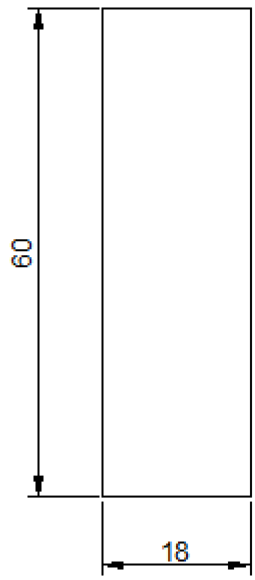
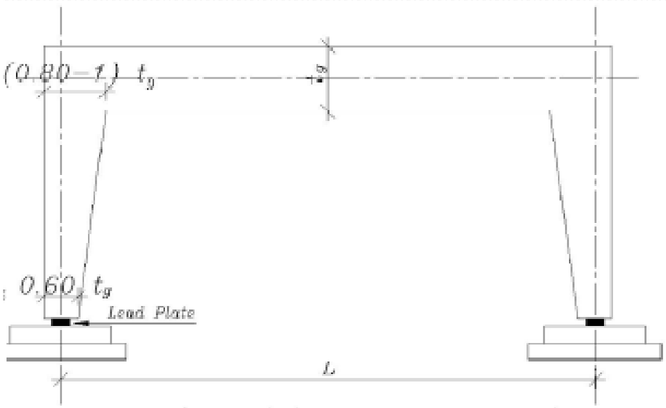
It's required to design the frame in the given area knowing that the structural system is flat roof frames

Material properties are
f_{cu}=35 N/mm^2
f_y=360 N/mm^2
$f_{y\,str}$=240 N/mm^2

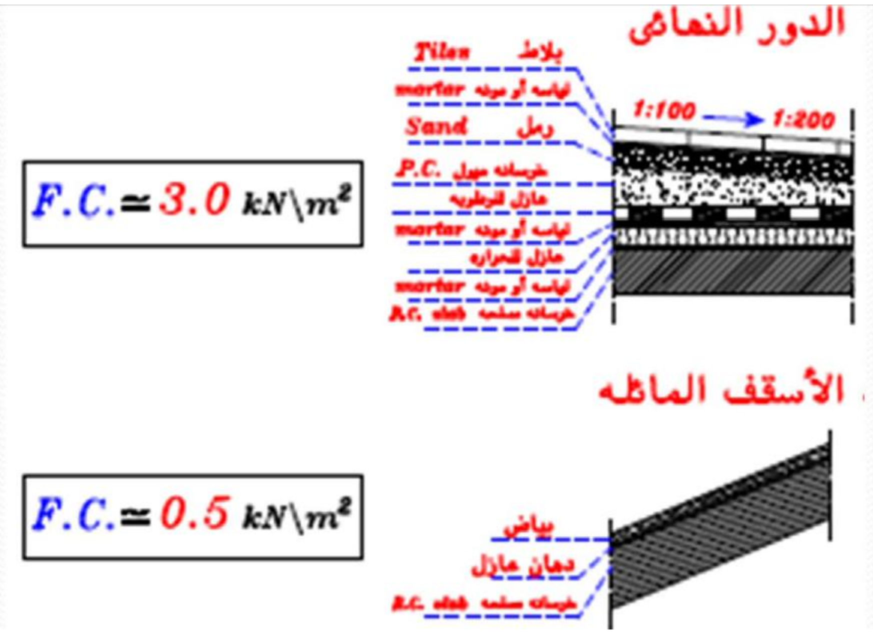


FROM DR. ISLAM LECTURES

Element	Dimensions
tg	L/(12-14)
bg	300-400 mm
Frame Spacing (S)	4.0-7.0 m
Column Thickness	(0.80-1.0)tg
Secondary Beam Thickness	Frame spacing (S)/(8-10)



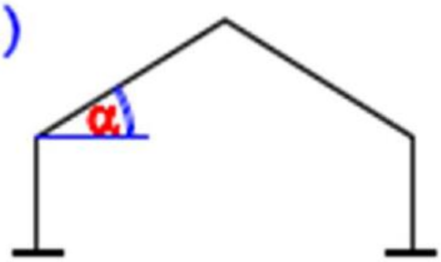
Sec. (X1–X1) (Concrete Dimensions)



Live Load. (L.L.)

