Assignment 1

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Geometry: $R = 4 m$

$a = 5,4 m$

$h = 3,5 m$

$n = 5$

Materials: concrete $C25/30 \Rightarrow 25 MPa$

steel $B500B \Rightarrow f_{yk} = 500 MPa$

Loads: $(g-g_0)_{floor,k} = 1,4 \ kN/m^2$

$(g-g_0)_{roof,k} = 1,4 \ kN/m^2$

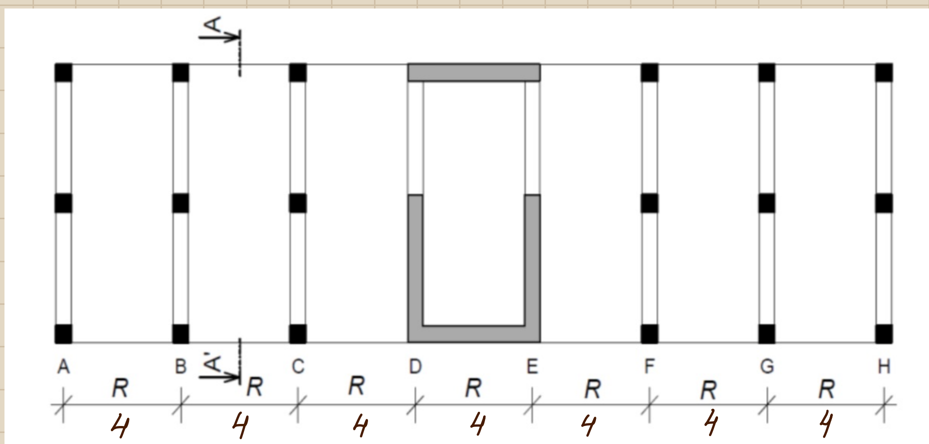
$q_{floor,k} = 4,9 \ kN/m^2$

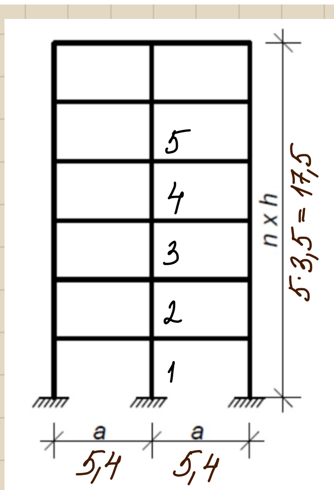
$q_{roof,k} = 0,75 \ kN/m^2$

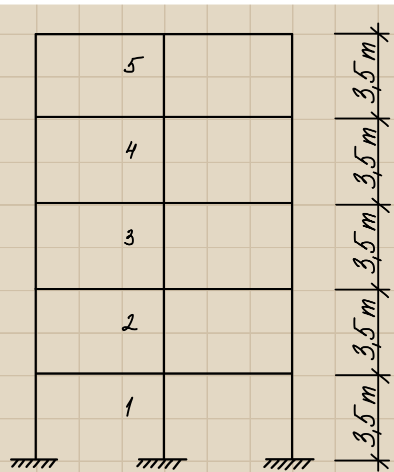
$g_{0,k}$ – calculate

Other: exposure class XC1 => carbonation

working life – 80 years







To find: depth of slab
beam dimensions (profile)
column dimensions

Slab

$L_s = 4m;\ h_s = \left(\frac{1}{30} \sim \frac{1}{25}\right) L_s = (133{,}3\ mm \sim 160\ mm) \Rightarrow$

$\Rightarrow$ design 150 mm

1) Concrete cover:

$C_{nom} = C_{min} + \Delta C_{dev}$

$C_{min} = \max(C_{min,b};\ C_{min,dur};\ 10\ mm)$

$C_{min,b} = 12\ mm$ (since we take rebars $\varnothing = 12\ mm$)

