VERA LÓPEZ GARCÍA
79111720 T

1st TASK: FRAME STRUCTURE

DATA

$l = 4'4\,m$; $a = 5'6\,m$; $h = 3'9\,m$; $n = 7$

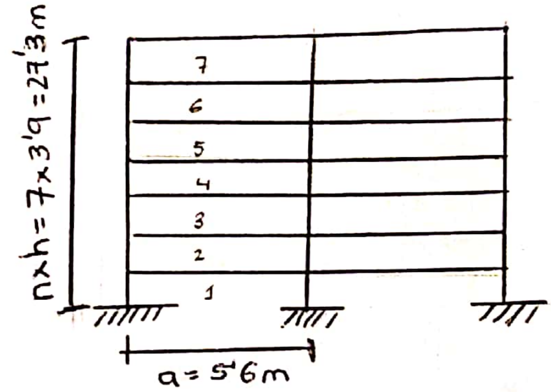
$(g-g_0)_{floor,k} = 1'1\ KN/m^2$; $(g-g_0)_{roof,k} = 1'9\ KN/m^2$.

$q_{floor,k} = 2'9\ KN/m^2$; P: XC1 ; Z: 50 years

Concrete: C 25/30
Steel: B500

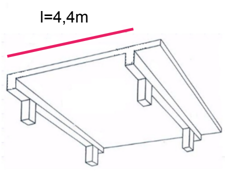
$q_{roof,k} = 0'75\ KN/m^2$

weight of the concrete = $25\ KN/m^3$



1. DEPTH OF THE SLAB (ONE-WAY SLAB)

1.1. Empirical estimation: $h_s = \left(\frac{1}{30} - \frac{1}{25}\right)\cdot l$



l = span of the slab = l = 4'4m

$h_s = \frac{1}{30} \cdot 4400 = 146'67\ mm$

$h_s = \frac{1}{25}\ 4400 = 176\ mm$

$h_s = 150\ mm - 180\ mm$. (Round slab dimension to 10 mm)

I will use $\underline{h_s = 180\,mm}$ (for bigger loads)

1.2. Effective depth (d): $d = h_s - c - \frac{\phi}{2}$

ϕ = 10mm for steel bars.

1.2.1. $c = c_{min} + \Delta c_{dev}$
$c_{min} = \max(c_{min,b}\ ;\ c_{min,dur}\ ;\ 10\,mm)$

$\Delta c_{dev} = 10\ mm$.
$c_{min,b} = 10\ mm$
$c_{min,dur}$: with S4 and XC1 → $c_{min,dur} = 15\ mm$

$\Delta c_{dev} = 10\,mm$ → technology allowance

$c_{min,b} = 10\,mm$ → Good mechanical bond between steel & concrete

$c_{min,dur}$ → we will use class S4 (for 50 years)

Values of $c_{min,dur}$ [mm]							
Structural class	Exposure class related to environmental conditions						
	X0	XC1	XC2/XC3	XC4	XD1/XS1	XD2/XS2	XD3/XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4 (for 50 years)	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

Now, according to the structural class table, with my concrete C 25/30, I should decrease class by 1:

Criterion	Structural class							
	Exposure class related to environmental conditions							
	X0	XC1	XC2	XC3	XC4	XD1/XS1	XD2/XS2	XD3/XS3
Working life 80 years	increase class by 1							
Working life 100 years	increase class by 2							
Concrete class	decrease class by 1 if concrete class is at least:							
	C20/25	C25/30	C30/37	C35/45	C40/50	C40/50	C40/50	C45/55
Member with slab geometry	decrease class by 1							
Special quality control of concrete	decrease class by 1							

Values of $c_{min,dur}$ [mm]							
Structural class	Exposure class related to environmental conditions						
	X0	XC1	XC2/XC3	XC4	XD1/XS1	XD2/XS2	XD3/XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4 (for 50 years)	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

