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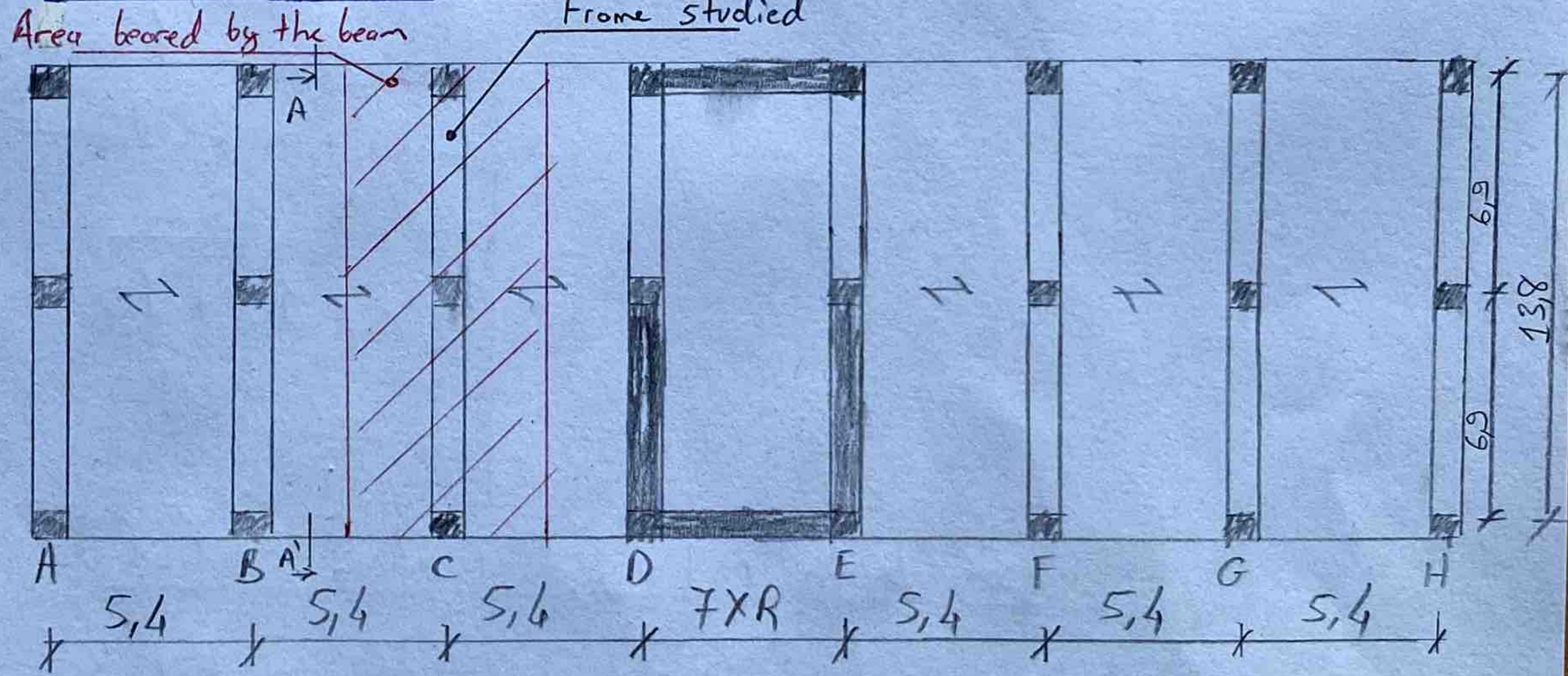
Task 1: Frame structure

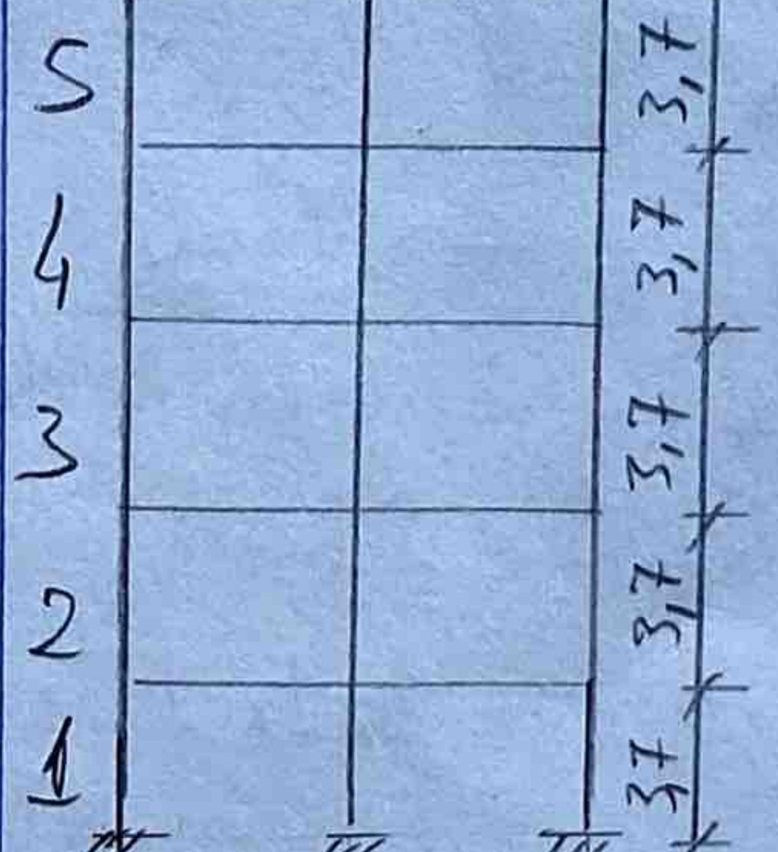
Input data:

R	5,4 m
a	6,9 m
h	3,7 m
n	5
(g-go) floor, k	1,6 kN/m²
(g-go) roof, k	1,8 kN/m²
q floor, k	3,9 kN/m²
q roof, k	0,75 kN/m²

P	XC2
Z	50
Concrete class	C30/37



Section A-A'



1. Preliminary design of the dimensions of the structure (slop depth, dimensions of the beam, dimensions of the column)

1.1. Slab

Emprical estimation

l: span of the slab

$$h_s = \left(\frac{1}{30} \sim \frac{1}{25}\right) l$$

$$h_s = \left(\frac{1}{30} \sim \frac{1}{25}\right) \cdot 5{,}4 = (0{,}18 \sim 0{,}216)$$

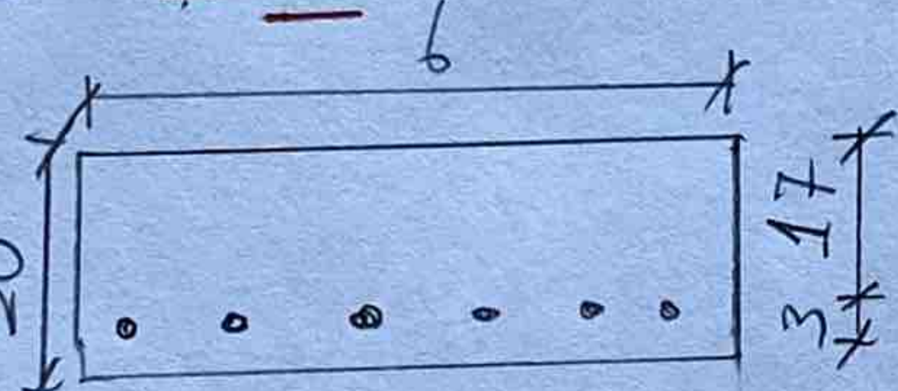
We choose 20 cm of depth for preliminary design