$C_{min,dur}$ determination:

default class S4

working life 80 years $\Rightarrow$ S5

concrete class C25/30 $\Rightarrow$ S4

slab $\Rightarrow$ S3

structural class S3 + exposure class XC1 $\Rightarrow C_{min,dur} = 10mm$

$C_{min} = \max(12mm;\ 10mm;\ 10mm) \Rightarrow 12\ mm$

$\Delta C_{dev} = 10mm$

$C_{nom} = 12 + 10 = 22\ mm$

$C \geq C_{nom} \Rightarrow$ design $C = 25\ mm$

2) Effective depth of slab (one-way)

$d_s = h_s - C - \frac{\varnothing}{2} = 150 - 25 - 6 = 119\ mm$

3) Assessment of the slab depth

We must verify that $\lambda \leq \lambda_d$

$$\lambda = \frac{L_s}{d_s} = \frac{4000}{119} \approx 33{,}61$$

$$\lambda_d = K_{c1} \cdot K_{c2} \cdot K_{c3} \cdot \lambda_{d,tab}$$

rectangular shape => $K_{c1} = 1$

$K_{c2} = \min(1, 7/L_s)$ => $K_{c2} = 1$

for K_{c3} we need to know $A_{s,req}$, and we can't obtain it without M_{Ed}, so I will take value from the presentation

$K_{c3} = 1{,}2$

concrete class 25/30 => $\lambda_{d,tab} = 24{,}1$

$$\lambda_d = 1 \cdot 1 \cdot 1{,}2 \cdot 24{,}1 = 28{,}92$$

$\lambda \nleq \lambda_d$ check failed!

checking by how much the condition was unsatisfied:

33,61 – 100%

28,92 – x%

$$x = \frac{28{,}92 \cdot 100}{33{,}61} \approx 86\%$$

condition unsatisfied by 14% => keeping the dimensions

<u>Beam</u>

$L_b = 5{,}4\,m$

$$h_b = \left(\frac{1}{15} \sim \frac{1}{12}\right) L_b = (360\,mm \sim 450\,mm)$$

$h_b \geq 2{,}5 h_s = 2{,}5 \cdot 150 = 375\,mm$ + multiple of 50

=> $h_b = 450\,mm$

$$b_b = \left(\frac{1}{3} \sim \frac{2}{3}\right) h_b = (133{,}3\,mm \sim 266{,}6\,mm)$$

multiple of 50

=> $b_b = 250\,mm$

$(g-g_0)_{floor,k} = 1{,}4\ kN/m^2$
$(g-g_0)_{roof,k} = 1{,}4\ kN/m^2$
$q_{floor,k} = 4{,}9\ kN/m^2$
$q_{roof,k} = 0{,}75\ kN/m^2$
$g_{0,k}$ – calculate

1) Load calculation

Slab loads:

$f_{s,d} = 14{,}3\ kN/m^2$

Load type	k-value [kN/m²]	γ	design value [kN/m²]
Self-weight	$25\ kN/m^3 \cdot 0{,}15m =$ 3,75	1,35	5,06
Dead load	1,4	1,35	1,89
Live load	4,9	1,5	7,35
Total			14,3

Roof loads:

$f_{r,d} = 8{,}08\ kN/m^2$

Load type	k-value [kN/m²]	γ	design value [kN/m²]
Self-weight	$25\ kN/m^3 \cdot 0{,}15m =$ 3,75	1,35	5,06
Dead load	1,4	1,35	1,89
Live load	0,75	1,5	1,125
Total			8,08

Beam self-weight:

$f_{b,d} = (0{,}45 - 0{,}15) \cdot 0{,}25 \cdot 25 \cdot 1{,}35 = 2{,}53\ kN/m$

To obtain the load for typical floor/roof in kN/m (per one meter of beam) we need to multiply previously calculated loads by the span of the slab:

$f_f = 14{,}3 \cdot 4 = 57{,}2\ kN/m$ – load of typical floor slab per 1m

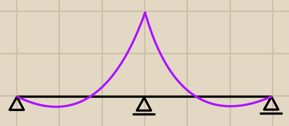
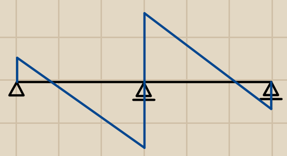
$f_r = 8{,}08 \cdot 4 = 32{,}32\ kN/m$ – load of roof slab per 1m

And we need to add beam self-weight to both values above to get the complete loading of typical floor/roof:

$f_f = 57{,}2 + 2{,}53 = 59{,}73\ kN/m$

$f_r = 32{,}32 + 2{,}53 = 34{,}85\ kN/m$

2) Estimated theoretical max. values – typical floor beam

$M_{Ed,max} = \frac{1}{8} \cdot f_f \cdot L_b^2 = \frac{1}{8} \cdot 59{,}73 \cdot 5{,}4^2 =$
$= 217{,}72 \ kNm$

$V_{Ed,max} = \frac{5}{8} \cdot f_f \cdot L_b = \frac{5}{8} \cdot 59{,}73 \cdot 5{,}4 =$
$= 201{,}59 \ kN$

3) Preliminary check of bending – typical floor

Relative bending moment: $\mu = \frac{M_{Ed,max}}{b_b \cdot d_b^2 \cdot f_{cd}}$

3.1) Effective depth of the beam d_b

To get the effective depth of the beam, we need to calculate the concrete cover for the beam and choose rebars and stirrup diameters.

Rebars – assume 16 to 22 mm => 16 mm

Stirrups – I will preliminarily assume 10 mm

Concrete cover:

$C_{nom} = C_{min} + \Delta C_{dev}$

$C_{min} = \max(C_{min,b}; C_{min,dur}; 10 \ mm)$

$C_{min,b} = 16 \ mm$ (since we take rebars $\varnothing = 16 \ mm$)

$C_{min,dur}$ determination:

default class S4

working life 80 years => S5

concrete class C25/30 => S4

no slab geometry => class remains S4

structural class S4 + exposure class XC1 => $C_{min,dur} = 15 \ mm$

$C_{min} = \max(16 \ mm; 15 \ mm; 10 \ mm) => 16 \ mm$

$\Delta C_{dev} = 10 mm$

$C_{nom} = 16 + 10 = 26 mm$

$C \geqslant C_{nom} \Rightarrow$ design $C = 30 mm$

Now that we have diameters of rebars and stirrups and have calculated the concrete cover, we can obtain d_b:

$$d_b = h_b - C - \varnothing_{sw} - \frac{\varnothing}{2} = 450 - 30 - 10 - 8 = 402 mm$$

Relative bending moment calculation:

$$\mu = \frac{M_{Ed,max}}{b_b \cdot d_b^2 \cdot f_{cd}} \qquad f_{cd} = \frac{f_{ck}}{\gamma_c} = \frac{25 MPa}{1,5} = 16,67 MPa$$

$$\mu = \frac{217,72}{0,2 \cdot 0,402^2 \cdot 16,67 \cdot 10^3} = 0,22$$

for $\mu = 0,22 \Rightarrow \xi = 0,315$

$0,315 < 0,4 \Rightarrow$ design is OK

4) Preliminary check of reinforcement ratio

Must satisfy the condition: $\rho_{s,rqd} \leqslant 0,04$

$$\rho_{s,rqd} = \frac{A_{s,rqd}}{A_c} = \frac{\frac{M_{Ed,max}}{\zeta \cdot d_b \cdot f_{yd}}}{b_b \cdot d_b}$$

from the table we get that for $\mu = 0,22$ the value of $\zeta = 0,874$

$$f_{yk} = 500 MPa; \; f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{500}{1,15} = 434,78 MPa$$

$$\rho_{s,rqd} = \frac{\frac{217,72}{0,874 \cdot 0,402 \cdot 434,78 \cdot 10^3}}{0,25 \cdot 0,402} = 0,009908$$