For Top roof.

الدور النهائى



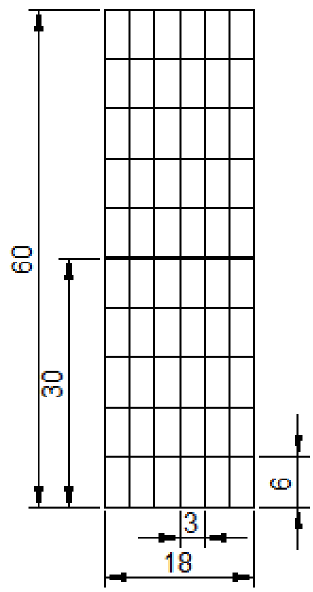
IF $\alpha > 20^{\circ} \longrightarrow L.L. = 0.5 \ kN\backslash m^2$

IF $\alpha \leqslant 20^{\circ} \longrightarrow L.L. = 1.0 \ kN\backslash m^2$

Solution

Take s = 6 m

As shown in fig.



1. Calculate slab thickness

$$t_{min} = \begin{cases} L/30 & \textit{simple span} \\ L/35 & \textit{continuous from one end} \\ L/40 & \textit{continuous from two ends} \end{cases}$$

where L is the effective span

But also remember

T > 80mm for static loading
T > 120 mm for dynamic loading

$$t_{min} = \frac{l}{35} = 85.71$$

Take t = 120 mm (conservatively)

2. Calculate slab loading

$g_s = \gamma_c t_s + Fl$

$g_s = 25 * 0.12 + 3 = 6\ \ kN/m^2$

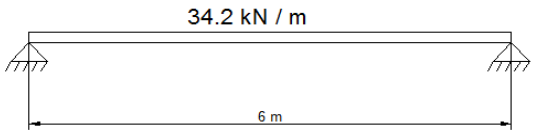
$w_{su} = 1.4\ g_s + 1.6\ p_s$

$w_{su} = 1.4 * 6 + 1.6 * 1 = 10\ kN/m^2$

3. Calculate secondary beam loading

$w_u = \ w_{ow} + \ w_{slab}$

$w_u = 1.4\ (\gamma_c * b * t) + w_{su} * spacing$



$w_u = 1.4\ (25 * 0.2 * 0.6) + (10 * 3) = 34.2\ kN/m$\

4. Frame dimensions

$$t_g = \frac{L}{12-14} = \frac{18}{12-14} = 1.5 - 1.286 = 1.4m$$

$t_g = 1.4m$

$t_{col.top} = (0.8-1)t_g = 1.2 - 1.4 = 1.4m$

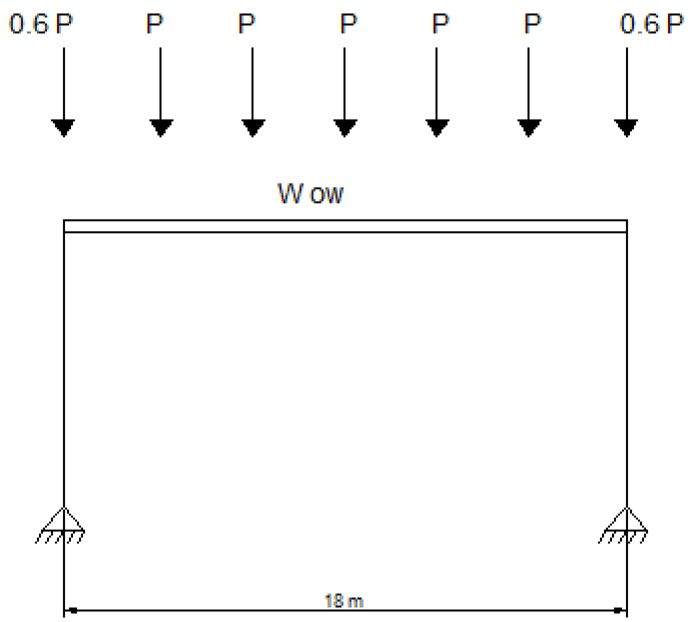
$t_{col.bot} = 0.6t_g = 0.6 * 1.4 = 0.84m$