Effective depth

$$d = h_s - c - \frac{\phi}{2}$$

$$d = 200 - 25 - \frac{10}{2}$$

$$d = 170 \text{ mm}$$



$c = c_{min} + \Delta c_{dev}$ ①

$\Delta c_{dev} = 10$ mm

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

$c_{min,b} = 10$ mm

$c_{min,dur} \longrightarrow$ Table

According to table:

$c_{min,dur} = 15$ mm

$c_{min} = \max(10\text{mm};\ 15\text{mm};\ 10\text{ mm})$

$c_{min} = 15$ mm

① $c = 15 + 10 = 25$ mm

Span/depth ratio (deflection control)

$$\lambda = \frac{l}{d} \overset{?}{\leq} \lambda_{lim}$$

$$\lambda = \frac{5{,}4}{0{,}17} \approx 31{,}8$$

The condition isn't satisfied.
So, we increase the depth of the slab: $h_s = 21$ cm

$$d_1 = h_s - c - \frac{\phi}{2} = 210 - 25 - \frac{10}{2} = 180 \text{ mm}$$

$$\lambda_1 = \frac{l}{d_1} = \frac{5{,}4}{0{,}18} = 30$$

$\lambda_{lim} = k_{c1} \cdot k_{c2} \cdot k_{c3} \cdot \lambda_{d,tab}$ ②

$k_{c1} = 1$

$k_{c2} = 1$

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

$\lambda_{d,tab} \longrightarrow$ Table

According to table (outer span; C30/37)

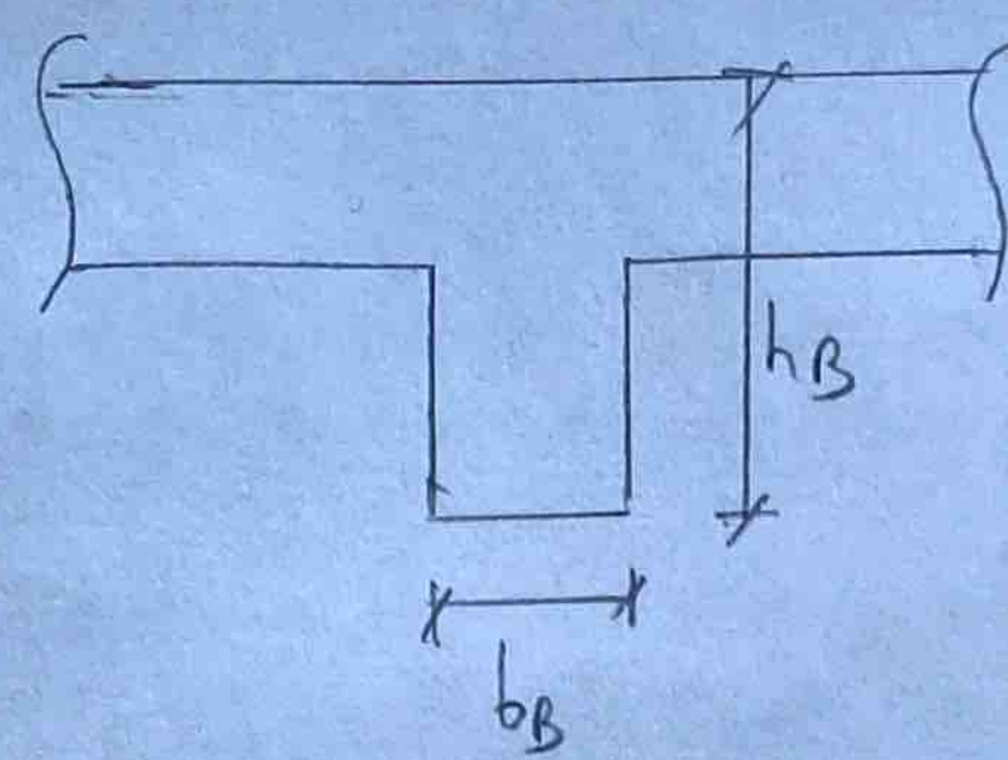
$\lambda_{d,tab} = 26$

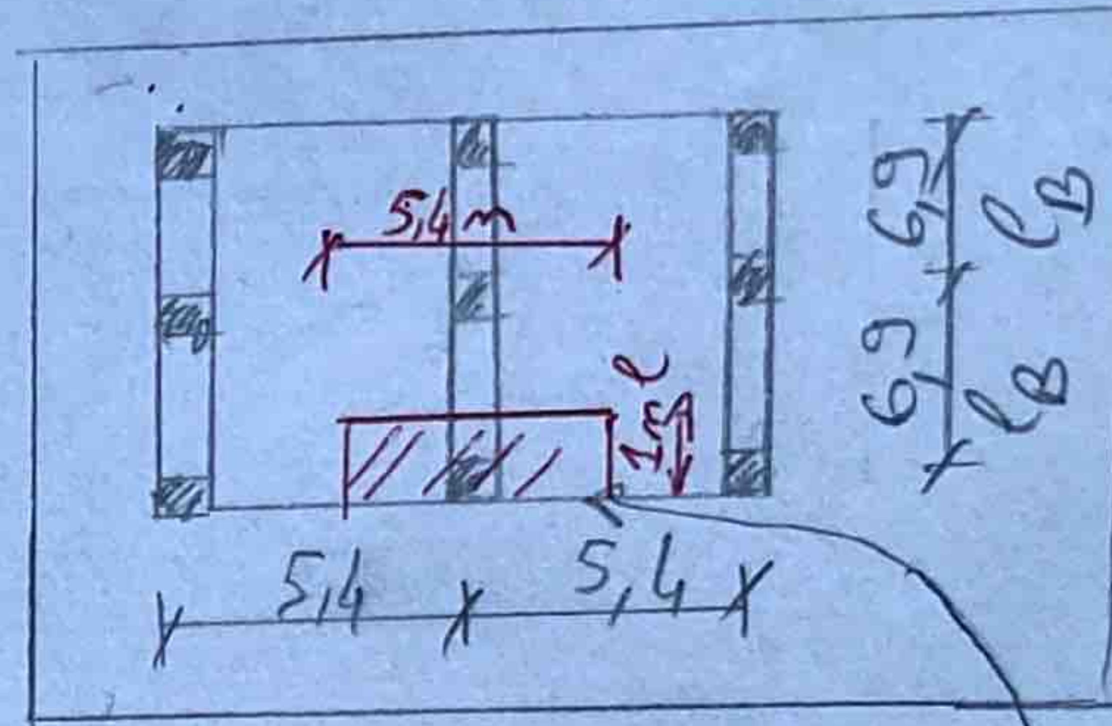
② $\lambda_{lim} = 1 \times 1 \times 1{,}2 \times 26 = 31{,}2$

$\lambda_1 < \lambda_{lim}$
(30) (31,2)

$$\boxed{h_s = 21 \text{ cm}}$$

1.2. Design of the beams





Emprical estimation

ℓ_B : span of the beam

$$h_B = \left(\frac{1}{12} \sim \frac{1}{10}\right) \ell_B$$

$$h_B = \left(\frac{1}{12} \sim \frac{1}{10}\right) \cdot 6{,}9 = (0{,}575 \sim 0{,}69)$$