$0,009908 < 0,04 \Rightarrow$ condition is satisfied, no need for change

5) Preliminary check of load-bearing capacity in shear

This condition must be satisfied: $V_{Rd,max} \geqslant V_{Ed,max}$

$$V_{Rd,max} = \nu \cdot f_{cd} \cdot b_b \cdot \zeta\, d_b \cdot \frac{ctg\theta}{1+ctg^2\theta}$$

We assume $ctg\theta = 1{,}5$

$\zeta = 0{,}874$

$$\nu = 0{,}6\left(1-\frac{f_{ck}}{250}\right) = 0{,}6\left(1-\frac{25}{250}\right) = 0{,}54$$

$$V_{Rd,max} = 0{,}54 \cdot 16{,}67 \cdot 10^3 \cdot 0{,}25 \cdot 0{,}874 \cdot 0{,}402 \cdot \frac{1{,}5}{1+1{,}5^2} = 364{,}93\ kN$$

$364{,}93 > 201{,}59$ => condition is satisfied, no need for change

6) Preliminary check of deflection

We must verify that $\lambda \le \lambda_d$

$$\lambda = \frac{L_b}{d_b} = \frac{5400}{402} \approx 13{,}43$$

$\lambda_d = K_{c1} \cdot K_{c2} \cdot K_{c3} \cdot \lambda_{d,tab}$

rectangular shape => $K_{c1} = 1$

$K_{c2} = \min\left(1, 7/L_b\right)$ => $K_{c2} = 1$

for K_{c3} we need to know $A_{s,prov}$, and we don't know it yet, so I will take the value from the presentation

$K_{c3} = 1{,}2$

To get $\lambda_{d,tab}$:

$\rho_{s,rqd} = 0{,}009908 \approx 0{,}01 = 1\%$

concrete class C25/30 => we must interpolate between these values (from table for simply supported beam):

0,5% – 18,5

1,5% – 13,5

$18{,}5 - 13{,}5 = 5$; $\frac{5}{2} = 2{,}5$ => $1\% = 13{,}5 + 2{,}5 = 16$

$\lambda_d = 1 \cdot 1 \cdot 1{,}2 \cdot 16 = 19{,}2$

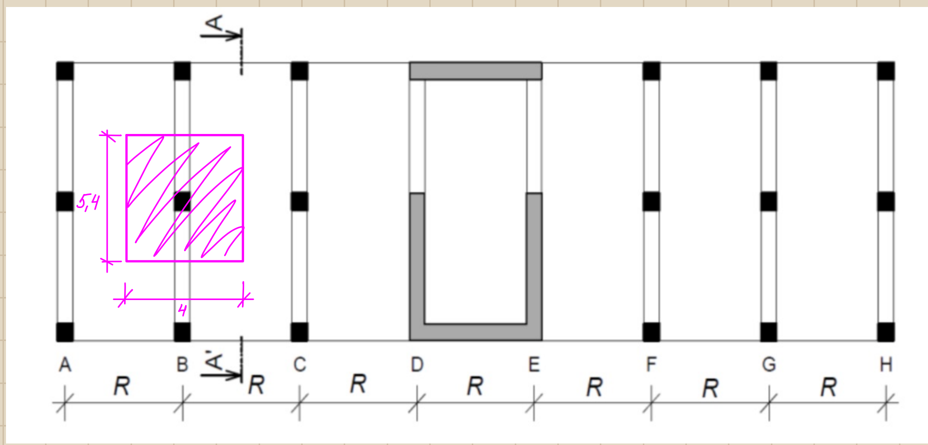
$13,43 < 19,2 \Rightarrow$ condition is fullfilled, no further changes

Column

To obtain the loading of the column, we must multiply previously obtained loads by the length of the beam (so far, the loads were taken in kN/m for beam calculation, but for the column they must be taken in kN for the whole tributing area).

