172) = 6{,}58$ kN

Assume 8 transverse cross bars with diameter 5,0 mm for half of the panel and the arrangement is shown below:

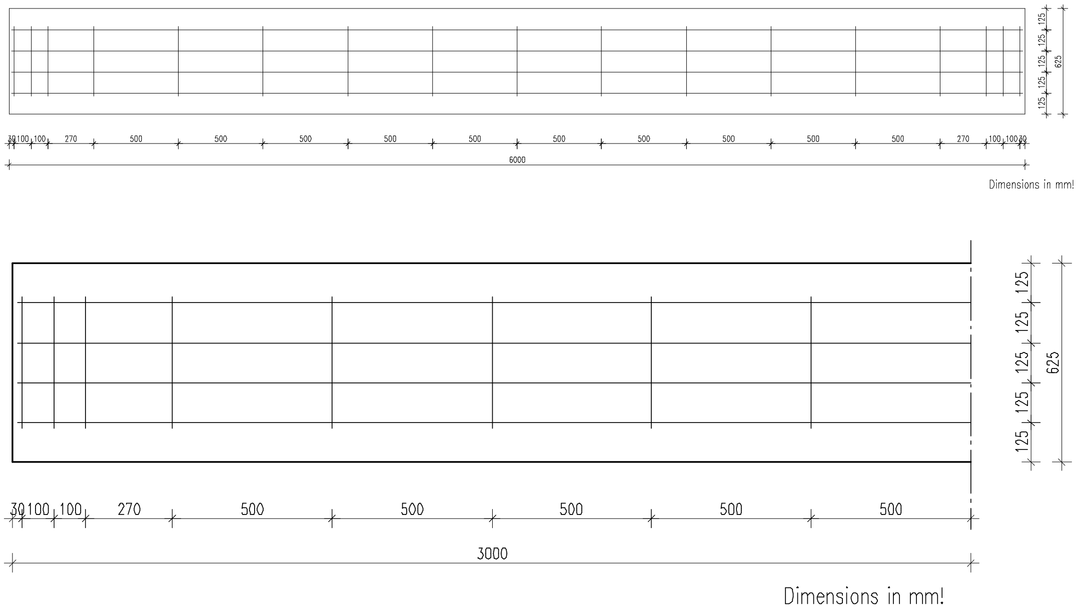


Figure 1: Reinforcement layout

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

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

$$= \frac{1,35 \cdot 1,3 \cdot (33,5 / 5,0)^{1/3} \cdot 0,85 \cdot 3,5}{1,44} \leq 2,2 \cdot \frac{3,5}{1,44}$$

Bond Class B1

$$= 6,84 \text{ MPa} > 5,35 \text{ MPa}$$

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

Design value for bearing strength at middle of span ($m = 1,04$; $n_p = 1$, transverse compression at support):

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

$$= \frac{1,35 \cdot 1,04 \cdot (33,5 / 5,0)^{1/3} \cdot 0,85 \cdot 3,5}{1,73} \leq 2,2 \cdot \frac{3,5}{1,73}$$

$$= 4,55 \text{ MPa} > 4,45 \text{ MPa}$$

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:

$$F_{RA,support} = 0{,}83 \cdot n_t \cdot Ø_t \cdot t_t \cdot f_{ld,support} \le 0{,}6 \cdot n_l \cdot F_{wg} / \gamma_s$$
$$= 0{,}83 \cdot 1 \cdot 5{,}0 \cdot 180 \cdot 5{,}35 \le 0{,}6 \cdot 4 \cdot 0{,}25 \cdot A_{sl} \cdot f_{yk} / \gamma_s$$
$$= 4{,}00 \text{ kN} < 7{,}38 \text{ kN}$$

$$F_{RA,max} = \sum \min\left[0{,}83 \cdot \phi_{tot} \cdot t_t \cdot f_{ld}(n_t); \frac{0{,}60 \cdot n_l \cdot n_t \cdot F_{wg}}{\gamma_S}\right]$$
$$= 43{,}93 \text{ kN} < 59{,}01 \text{ kN}$$