We choose 60 cm of depth for preliminary design

Condition of stiffness of the beam

$h_B \overset{?}{\geq} 2{,}5 h_s$ | $2{,}5 \times 21 = 52{,}5$ cm

60 cm > 52,5 cm

The condition is satisfied.

$$b_B = \left(\frac{1}{3} \sim \frac{2}{3}\right) h_B$$

$$b_B = \left(\frac{1}{3} \sim \frac{2}{3}\right) 60$$

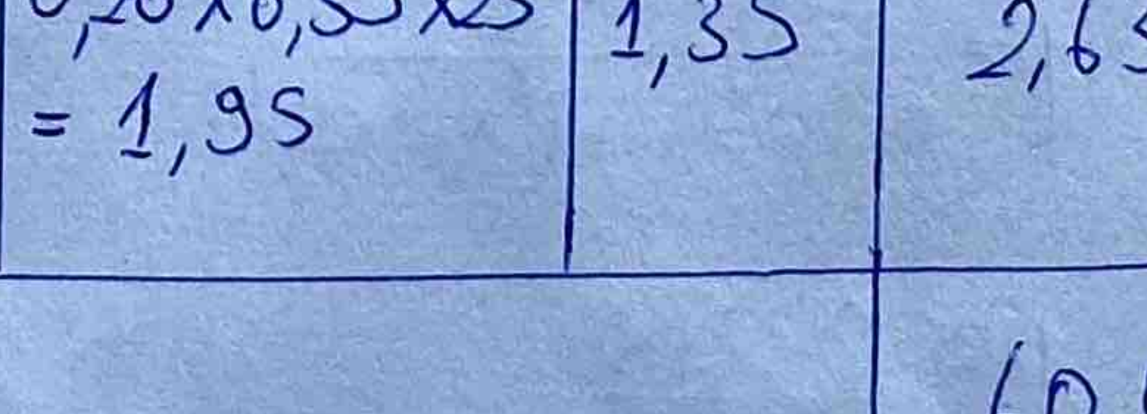
$$b_B = (20 \sim 40)$$

$$b_B = 20 \text{ cm}$$

1.2.1. Last floor beam

Type	Name	f_k [kN/m²]	γ_F	fd [kN/m²]	fd [kN/ml]
Permanent (Dead load)	(g-g0) roof, k	1,8	1,35	2,43	13,12
	Self-weight of the slab	25 x 0,21 = 5,25	1,35	7,09	38,29
Variables (Live load)	$q_{roof,k}$	0,75	1,5	1,12	6,05

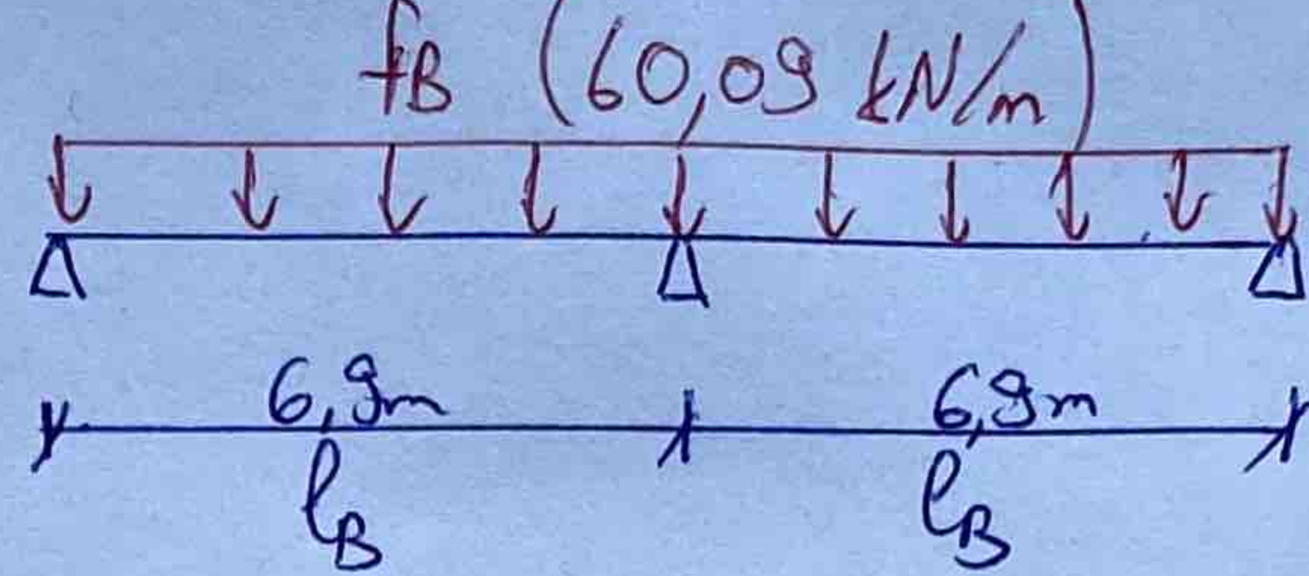
Section Beam :



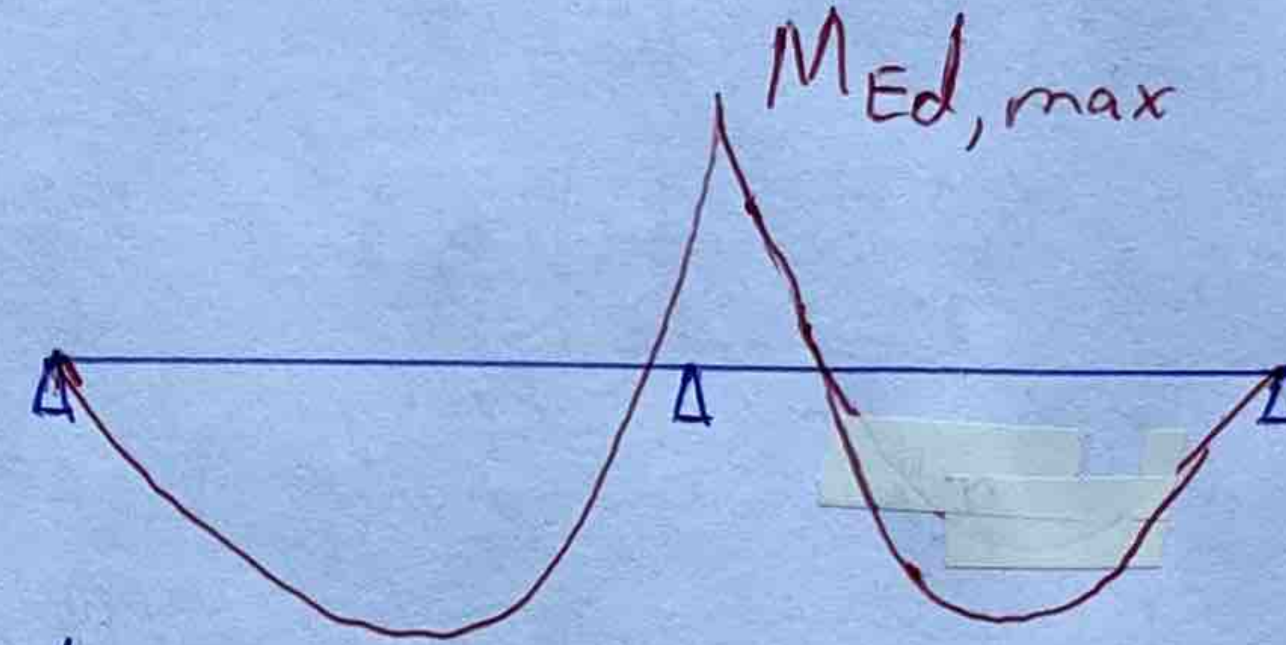
Type	Name	fk [kN/ml]	γ_F	fd [kN/ml]
Permanent (Dead load)	Self-weight of the beam	0,20 x 0,39 x 25 = 1,95	1,35	2,63
TOTAL Σ				60,09