$F_f = 59,73 \cdot 5,4 = 225,34$ kN – load on the regular column

$F_r = 34,85 \cdot 5,4 = 188,19$ kN – load on the column holding the roof



We must also consider the column self-weight. Since the floor height is the vertical distance between floor surfaces of adjacent floors, so the height of the column would be:

$L_c = h - h_b = 3,5 - 0,45 = 3,05$ m

Assume the column dimensions 300x300mm. Column self-weight:

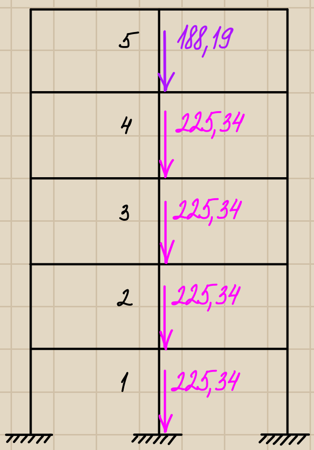
$F_c = 0,3 \cdot 0,3 \cdot 3,05 \cdot 25 \cdot 1,35 = 9,26$ kN

Load at the foot N_{Ed}:

$N_{Ed} = F_r + 4 \cdot F_f + 5 \cdot F_c = 188,19 + 4 \cdot 225,34 + 5 \cdot 9,26 = 1135,85$ kN

The design load must satisfy the condition: $N_{Rd} \geq N_{Ed}$

$N_{Rd} = 0,8 A_c \cdot f_{cd} + A_s \cdot \sigma_s$



We suppose that reinforcement area $A_s = 0{,}02 A_c$. Substitute this value in the above equation:

$$0{,}8 A_c \cdot f_{cd} + 0{,}02 A_c \cdot \sigma_s \geq N_{Ed}$$

We can derive condition for column cross-sectional area:

$$A_c \geq \frac{N_{Ed}}{0{,}8 f_{cd} + 0{,}02 \sigma_s} \Rightarrow A_c \geq \frac{1135{,}85}{0{,}8 \cdot 16{,}67 \cdot 10^3 + 0{,}02 \cdot 400 \cdot 10^3}$$

$$A_c \geq 0{,}053 \ m^2$$

For column $0{,}3m \times 0{,}3m$ the area is $0{,}09 \ m^2$, which is significantly on the safer side. Let's try to decrease the dimensions to $0{,}25m \times 0{,}25m$.

Weight of the column is then:

$$F_c = 0{,}25 \cdot 0{,}25 \cdot 3{,}05 \cdot 25 \cdot 1{,}35 = 6{,}43 \ kN$$

$$N_{Ed} = F_r + 4 \cdot F_f + 5 \cdot F_c = 188{,}19 + 4 \cdot 225{,}34 + 5 \cdot 6{,}43 = 1121{,}72 \ kN$$

$$A_c \geq \frac{1121{,}72}{0{,}8 \cdot 16{,}67 \cdot 10^3 + 0{,}02 \cdot 400 \cdot 10^3} \Rightarrow A_c \geq 0{,}053 \ m^2$$

