جامعة بيرزيت

BIRZEIT UNIVERSITY

Faculty of Engineering and Technology

Civil Engineering Department

Reinforced Concrete Design 2

ENCE436

Project Assignment

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Seven Stories - Outer dimensions (45m X 42m)

$W_{LL} = 5 KN/m^2 = 0.5 \; t/m^2$

W_{DL} = Weight of the slab.

Column Size (70 X 70 cm^2).

The building braced in both directions.

Properties of the materials ($f_c' = 28$ MPa , $f_y = 420$ MPa).

Slab Type: two-way slab on beams. (Solid).

Try Spans | 6 @ 7.5 m in 45 m direction
and 7 @ 6m in 42 m direction.

- we should find the thickness of the slab and α_{fm} to check direct design method requirements.

* assume $\alpha \geq 2$.

$L_n = 7.5 - 0.30 = 7.2$ m

$L_n \geq 0.65 (L_1) = 4.875$ m

$S_n = 6 - 0.35 = 5.65$ m.

$\beta = L_n / S_n = 1.274$

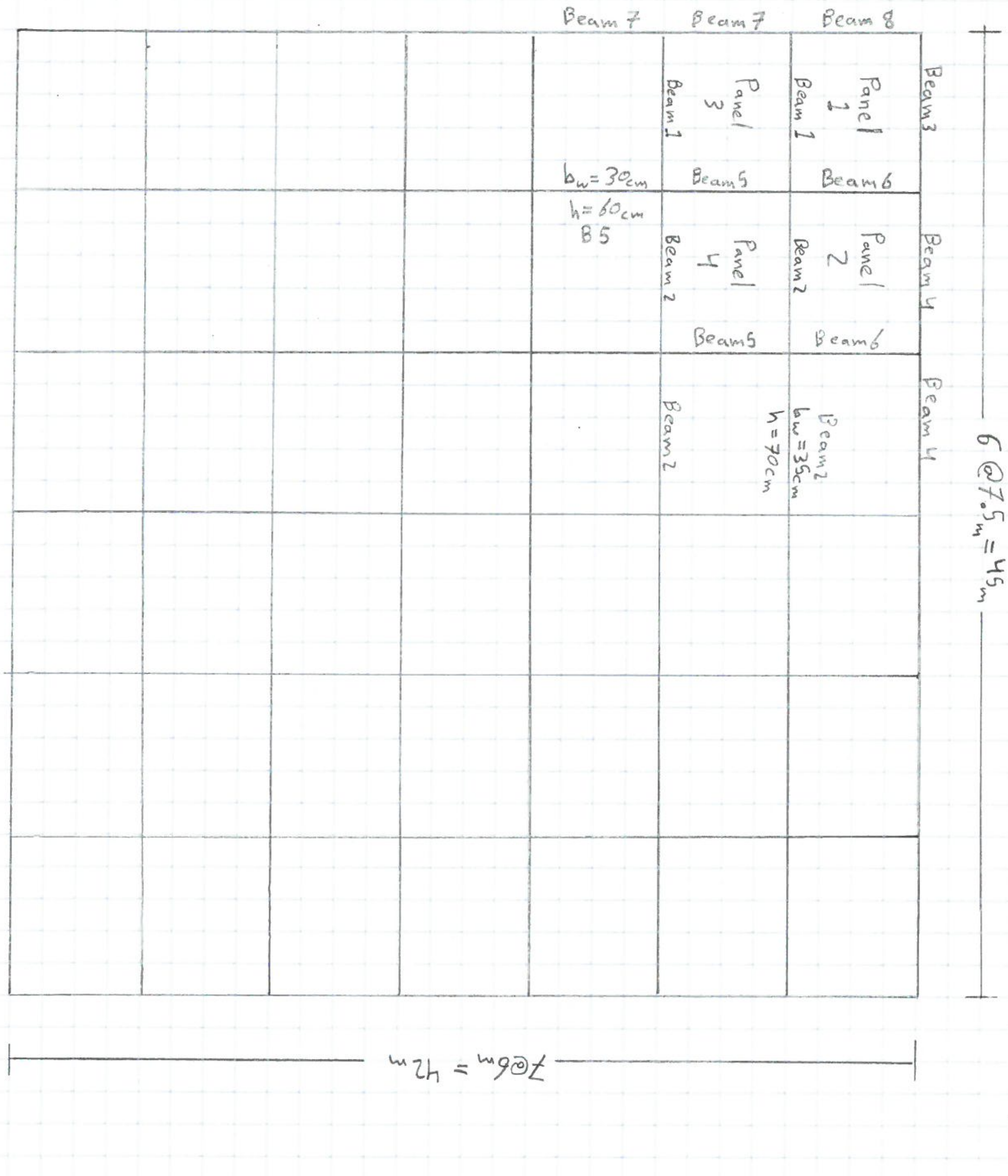
width of beam in short direction. = 30 cm, h = 60cm

width of beam in long direction. = 35 cm., h = 70cm

→ Table ACI - 8.3.1.2:

$$h_{min} = \max \left\{ 90 \text{ mm}, \; \frac{L_n \left(0.8 + f_y/1400\right)}{36 + 9\beta} = \frac{(720)\left[0.8 + \frac{420}{1400}\right]}{36 + 9(1.274)} = 16.7 \text{cm} \geq 9 \text{cm} \right.$$

$h_{min} = 17$ cm.



Beam 7
Beam 7
Beam 8
Beam 1
Panel 3
Beam 1
Panel 1
Beam 3
$b_w = 30_{cm}$
Beam 5
Beam 6
$h = 60_{cm}$
B5
Beam 2
Panel 4
Beam 2
Panel 2
Beam 4
Beam 5
Beam 6
Beam 2
$h = 70cm$
$b_w = 35_{cm}$
Beam 2
Beam 4
$6 @ 7.5_m = 45_m$
$7 @ 6_m = 42_m$

- To find α in Panels. (Ratio of Flexural stiffnesses of beam to slab).
determine α for Panels 1, 2, 3 and 4. only.
before it we should determine α for all beams around these Panels. ($B_1 - B_8$).

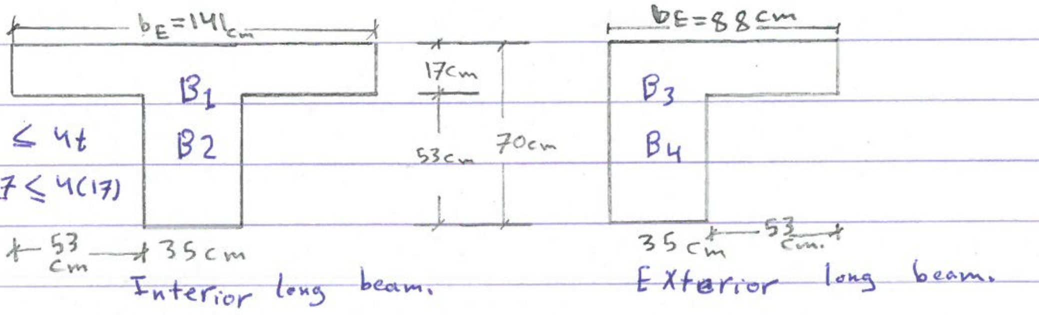
ⓐ Interior long beams – B_1, B_2

$h = 70$ cm, $b_w = 35$ cm, slab thickness = 17 cm.

$h - t \leq 4t$

$70 - 17 \leq 4(17)$

Controls = 53 cm.



$b_E/b_w = 4.028$, $t/h = 0.243$

$K = 1.7722$ from equation 16.4.2 b.

$I_b = K \frac{b_w h^3}{12} = 1.773 * 10^6 \text{ cm}^4$

$I_s = (600)(17)^3/12 = 245\,650 \text{ cm}^4$

$\alpha = \frac{E_{cb} I_b}{E_{cs} I_s}$, $E_{cb} = E_{cs}$ $\Rightarrow \alpha = 7.22$ | for B_1, B_2.

ⓑ Exterior long Beams B_3, B_4.

$\frac{b_E}{b_w} = \frac{88}{35} = 2.51$, $t/h = 0.243$

eq. 16.4.2 b $\Rightarrow K = 1.483$

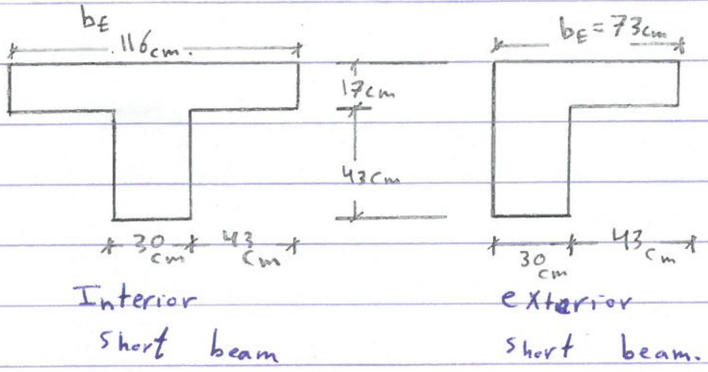
$I_b = K b_w \frac{h^3}{12} = 1.4836 \times 10^6 \text{ cm}^4$

$I_s = \frac{(300 + 35/2)(17)^3}{12} = 129\,989.8$

$\alpha = 11.41$ for B_3, B_4.

ⓒ Interior Short Beams B5 - B6

$h = 60\ cm$, $b_w = 30\ cm$, $slab = 17\ cm$.



$h - t = 60 - 17 = 43cm < 4t.$

$4(t) = 68cm$

$b_E/b_w = 3.867$, $t/h = 0.283$ $\Rightarrow K = 1.7558$

$I_b = (0.948155)10^6 cm^4$, $I_s = \frac{1}{12}(750)(17)^3 = 307062.5\ cm^4$

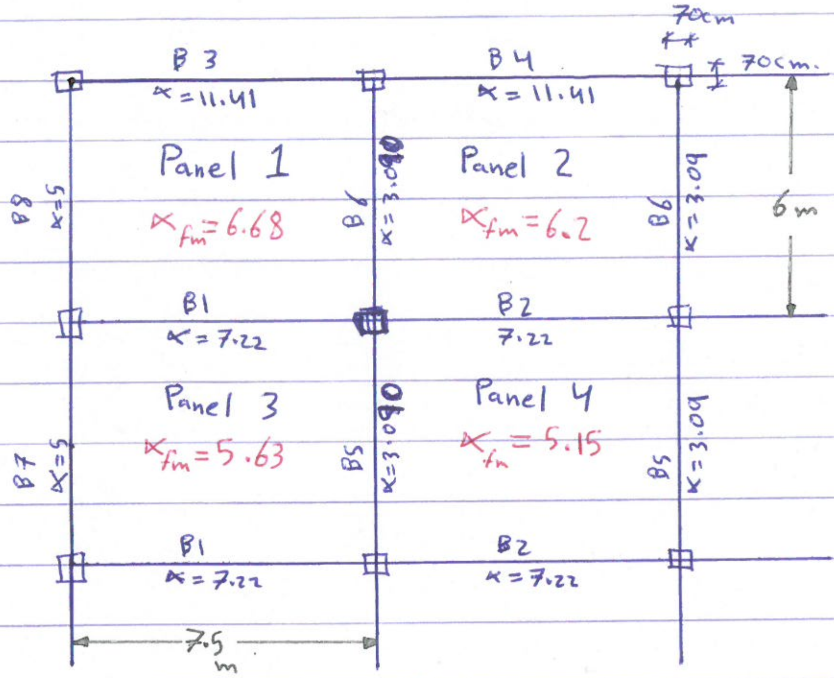
$\alpha = 3.088$ for B5, B6

ⓓ Exterior Short Beams. B7, B8.

$b_E/b_w = 2.433$, $t/h = 0.283$ $\Rightarrow K = 1.4775$

$I_b = 797\,850\ cm^4$, $I_s = \frac{1}{12}(750/2 + 30/2)(17)^3 = 159\,672.5\ cm^4$

$\alpha = 4.99 \approx 5.0$

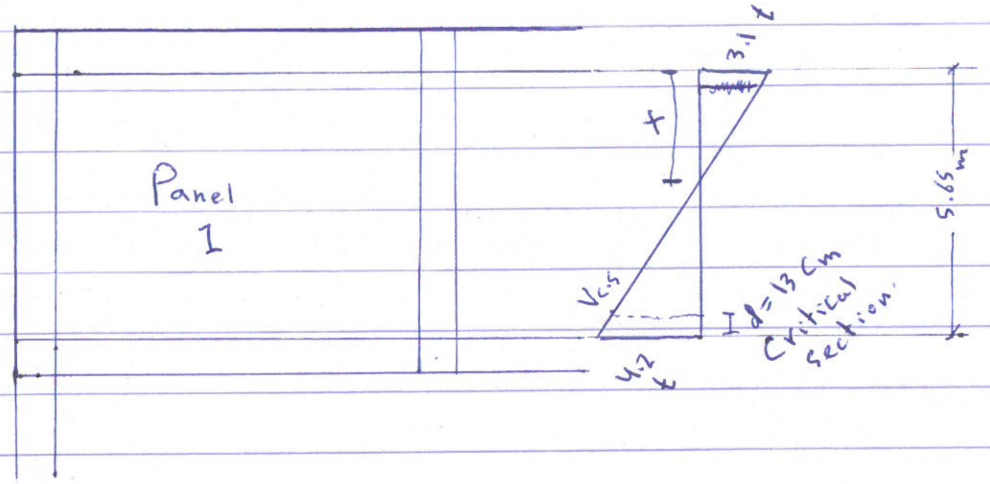


The ratio of flexural stiffness of beam to slab (α) > 2 in each Panel So the assumption is correct.

and the slab thickness = 17 cm. as determinate from (ACI -8.3.1.2).

$\Longrightarrow$ DL $\Rightarrow W_{DL} = (0.17)(2.4) = 0.408 \; t/m^2$

- Checking the Adequacy of the Slab Thickness for Shear.



$V_f = 1.15 \frac{w_u (L_n)}{2} = 4.2 \; t$

$(5.65)(1.29) - 4.2 = 3.1 \; t$

$d_{slab} = 17 - 4 = 13$ cm

$\phi V_c = \phi \, 0.17 \sqrt{f_c'} \, b d$

$= (0.75)(0.17) \frac{\sqrt{28}}{100} (100)(13)$

$= 8.77 \; t$

$\frac{4.2}{5.65 - x} = \frac{3.1}{x} \Rightarrow x = 2.4$ m

$\frac{V_{critical\ section}}{3.25 - d} = \frac{V_f}{3.25}$

$V_{c.s} = 4.032 \; t$

$\therefore \; \underset{8.77t}{\phi V_c} > \underset{4.032t}{V_u}$

- Check Direct Design method requirements:

1- Number of spans more than two in each direction. ✓

2- Rectangular Panel with ratio $\left(\frac{7.5}{6} = 1.25 \leq 2.0\right)$

3- equal spans lengths. (differ. $= 0 < 1/3$ of longer span).

4- offset of columns $= 0 \leq 10\%$ of span.