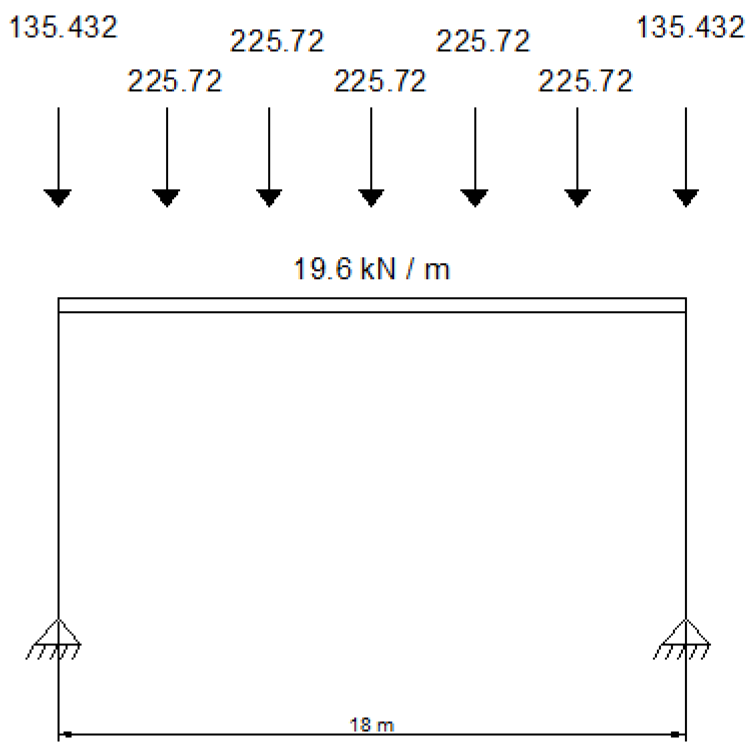
5. Frame loads and straining actions

Loads subjected to the frame are self-weight (distributed load) + secondary beam loads (concentrated load)

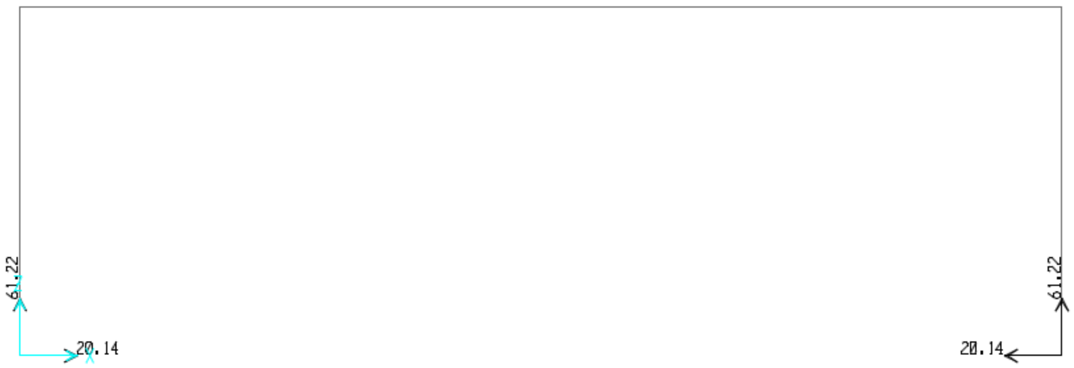
$w_{ow} = 1.4(\gamma_c * b * t) = 1.4 * 25 * 0.4 * 1.4 = 19.6$ kN/m