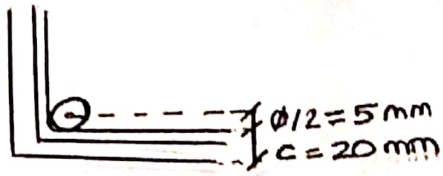
Finally, my $C_{min,dur} = 10$ mm

$C_{min} = max\{10\text{ mm};\ 10\text{mm};\ 10\text{ mm}\} = 10$ mm

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

$C = 10 + 10$

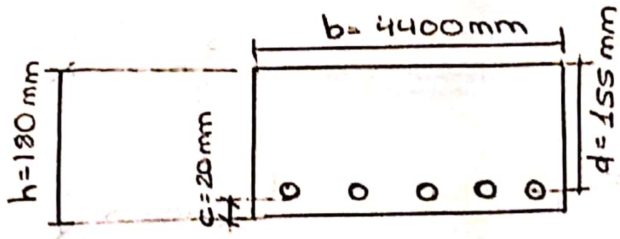
$c = 20$ mm



$\rightarrow d = h_s - c - \frac{\emptyset}{2}$

$d = 180 - 20 - 5$

$d = 155$ mm



1.3. Span/depth ratio (deflection control)

$$\lambda = \frac{l}{d} \leq \lambda_{lim} = K_{c1} K_{c2} K_{c3} \lambda_{d,tab}$$

$K_{c1} = 1.0 \rightarrow$ Effect of shape

$K_{c2} = 1.0 \rightarrow$ Effect of span

$K_{c3} = 1.2 \rightarrow$ Effect of Reinforcement

$\lambda_{d,tab} \rightarrow$ TABLE:

$\downarrow$

using ρ: 0'5% & the outer span (more disadvantaged)

$\lambda_{d,tab}$ for outer span of the continuous beam/slab

	Concrete class						
ρ	12/15	16/20	20/25	25/30	30/37	40/50	50/60
0,5 %	19,0	20,5	22,1	24,1	26	33,5	41,5
1,5 %	15,9	16,4	16,9	17,6	18	19,5	20,8

$\lambda_{d,tab} = 24'1$. $\rightarrow \lambda_{lim} = 1 \times 1 \times 1'2 \times 24'1$

$\lambda_{lim} = 28'92$ mm

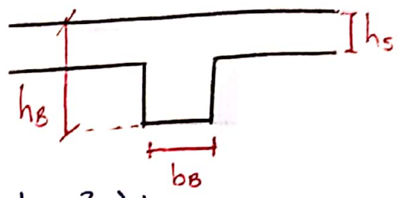
l = span of the slab

And, $\lambda = \frac{l}{d}$; $l = 4400$ mm

$$\lambda = \frac{4400}{155} = 28'38$$

Because $\lambda < \lambda_{lmt}$, $28'38 < 28'92$ we can omitte detailed calculations of deflections

CONCLUSION: The slab is designed with thickness 180mm, cover depth 20mm, reinforcing bars diameter 10 mm. Its effective depht is 155 mm.



2. DESING OF THE BEAM

2.1. Empirical estimation

$$h_B = \left(\frac{1}{12} \sim \frac{1}{10}\right) L_B \quad ; \quad b_B = \left(\frac{1}{3} \sim \frac{2}{3}\right) h_B$$

To reach sufficient stiffness of the beam:

$$h_B \geq 2'5 h_s$$

L_B = span of the beam

$L_B = a$

$L_B \equiv a = 5600$ mm. So: $h_B = \left(\frac{1}{12} \sim \frac{1}{10}\right) \cdot 5600$

$$h_B = (466'67 \sim 560)\ \text{mm}$$

From before, I obtened $h_s = 180$ mm. So, if we want to fullfill the condition of stiffness:

$$2'5 \cdot h_s = 2'5 \cdot 180 = 450\ \text{mm} \leq 560\ \text{mm} \checkmark$$