1.2.1.1. Preliminary check of the last floor beam

Modelisation



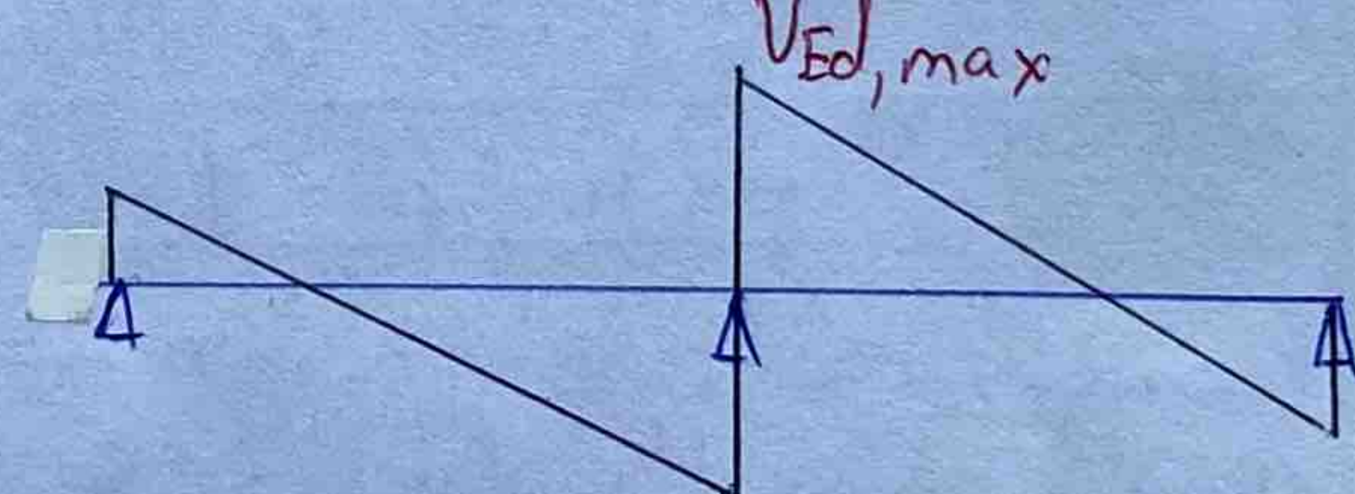
Theorical bending moment



$$M_{Ed,max} = \frac{1}{8} f_B \cdot l_B^2$$

$$= \frac{1}{8} \cdot 60{,}09 \times 6{,}9^2 = \underline{357{,}6 \text{ kN.m}}$$

Theorical shear force



$$V_{Ed,max} = \frac{5}{8} f_B \cdot l_B$$

$$= \frac{5}{8} \cdot 60{,}09 \cdot 6{,}9 = \underline{259{,}1 \text{ kN}}$$

Preliminary check of bending

$$\mu = \frac{M_{Ed,max}}{b_B \cdot d_B^2 \cdot f_{cd}}$$

$$\mu = \frac{357{,}6 \times 10^3}{0{,}20 \cdot 0{,}567^2 \times 20 \times 10^6}$$

$$\mu \simeq \underline{0{,}278}$$

$M_{Ed,max} = 357{,}6$ kN.m

$d_B = h_B - c - \frac{\phi}{2}$; $\phi = 16$ mm (hypothesis)

$= 600 - 25 - \frac{16}{2}$

$= \underline{567 \text{ mm}}$

$b_B = \underline{200 \text{ mm}}$

$f_{cd} = \frac{f_{ck}}{\gamma_c} = \frac{30}{1{,}5} = \underline{20 \text{ MPa}}$

μ	ξ
0,270	0,402
0,280	0,421

$\xi > 0,40$, we have to increase h_B and/or b_B

We change b_B by 25 cm

2nd verification

$$\mu = \frac{M_{Ed,max}}{b_B \cdot d_B^2 \cdot f_{cd}} = \frac{357,6 \times 10^3}{0,25 \times 0,567^2 \times 20 \times 10^6} = 0,222$$

μ	ξ
0,220	0,315
0,230	0,331

$\xi \in \langle 0,15 - 0,40 \rangle$ design is correct.

Preliminary check of reinforcement ratio.

μ	ζ
0,220	0,874
0,230	0,867

For $\mu = 0,222$

$$\zeta = 0,867 + \left(\frac{0,874 - 0,867}{0,220 - 0,230} \times (-0,008) \right) \simeq 0,872$$

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

$M_{Ed,max} = 357,6$ kN.m

$\zeta = 0,872$

$d_B = 567$ mm

$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{500}{1,15} = 435$ MPa

$$\rho_{s,rqd} = \frac{\frac{357,6 \times 10^3}{0,872 \cdot 0,567 \cdot 435 \times 10^6}}{0,25 \times 0,567}$$

$\simeq 0,012 = 1,2\,\%$

$\rho_{s,rqd} < 0,04$ the reinforcorcement ratio is checked.

Preliminary check of load-bearing capacity in shear

$$V_{Rd,max} = \nu \cdot f_{cd} \cdot b_B \cdot \zeta \cdot d_B \cdot \frac{\cot\theta}{1+\cot^2\theta} \overset{?}{\geq} V_{Ed,max}$$

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

$\nu = 0{,}6 \times \left(1 - \frac{30}{250}\right) = \underline{0{,}528}$

$b_B = 250$ mm

$\zeta = 0{,}872$

$\cot\theta = 1{,}5$ (pr? $\theta = 45°$? $\cot\theta = 1$?)

$d_B = 567$ mm

$$V_{Rd,max} = \left(0{,}528 \times 20 \times 10^6 \times 0{,}25 \times 0{,}872 \times 0{,}567 \times \frac{1{,}5}{1+1{,}5^2}\right) \times 10^{-3}$$

$\simeq \underline{602{,}4 \text{ kN}}$

$V_{Rd,max} > V_{Ed,max}$ The shear is checked.
(602,4 kN) (259,1 kN)

Span/depth ratio (deflection control)

ρ	Concrete class C30/37
0,5%	26
1,5%	18

$$\lambda_{d,tab\,1,2\%} = 26 + \left(\frac{26-18}{0{,}5-1{,}5} \times 0{,}7\right) = \underline{20{,}4}$$