5- Gravity loads only, uniform, service $LL \leq 2DL$

$$DL = \underset{\text{thickness of slab}}{0.17\,m}\ \underset{t/m^3}{(2.4)} = 0.408\ t/m^2 \qquad 0.5 \leq (2)(0.408)\ ✓$$

6- $0.2 \leq (L_1^2/\alpha_1)/(L_2^2/\alpha_2) \leq 5.0$

- Check Limitation 6 for Panels: $L_1 = 7.5\,m$, $L_2 = 6\,m$

Panel (1) ⇒
$$\begin{cases} \alpha_1 = \dfrac{11.41+7.22}{2} = 9.315, & \dfrac{(L_1)^2}{\alpha_1} = \dfrac{(7.5)^2}{9.315} = 6.038 \\ \alpha_2 = \dfrac{5+3.09}{2} = 4.045, & \dfrac{(L_2)^2}{\alpha_2} = 8.9 \end{cases}$$

The ratio $= 1.47$
$0.2 \leq \quad \leq 5$

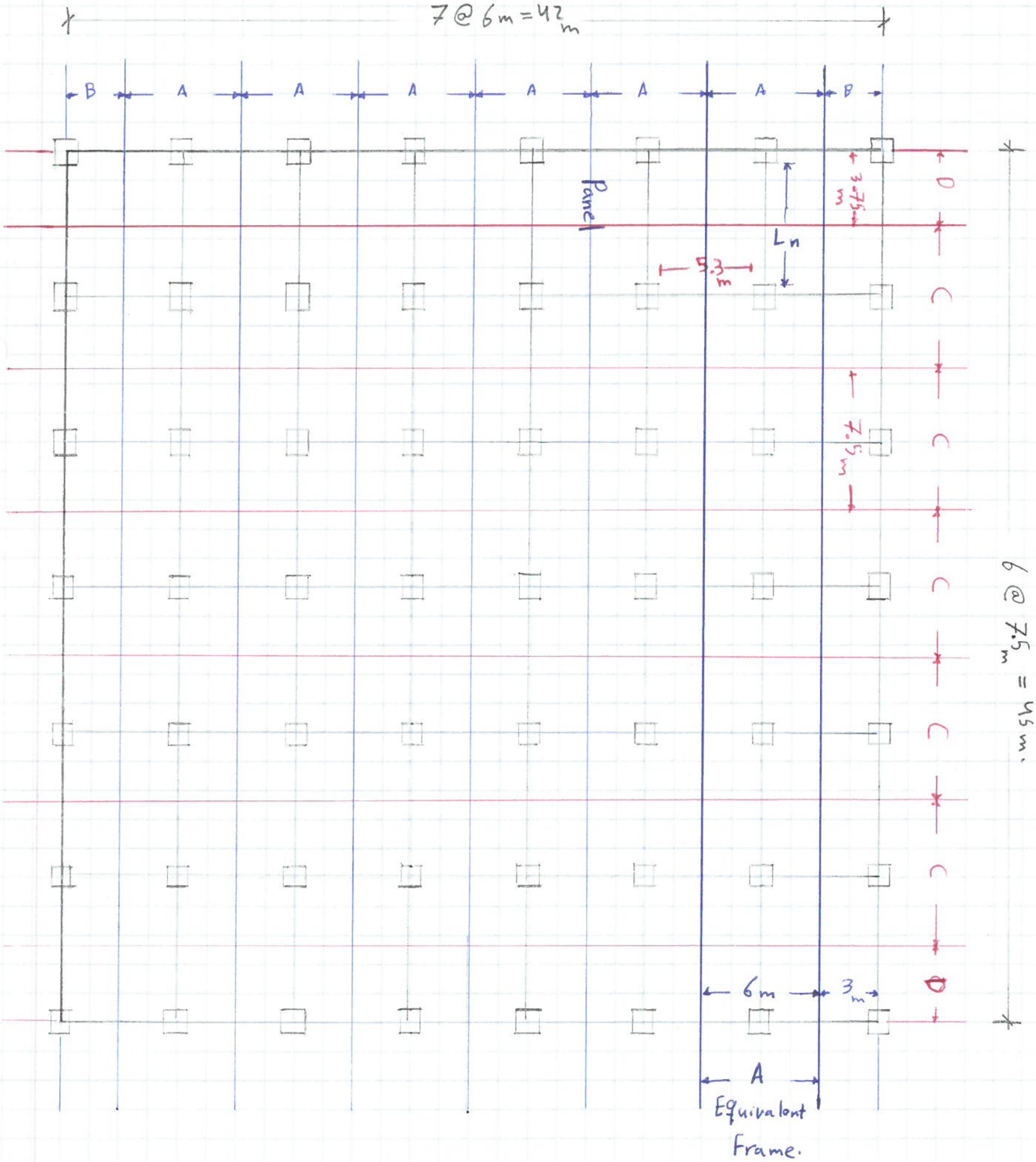
Panel (2) ⇒
$$\begin{cases} \alpha_1 = 9.315, & L_1^2/\alpha_1 = 6.038 \\ \alpha_2 = 3.09, & L_2^2/\alpha_2 = 11.65 \end{cases} \Rightarrow \text{The ratio} = 1.93 \quad 0.2 \leq \quad \leq 5$$

Panel (3) ⇒
$$\begin{cases} \alpha_1 = 7.22, & L_1^2/\alpha_1 = 7.79 \\ \alpha_2 = 4.045, & L_2^2/\alpha_2 = 8.9 \end{cases} \qquad \text{ratio} = 0.87 \quad 0.2 \leq \quad \leq 5$$

Panel (4) ⇒
$$\begin{cases} \alpha_1 = 7.22, & L_1^2/\alpha_1 = 7.79 \\ \alpha_2 = 3.09, & L_2^2/\alpha_1 = 11.65 \end{cases} \qquad \text{ratio} = 0.67 \quad 0.2 \leq \quad \leq 5$$

all ratios between 0.2 and 5.

- All requirements of the direct design method are Satisfied.


7 @ 6m = 42 m
B
A
A
A
A
A
A
B
Panel
3.75 m
Ln
5.3 m
7.5 m
D
C
C
C
C
C
D
6 @ 7.5 m = 45 m.
6m
3 m
A
Equivalent Frame.

- Determine M_o for each span of the four equivalent rigid frame and the longitudinal distribution.

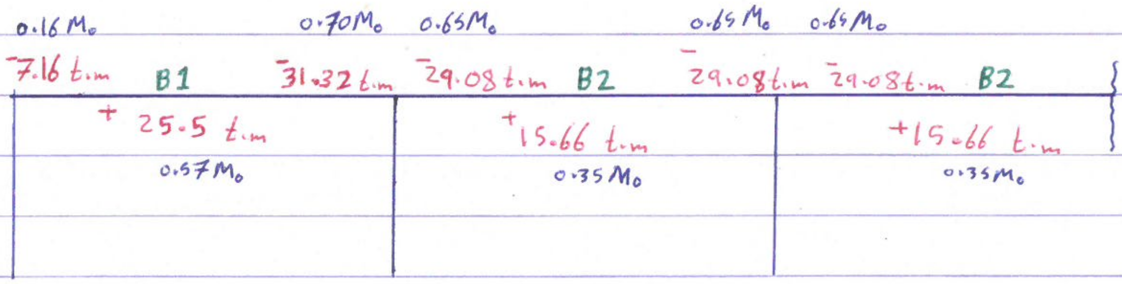
$$W_u = 1.2\ DL + 1.6\ LL = (1.2)(0.408) + (1.6)(0.5) = 1.29\ t/m^2$$

- For frame A : $L_n = 7.5 - 0.7 = 6.8\ m$

$L_2 = 6\ m$

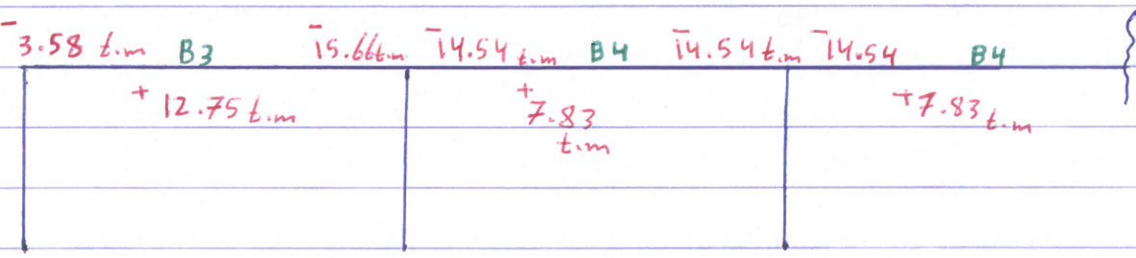
$M_o = 1/8\ W_u L_2 L_n^2 = 44.74\ t.m$ (in all spans in frame A).

fig. 16.8.2



- For frame B : $L_n = 6.8\ m$, $L_2 = 3\ m$

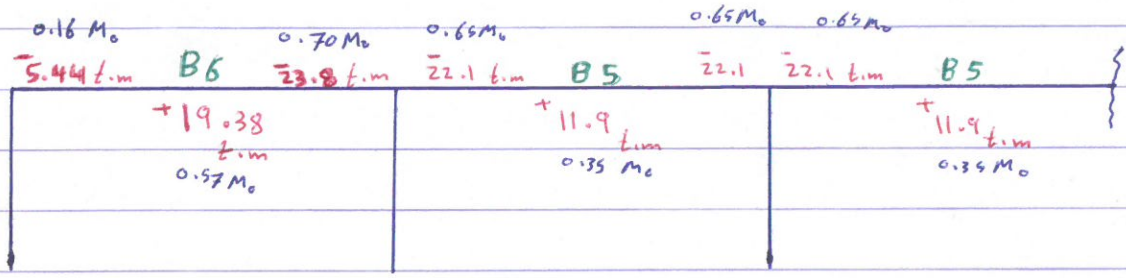
$M_o = 22.37\ t.m$ (all spans in frame B).



- For frame C : $L_n = 6 - 0.7 = 5.3$ m

$L_2 = 7.5$ m

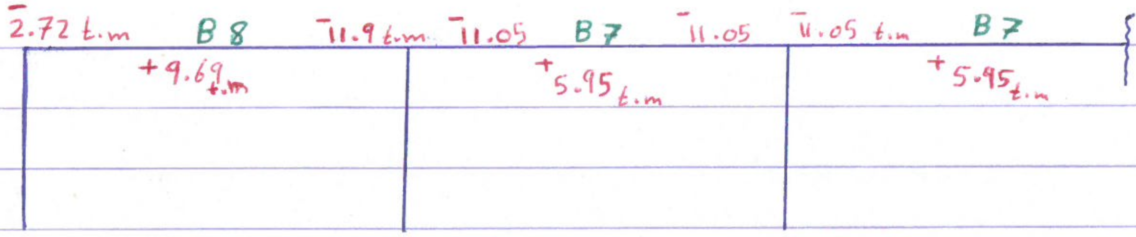
$M_o = \frac{1}{8} W_u \; L_2 \; (L_n)^2 = 34$ t.m (in all spans in frame C)



- for frame D : $L_n = 5.3$ m

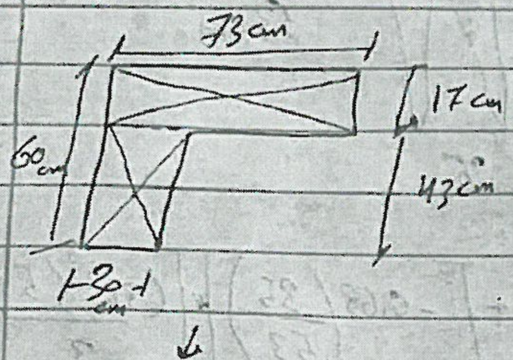
$L_2 = 3.75$ m

$M_o = 17$ t.m (in all spans in frame D).

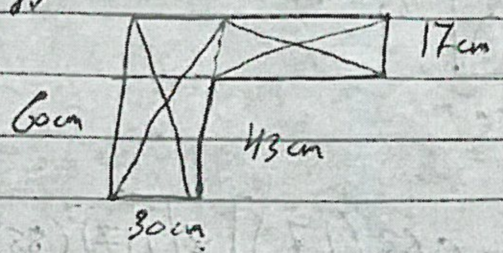


torsional Constant C for the edge beams.

Short direction : Beam B7, B8 - edge



or



$$C = \left[1 - 0.63\left(\frac{17}{73}\right)\right] \times \left[\frac{(17)^3(73)}{3}\right] + \left[1 - 0.63\left(\frac{30}{43}\right)\right] \times \left[\frac{(30)^3(43)}{3}\right]$$

$$= 1.02 \times 10^5 \ cm^4 + 2.17 \times 10^5 \ cm$$

$$= 3.19 \times 10^5 \ cm^4$$

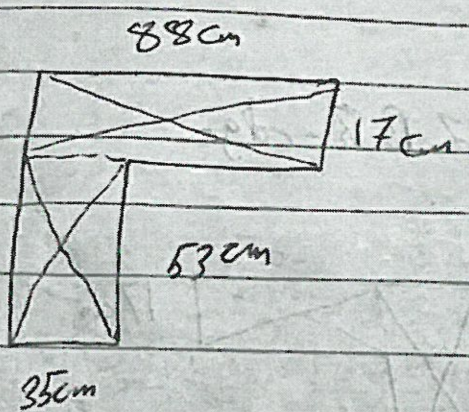
or

$$C = \left[1 - 0.63\left(\frac{17}{43}\right)\right] \times \left[\frac{(17)^3(43)}{3}\right] + \left[1 - 0.63\left(\frac{30}{\cancel{30}60}\right)\right] \times \left[\frac{(30)^3(60)}{3}\right]$$

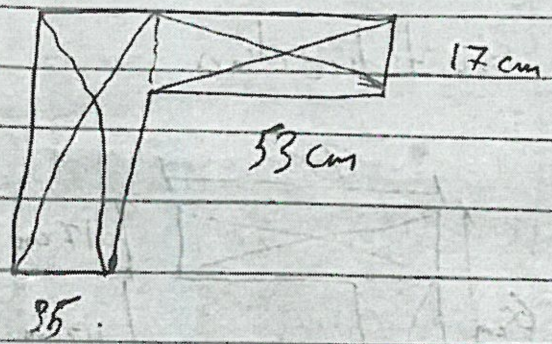
$$= 5.26 \times 10^4 \ cm^4 + 3.70 \times 10^5 \ cm^4$$

$$= \underline{\underline{4.23 \times 10^5 \ cm^4}}$$

Long direction: Beams: B_3, B_4 - edge



or



$$C = \left[1 - 0.63\left(\frac{17}{88}\right)\right] \times \left[\frac{(17)^3(88)}{3}\right] + \left[1 - 0.63\left(\frac{35}{53}\right)\right] \times \left[\frac{(35)^3(53)}{3}\right]$$

$$= 1.27 \times 10^5 \text{ cm}^4 + 4.42 \times 10^5 \text{ cm}^4$$

$$= 5.69 \times 10^5 \text{ cm}^4$$

or

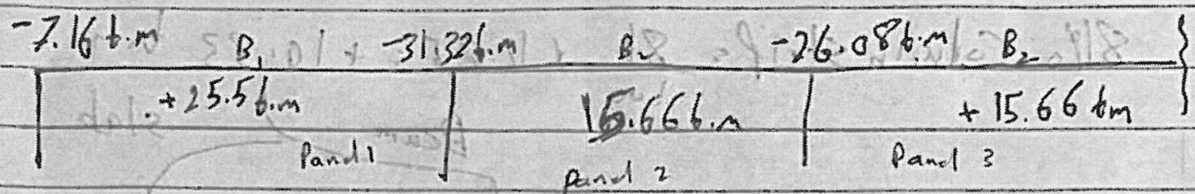
$$C = \left[1 - 0.63\left(\frac{17}{53}\right)\right] \times \left[\frac{(17)^3(53)}{3}\right] + \left[1 - 0.63\left(\frac{35}{70}\right)\right] \times \left[\frac{(35)^3(70)}{3}\right]$$

$$= 6.93 \times 10^4 \text{ cm}^4 + 6.85 \times 10^5 \text{ cm}^4$$

$$\underline{C = 7.54 \times 10^5 \text{ cm}^4}$$

Lateral Distribution of moment :

for frame A :



Panel 1

for negative moment -7.16 t.m (edge)

$L_2 = 6^m$, $L_1 = 7.5^m$, $\alpha_1 = 7.22$, $C = 4.23 \times 10^5$ cm

$L_2/L_1 = 0.80$

$\alpha_1 L_2/L_1 = 5.78$

$I_s = \frac{(600)(17^3)}{12} = 2.457 \times 10^5$ cm^4

$B_t = C/2I_s = 0.86$

	0.5	0.8	1
$B_t = 0$	100%	100%	100%
$B_t = 0.86$		93.5 %	
$B_t \geq 2.5$	90%	81%	75%

$\therefore$ 93.5 % Column Strip $= \frac{93.5}{100} \times -7.16 = -6.69$ t.m

85% Beam | 15% Slab

$\frac{85}{100} \times 6.69 = -5.69$ t.m

$\frac{15}{100} \times 6.69 = -1$ t.m

100 - 93.5 = 6.5 %

6.5% middle strip $= \frac{6.5}{100} \times -7.16$

$= 0.47$ t.m

for positive moment + 25.5 t.m

	0.5	0.8	1
$\alpha_1 L_2/L_1 \geq 1$	90	81 %	75

$\therefore$ 81 % Column strip = $\frac{81}{100} \times 25.5 = +20.66$ t.m

85 % Beam → $\frac{85}{100} \times 20.66 = +17.56$ t.m

15 % slab → $\frac{15}{100} \times 20.66 = 3.1$ t.m

100 − 81 = 19 % ~~for~~ for middle strip

$\frac{19}{100} \times 25.5 = +4.85$ t.m

for negative moment −31.32 t.m (Interior).