So, we take $\underline{h_B = 600\ \text{mm}}$ (Rounding to 50mm)

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

$$b_B = (186'67 \sim 373'33)\ \text{mm}.$$

Rounding the beam to 50mm $\rightarrow$ $\underline{b_B = 400\ \text{mm}}$

3. PRELIMINARY CHECK OF THE BEAM

We need to calculate the theoretical maximum values of internal forces for the three differents loads:

1. permanent load of typical floor: $(g - g_0)_{floor,K} = 1'1$ KN/m²
2. permanent load of the roof: $(g - g_0)_{roof,K} = 1'9$ KN/m²
3. Life load of typical floor: $q_{floor,K} = 2'9$ KN/m²
4. Life load of the roof: $q_{roof,K} = 0'75$ KN/m²

For permanent load, the partial safety coefficients for actions (ULS) is $\gamma = 1'35$; and for variable load is $\gamma = 1'5$. for unfavourable effect.

I will begin with LOAD from a FLOOR SLAB

PERMANENT	K-value [KN/m²]	γ	d-value [KN/m²]
Self-weight $25\,KN/m^3 \cdot 0'18\,m$	4'5	1'35	6'075
$(g-g_0)_{floor,K}$	1'1	1'35	1'485
VARIABLE			
Live: $q_{floor,K}$	2'9	1'5	4'35
	8'5 KN/m²		11'91 KN/m²

- Total value of load of a floor structure is 8'5 KN/m². For further calculations the design value of 11'91 KN/m² will be use.

Now, the LOAD of the ROOF but without the self-weight

PERMANENT	K-value [KN/m²]	γ	d-value [KN/m²]
$(g-g_0)_{roof,K}$	1'9	1'35	2'565
Self-weight: $25\frac{KN}{m^3} \cdot 0'18$	4'5	1'35	6'075
VARIABLE			
Live: $q_{roof,K}$	0'75	1'5	1'125
	7'15 KN/m²		9'765 KN/m²

- The total value of load of the roof structure is 7'15 KN/m². For further calculations the desing value it will be use it's 9'765 KN/m²

Now, to obtain f_B, the load of one floor is going to be multiply by the loading width of the beam:

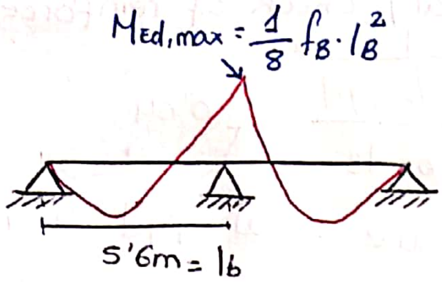
The span of the beam: $L = a = 4'4\,m$

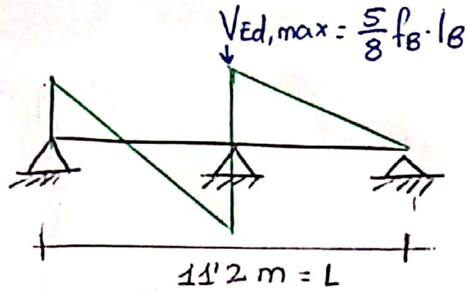
So: $11'91 \frac{KN}{m^2} \cdot 4'4\,m = 52'4\ KN/m$

And the self-weight of the beam:

$(0'6-0'18) \cdot 0'4 \cdot 25 \cdot 1'35 = 5'67\ KN/M$

So, $f_b = 52'4 + 5'67 \cong 58\ KN/m$





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

$M_{Ed,max} = \frac{1}{8} \cdot 58 \cdot 5'6^2$

$M_{Ed,max} = 227'4$ KN·m

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

$V_{Ed,max} = \frac{5}{8} \cdot 58 \cdot 5'6$

$V_{Ed,max} = 203$ KN

3.2. Preliminary check of bending (ξ)

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

PREVIOUS CONSIDERATIONS

1. C25/30 ↗ $f_{ck} = 25$ MPa ↘ $f_{cd} = \frac{25}{1'5} = 16'67$ MPa

2. B500 ↗ $f_{yk} = 500$ MPa ↘ $f_{yd} = \frac{500}{1'15} = 434'78$ MPa

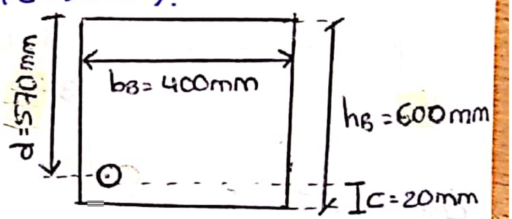
For d_B it is consider that the diameter of rebars 16-22 mm: I will take 20 mm

3. Effective height of the beam (d_B) considering a diameter of the rebars 20 mm ($\emptyset = 20$ mm) and a minimum cover depth of 20 mm (c = 20mm).

$d_B = h_B - c - \emptyset/2$

$d_B = 600 - 20 - 20/2$

$d_B = 570$ mm $= 0'57$ m



1 MPa = 1000 KN/m²

Therefore: $\mu = \dfrac{M_{Ed,max}}{b_B \cdot d_B^2 \cdot f_{cd}} = \dfrac{227'4}{0'4 \cdot 0'57^2 \cdot 16'67e^3} = 0'1$

$\mu = 0'1$, so looking in the table:

μ	ω	ξ	ζ
0,090	0,0945	0,118	0,953
0,100	0,1056	0,132	0,947
0,110	0,117	0,146	0,942
0,120	0,128	0,160	0,936
0,130	0,140	0,175	0,930

$\xi = 0'132$

$0'132 = \xi < 0'15$ h_B and/or b_B should be decrease.

So with $b_B = 0'35$ m $\rightarrow \mu = \dfrac{227'4}{0'35 \cdot 0'57^2 \cdot 16'67e^3} = 0'12$

$\mu = 0'12 \rightarrow \xi = 0'16$

$0'15 < \xi = 0'16 < 0'4$ ✓

3.3. Preliminary check of reinforcement ratio

$$\rho_{s,rqd} = \frac{\frac{M_{Ed,max}}{\zeta d_B \cdot f_{yd}}}{b_B d_B} \leq 0'04$$

From before, we know $f_{yd} = 434'78$ MPa and we take ζ from the table:

μ	ω	ξ	ζ
0,090	0,0945	0,118	0,953
0,100	0,1056	0,132	0,947
0,110	0,117	0,146	0,942
0,120	0,128	0,160	0,936
0,130	0,140	0,175	0,930

$\zeta = 0'936$

So: $$\rho_{s,rqd} = \frac{\frac{227'4}{0'936 \cdot 0'57 \cdot 434'78e^3}}{0'35 \cdot 0'57} = 0'005 \leq 0'04 \checkmark$$

3.4. Preliminary check of load-bearing capacity in shear (compression diagonals)

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

$\cot\theta = 1'5$ because cracks open at $45°$

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

$$V_{Rd,max} = 0'6\left(1 - \frac{25}{250}\right) \cdot 16'67e^3 \cdot 0'35 \cdot 0'936 \cdot 0'57 \cdot \frac{1'5}{1+1'5^2}$$

$V_{Rd,max} = 775'8$ kN $\geq V_{Ed,max} = 334$ kN $\checkmark$

3.5. Deflection control.

$$\lambda = \frac{l}{d_B} \leq \lambda_{lim} = K_{c1} K_{c2} K_{c3} \lambda_{d,tab}$$