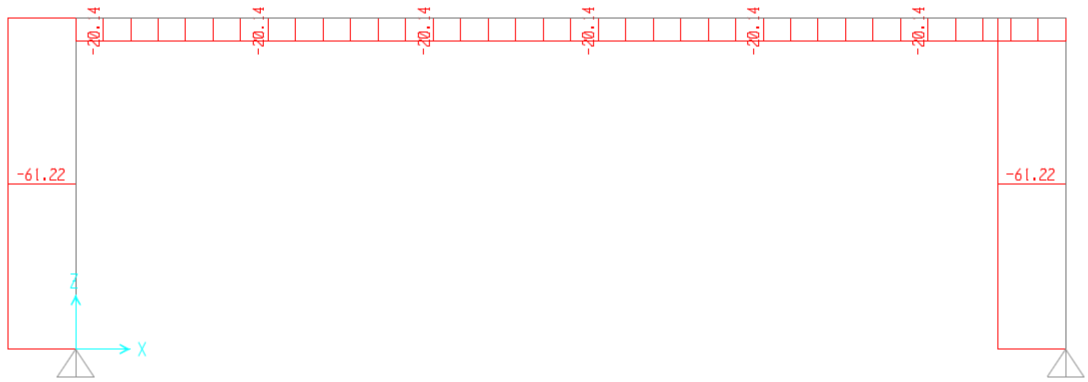
Load p will determined at critical section (at second support)
where Pu = 1.10 x Wu x S
Pu =1.10 x 34.2 x 6=225.72 kN