	0.5	0.8	1
$\alpha_1 L_2/L_1 \geq 1$	90	~~75~~ 81	75

81 % Column strip $\frac{81}{100} \times -31.32$

= −25.37

Beam 85 % → −21.56 t.m

slab 15 % → −3.81 t.m

19 % for middle strip

$= \frac{19}{100} \times -31.32 = -5.95$ t.m for Second Panel (B2)

for positive moment + 15.66 t.m

$\alpha = 7.22$, $L_2 = 6$ m , $L_1 = 7.5$ m

$L_2/L_1 = 0.80$, $\alpha L_2/L_1 = 5.78 > 1$

0.5	0.8	1
90%	81%	75%

$\therefore$ 81% Column strip $= \frac{81}{100} \times 15.66$

$= {}^{+}12.68$ t.m

Beam — slab

19% for middle strip

$\frac{19}{100} \times 15.66 = {}^{+}2.98$ t.m

$\frac{85}{100} \times 12.68 = {}^{+}10.78$ t.m

$\frac{15}{100} \times 12.68 = {}^{+}1.90$ t.m

for negtive moment = 26.08 t.m

81% Column strip $= \frac{81}{100} \times 26.08 = -21.12$ t.m

Beam — slab

$\frac{85}{100} \times 21.12 = 17.95$ t.m

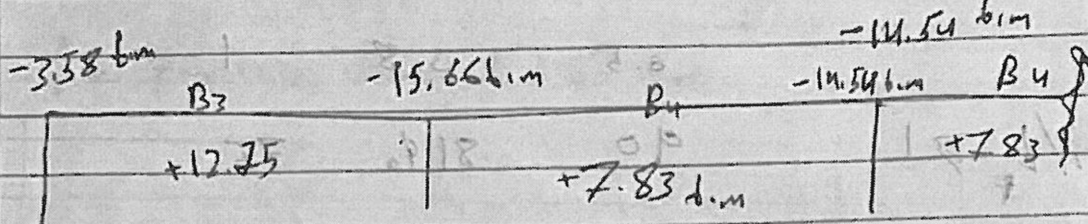
$\frac{15}{100} \times 21.12 = -3.17$ t.m

19% for middle strip

$= \frac{19}{100} \times 26.08 = -4.96$ t.m

and so for — — — (same)

for frame B:.



for negative moment: -3.58 t.m (edge)

$L_2 = 6m$, $L_1 = 7.5 m$, $\alpha_1 = 11.41$, $C = 4.23 \times 10^5$ cm^4

$L_2/L_1 = 0.80$, $\alpha_1 L_2/L_1 = 9.13 \geq 1$ $\beta_t = \frac{C}{2 I_s} = 1.63$

$I_s = \frac{(317.5)(17^3)}{12}$

	0.5	0.8	1
$\beta_t = 0$	100 %	100 %	100 %
$\beta_t = 1.63$		87.61 %	
$\beta_t \geq 2.5$	90 %	81 %	75 %

∴ 87.61 % Coulumn strip

$= \frac{87.61}{100} \times -3.58 = -3.14$ t.m

Beam — slab

$\frac{85}{100} \times -3.14 = 2.67$ t.m

$\frac{15}{100} \times -3.14 = 0.47$ t.m

$100 - 87.61 = 12.39$ %

for middle strip

$= \frac{12.39}{100} \times -3.58 = 0.44$ t.m for positive moment +12.75 t.m

	0.5	0.8	1
$\alpha L_2/L_1 \geq 1$	90%	81%	75%

81% Column strip = $\frac{81}{100} \times 12.75 = +10.33$ t.m

Beam — slab

$\frac{85}{100} \times 10.33$ — $\frac{15}{100} \times 10.33$

$= +8.78$ t.m — $= +1.55$ t.m

19% for middle strip

$\frac{19}{100} \times 12.75 = +2.42$ t.m

for Negitive moment −15.66 t.m :

81% Column strip = $\frac{81}{100} \times -15.66 = -12.68$ t.m

Beam — slab

$\frac{85}{100} \times -12.68$ — $\frac{15}{100} \times -12.68$

19% for middle strip

$= -10.78$ t.m — $= -1.90$ t.m

$\frac{19}{100} \times -15.66$ t.m. $= -2.98$ t.m for second panel (B_4):

for positive moment +7.83 t.m

$L_2/L_1 = 0.8 \quad \alpha\, L_2/L_1 = 9.13 \geq 1$

	0.5	0.8	1
	90	81	75

81 % Coulmn strip

$\frac{81}{100} \times 7.83 = +6.34$ t.m

Beam 85% → + 5.39 t.m

slab 15% → +0.95 t.m

19 % for middle strip $= \frac{19}{100} \times 7.83 = +1.49$ t.m

for negative moment -14.54 t.m

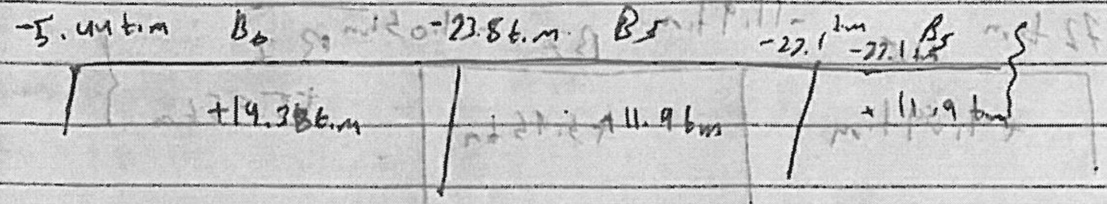
81 % Coulmn strip $= \frac{81}{100} \times -14.54 = -11.78$ t.m

Beam: $\frac{85}{100} \times -11.78 = -10.01$ t.m

slab: $\frac{15}{100} \times -11.78 = -1.77$ t.m

19 % for middle strip $= \frac{19}{100} \times -14.54 = -2.76$ t.m

for frame C :



$\alpha = 3.09$, $L_2 = 7.5\,m$, $L_1 = 6\,m$, $C = 7.54 \times 10^5\,cm^4$

$L_2 / L_1 = 1.25$ $\alpha\, L_2 / L_1 = 3.86 \geq 1$ $I_s = \frac{(750)(12)^3}{12} = 3.07 \times 10^5\,cm^4$

$B_6 = \frac{C}{2I} = 1.23$

for negative moment -5.44 t.m (edge)

	1	1.25	2
$B_6 = 0$	100%	100%	100%
$B_6 = 1.23$		84.01%	
$B_6 \geq 2.5$	75%	67.5%	45%

∴ 84.01% Column strip

$\frac{84.01}{100} \times -5.44 = -4.57$ t.m

Beam 85% → -3.88 t.m

slab → -0.69 t.m

100% - 84.01% = 15.99%

∴ 15.99 for middle strip

$= \frac{15.99}{100} \times -5.44 = 0.87$ t.m for positive moment $+19.38\ t.m$

$\alpha L_2/L_1 \geq 1$	1.	1.25	2
	75%	67.5%	45%

∴ 67.5% for coulmn strip

$\frac{67.5}{100} \times 19.38 = +13.08\ t.m$

Beam 85% | slab 15%

$+11.12\ t.m$ | $+1.96\ t.m$

$100 - 67.5 = 32.5\%$ for middle strip

$\frac{32.5}{100} \times 19.38 = +6.30\ t.m$

for negtive moment $-23.8\ t.m$

67.5% for coulmn strip $= \frac{67.5}{100} \times 23.8 = -16.07\ t.m$

Beam | Slab

$\frac{85}{100} \times -16.07 = -13.66\ t.m$ | $\frac{15}{100} \times -16.07 = -2.41\ t.m$

32.5% for middle strip

$\frac{32.5}{100} \times 23.8\ t.m$

$= -7.74\ t.m$

for Second panel (B5):

for positive moment +11.9 t.m

1.25   2

~~75%~~ 67.5%   45%

67.5% Column strip

$\alpha_1 L_2 / L_1 > 1$

$\frac{67.5}{100} \times 11.9 = 8.03$ t.m

Beam 85%   slab 15%

; 32.5% for middle strip   +6.83 t.m   1.20 t.m

$\frac{32.5}{100} \times 11.9 = +3.87$ t.m

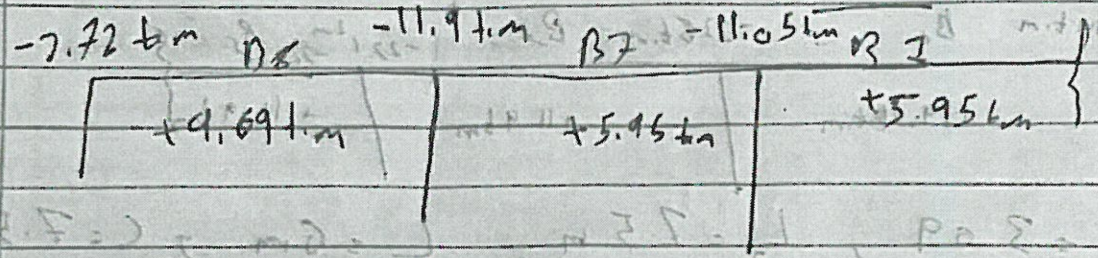
for negative moment −22.1 t.m

67.5% for Column strips $= \frac{67.5}{100} \times -22.1 = 14.92$ t.m

Beam 85%   slab 15%

32.5% for middle strip   −12.68 t.m   −2.24 t.m

$\frac{32.5}{100} \times -22.1 = -7.18$ t.m for frame D:



$\alpha = 5$, $L_2 = 7.5$, $L_1 = 6m$, $C = 7.54 \times 10^5\ cm^4$

$L_2/L_1 = 1.25$, $\alpha L_2 / L_1 = 6.25 \geq 1$ $\quad I_s = \frac{(315)(17)^3}{12}$

$B_t = \frac{C}{2I_s} = 2.9$

	1	1.25	2
for negative moment (-2.72 t.m) $B_t \geq 2.5$	75%	67.5%	45%

∴ 67.5% for column strip

$= \frac{67.5}{100} \times -2.72 = -1.84$ t.m

Beam / slab

85% → -1.56 t.m

15% → - 0.28 t.m

∴ 32.5% for middle strip $= \frac{32.5}{100} \times -2.72$

$= -0.88$ t.m

for Positive moment +9.69 t.m

$\alpha L_2/L_1$, γ_t, 1	1	1.25	2
	75	67.5	45

∴ 67.5 % for Column Strip

$= \frac{67.5}{100} \times +9.69 = +6.54$ t.m

Beam 85% → +5.56 t.m

Slab 15% → +0.98 t.m

∴ 32.5% for middle Strip

$\frac{32.5}{100} \times +9.69 = +3.15$ t.m

for negative moment -11.9 t.m ∴

67.5 % for Column Strip

$\frac{67.5}{100} \times -11.9 = -8.03$ t.m

Beam 85% → -6.83 t.m

Slab 15% → -1.20 t.m

∴ 32.5% for middle Strip

$\frac{32.5}{100} \times -11.9 = -3.87$ t.m for second Panel (B7).

~~for~~ the same

for Positive moment +5.95 t.m

67.5% for Column strip

$\frac{67.5}{100} \times +5.95 = +4.02$ t.m

Beam 85% | Slab 15%

+3.42 t.m | +0.60 t.m

∴ 32.5% for middle strip

$\frac{32.5}{100} \times +5.95 = +1.93$ t.m

for negtive moment -11.05 t.m $\alpha\, l_2/l_1 \geq 1$

67.5% for Column strip $= \frac{67.5}{100} \times -11.05 = -7.46$ t.m

Beam 85% | Slab 15%

-6.34 t.m | -1.12 t.m

32.5% for middle strip

$\frac{32.5}{100} \times -11.05 = -3.59$ t.m

and so for — -

Fram A :.

total width = 6m, Coulmn strip width = 3 m, middle strip width = 3 m

	Exterior span			Interior span	
	Ex. -Vc	+Vc	-Vc	-Vc	+Vc
total moment	-7.16	+25.5	-31.32	-26.08	+15.66
Beam	-5.69	+17.56	-21.56	-17.95	+10.78
slab	-1	+ 3.1	- 3.81	-3.17	+1.90
middle strip slab	-0.47	+ 4.85	-5.95	-4.96	+ 2.98

Fram B:.

total width = 3m, Coulmn strip width = 1.5m, middle strip width = 1.5m

	Exterior span			Interior span	
	Ex. -Vc	+Vc	-Vc	-Vc	+Vc
total moment :.	-3.58	+12.75	-15.66	-14.54	+7.83
Beam :	-2.67	+8.78	-10.78	-10.01	+5.39
slab,	-0.47	+1.55	-1.90	-1.77	+0.95
middle strip slab	-0.44	+2.42	-2.98	-2.76	+1.49

Fram C:

total width = 7.5m, Coulmn stripwidth = 3m, middle stripwidth = 4.5m

	Exterior span			interior span	
	Ex. -Vc	+Vc	-Vc	-Ve	+Ve
total moment :	-5.44	+19.38	-23.8	-22.1	+11.9
Beam :	-3.88	+11.12	-13.66	-12.68	+6.83
slab :	-0.69	+1.96	-2.41	-2.24	+1.20
middle stripslab :	-0.87	+6.30	-7.74	-7.18	+3.87

Fram D:

total width = 3.75m, Coulmn stripwidth = 1.5m, middle stripwidth = 2.25m

	Exterior span			Interior span	
	Ex. -Vc	+Vc	-Vc	-Vc	+Vc
total moment :	-2.72	+9.69	-11.9	-11.05	+ 5.95
Beam :	-1.56	+5.56	-6.83	-6.34	+3.42
slab	-0.28	+0.98	-1.20	-1.12	+0.60
middle stripslab :	-0.88	+3.15	-3.87	-3.59	+1.93

*Design of slab reinforcement:-

$$\boxed{d = 17 - 4 = 13\,cm}$$

→ $A_{s\,min} = 0.0018\,bh = 0.0018(100)(17) = 3.06\,cm^2$

→ $A_{s\,max} = 0.01806\,bd = 0.01806(100)(13) = 23.478\,cm^2$

$S_{max} \le \begin{cases} 2h = 2\times17 = 34\,cm & \checkmark \text{ controls} \\ 45\,cm \end{cases}$

* Fram A & C:-

• Exterior Span:-

→ Middle strip:-

$\boxed{\text{• Exterior negative moment:-}}$

→ $M_A = -0.47\ t.m$ → $M_C = -0.87\ t.m$

→ $M_{A_u} = \frac{0.47}{2(1.5)} = 0.156\ \frac{t.m}{m}$ → $M_{C_u} = \frac{0.87}{2(2.25)} = 0.193\ \frac{t.m}{m}$

$M_{cu} > M_{Au}$

→ design at $M_{Cu} = 0.193\ \frac{t.m}{m}$

• $M_n\ req = \frac{0.193}{0.9} = 0.214\ t.m$

• $R_n\ req = \frac{0.214\times100}{100(13)^2} = 0.00127$

• $m = f_y/0.85 f'_c = 17.64$

• $\rho_{req} = \frac{1}{17.64}\left(1 - \sqrt{1 - \frac{2(17.64)(0.00127)}{4.2}}\right) = 0.003$

• $A_s\ req = \rho_{req}\,bd = 0.39\,cm^2 < A_{s\,min}$, use $A_{s\,min}$

• #bars $= \frac{3.06}{\frac{\pi}{4}(1.2)^2} = 2.7$ use $3\phi12$

• Cheack spacing $= \frac{100}{3} = 33.3\,cm < S_{max}$ ✓

use $\phi12$ @ 30 cm

⇒ The other design are calculated In the Same way using Excel Sheet :-

$\boxed{\text{Exterior positive moment :-}}$

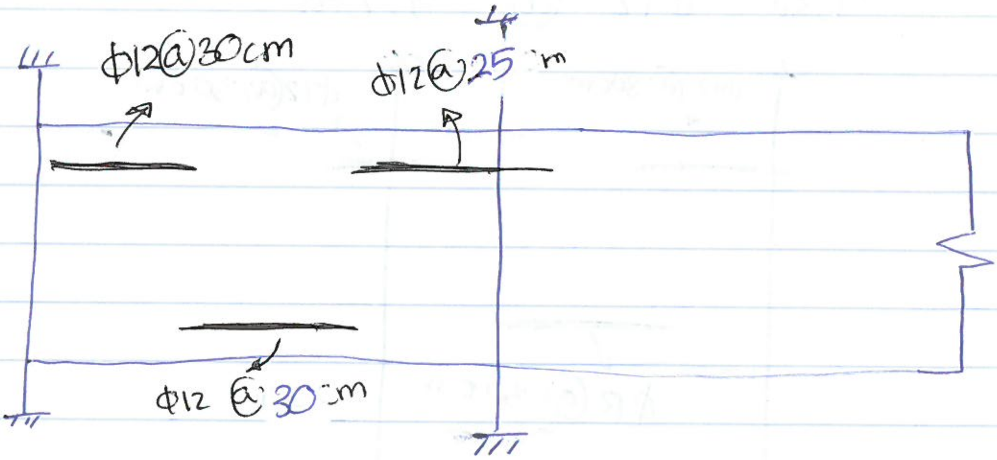
$M_A = 4.85$ t.m $\qquad M_C = 6.3$ t.m

Use $\phi 12$ @ 30 cm

$\boxed{\text{Negative Interior moment :-}}$

$M_A = -5.95$ t.m $\qquad M_C = -7.74$ t.m

Use $\phi 12$ @ 25 cm



→ Exterior span (coulmn Strip):-

$\boxed{\text{• Negativ moment}}$

$M_A = -1\ t.m$ $M_C = -0.69\ t.m$

Use $\phi 12$ @ 30 cm

$\boxed{\text{• Positive moment}}$

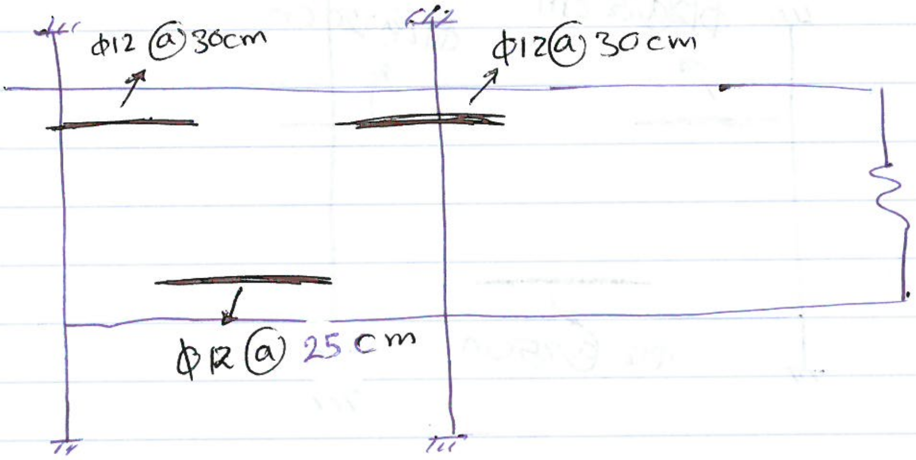
$M_A = 3.1\ t.m$ $M_C = 1.96\ t.m$

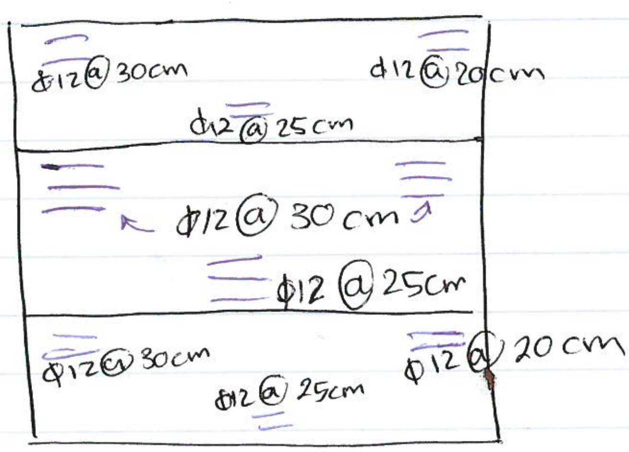
Use $\phi 12$ @ 25cm

→ Interior Negative momen to the (Column Strip):