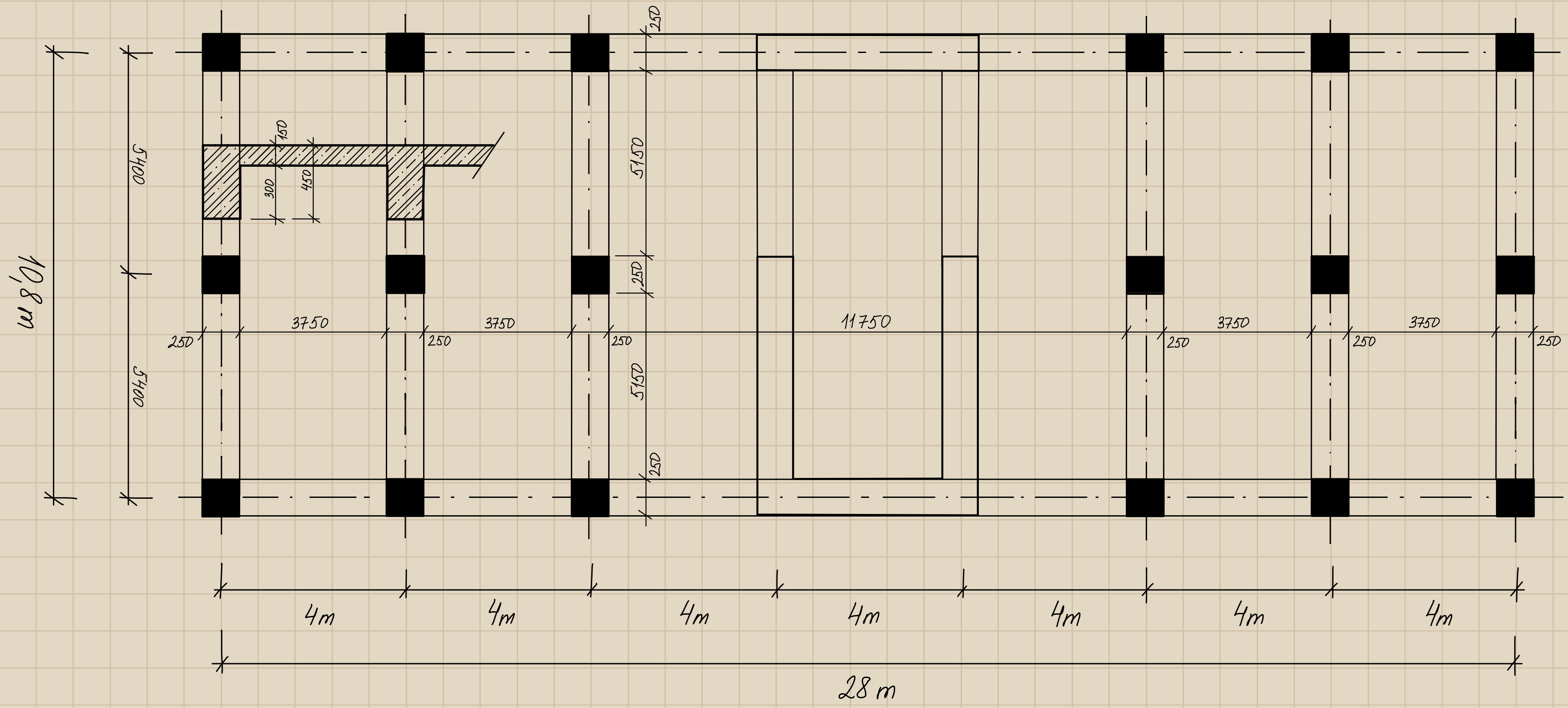
$$0{,}25 \cdot 0{,}25 = 0{,}0625 \ m^2 > 0{,}053 \ m^2$$

Now let's check if this column would produce enough resistance:

$$N_{Rd} = 0{,}8 A_c \cdot f_{cd} + 0{,}02 A_c \cdot \sigma_s$$

$$N_{Rd} = 0{,}8 \cdot 0{,}0625 \cdot 16{,}67 \cdot 10^3 + 0{,}02 \cdot 0{,}0625 \cdot 400 \cdot 10^3 = 1333{,}5 \ kN$$

$1333{,}5 > 1121{,}72 \Rightarrow$ satisfied, can keep the dimensions



10,8 m
5400
5400
150
300
450
5150
5150
250
250
250
3750
3750
3750
3750
11750
4m
4m
4m
4m
4m
4m
4m
28 m