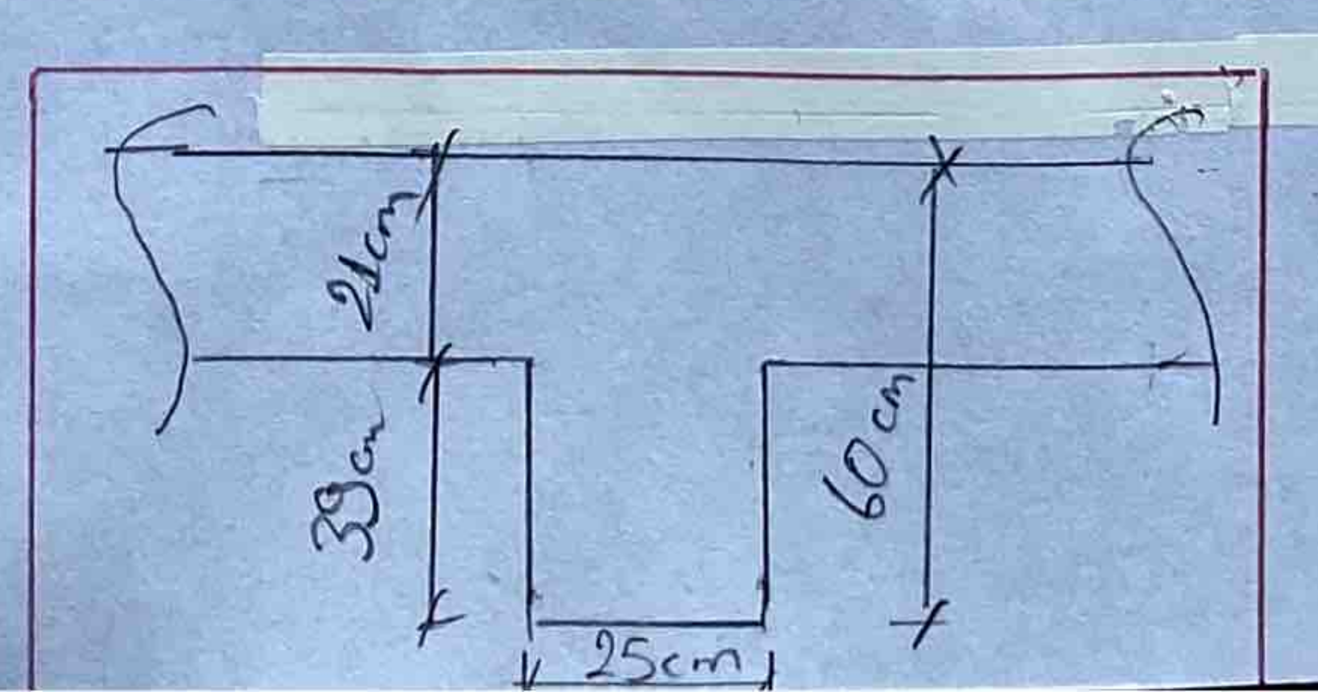
$\lambda = \frac{l}{d} \overset{?}{\leq} \lambda_{lim}$

$\lambda = \frac{6{,}9}{0{,}567} = 12{,}2$

$\lambda < \lambda_{lim}$
(12,2) (24,5)

$\lambda_{lim} = k_{c1} \cdot k_{c2} \cdot k_{c3} \cdot \lambda_{d,tab\,1,2\%}$
$= 1 \times 1 \times 1{,}2 \times 20{,}4$
$= \underline{24{,}5}$

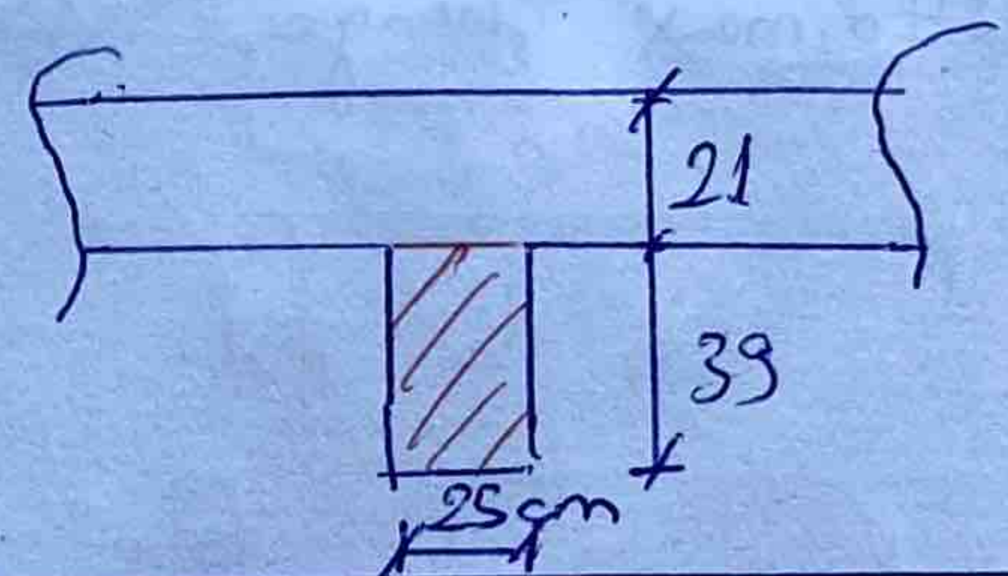
Final Section of the beam of the last floor



1.2.2 Typical floor beam

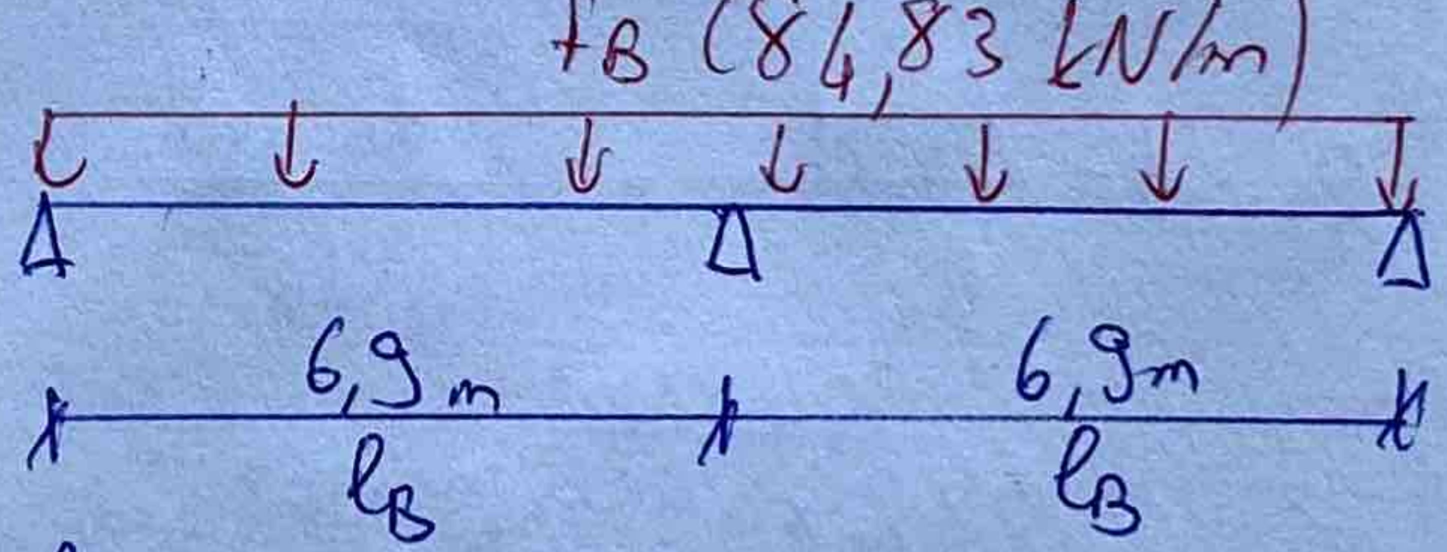
Type	Name	fk [kN/m²]	γ_F	fd [kN/m²]	fd [kN/ml]
Permanent (Dead load)	(g-go) floor, k	1,6	1,35	2,16	11,66
	Self weight of the slab	2,5x0,21 =5,25	1,35	7,09	38,29
Variables (Live load)	q floor, k	3,9	1,5	5,85	31,59