$M_A = -1.9\ t.m$ $M_C = -2.41\ t.m$

Use $\phi 12$ @ 30 cm





→ Interior span:-

• Middle strip:-

• Negative moment:-

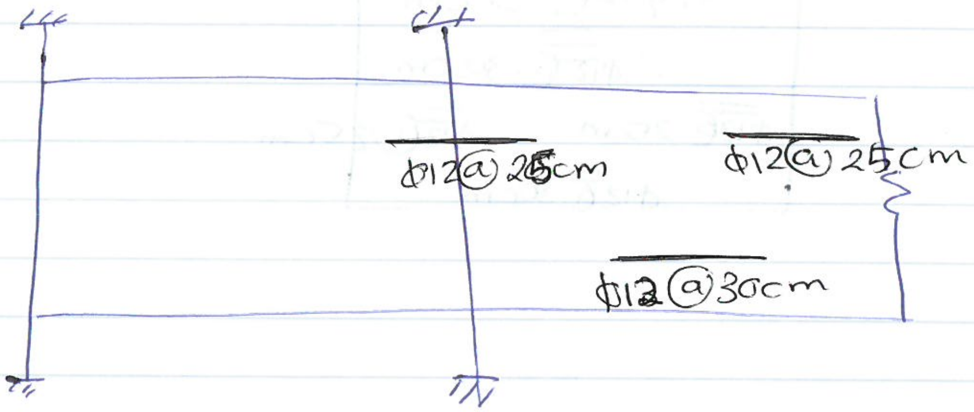
$M_A = -4.96$ t.m $\qquad M_C = -7.18$ t.m.

Use $\phi 12$ @ 25 cm

• positive moment:-

$M_A = +2.98$ t.m $\qquad M_C = 3.87$ t.m

Use $\phi 12$ @ 30 cm



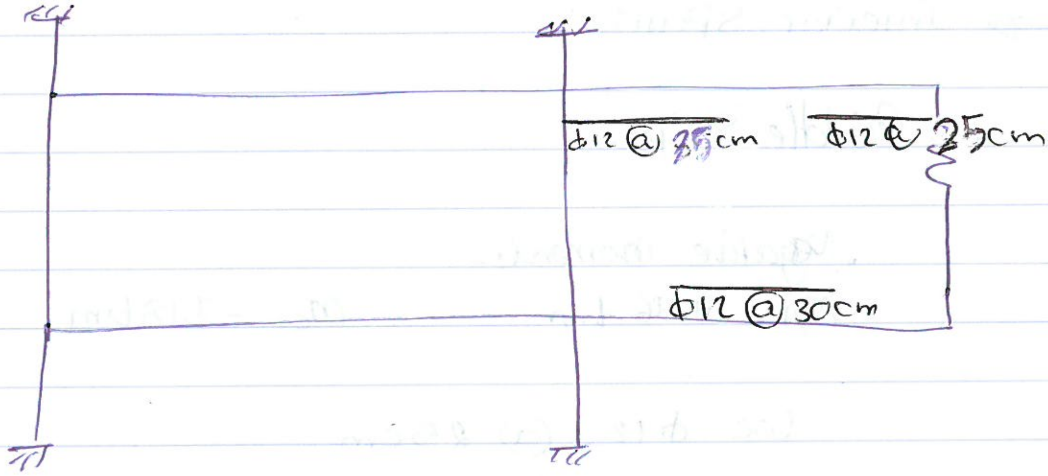
→ column strip:-

Negative moment $\quad$ • $M_A = -3.17$ t.m $\qquad$ • $M_C = -2.24$ t.m

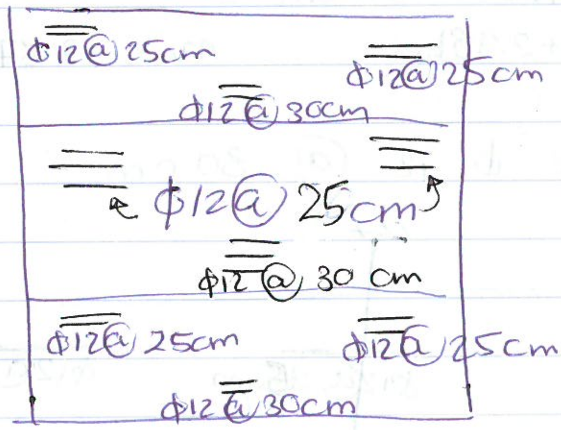
Use $\phi 12$ @ 25 cm

positive moment $\quad$ • $M_A = 1.9$ t.m $\qquad$ • $M_C = 1.2$ t.m

Use $\phi 12$ @ 30 cm



ϕ12 @ 25cm
ϕ12 @ 25cm
ϕ12 @ 30cm



ϕ12 @ 25cm
ϕ12 @ 25cm
ϕ12 @ 30cm
ϕ12 @ 25cm
ϕ12 @ 30 cm
ϕ12 @ 25cm
ϕ12 @ 25cm
ϕ12 @ 30cm

* Reinforcement for fram B and D:

* Exterior span:

⇒ Middle strip :

* Exterior neg. moment "B" $= \dfrac{-0.44}{2} = \dfrac{-0.22}{0.75}$

$$(Mu)_B = -0.293 \text{ t.m}$$

* Exterior neg. moment "D" $= \dfrac{-0.88}{2} = \dfrac{-0.44}{1.125}$

$$(Mu)_D = -0.391 \text{ t.m}$$

* Take $(Mu)_D = -0.391$ t.m

- $(Mn)_{req.} = \dfrac{0.391}{0.9} = 0.434$ t.m

- $(Rn)_{req.} = \dfrac{(0.434) * 100}{100 * (13)^2} = 0.00257$

- $\rho_{req.} = \dfrac{1}{17.64}\left[1 - \sqrt{1 - \dfrac{2(17.64)(0.00257)}{4.2}}\right]$

$= 0.000615$

- $(As)_{req.} = \rho_{req.}\, bd = 0.8 \text{ cm}^2 < (As)_{min.}$

∴ use $(As)_{min.}$

- $\# \text{ of bars} = \dfrac{3.06}{\pi(1.2)^2} = 2.7$,, use $3 \varnothing 12$

  - Check spacing: $S = \dfrac{100}{3} = 33.3 \text{ cm} < S_{max}$

    use $\varnothing 12$ @ 30 cm

* By using Excel Sheet :

- For Exterior span "Middle strip" for positive moment :

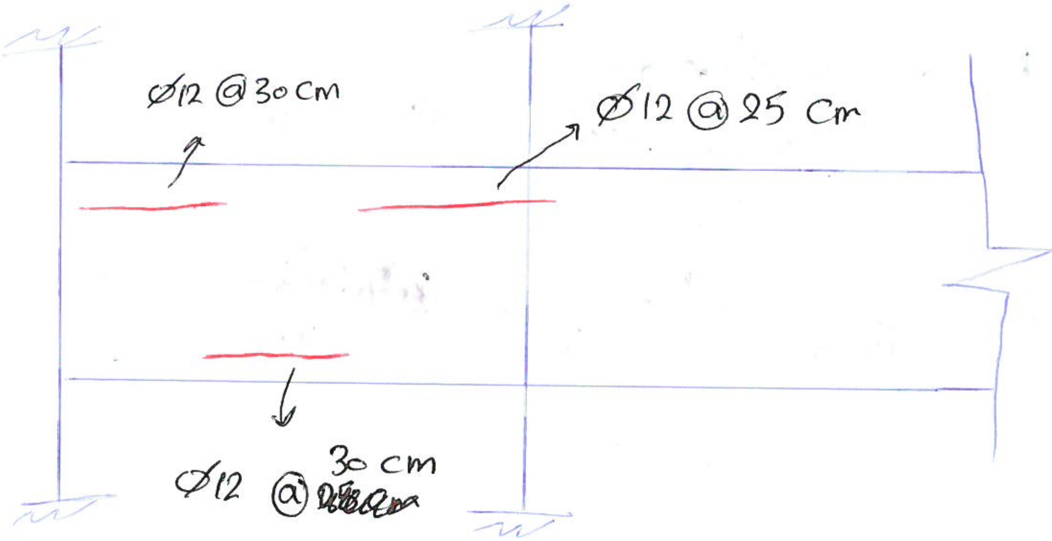
  $(M_u)_B = 2.42 \text{ t.m}$,, $(M_u)_D = 3.15 \text{ t.m}$

  use $\varnothing 12$ @ 30 cm

- Negative interior moment "middle strip" :

  $(M_u)_B = -2.98 \text{ t.m}$,, $(M_u)_D = -3.87 \text{ t.m}$

  use $\varnothing 12$ @ 25 cm



* Exterior span (column strip):

- for negative moment:

$(M_u)_B = -0.47$ t.m ,, $(M_u)_D = -0.28$ t.m

use $\phi 12$ @ 30 cm

- for positive moment:

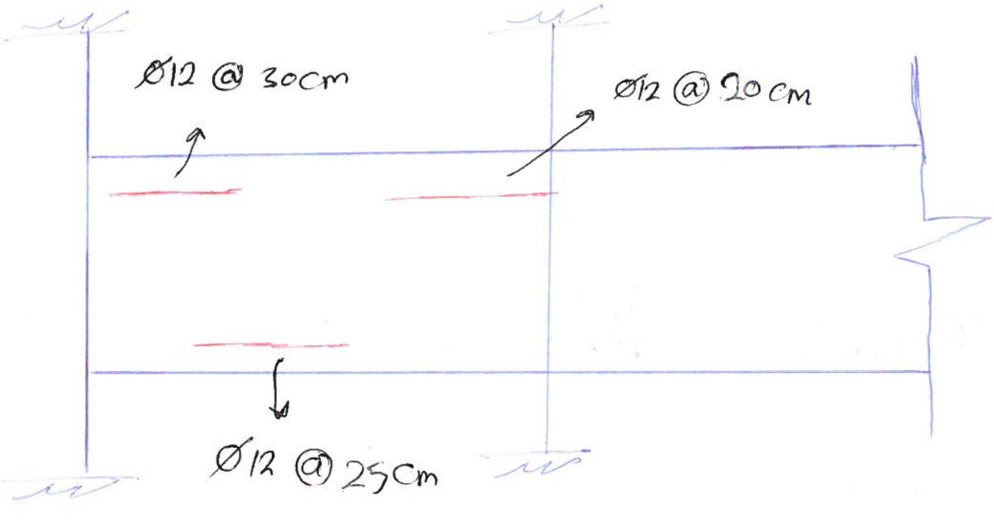
$(M_u)_B = 1.55$ t.m ,, $(M_u)_D = 0.98$ t.m

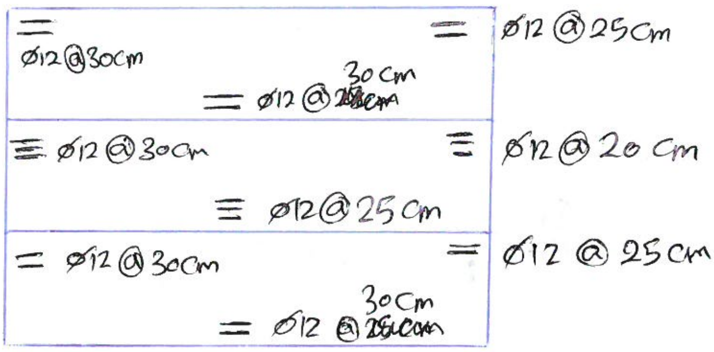
use $\phi 12$ @ 25 cm

* Interior negative moment for column strip:

$(M_u)_B = -1.9$ t.m ,, $(M_u)_D = -1.2$ t.m

use $\phi 12$ @ 20 cm





* Interior Span:

Middle strip:

- Negative moment:

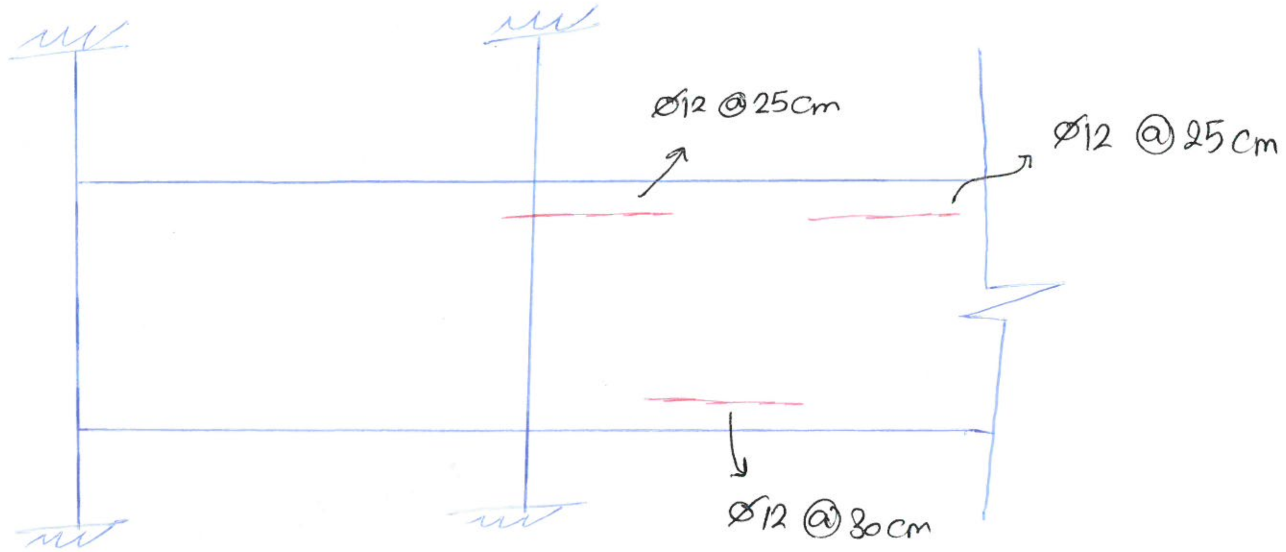
$(M_u)_B = -2.76$ t.m ,, $(M_u)_D = -3.59$ t.m

use Ø12 @ 25 cm

- Positive moment:

$(M_u)_B = 1.49$ t.m ,, $(M_u)_D = 1.93$ t.m

use Ø12 @ 30 cm



* Interior span "Column Strip":

- negative moment:

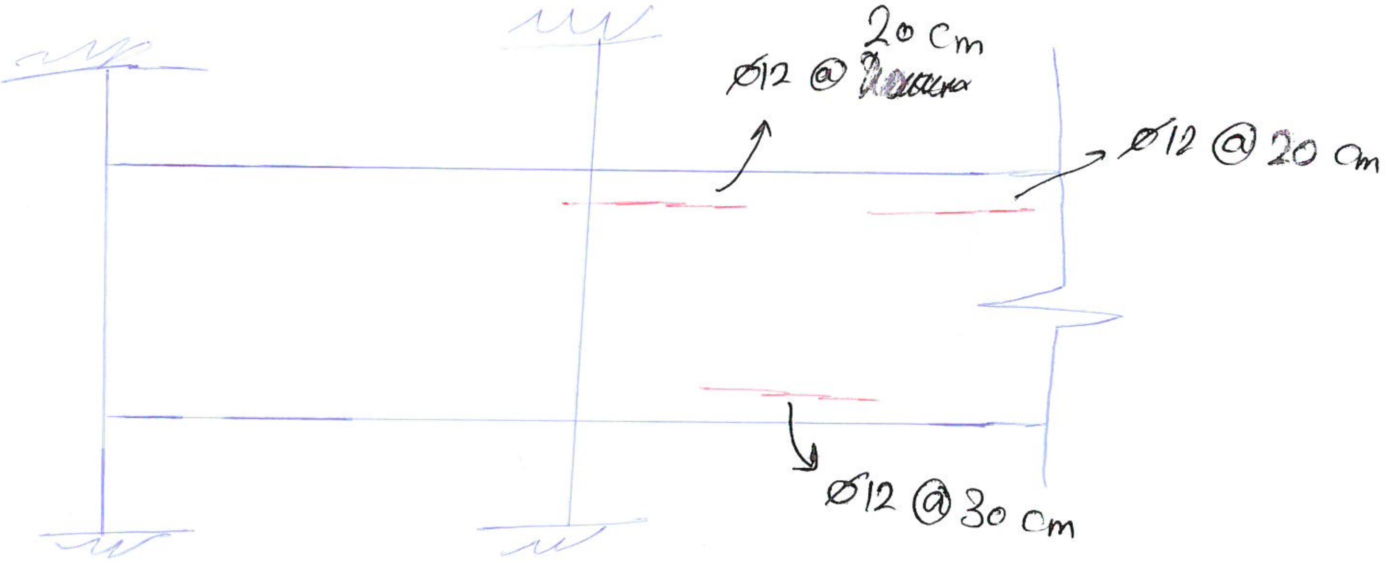
$(M_u)_B = -1.77$ t.m ,, $(M_u)_D = -1.12$ t.m

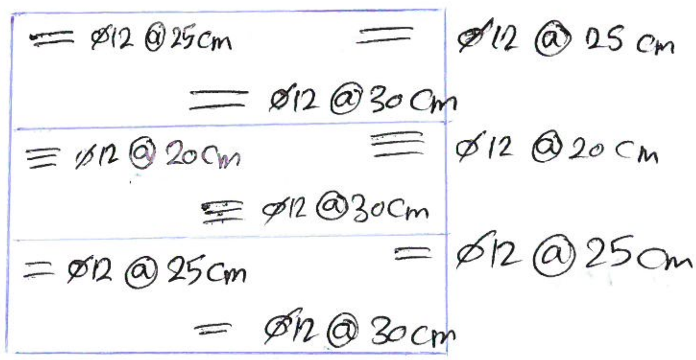
use Ø12 @ 20 cm

- Positive moment:

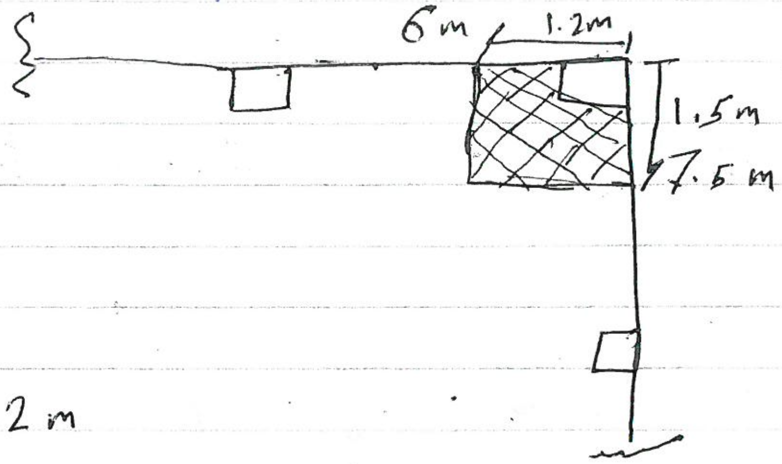
$(M_u)_B = 0.95$ t.m ,, $(M_u)_D = 0.6$ t.m

use Ø12 @ 30 cm


Ø12 @ 20 cm
Ø12 @ 20 cm
Ø12 @ 30 cm


Ø12 @ 25cm
Ø12 @ 25 cm
Ø12 @ 30 Cm
Ø12 @ 20Cm
Ø12 @ 20 Cm
Ø12 @30Cm
Ø12 @ 25Cm
Ø12 @ 25Cm
Ø12 @ 30cm

** Reinforcment of Slab Edge:-

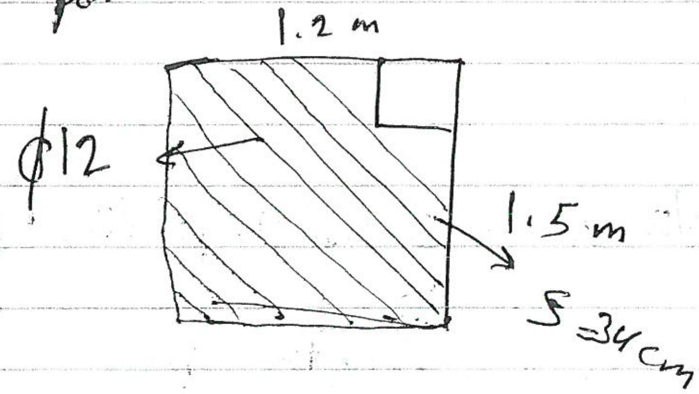


$\frac{6}{5} = 1.2\ m$

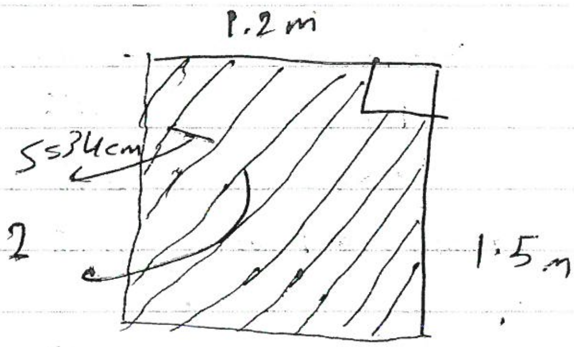
$\frac{7.5}{5} = 1.5\ m$ we use ϕ 12

max spacing = 2h = 2(17) = 34 cm

for top bar



for bottom bar



this distribution is the same
for all corners of the building

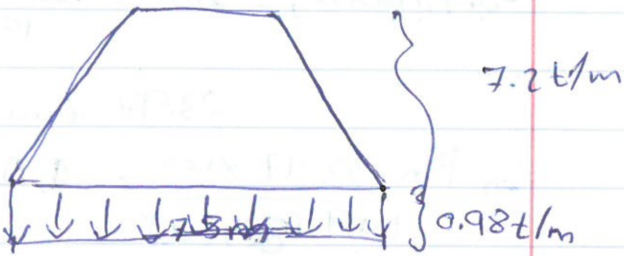
* Checking if the preliminary size of the:

[1] 35x70 cm for B_1 & B_2

→ Max B.M on Beams = -13.66 t.m

$$M_{Max\ Neg} = 13.66 + \frac{(0.98)(7.2)^2}{10}$$

$$= 18.74 \text{ t.m}$$



7.5 m
−0.3 m
= 7.2 m

→ d = h − cover = 70 − 4 = 66 cm

→ $R_n = \frac{18.74 \times 100}{0.90 \times (66)^2 (0.35)} = 0.0135$

→ m = 17.65

→ $\rho_{req} = \frac{1}{17.65}\left(1 - \sqrt{1 - \frac{2(0.0195)(17.65)}{4.2}}\right) = \cancel{0.0029}\ 3.35 \times 10^{-3}$

→ Total factored load on the Beam:-

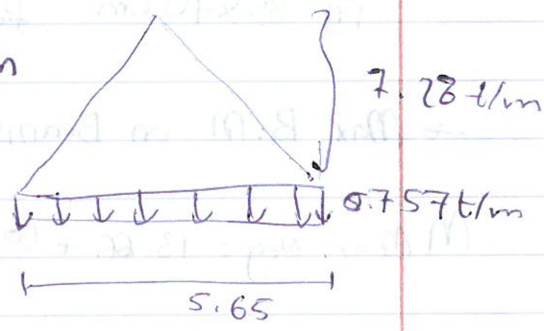
$$\frac{(7.2)(7.2)}{2} + \frac{0.98(7.5)}{2} = 29.742 \text{ t.} = 297.2 \text{ kN}$$

→ $V_u = \frac{297.2}{0.75(0.35)(0.66)} = 1715.44 \text{ kN/m}^2$

$V_{all} = \frac{0.75\sqrt{28}}{6} \times 1000 = 661.4 \text{ kN/m}^2$

this mean that the dimension of the beam should be increased.

[2] 30x73 cm for $B_5 . B_6$



→ Max B.M on Beams = -21.56 t.m

→ $M_{max\ neg} = 21.56 + \frac{0.757(5.65)^2}{10}$

$= 23.97$ t.m

→ $R_n = \frac{23.97 \times 100}{(0.9)(30)(66)} = 0.01540$

→ $\rho_{req} = \frac{1}{17.65}\left(1 - \sqrt{1 - \frac{2(17.65)(0.01528)}{4.2}}\right) = 3.78 \times 10^{-3}$

→ Total factored load on the Beam:-

$\frac{5.65(7.28)}{2} + \frac{(0.757)(6)}{2} = 22.837\ t = 228.37$ KN

→ $V_u = \frac{228.37}{(0.75)(666)(0.3)} = 1537.8$ KN/m²

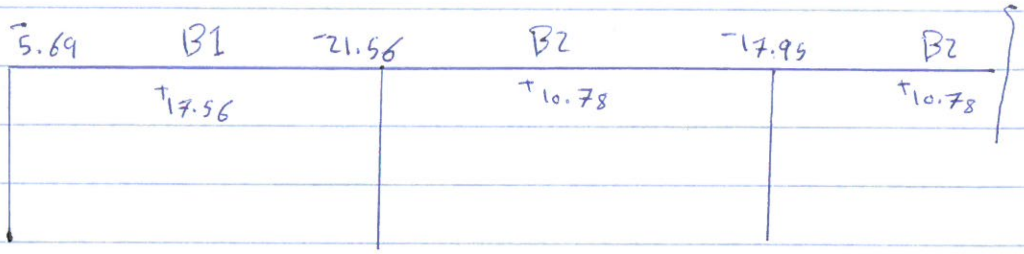
$V_{all} = \frac{\phi}{6}\sqrt{f_c'} = 661$ KN/m²

This mean that the dimension of the beam should be Increased.

- Design of beams:

· For frame A

moments at beam: (t.m)



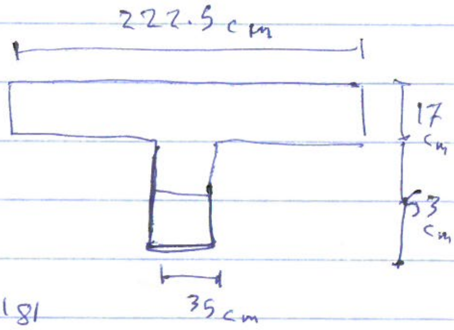
ACI (6.3.2.1) Table:

effective flange width (each side of web) $= \min \begin{cases} 8h = (8)(17) = 136\ cm, & \frac{S_w}{2} = \frac{600}{2} = 300\ cm, & \frac{L_n}{8} = \frac{750}{8} = 93.75\ cm \end{cases}$

$= 93.75\ cm.$

$b_e = 2(w_f) + b_w = 222.5\ cm$

- Design for negative moment (maximum):



$M_u = -21.56\ t.m = -211.5\ KN.m$

design as rectangle section.

assume $\phi = 0.9$, 1 layer $\phi 22$, $d = 63.9\ cm$

$\Rightarrow$ design aids: Table(A.4): $\rho_{min} = 0.0033$, $\rho_{0.005} = 0.0181$

$R = \frac{M_u}{\phi B d^2} = \frac{(211.5)10^6}{(0.9)(350)(639)^2} = 1.644\ MPa$

Table (A.5a) $\Rightarrow \rho = 0.004064 \Rightarrow A_s = \rho b d = 9.09\ cm^2 \Rightarrow 3\phi 22 \rightarrow A_s = 11.4\ cm^2$

- Check spacing: $S_{min} = \max(4/3\, d_{agg} = 2.533\ cm,\ 2.5\ cm,\ d_b = 2.2\ cm) = 2.533\ cm$

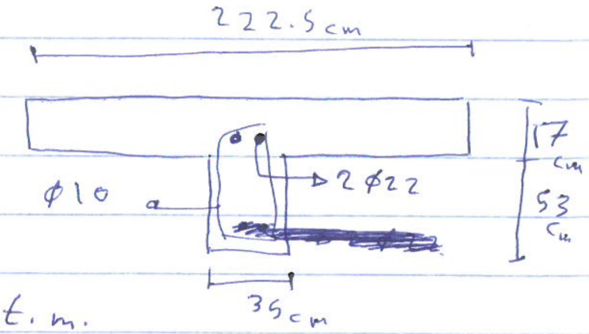
$S = \frac{35 - 5.5 - 3(2.2)}{2} = 9.2 > S_{min}$ ✓

- Check ϕ: $\rho = \frac{11.4}{(63.9)(35)} = 0.005097 < \rho_{0.005} = 0.0181$ $\therefore \phi = 0.9$ ✓

- Check moment Capacity:

$T = C \Rightarrow A_s f_y = 0.85\ f_c'\ ab$

$a = \frac{(11.4)(4.2)}{(0.85)(0.28)(35)} = 5.75\ cm$



$\phi M_n = \phi A_s f_y (d - a/2)$

$\phi M_n = 2629.73\ t.cm$

$\phi M_n = 26.3\ t.m > M_u = 21.56\ t.m.$

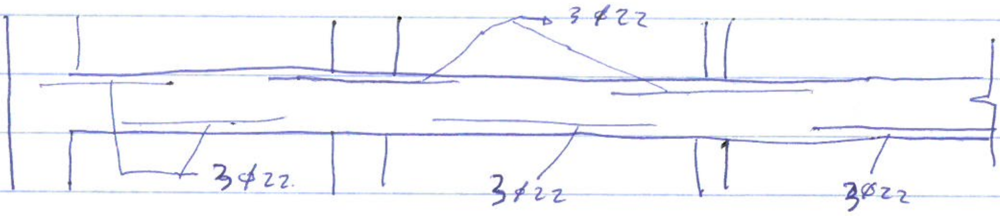
- Check Crack Control:

$\Rightarrow$ design aids (A.8) : Cover = 40 mm $\Rightarrow$ min. # bars $\phi 22$ = 2 < 3 bars we used. ✓

* minimum # bars for Crack Control requirments = 2.
determine moment Capacity for 2 bars $\phi 22$

$T = C \Rightarrow a = 3.83\ cm \Rightarrow \phi M_n = 1780\ t.cm = 17.8\ t.m.$

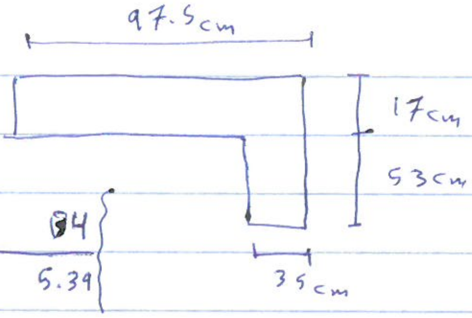
We Can use $3\phi 22$ for other moment Values.

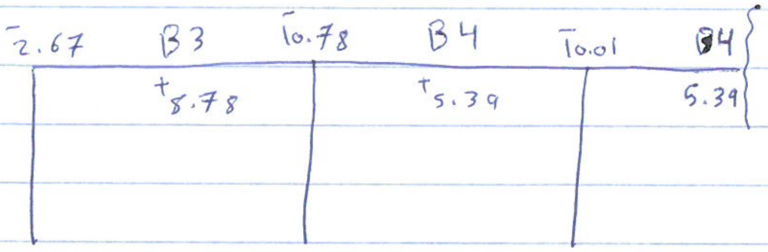


- Check ρ for $2\phi 22$: $\rho = \frac{2(3.8)}{(35)(70)} = 0.0031 < \rho_{min} = 0.0033$

So we need at minimum $3\phi 22$ for all beam at differ. moments.