0.6 P
P
P
P
P
P
0.6 P
W ow
18 m

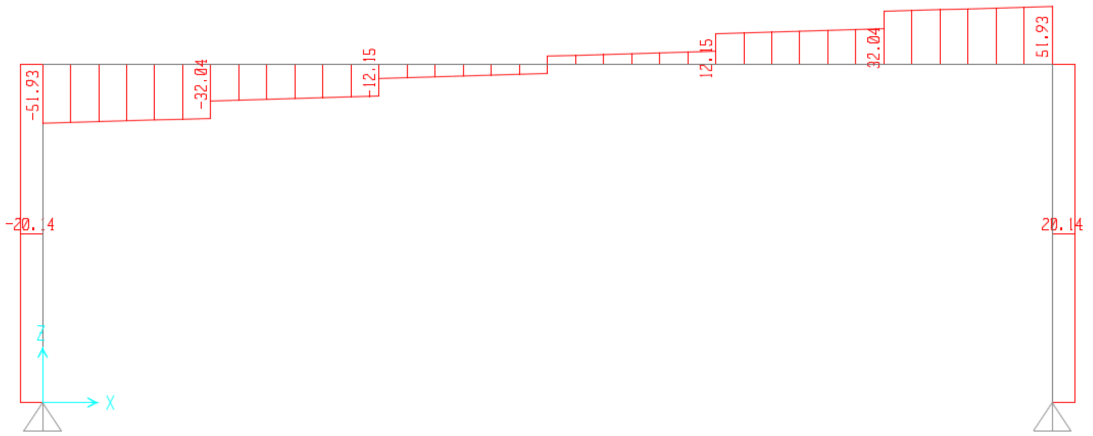

135.432
225.72
225.72
225.72
225.72
225.72
135.432
19.6 kN / m
18 m

reactions

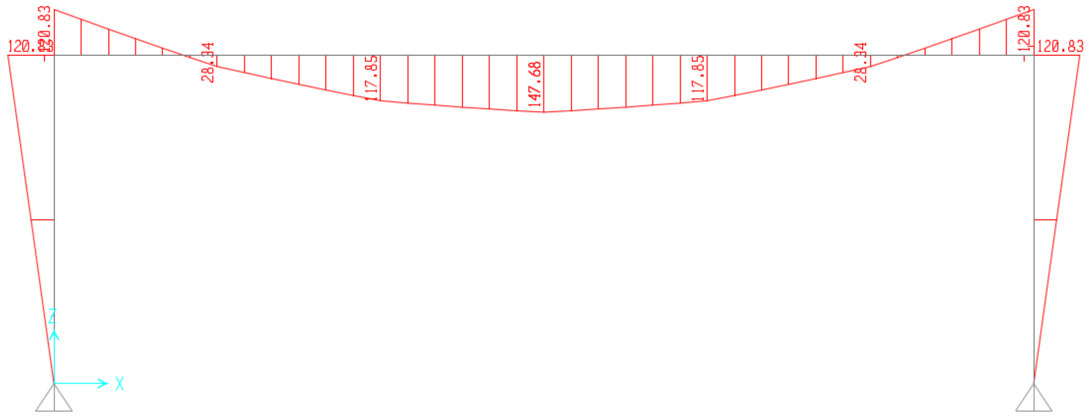




N.F.D



S.F.D



B.M.D

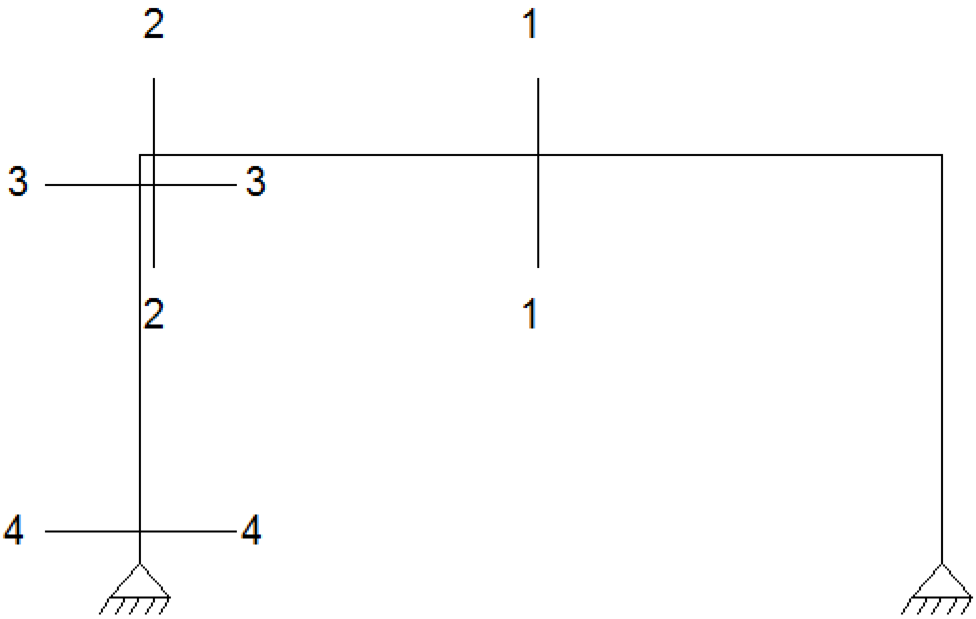
we use accurate analysis for this frame alternatively we can convert the concentrated loads to distributed load by divide the summation of concentrated force by span length and use the following formula to find horizontal reaction

$$H_a = H_b = \frac{w_u \times L^2}{4 \times h \times N}$$

$$\text{where} \quad K = \frac{I_b}{I_c} \times \frac{h}{L} \quad \text{and} \quad N = 2K + 3$$

Then apply equilibrium equation to find vertical reaction

Critical sections



6. Design of sec 1-1

Mu=1476.8 kN/m
Pu=201.4 kN (comp.)