Section Beam

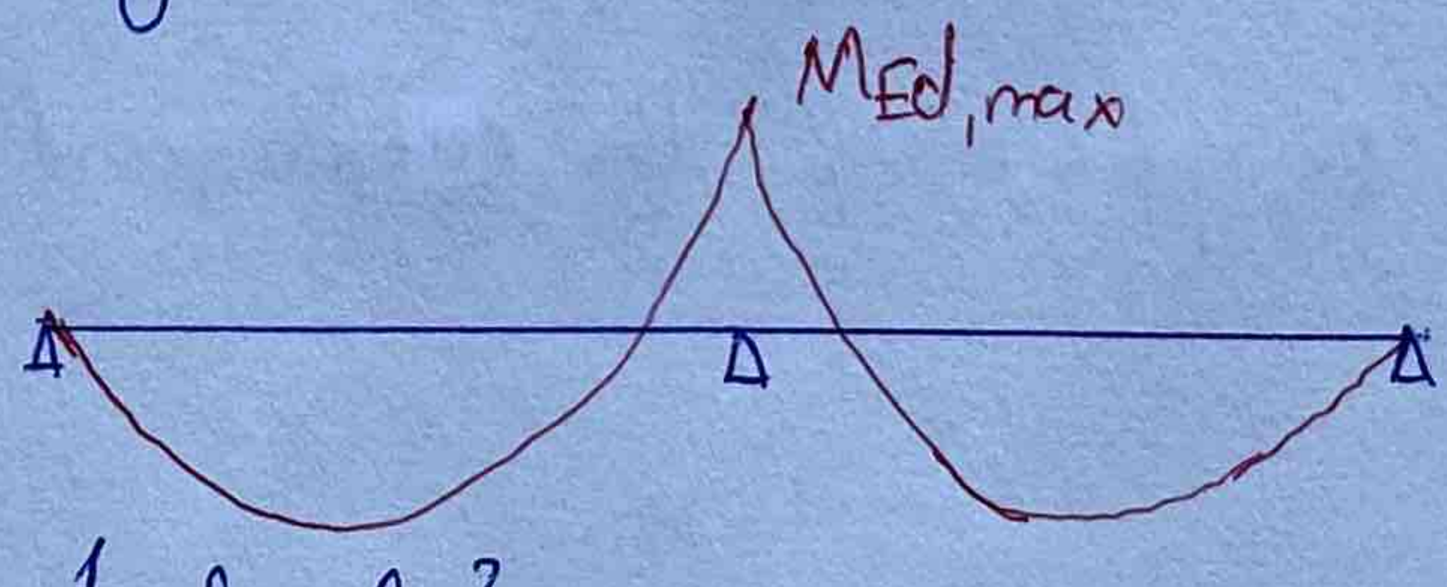


Type	Name	fk [kN/ml]	γ_F	fd [kN/ml]
Permanent (Dead load)	Self-weight of the beam	0,25x0,39x25 ≈2,44	1,35	3,29
Σ Total				84,83

1.2.2.1 Preliminary check of the typical floor beam



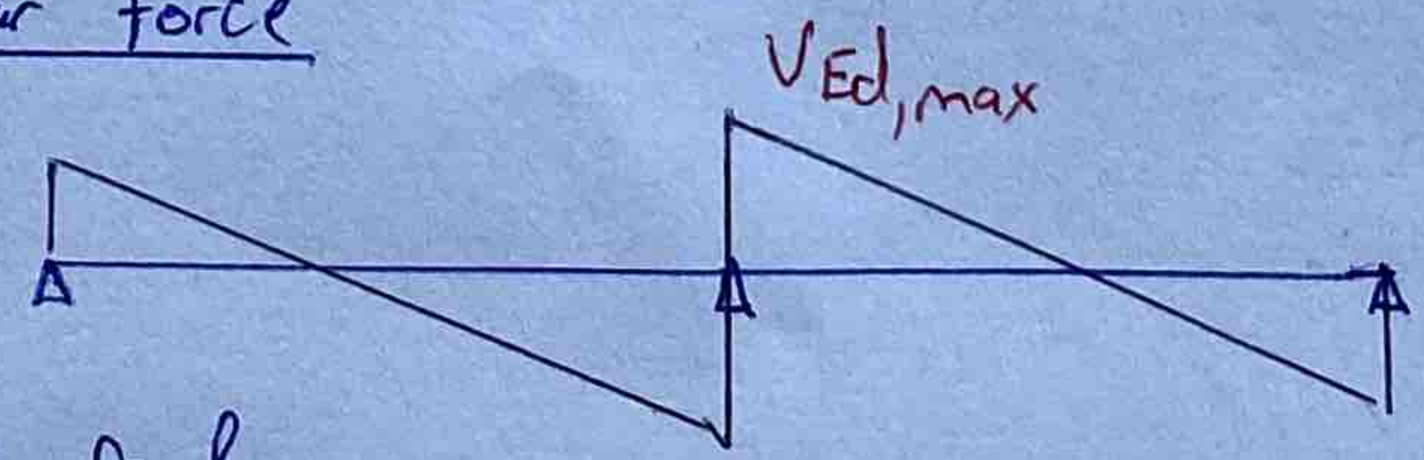
Theorical bending moment



$$M_{Ed,max} = \frac{1}{8} \cdot f_B \cdot l_B^2$$

$$= \frac{1}{8} \cdot 84{,}83 \cdot 6{,}9^2 = \underline{504{,}8 \ kN.m}$$

Theorical shear force



$$V_{Ed,max} = \frac{5}{8} f_B \cdot l_B$$

$$= \frac{5}{8} \cdot 84{,}83 \cdot 6{,}9 = \underline{365{,}8 \ kN}$$

Preliminary check of bending

$$\mu = \frac{M_{Ed,max}}{b_B \cdot d_B^2 \cdot f_{cd}} = \frac{504{,}8 \times 10^3}{0{,}25 \times 0{,}567^2 \times 20 \times 10^6} = \underline{\underline{0{,}314}}$$

μ	ξ
0,310	0,479
0,320	0,50

$\xi > 0{,}60$, we have to change h_B and/or b_B

We change b_B by 30 cm

2nd verification

$$\mu = \frac{M_{Ed\,max}}{b_B \cdot d_B^2 \cdot f_{cd}} = \frac{504{,}8 \times 10^3}{0{,}30 \times 0{,}567^2 \times 20 \times 10^6} \simeq \underline{\underline{0{,}262}}$$

μ	ξ
0,260	0,386
0,270	0,402

$\xi \in \langle 0{,}15 - 0{,}40 \rangle$ design is now correct.

Preliminary check of reinforcement ratio

μ	ζ
0,260	0,846
0,270	0,839

For $\mu = 0{,}262$

$$\zeta = 0{,}846 + \left(\frac{0{,}846 - 0{,}839}{0{,}260 - 0{,}270} \times 0{,}002 \right) \simeq \underline{\underline{0{,}844}}$$

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

$$= \frac{\frac{504{,}8 \times 10^3}{0{,}844 \times 0{,}567 \times 435 \times 10^6}}{0{,}30 \times 0{,}567} \simeq 0{,}014 = 1{,}4\,\%$$

$\rho_{s,rqd} < 0{,}04$ the reinforcement ratio is checked.