- for Frame B:



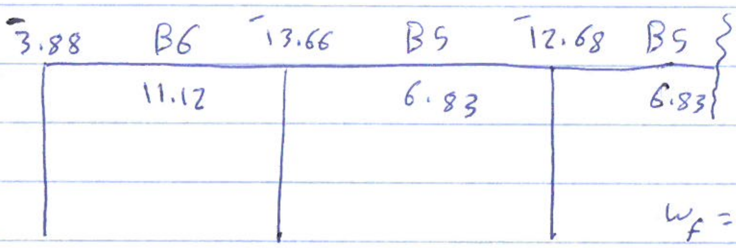


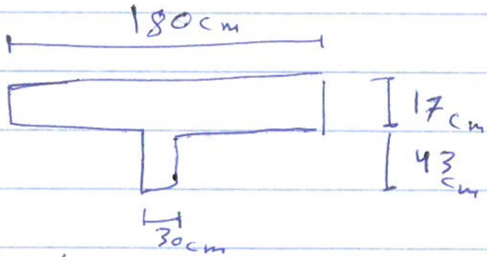
W_{flange} (one side of web) $= \left\{ 6h = 102 \;,\; \frac{S_w}{2} = 300 \;,\; \frac{L_n}{12} = 62.5 \right\} = 62.5\,cm$

$b_e = 62.5 + 35 = 97.5\ cm.$

use $3\ \phi\ 22$ (all checks ✓) for all spans.

for frame C:





$w_f = \left\{ 136 \;,\; \frac{750}{2} \;,\; \frac{600}{8} \right\} = 75\,cm$

- Check spacing: $d = 53.9\ cm$

$S = 30 - 10 - 2(2.2) = 15.6\,cm > S_{min}$

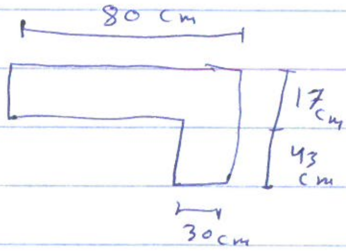
- check $\phi = 0.9$: $\rho = \frac{2(3.8)}{(53.9)(30)} = 0.0047 > \rho_{min}$, $< \rho_{0.005}$ $\therefore \phi = 0.9$

- check crack control: min # bars = 2. ✓

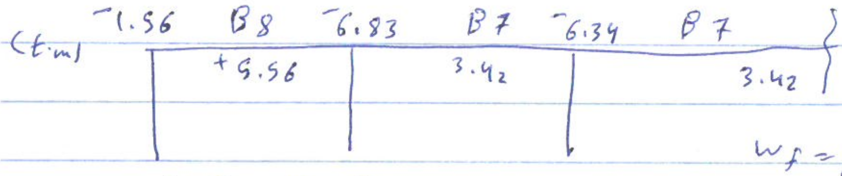
- check moment capacity: $T = C \Rightarrow a = 4.47\,cm$

$\phi M_n = 14.8\ t.m. >$ all moment at frame C.

$2\ \phi 22$ ✓



- for frame D:



$w_f = \left\{ 102 \;,\; \frac{750}{2} \;,\; \frac{600}{12} \right\} = 50\,cm$

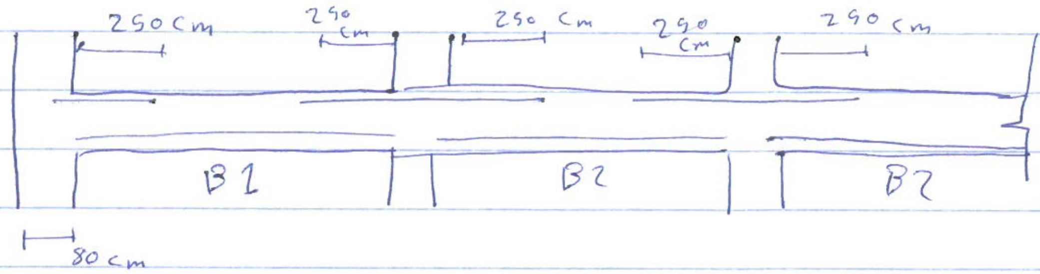
use $2\ \phi\ 22$ all checks as frame D

for all spans in this frame.

Beam Reinforcement:

Frame A:

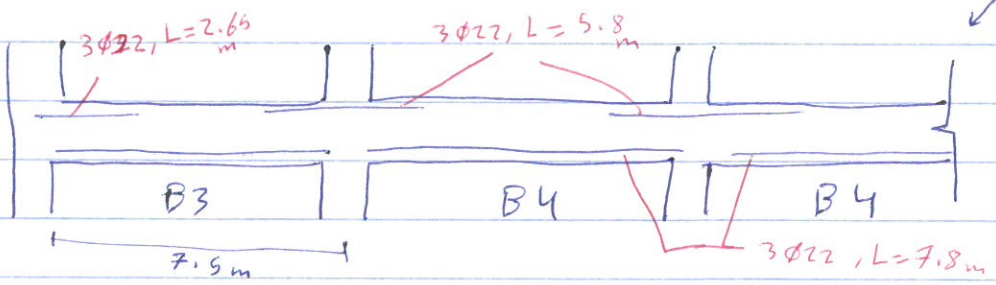
3 Ø22
in the top
and bottom



Same
reinforcement

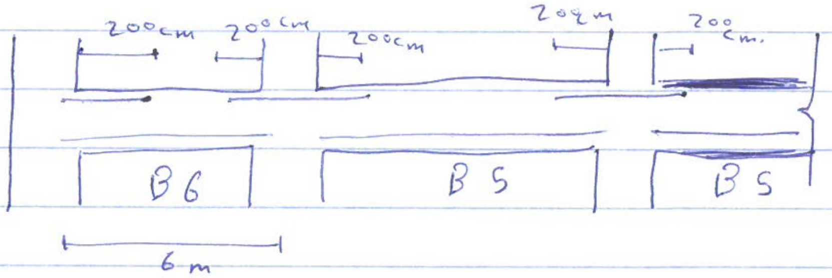
Frame B:

3 Ø22
in the top
and bottom
reinforcement.



frame C:

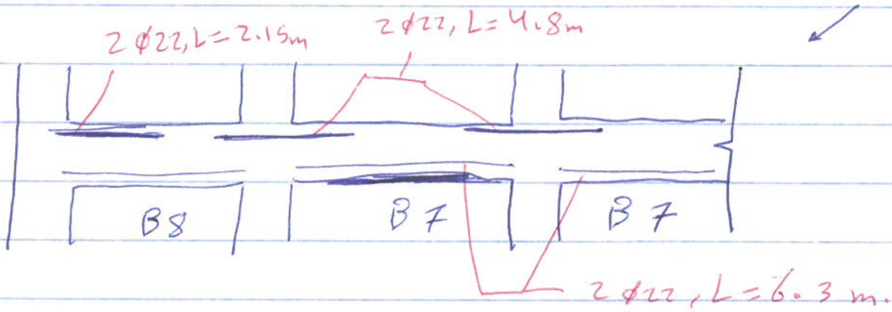
2 Ø22
in the top
and bottom



Same
reinforement
details

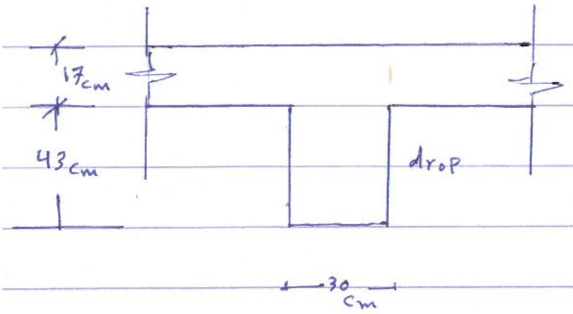
frame D:

2 Ø 22
in top
and bottom



- Determine Loads in each beam:

- in short direction:



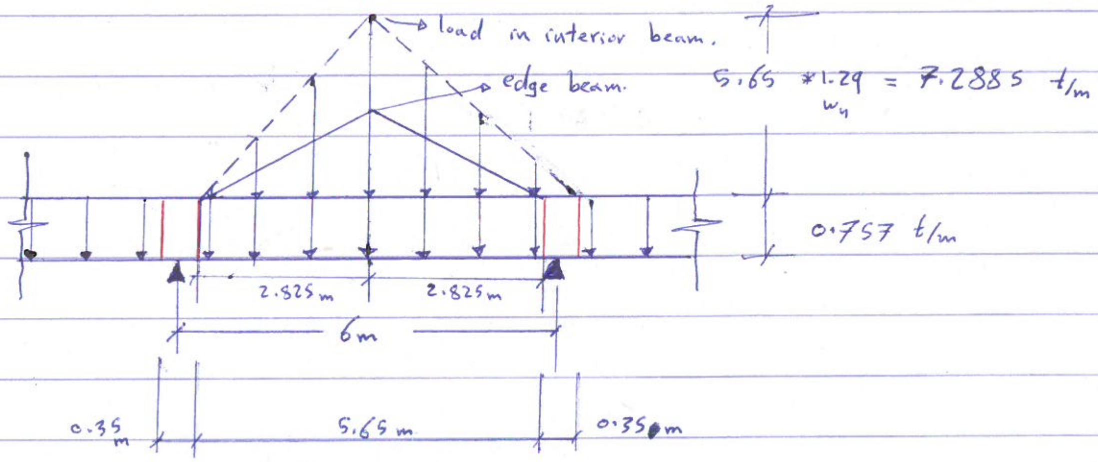
$w_{drop} = (0.43)(0.3)(2.4) = 0.31\ t/m$

$w_{drop,\ factored} = (1.2)(0.31) = 0.37\ t/m$

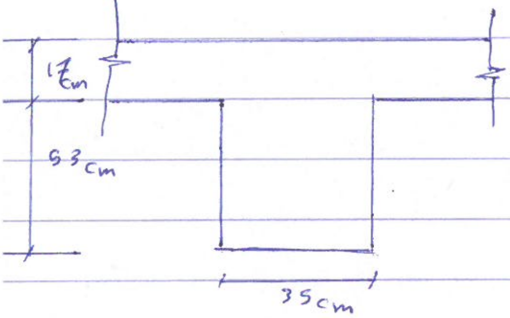
for the thick slab:

$w_u = (0.3)(1.29) = 0.387\ t/m$

$w,\ Total = 0.37 + 0.387 = 0.757\ t/m.$



in long direction:

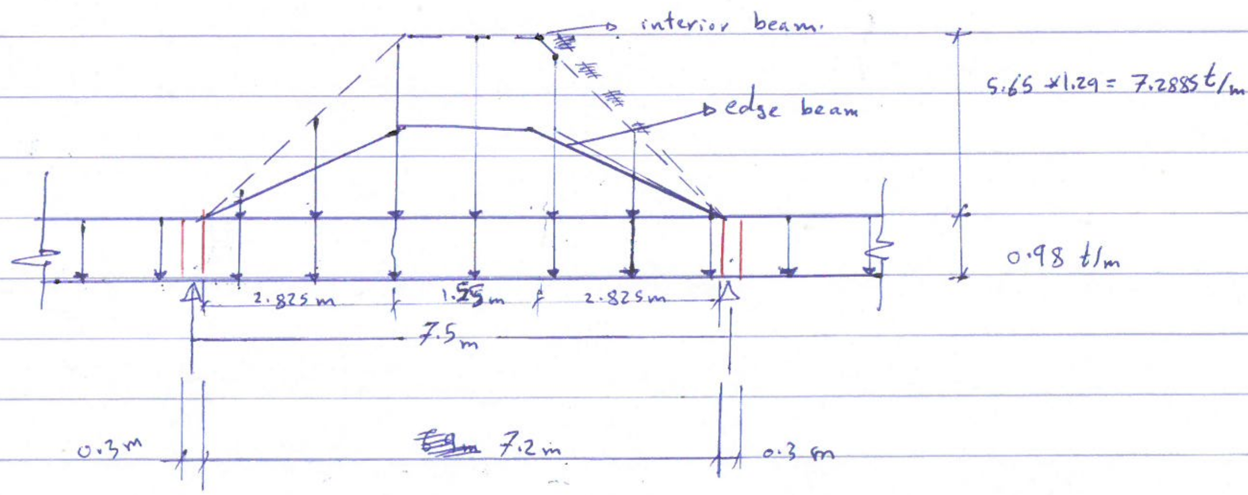


$w_{drop} = (0.53)(0.35)(2.4) = 0.4452\ t/m$

$w_{drop,\ factored} = 0.53\ t/m.$

thick slab $= (0.35)(1.29) = 0.45\ t/m.$

Total $= 0.98\ t/m.$



- Design of Columns :

$W_{LL,factored} = (1.6)(0.5) = 0.8 \ t/m^2$, Column dim. $70_{cm} \times 70_{cm}$

$W_{DL,factored} = (1.2)(0.408) = 0.49 \ t/m^2$

Story height $= 6$ m , $\rho \approx 2\%$

- for Top Floor : [interior Column]

Self weigh of Coulmn $= (0.7)(0.7)(2.4)\left(6 - \frac{0.6+0.7}{2}\right) = 6.3 \ t$

$W_{beam} = (0.98)(7.5) + (0.757)(6) = 11.9 \ t$

Load from short beam (interior beam) $= (7.2885)(5.65)(0.5) = 20.6 \ t$

Load from Long beam (interior beam) $= (2.825)(7.2885) + (7.2885)(1.55)$

$= 31.89 \ t$

Total $(P_u) = 70.7 \ t$

$P_n = \frac{70.7}{0.65} = 108.7 \ t$

-ACI 13.6.9 : $M_u = 0.07\left[(q_{D_u} + 0.5 q_{L_u}) L_2 L_n^2 - q'_{D_u} L_2' (L_n')^2\right]$

Prime → refer to short span. , q_{D_u} → DL, factored , q_{L_u} → LL factored

$L_2 = 7.5$ m → $L_n = 7.5 - 0.7 = 6.8$ m

$L_2' = 6$ m → $L_n' = 5.3$ m

⇒ $M_u = 15.8$ t.m → assume all Columns same E_c

↳ in each direction. ∴ M_u distributed equally for above and bottom Columns in bottom floor.

And the Columns in the top floor resist all M_u.

$M_{u,req.} = M_{u_1} + 0.55 M_{u_2}\left(\frac{h}{b}\right)$, $h = b = 0.7$ m.

$= 15.8 + (0.55)(15.8)$

$= 24.5$ t.m

$e = \frac{M_{ueq}}{P_u} = \frac{(24.5)100}{70.7} = 34.70$ cm

[Top floor : interior Column]

- Preliminary Size :

$$A_g = \frac{P_n}{(0.85)(0.28)(0.6)} = \frac{108.7}{(0.85)(0.28)(0.6)} = 27.6 \times 27.6$$

Try 30 cm X 30 cm

use $\phi 2.2$

$d = 30-4-1-1.1 = 23.9$ cm

$d' = 6.1$ cm

$\gamma = \frac{d-d'}{h} = \frac{23.9-6.1}{30} = 0.593$

at least $\gamma = 0.6$

Try 35 X 35 $\Rightarrow \gamma = \frac{(35-6.1)-(6.1)}{35} = 0.65$

$K_n = \frac{P_{n,req}}{f_c' A_g} = \frac{108.7}{(0.28)(35)^2} = 0.32$

$M_n = K_n(e/h) = 0.314$

from Chart (3.2.1) : $\gamma = 0.6 \Rightarrow \rho = \gg 0.08$

Try 40 X 40 : $\gamma = \frac{40-6.1-6.1}{40} = 0.695$

$K_n = 0.24$ → Chart (3.2.1) : ($\gamma = 0.6$) : $\rho = 0.042$

$R_n = 0.21$ Chart (3.2.2) : $\gamma = 0.7$: $\rho = 0.033$

Try 45 x 45 : $\gamma = 0.73$

$K_n = 0.19$, $M_n = 0.148$ → Chart (3.2.2) ($\gamma = 0.7$) : $\rho = 0.018$

Chart (3.2.3) $\gamma = 0.8$: $\rho = 0.016$

→ $\rho = 0.0174 \Rightarrow A_s = 35.24$ cm^2 → # bars $\phi 22 = 9.3$

# of bars $\phi 22 = 10$ bar → $\rho = 0.019 \approx 2\%$

[Top floor : interior Column].