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

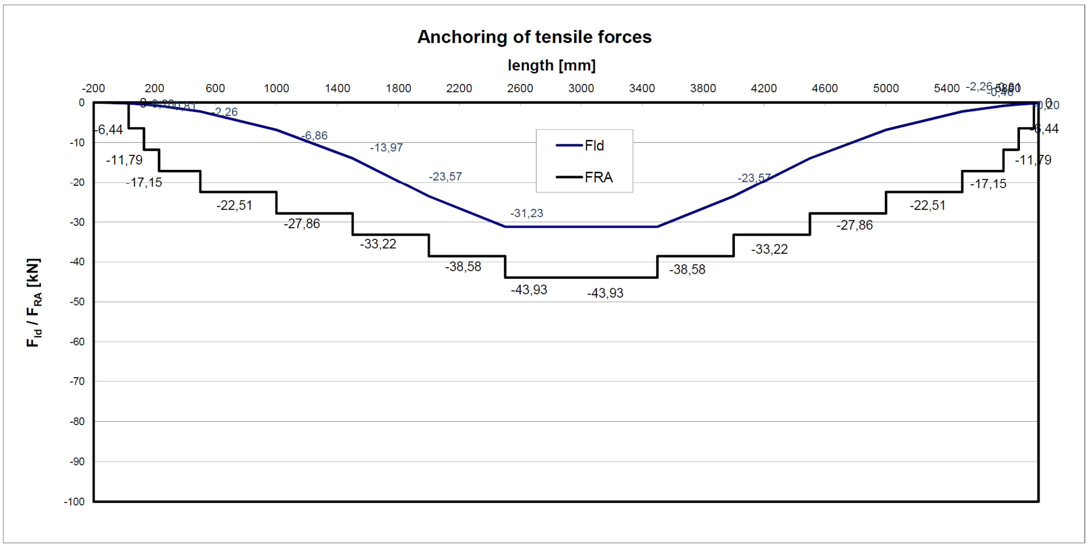


Figure 2: Anchorage of tensile forces

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 16 cross bars with Ø 5,0 mm (for whole panel).

Welding Strength Class S1

# 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, $M_{Sd2} < M_{cr}$, therefore, the slab is considered to be uncracked condition.	
## *7.1 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: $I_{c,brutto} = \frac{b \cdot h^3}{12} + n \cdot (4 \cdot \pi \cdot (ø_1/2)^4 / 4 + 4 \cdot \pi \cdot (ø_2/2)^4 / 4) = 41672{,}48 \text{ cm}^4$	
Both layers of the longitudinal reinforcement can be fully taken into account to determine the moment of inertia.	
Parts of moment of inertia from consideration of the reinforcement: $A_{s1} = A_{s2} = 1{,}13 \text{ cm}^2$	
As the reinforcement layout is symmetrical the centre of gravity is	
$y_s = 10{,}00$ cm	
$I_{ST} = b \cdot h \cdot (\frac{h}{2} - y_s)^2 + n \cdot (A_{s1} \cdot (y_{s1} - y_s)^2 + A_{s2} \cdot (y_{s2} - y_s)^2)$ $= 1250 \cdot (10{,}0 - 10{,}0)^2 + 114{,}3 \cdot (1{,}13 \cdot (2{,}8 - 10{,}0)^2 + 1{,}13 \cdot (17{,}2 - 10{,}0)^2)$ $= 13391{,}21 cm^4$ $E_{cm} \cdot I_{ci} = E_{cm} \cdot (I_{C;BRUTTO} + I_{st}) = 1750 \cdot (41672{,}5 + 13391{,}2) \cdot 10^{-8}$ $= 0{,}964 MNm^2$	
Deflection due to load combination 2 (frequent action combinations):	
$y_{el} = \frac{5}{48} \cdot \frac{M_{Sd2} \cdot L_{eff}^{\;2}}{E_{cm} \cdot I_{ci}} = \frac{5}{48} \cdot \frac{0{,}00026 \cdot 5{,}883^2}{0{,}964} = 0{,}0010 m$ $y_{el} = 0{,}0010 m = 0{,}10 cm < 2{,}35 cm = \frac{L_{eff}}{250}$	

General note: The limit value for the maximum deflection may be found in a national application document. ***7.2 Long-term deflection*** No long term effects!	

Annex A

$$1000 \cdot m_d = \frac{1000 \cdot M_{Sd1} \cdot \gamma_c}{\alpha \cdot f_{ck} \cdot A_c \cdot d}$$

$$A_s = A_c \cdot \varpi \cdot \frac{\alpha \cdot f_{ck} \cdot \gamma_S}{\gamma_c \cdot f_{yk}}$$

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