Preliminary check of load bearing capacity in shear

$$V_{Rd,max} = \nu \cdot f_{cd} \cdot b_B \cdot S \cdot d_B \cdot \frac{\cot\theta}{1+\cot^2\theta} \overset{?}{\geq} V_{Ed,max}$$

$$V_{Rd,max} = \left(0{,}528 \times 20 \times 10^6 \times 0{,}30 \times 0{,}844 \times 0{,}567 \times \frac{1{,}5}{1+1{,}5^2}\right) \times 10^{-3}$$

$$\simeq \underline{700{,}0 \text{ kN}}$$

Shear resistance is checked.

$V_{Rd,max} > V_{Ed,max}$
$(700 \text{ kN}) \quad (365{,}8 \text{ kN})$

Span/depth ratio (deflection control)

	Concrete classe
ρ	C30/37
0,5%	26
1,5%	18

$$\lambda_{d,tab\,1,4\%} = 26 + \left(\frac{26-18}{0{,}5-1{,}5} \times 0{,}9\right) \simeq \underline{18{,}8}$$

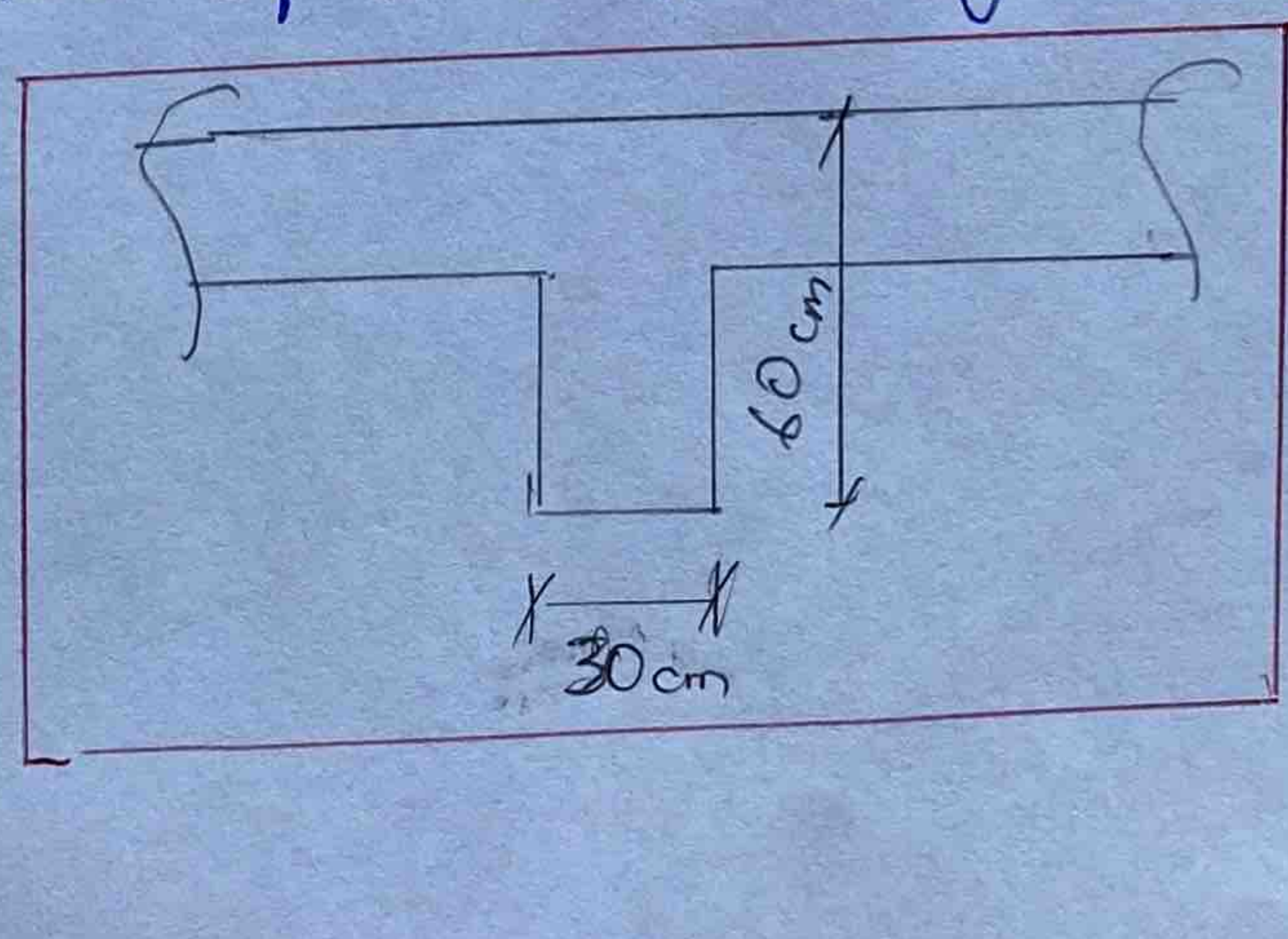
$$\lambda = \frac{l}{d} \overset{?}{\leq} \lambda_{lim}$$

$$\lambda = \frac{6{,}9}{0{,}567} = 12{,}2$$

$$\lambda_{lim} = k_{c1} \cdot k_{c2} \cdot k_{c3} \cdot \lambda_{d\,tab\,1,4\%} = 1 \times 1 \times 1{,}2 \times 18{,}8 \simeq \underline{22{,}6}$$

$\lambda < \lambda_{lim}$
$(12{,}2) \quad (22{,}6)$

Final section of beam for a typical floor



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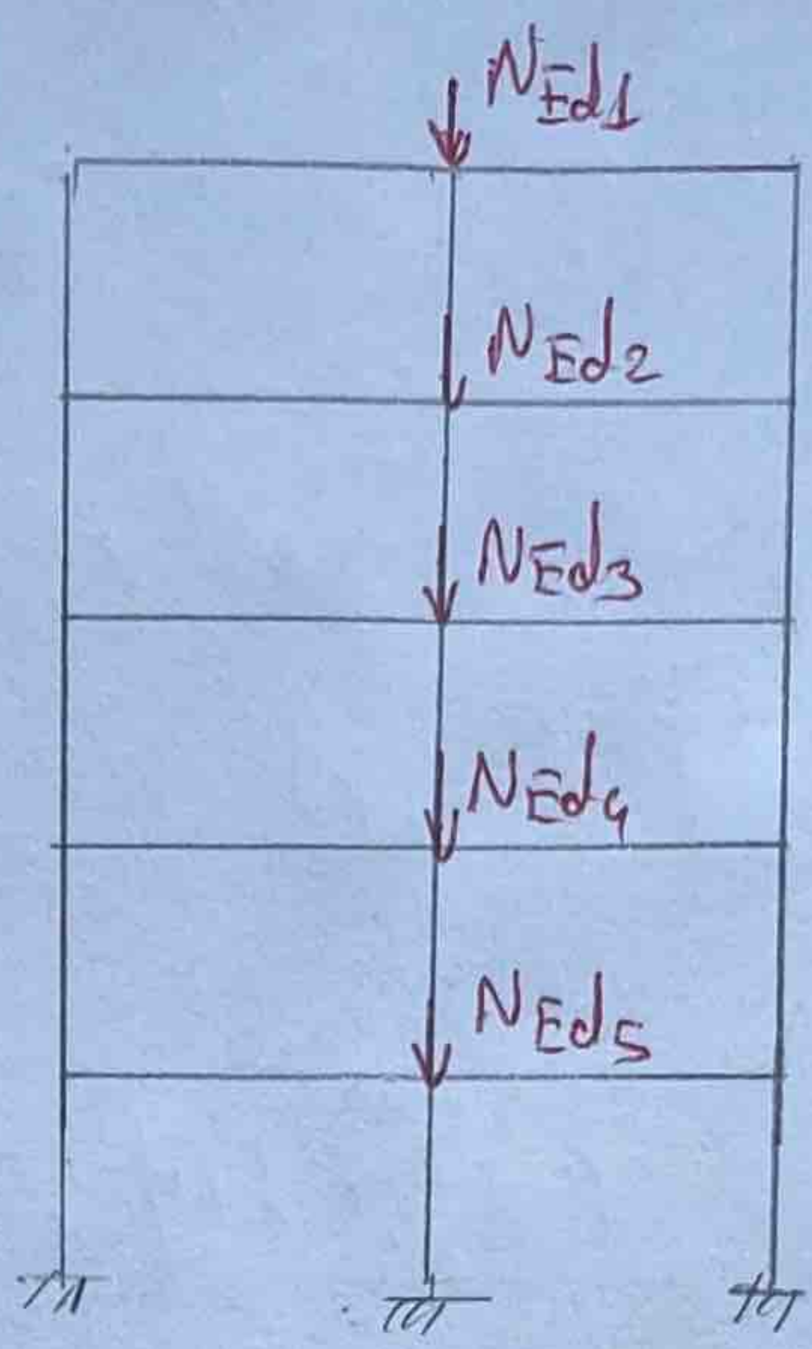
Design of the column