- Check for Concentric load Capacity:

$$P_o = 0.85\, f_c'\, A_g + (f_y - 0.85 f_c')\, A_s$$

$$= (0.85)(0.28)(45)^2 + (4.2 - (0.28)(0.85))(10)\left(\frac{(2.2)^2}{4}(3.14)\right)$$

$$= 632.5 \ t$$

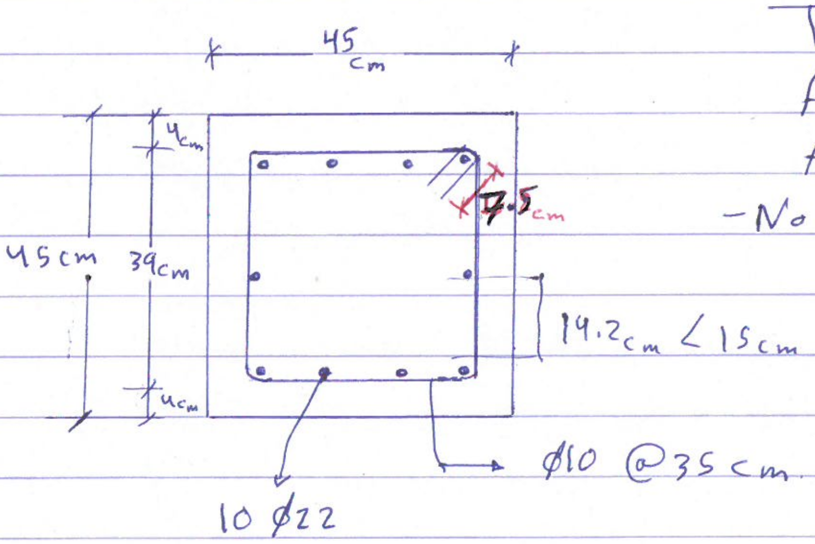
$$\phi r P_o = (0.65)\underset{\text{Tied Column.}}{(0.8)}(632.5) = 328.89 \ t > P_u = 70.7 t$$

- longitudinal bar spacing: $S_{min} = \max\left\{4_{cm},\ 1.5 d_b = 3.3\right\} = 4_{cm}$

$$S = \frac{45 - 2(4+1) - 4(2.2)}{3} = 7.07 \ cm > s_{min}.$$

- Spacing between ties $= s_{max} = \min\left\{16 d_b = 35.2,\ 48 d_t = 48,\ \underset{=45}{\text{least dim.}}\right\}$

$$S = 35.2_{cm} \approx 35 \ cm.$$



Tied Column

$f_c' = 0.28 \ t/cm^2$

$f_y = 4.2 \ t/cm^2$

- No additional ties/links needed.

[Top Floor: interior Column].

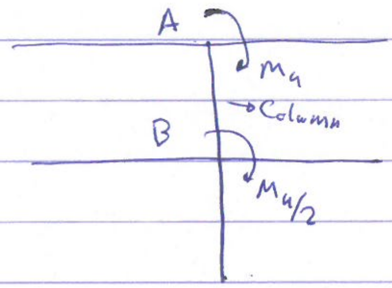
- Slenderness effects check:

$L_c = 6m$, $L_u = 6 - \frac{0.6+0.7}{2} = 5.35\ m$

$L_g = 7.5\ m$, double Curvature

→ long beam $M_u = 15.8\ t.m$

Top floor



$$\Psi_A = \frac{\Sigma\,[E_c\, I_c / L_c]}{\Sigma\,[E_g\, I_g / L_g]} = \frac{239\,203/6}{(2)\left[\frac{350146}{7.5}\right]} = 0.427$$

$$I_c = (0.7)\frac{1}{12}BH^3 = (0.7)\frac{1}{12}(45)^4 = 239\,203\ cm^4$$

$$I_g = (0.35)\frac{1}{12}BH^3 = (0.35)\frac{1}{12}(35)(70)^3 = 350146\ cm^4$$

$E_c = E_g$

$\Psi_B = 2\Psi_A = 0.854$ ⇒ alignment Chart for braced frame : $K = 0.71$

$$\frac{KL_u}{r} = \frac{KL_u}{(0.3)h} = 28.14 < \left(\frac{KL_u}{r}\right)_{limit} = 40$$

$$\left(\frac{KL_u}{r}\right)_{limit} = \min\left\{\begin{array}{l} 34 + 12\left(\frac{M_{1ns}}{M_{2ns}}\right) = 34 + 12\,(0.5) = 40 \\ 40 \end{array}\right\} = 40$$

double Carv.

∴ Slenderness affects ignored.

- Top floor [edge Column].

S.w of Column = 6.3 t

$W_{beam} = \frac{1}{2}(0.98)(7.5) + \frac{1}{2}(0.757)(6) = 5.95\ t$

Load from short beam (extirior beam) $= \frac{1}{2}\left(\frac{7.2885}{2}\right)(2.825) = 5.15\ t$

Load " long " " edge " $= \frac{1}{2}\left(\frac{7.2885}{2}\right)(2.825) + \frac{1}{2}\left(\frac{7.2885}{2}\right)(1.55)$

$= 7.97\ t$

$P_u =$ Total $= 25.37\ t \Rightarrow P_n = 39\ t$

For edge Column we design for $(0.3\ M_o)$

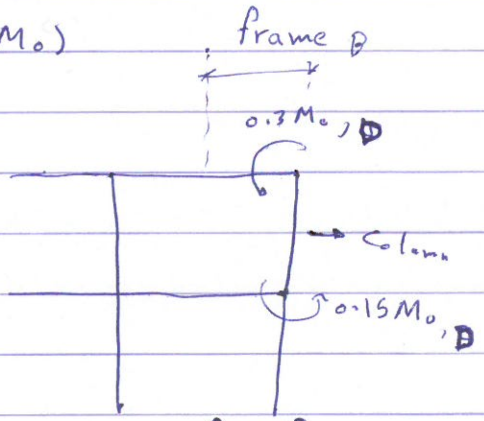
$0.3\ M_{o,B} = (0.3)(22.37) = 6.7\ t.m$

$0.3\ M_{o,D} = (0.3)(17) = 5.1\ t.m$

$M_{u\,req.} = 6.7 + (0.55)(1)(5.1)$

$= 9.5\ t.m$

$e = \frac{M_{u,req}}{P_n} = 37.5\ cm$



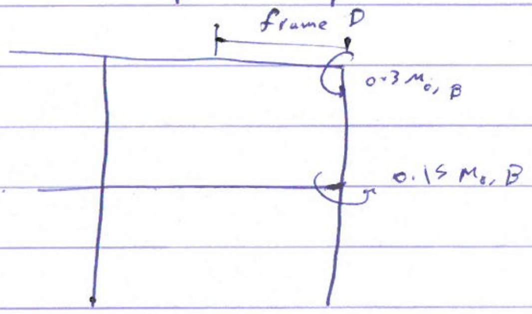
- Preliminary Size:

$A_g = \frac{P_n}{(0.85)(0.28)(0.4)} = 409 = 20.24 \times 20.24$

$\approx 25cm \times 25cm$

use $\phi 22 \Rightarrow \gamma = \frac{25 - 2(6.1)}{25} = 0.512$

at least $\gamma = 0.6$



$\Rightarrow$ Try $30_{cm} \times 30_{cm}$

$\gamma = 0.593$

$\Rightarrow$ Try $35 \times 35 \Rightarrow 0.65 = \gamma$.

$K_n = \frac{39}{f_c' A_g} = 0.11$

$R_n = 0.122$

from Chart (3.2.1): $\rho = 0.018$, $\gamma = 0.6$

from Chart (3.2.2): $\rho = 0.015$, $\gamma = 0.7$

$\Rightarrow \rho = 0.0165 = 1.65\% \approx 2\%$

$\Rightarrow$ Try $40 \times 40 \Rightarrow \gamma = 0.695$

$K_n = 0.087$

$R_n = 0.0816$

Chart (3.2.1) : $\rho = << 0.02$

Take $35_{cm} \times 35_{cm} \rightarrow A_s = 20.2\ cm^2 \Rightarrow$ # bars $\phi 22 = 5.3 \approx 6$ bars.

$\Rightarrow \rho = 0.0186$

[Top floor : edge Column]

- check for Concentric load Capacity:

$6\phi 22 \quad (35 \times 35)$

$P_o = 0.85 f_c' A_g + (f_y - 0.85 f_c') A_s$

$= 381.87\ t$

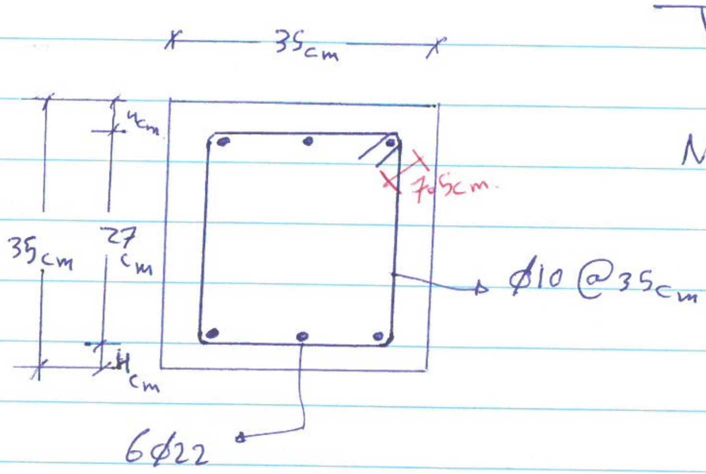
$\phi r P_n = 198.6\ t > P_u = 25.37 t$

- longitudinal bars Spacing: $S_{min} = 4 cm.$

$$S = \frac{35-10-3(2.2)}{2} = 9.2\ cm > S_{min}$$

- Spacing between ties $= S_{max} = \min\{16 d_b, 48 d_t, 35\}$

$S = 35\ cm$



Tied Column

($35_{cm} \times 35_{cm}$)

No additional ties/links.

[Top Floor: edge Column]

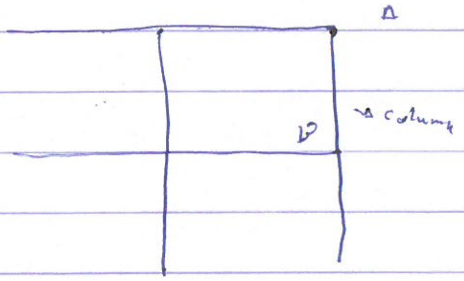
- Slenderness effects check:

$L_c = 6m$, $L_u = 5.35$ m

$L_g = 7.5m$, double Curvature

$I_c = 149\ 333\ cm^4$

$I_g = 350\ 146\ cm^4$



$$\Psi_A = \frac{149\ 333/6}{350146/7.5} = 0.53$$

$$\Psi_B = \frac{(2)\ (149333/6)}{350146/7.5} = 1.07$$

$\Rightarrow K = 0.73$

$$\frac{KL_u}{r} = \frac{KL_u}{(0.3)(0.35)} = 37.2 \quad < \left(\frac{KL_u}{r}\right)_{limit}$$

$$\left(\frac{KL_u}{r}\right)_{limit} = \min \begin{Bmatrix} 34 + 12\,(0.5) \\ 40 \end{Bmatrix} = 40cm$$

$\therefore$ Slenderness affects ignored.

– Bottom Floor : [interior Column]

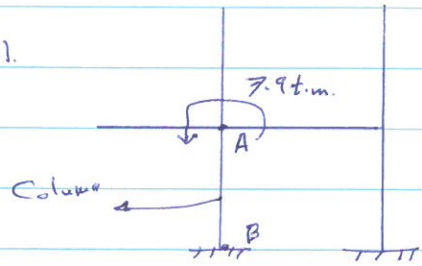
$$P_u = P_{u,\ \substack{Top\ f. \\ int.\ Column}} \times (7_{Stories}) = (70.7)(7) = 495\ t$$

$M_u = 15.8\ t.m \Rightarrow$ distributed equally

M_u at the Column $= 7.9$ t.m. (in each direction).

$$M_{u,req.} = M_{u1} + 0.55\ M_{u2} \left(\cancel{\tfrac{h}{b}}\right) \quad ①$$

$$= 7.9 + (0.55)(7.9) = 12.245\ t.m.$$



→ design as Concentric :

$$P_n = \frac{P_u}{0.65} = 761.5\ t \quad , \quad P_{o_{req.}} = \frac{P_u}{(0.65)(0.8)} = 952\ t.$$

$$P_o = 0.85 f_c'\ A_g + (f_y - 0.85 f_c')\ A_s \quad , A_s = \rho A_g$$

$$952 = (0.85)(0.28)\ A_g + (4.2 - 0.85(0.28))\ (0.02)\ A_g$$

$A_g = 3000\ cm^2 \rightarrow (54.78)^2 \approx 55_{cm} \times 55_{cm} \Rightarrow A_g = 3025\ cm^2$

$\Rightarrow A_s = 60.5\ cm^2 \Rightarrow$ use $\phi 22$

$\Rightarrow 16\ \phi\ 22 \Rightarrow A_s = 60.8\ cm^2 \Rightarrow \rho = 2.01\ \% \approx 2\%$

$\Rightarrow P_o = 960.8\ t$

– Longitudinal bars spacing : $S_{min} = 4\ cm.$

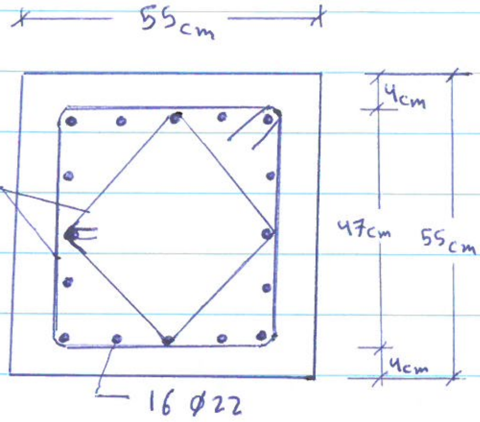
$$S = \frac{55 - 2(5) - 5(2.2)}{4} = 8.5\ cm \; \begin{matrix} > S_{min} \\ < 15\ cm. \end{matrix}$$

– Spacing between ties :

$$S_{max} = \min \{35.2,\ 48,\ 55\} \approx 35\ cm.$$

[Bottom Floor: interior Column].

-Check by Bresler formula:



Tied Column (55 X 55 cm)

$$e_x = e_y = \frac{M_u}{P_u} = \frac{(7.9)(100)}{495} = 1.6 \text{ cm}$$

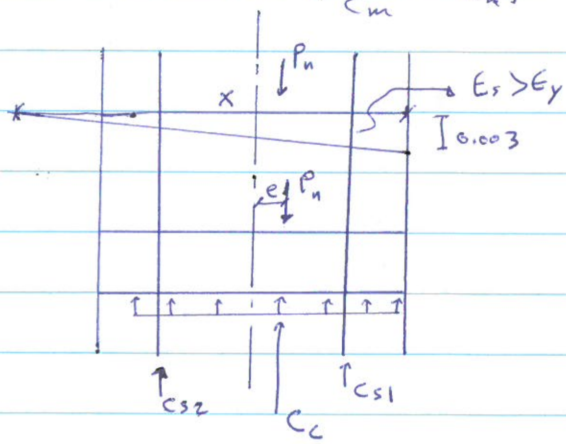
→ find $P_{n,x}$ at $e = 1.6$ cm

Compresion Controls, $\beta = 0.85$

$d = 48.9$ cm, $E = 200$ GPa, $A_{bar} = 3.8 \text{ cm}^2$

assume all steel in Compresion

steel in right Yield.



$$\epsilon_{s2} = \frac{0.003}{x}(x - 48.9)$$

$$f_{s2} = 6 - \frac{293.4}{x}$$

$$C_{s1} = (5)(3.8)(f_y - 0.85 f_c') = 75.3 \text{ t}$$

$$C_{s2} = (5)(3.8)\left(6 - \frac{293.4}{x} - 0.85(0.28)\right) = 109.5 - \frac{5575}{x}$$

$$C_c = 0.85 f_c' ab = 11.13x$$

$$\sum F_y = 0 \Rightarrow P_n = C_{s1} + C_{s2} + C_c = 11.13x + 184.8 - \frac{5575}{x}$$

$$\sum M_{P.c} = 0 \Rightarrow (1.6)P_n = (21.4)(C_{s1} - C_{s2}) + C_c\left[27.5 - \frac{0.85}{2}x\right]$$

$$(17.8)x + 295.7 - \frac{8920}{x} = -732 + \frac{119305}{x} + 306x - 4.73x^2$$

$$(4.73)x^2 - 288.2x + 1027.7 - \frac{128225}{x} = 0$$

$x = 64.13$ cm → $a = 54.5$ cm

The assumption is Correct. $x > d$.

$P_{n,x} = 811.7$ t

$$\frac{1}{P_n} = \frac{1}{P_{n,x}} + \frac{1}{P_{n,y}} - \frac{1}{P_o}$$

$P_{n,x} = P_{n,y}$

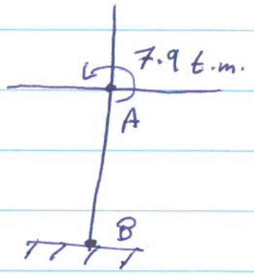
⇒ $P_n = 702.66$ t

$\phi P_n = 456.7$ t $< P_u = 495$ t

we should increase the dimension.

- Slenderness effects check: $L_c = 6$ m, $L_u = 5.35$ m, $L_g = 7.5$ m

double Curvature, $M_u = 7.9$ t.m



$$\psi_A = \frac{2(0.7)\left(\frac{1}{12}(55)^4\right)/6}{2(0.35)\left(\frac{1}{12}(35)(70)^3\right)/7.5}$$

$\psi_A = 1.9$, $\psi_B = 0$ (fixity restrain all moment).

$K = 0.65$ → $\frac{KL_u}{r} = 21 < \left(\frac{KL_u}{r}\right)_{limit} = 40$

∴ No slenderness effects.

[Bottom Floor interior Column].