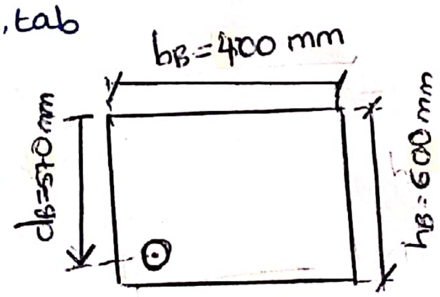
Same procedure than the span

$l_B = a = 5600$ mm

$d_B = h_B - c - \varnothing/2$

$d_B = 600 - 20 - 20/2$

$d_B = 570$ mm



$\lambda = \frac{5600}{570} = 9'824$

0'5% → 24'1
1'5% → 17'6
1 → 6'5
'495 − 3'2175

$K_{c1} = 1.0$ → Effect of shape
$K_{c2} = 1.0$ → Effect of span
$K_{c3} = 1.2$ → Effect of reinforcement

$\lambda_{d,tab}$ → TABLE for outer span with $\rho_{s,rqd}$ calculated

→ $\lambda_{ad,tab}$ for $\rho_{s,rqd} = 0'005$

$\lambda_{d,tab}$ for outer span of the continuous beam/slab

	Concrete class						
ρ	12/15	16/20	20/25	25/30	30/37	40/50	50/60
0,5 %	19,0	20,5	22,1	24,1	26	33,5	41,5
1,5 %	15,9	16,4	16,9	17,6	18	19,5	20,8

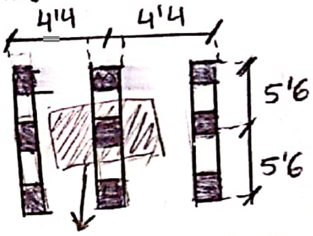
$\lambda_{ad,tab} = 27'3$, (interpolating)

SO: $\lambda_{limt} = 1'0 \cdot 1'0 \cdot 1'2 \cdot 27'3 = 32'8$

$\lambda = 9'03 \leq \lambda_{limt} = 32'8$ ✓ condition checked.

CONCLUSION: The beam is designed with thickness 600 mm and a with of 350 mm

4. DIMENSIONS OF THE COLUMN



Tributing area
$A = 4'4 \times 5'6 = 24'64 \, m^2$

4.1. Loads

From previous calculations (Page 4):
FLOOR desing load = 11'91 KN/m²
ROOF desing load = 9'765 KN/m²

Load from the slab

7x typical floor $7 \cdot 24'64 m^2 \cdot 11'91 \, KN/m^2 = 293'5 \, KN$
1x roof $1 \cdot 24'64 m^2 \cdot 9'765 KN/m^2 = 240'6 \, KN$

534'1 KN

Load from the beam

$(h_B - h_S) \cdot b_B \cdot 25 \frac{KN}{m^3}$ → $(0'6 - 0'18) \cdot 0'5 \cdot 25 \, KN/m^3 = 0'235 \cdot 25 = 5'875 \, KN/m$

$(4'4 + 5'6) m \cdot 5'875 \, KN/m = 58'75 \, KN$
x 7 floors

411'25 KN

Estimated self weight of the column ≃ 25 KN

→ $N_{Ed} = 534'1 + 411'25 + 25 \cong 970'4 \, KN$

4.2. $N_{Rd} = 0'8 \cdot A_c \cdot f_{cd} + A_s \cdot \sigma_s \geq N_{Ed}$

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

Estimation for $\sigma_s = 400$ MPa

$$A_c \geq \frac{970'4}{0'8 \cdot 16'67e^3 + 0'02 \cdot 400e^3}$$

$A_c \geq 0'045 m^2$ (minimun area)

4.2.2. Check the condition:

$A_s \simeq 0'02 \cdot A_c$ (Estimation)