Remember

If $\frac{p_u}{f_{cu\,b\,t}} < 0.04$ normal force is negligible

$\frac{p_u}{f_{cu\,b\,t}} = \frac{201.4*1000}{35*400*1400} = 0.01 < 0.04 \xrightarrow{yields}$ Neglect normal force

Sec. subjected to pure bending moment

d=t-100=1400-100=1300mm

$$d = c_1\sqrt{\frac{M_u}{f_{cu\,b}}}$$

$$1300 = c_1\sqrt{\frac{1476.8 * 10^6}{35 * 400}}$$

C1=4.00
J=0.804

$$A_s = \frac{M_u}{f_y\, j\, d} = \frac{1476.8 * 10^6}{360 * 0.804 * 1300} = 3924\ mm^2$$

$$A_{s.min} = smaller\ of\ \begin{cases} \frac{0.225\sqrt{f_{cu}}}{f_y} * b * d \\ 1.3As \end{cases}$$

$$A_{s.min} = \frac{0.225\sqrt{35}}{360} * 400 * 1300 = 1922.72\ mm^2$$

$$A_s = 3924\ \ mm^2$$

Use 8 ϕ25 (safe and economic)

7. Design of sec 2-2

Mu=1208.3 kN/m
Pu=201.4 kN (comp.)

Remember

If $\frac{p_u}{f_{cu\,b\,t}} < 0.04$ normal force is negligible

$\frac{p_u}{f_{cu\,b\,t}} = \frac{201.4*1000}{35*400*1400} = 0.01 < 0.04 \xrightarrow{yields}$ Neglect normal force

Sec. subjected to pure bending moment

d=t-100=1400-100=1300mm

$$d = c_1\sqrt{\frac{M_u}{f_{cu\,b}}}$$

$$1300 = c_1\sqrt{\frac{1208 * 10^6}{35 * 400}}$$

C1=4.426
J=0.8164

$$A_s = \frac{M_u}{f_y\, j\, d} = \frac{1208.3 * 10^6}{360 * 0.8164 * 1300} = 3162.47\ mm^2$$

$$A_{s.min} = smaller\ of \begin{cases} \dfrac{0.225\sqrt{f_{cu}}}{f_y} * b * d \\ 1.3As \end{cases}$$

$$A_{s.min} = \frac{0.225\sqrt{35}}{360} * 400 * 1300 = 1922.72mm^2$$

$$A_s = 3162.47\ mm^2$$

Use 7 ϕ25 (safe)

8. Design of sec 3-3

The system is unbraced in both direction in-plane and out-of-plane

Buckling out- of-plane

$H_0 = \frac{6-0.5}{2} = 2.75m$ **(due to wall beam)**

Top case 1
Bott. Case 1

$$H_e = k * H_0$$

$$H_e = 1.2 * 2.75 = 3.3m$$

$$\lambda = \frac{H_e}{b} = \frac{3.3}{0.4} = 8.25 < 10 \xrightarrow{yields} no\ add\ moment$$

Buckling in-plane

$$H_0 = 6m$$

Top case 1
Bott . Case 3

$$H_e = k * H_0$$

$$H_e = 1.6 * 6 = 9.6m$$

$$\lambda = \frac{H_e}{t_{avg}} = \frac{9.6}{1.12} = 8.57 < 10 \xrightarrow{yields} no\ add\ moment$$