$\downarrow = 2 \times V_{Ed\,max}$

$$N_{Ed,1} = 2 \times V_{Ed,max1} = 2 \times 259{,}1 = 518{,}2 \text{ kN}$$

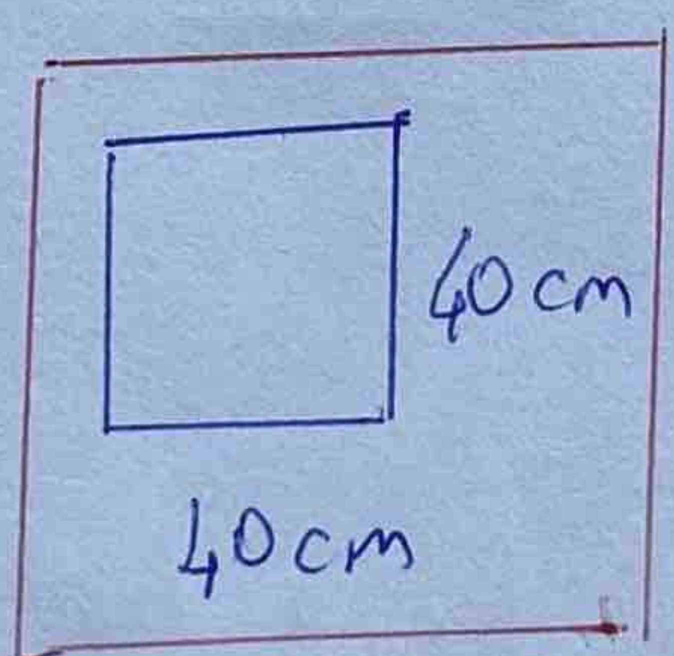
$$N_{Ed2} = N_{Ed,3} = N_{Ed4} = N_{Ed5} = 2 \times V_{Ed,max2} = 2 \times 365{,}8 = 731{,}6 \text{ kN}$$

$$\sum N_{Ed_n} = N_{Ed} = 518{,}2 + (4 \times 731{,}6) = 3444{,}6 \text{ kN}$$

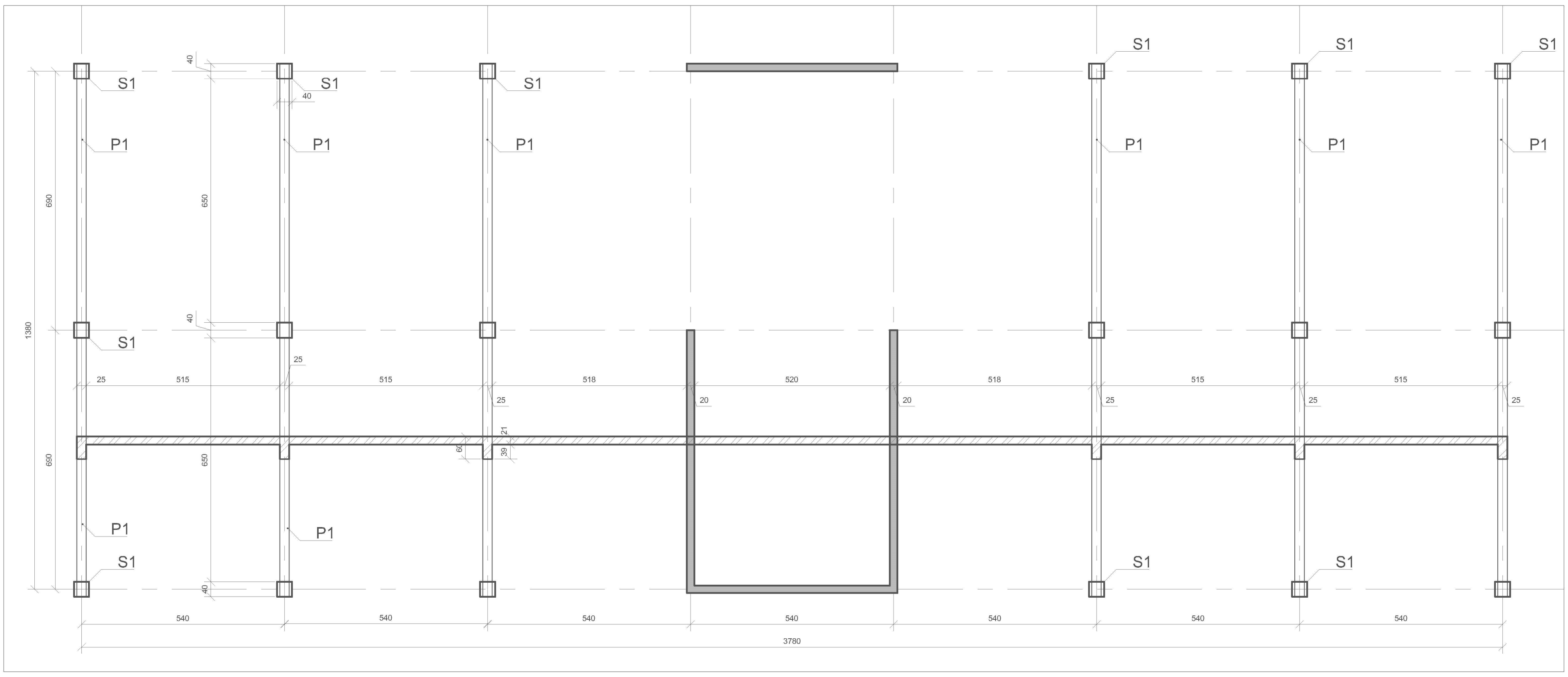
(Note: we didn't take into account the self-weight of column). because it's negligible.

$$A_c \geq \frac{N_{Ed}}{0{,}8 f_{cd} + 0{,}02 \sigma_s} = \frac{3444{,}6 \times 10^3}{(0{,}8 \times 20 \times 10^6) + (0{,}02 \times 400 \times 10^6)} \simeq 0{,}143$$

$$a = \sqrt{A_c} = \sqrt{0{,}143} \simeq 0{,}38 \text{ m}$$



Structural plan of last floor (roof)



Structural plan of a typical floor