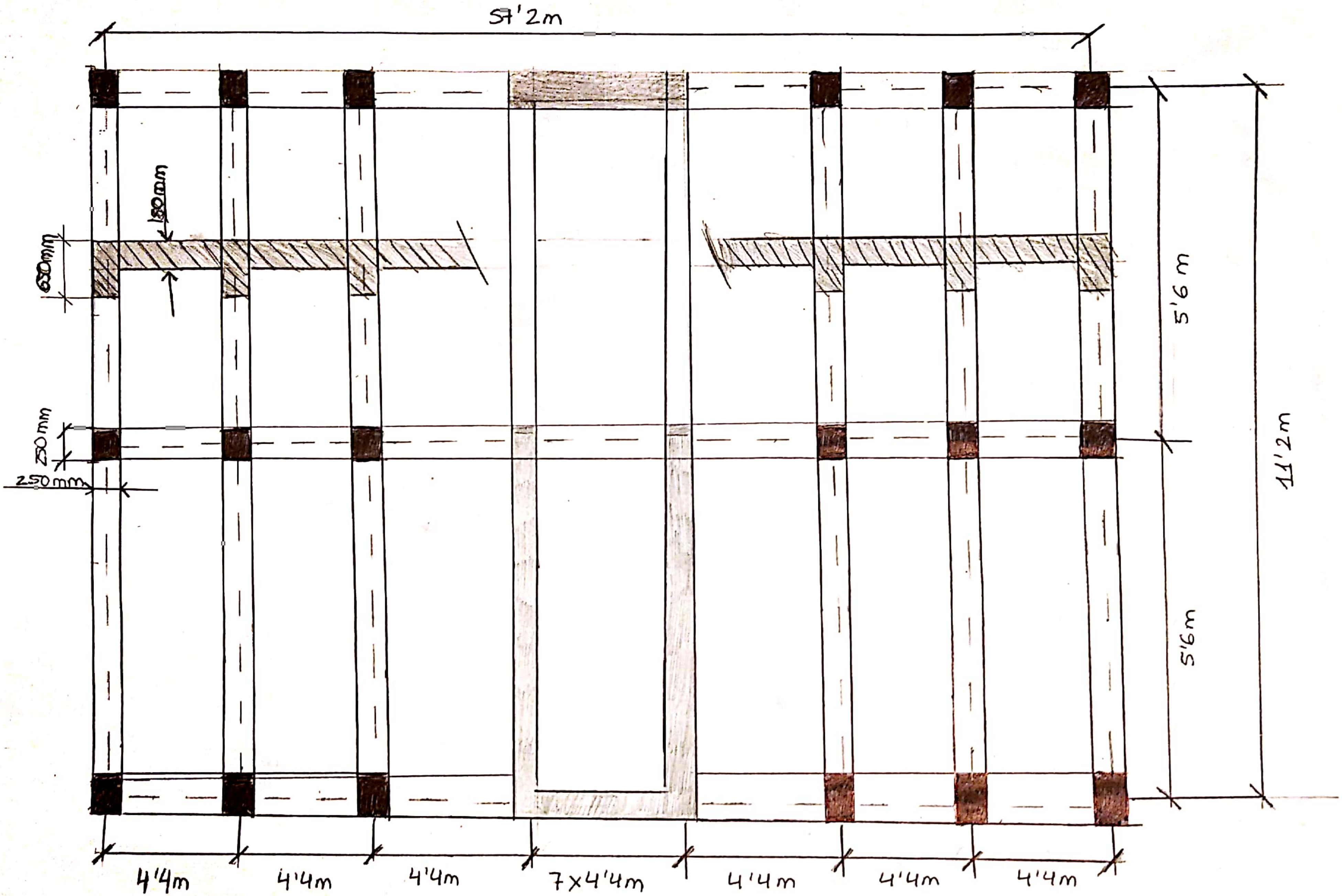
$N_{Rd} = 0'8 \cdot 0'045 \cdot 16'67e^3 + 0'02 \cdot A_c \cdot 400e^3$

$N_{Rd} = 960'12$ KN $\ngeq N_{Ed} = 970'4$ KN

So the dimensions of the column:

250 mm x 250 mm

→



57'2m
150mm
650mm
250mm
250mm
5'6m
11'2m
5'6m
4'4m
4'4m
4'4m
7x4'4m
4'4m
4'4m
4'4m