Sec subjected to

M=1208.3 kN.m

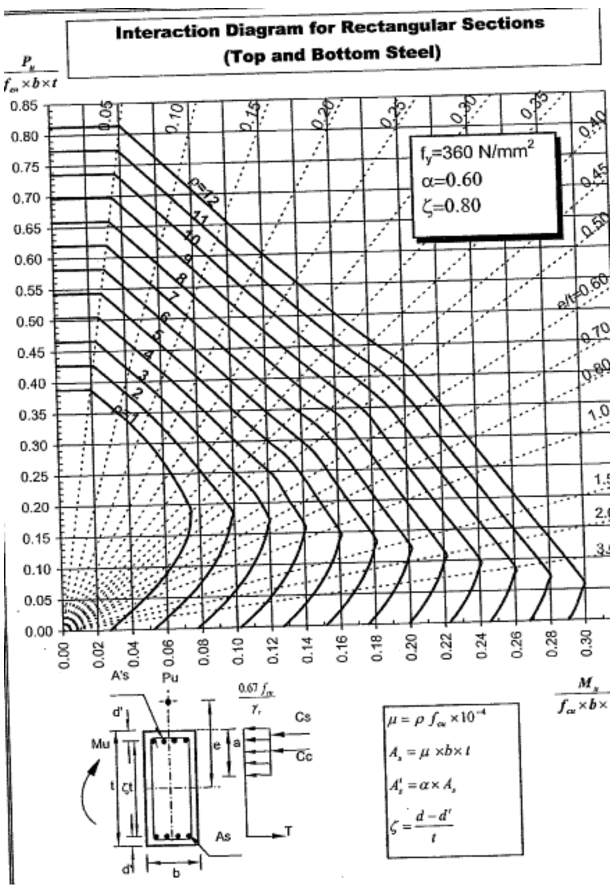
Pu=612.2 kN (comp.)

$\frac{p_u}{f_{cu\,b\,t}} = \frac{612.2*1000}{35*400*1400} = 0.0416 > 0.04 \xrightarrow{yields}$ Normal force considered

$$\frac{M_u}{f_{cu}\,b\,t^2} = \frac{1208.3*10^6}{35*400*1400^2} = 0.044$$

$$\xi = \frac{t-2cov.}{t} = \frac{1400-2*80}{1400} = 0.886$$

Use interaction diagram with $f_y = 360, \alpha = 0.6,$
$\xi = 0.8$ (conservatively)



$\rho = 1$

$\mu = \rho * f_{cu} * 10^{-4} = 1 * 35 * 10^{-4} = 0.0035$

$A_s = \mu * b * t = 0.0035 * 400 * 1400 = 1960\ mm^2$

$A_{s\backslash} = \alpha A_s = 0.6 * 4557 = 1176\ mm^2$

$A_{s.tot.} = A_s + A_{s\backslash} = 4557 + 2734.2 = 3136 mm^2$

$$A_{s.min} = greater\ of \begin{cases} 0.6\%\ b\ t = \dfrac{0.6}{100} * 400 * 1400 = 3360 mm^2 \\ 0.8\ Ac\ eff = \dfrac{0.8}{100} * 350 * 1300 = 3640 mm^2 \end{cases}$$

(note: For long column μ_{min} =0.25 +0.052 λ)

$A_{s.tot.} = 3640\ mm^2$

9. Design of sec 4-4

Pu act=612.2 kN

$A_{s.min} = 0.008bt = 3640 mm^2$

P u all =0.35 fcu Ac +0.67 fy Asc
= (0.35*35*400*1300+0.67*360*2600)/1000
=7238.32kN >612.2 kN ok

Use 6 ϕ25
(note: we use min. steel in this section and if min steel is not safe use ordinary design steps of short column)

10. Design for shear

$$Qu = Ya - Pedge - Wu\left(\frac{t_{top}}{2} + \frac{d}{2}\right)$$
$$Qu = 612.2 - 92.88 - 14.7\left(\frac{1.4}{2} + \frac{1.4}{2}\right) = 498.74\ kN$$

$$qu = \frac{Qu}{b\ d} = \frac{498.74 * 1000}{400 * 1300} = 0.96$$

$$qcu = 0.24\sqrt{\frac{fcu}{1.5}} = 0.24\sqrt{\frac{35}{1.5}} = 1.15$$

qu<qcu

use As min

$$A_{smin} = \frac{0.4}{fy} * b * s = \frac{0.4}{240} * 400 * 200 = 133.33mm^2$$

As 1 branch=100/2=66.67

Choose $5\phi 10 \ \ /m$