Try 60×60 (cm × cm) for interior Column in bottom floor:

$P_u = 495\ t$, $M_u = 7.9\ t \cdot m$

$P_n = 761.5\ t$, $P_o = 952\ t.$

$\rho = 0.02 \Rightarrow A_s = \rho A_g = 72\ cm^2$

$18.95\ \phi 22 \approx 20\ \phi 22$

$A_s = 76\ cm^2$

$P_o = 1157.8\ t$

- Longitudinal bar spacing: $S_{min} = 4\ cm$, $S = \frac{60-10-6(2.2)}{5} = 7.36\ cm$ ✓.
- Spacing between ties: $S = 35\ cm.$

$e_x = e_y = \frac{M_u}{P_n} = 1.6\ cm$

$P_{n,x} = ?$

Compression Controls/d = 53.9 cm.

assume all steel in compression

$\epsilon_{s2} = \frac{0.003}{x}(x - 53.9) \Rightarrow f_{s2} = 6 - \frac{323.4}{x}$

$C_{s1} = (6)(3.8)(f_y - 0.85 f_c') = 90.3\ t$

$C_{s2} = (6)(3.8)(6 - \frac{323.4}{x} - 0.85 f_c') = 131.4 - \frac{7373}{x}$

$C_c = 12.14\ x$

$\Sigma F_y = 0 \Rightarrow P_{n/x} = C_{s1} + C_{s2} + C_c = 12.14x + 221.7 - \frac{7373}{x}$

$\Sigma M_{pc} = 0 \Rightarrow (1.6) P_n = (C_{s1} - C_{s2})(23.9) + C_c\left[30 - \frac{0.85}{2}x\right]$

$19.4x + 354.7 - \frac{11797}{x} = -982.3 + \frac{176215}{x} + 364.2x - 5.16x^2$

$5.16x^2 - 344.8x + 1337 - \frac{188012}{x} = 0 \Rightarrow x = 70.48\ cm$

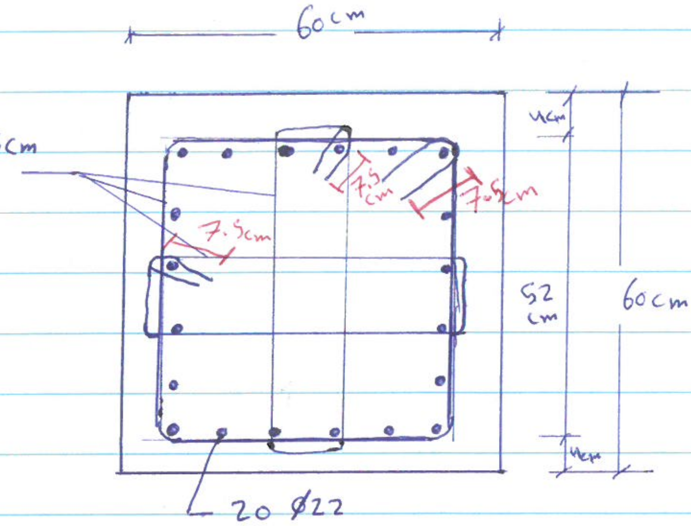
$\Rightarrow P_{n,x} = P_{n,y} = 972.7\ t$ The assumption is Correct.

$\frac{1}{P_n} = \frac{1}{P_{n,x}} + \frac{1}{P_{n,y}} - \frac{1}{P_o} \Rightarrow 838.65 = P_n$

$\phi P_n = 545\ t > P_u = 495\ t$ ✓

Interior Column
[bottom floor].

Trial Column (60 X 60)
cm cm



- slendrness effects check:

$$\Psi_A = \frac{(2)(0.7)\left(\frac{1}{12}(60)^4\right)/6}{(2)(0.35)\left(\frac{1}{12}(35)(70)^3\right)/7.5} = 2.7$$

$$\Psi_B = 0$$

$$\Rightarrow K = 0.67 \rightarrow \frac{KL_u}{r} = 20 < \left(\frac{KL_u}{r}\right)_{limit} = 40$$

$\therefore$ No slenderness effects

- Bottom floor: [edge Column]

Corner Column.

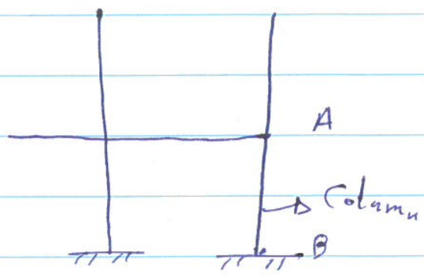
$P_u = P_{u,top,\,\text{edge Column}}\,(7) = (25.37)(7) = 177.6\ t$

$0.15\,M_{o,B} = 3.35\ t.m$

$0.15\,M_{o,D} = 2.55\ t.m$

$M_{u,req.} = 3.35 + (0.55)(2.55)(1) = 4.75\ t.m$

$e = \dfrac{4.75}{177.6} = 2.68\ cm$



- design as Concentric load:

$P_n = 273.2\ t \quad , P_o = 341.5\ t$

$P_o = 0.85\,f'_c A_g + (f_y - 0.85(f'_c))\,\rho A_g \quad , \rho = 0.02$

$A_g = 32.8 \times 32.8 \approx 35_{cm} \times 35_{cm}$

$A_g = 1225\ cm^2 \Rightarrow A_s = 24.5\ cm^2 \Rightarrow 6.5$ bar $\Rightarrow$ 8 bar $\phi 22$

$A_s = 30.4\ cm^2 \Rightarrow P_o = 412\ t$

- Longitudinal bars Spacing: $S_{min} = 4\ cm$

$S = \dfrac{35 - 2(4+1) - (3)(2.2)}{2} = 9.2\ cm > S_{min}$

- Spacing between ties: $S_{max} = \min\{16 d_b = 35.2\ ,\ 48 d_t\ ,\ 35\} = 35\ cm.$

[Bottom Floor: edge Column]

- Check by Bresler formula:

$e_x = \frac{(3.35)100}{177.6} = 1.88\ cm \Rightarrow P_{n,y} = ?$

$e_y = \frac{(2.55)100}{177.6} = 1.4\ cm. \Rightarrow P_{n,x} = ?$

- Compression Controls.

$\beta = 0.85,\ d = 35 - 4 - 1 - \frac{2.2}{2} = 28.9\ cm.$

assume all steel will ~~yield~~ Compresion

$\epsilon_{s2} = \frac{0.003}{X}(X - 28.9) \Rightarrow F_{s2} = 6 - \frac{173.4}{X}$

$C_{s1} = (3)(3.8)(4.2 - 0.85(0.28)) = 45.17\ t$

$C_{s2} = (3)(3.8)\left(6 - \frac{173.4}{X} - 0.85(0.28)\right) = 65.7 - \frac{1976.8}{X}$

$C_c = (0.85)\ f_c'\ (0.85X)(35) = 7.08X$

$\Sigma F_y = 0 \Rightarrow P_{n,x} = C_{s1} + C_{s2} + C_c$

$P_{n,x} = 110.9 - \frac{1976.8}{X} + 7.08X$

$\Sigma M_{pc} = 0 \Rightarrow P_{n,x}(1.4) = (C_{s1} - C_{s2})(11.4) + C_c\left[17.5 - \frac{0.85}{2}X\right]$

$9.912X + 155.3 - \frac{2767.5}{X} = \bar{2}34 + \frac{22535.5}{X} + 123.9X - 3.01X^2$

$3.01X^2 - 113.99X + 389.3 - \frac{25303}{X} = 0 \Rightarrow X = 39.91\ cm > d$

The assumption is correct

$\Rightarrow P_{n,x} = 344\ t$

$\Sigma M_{P.c} = 0 \Rightarrow P_{n,y}(1.88) = (C_{s1} - C_{s2})(11.4) + \left[17.5 - \frac{0.85}{2}X\right]C_c$

$13.3X + 208.5 - \frac{3716}{X} = \bar{2}34 + \frac{22535.5}{X} + 123.9X - 3.01X^2$

$3.01X^2 - 110.6X + 442.5 - \frac{26251.5}{X} = 0 \Rightarrow X = 38.76\ cm > d.$

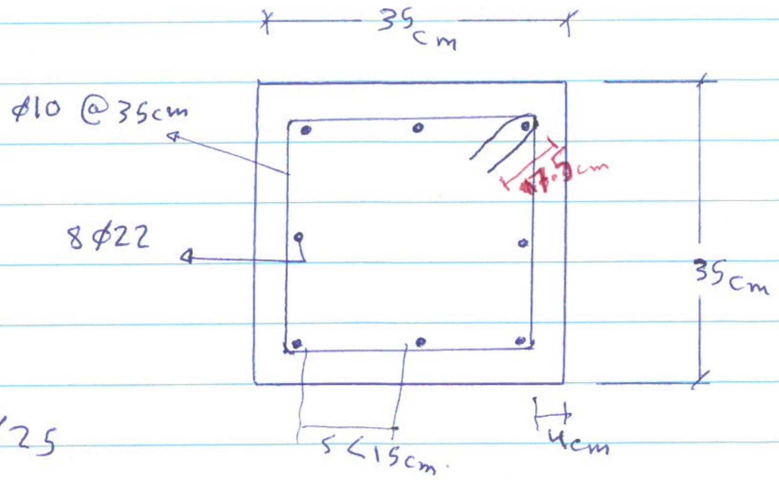
$\Rightarrow P_{n,y} = 334.3\ t$

$\frac{1}{P_n} = \frac{1}{P_{n,y}} + \frac{1}{P_{n,y}} - \frac{1}{P_o} = \frac{1}{334.3} + \frac{1}{344} - \frac{1}{412}$

$P_n = 288.1\ t \Rightarrow \phi P_n = 187.3\ t > P_u = 177.6\ t.$

[Bottom Floor: edge Column]

Tied Column ($35_{cm} \times 35_{cm}$)



- Slenderness effects Check:

$$\Psi_A = \frac{(2)(0.7)\left(\frac{1}{12}\right)(35)^4/6}{(0.35)\left(\frac{1}{12}\right)(35)(70)^3/7.5} = 0.625$$

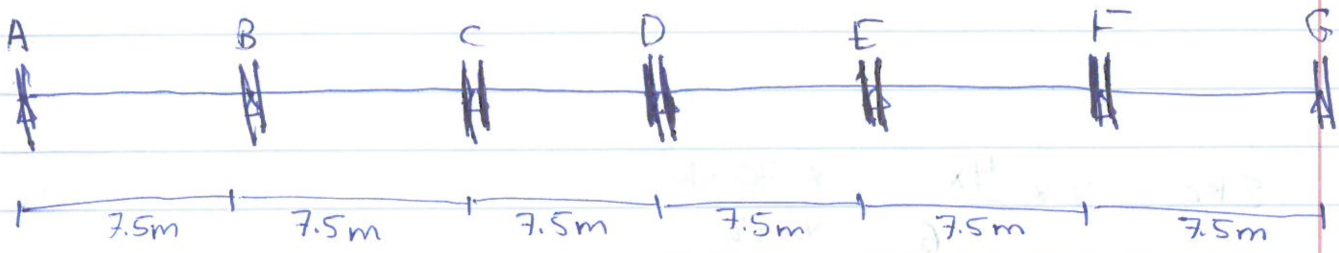
$\Psi_B = 0$

$$\Rightarrow K = 0.6 \rightarrow \frac{K L_u}{r} = \frac{(0.6)(5.35)}{(0.3)(0.35)} = 30.6 < \left(\frac{K L_u}{r}\right)_{limit} = 40$$

$M_{n1s} = M_{n2s}$

$\therefore$ No slenderness effects.

(d) Frame (A)



$LL = 0.5\ t/m^2$

$DL = 0.408\ t/m^2$

$LL > \frac{3}{4}(DL)$

$0.5 > \frac{3}{4}(0.408)$

$0.5 > 0.306$ → we have to Use 3 load Cases:-

$\min = 1.2\,DL$

$\max = 1.2\,DL + 1.6(0.75\,LL)$

$\max = 1.2\,DL + 1.6\,LL$

* Because of the required we will used ~~the one~~ thired load Case:

$W_{max} = 1.2\,DL + 1.6\,LL$

$= 1.2(0.408) + 1.6(0.5) = 1.289\ t/m^2$

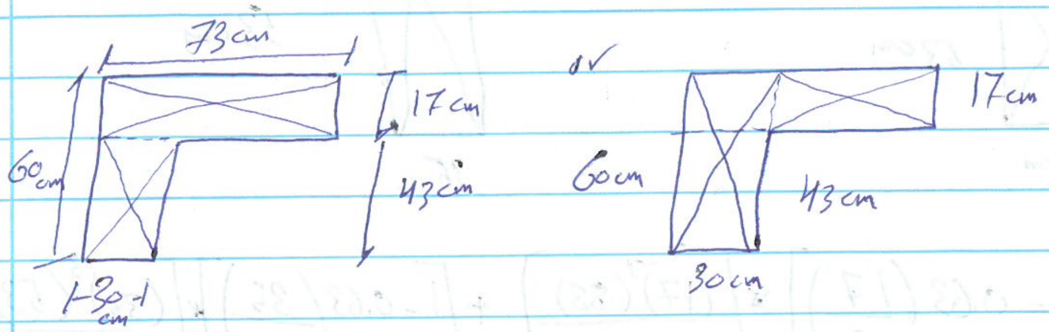
* Relative moment of Inertia:-

$I_1 = \frac{1}{12}(600)(17)^3 = 245650\ cm^4$

$I_2 = \frac{I_1}{\left(1 - \frac{c_2}{L_2}\right)^2} = \frac{245650}{\left(1 - \frac{76}{600}\right)^2} = 314823.7\ cm^4$

torsional Constant c for the edge beams.

Short direction :   Beam B7, B8 - edge



$$C = \left[1 - 0.63\left(\frac{17}{73}\right)\right] \times \left[\frac{(17)^3(73)}{3}\right] + \left[1 - 0.63\left(\frac{30}{43}\right)\right] \times \left[\frac{(30)^3 \cdot (43)}{3}\right]$$

$$= 1.02 \times 10^5 \ cm^4 + 2.17 \times 10^5 \ cm$$

$$= 3.19 \times 10^5 \ cm^4$$

or

$$C = \left[1 - 0.63\left(\frac{17}{43}\right)\right] \times \left[\frac{(17)^3(43)}{3}\right] + \left[1 - 0.63\left(\frac{30}{60}\right)\right] \times \left[\frac{(30)^3(60)}{3}\right]$$

$$= 5.26 \times 10^4 \ cm^4 + 3.70 \times 10^5 \ cm^4$$

$$= \underline{4.23 \times 10^5} \ cm^4$$

- Exterior Equ. Column : Frame A

$$\frac{1}{K_{ec}} = \frac{1}{\Sigma K_c} + \frac{1}{K_t}$$

$$\rightarrow \Sigma K_c = (2) \frac{4EI}{L_{c-c}} = \frac{(2)(4) E_c}{600} \left[\frac{70^4}{12}\right] = 26\,677.8\ E$$

$$\rightarrow K_t = \Sigma \frac{9EC}{L_2 (1 - c_2/L_2)^3} \left(\frac{I_{sb}}{I_s}\right) = \frac{(2)(9) E * 4.23 * 10^5}{600 (1 - \frac{70}{600})^3} \left(\frac{260258.99}{245650}\right)$$

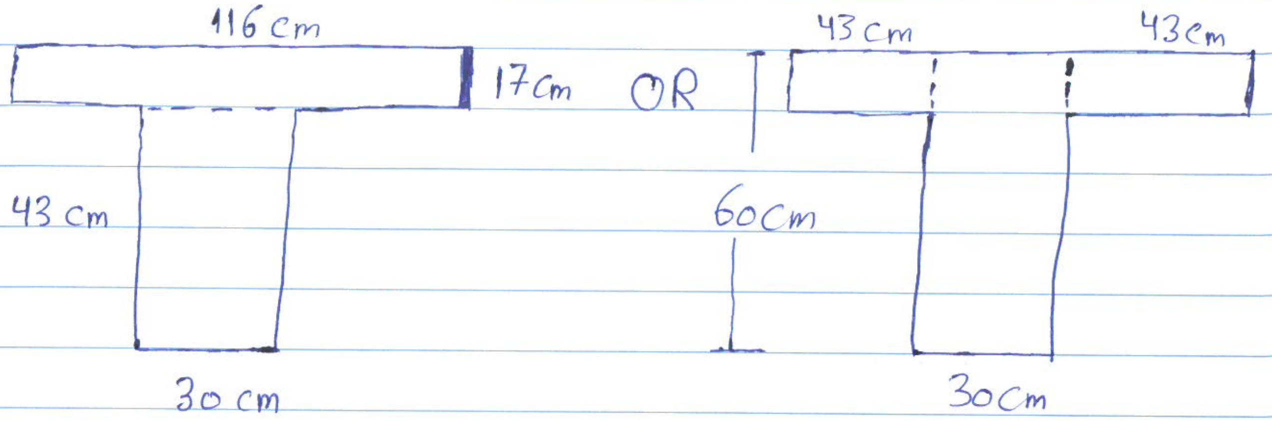
$C_{,\text{for } b_7 - b_8} = 4.23 * 10^5\ cm^4$ $= 195\,063.16\ E$

$I_{sb} \rightarrow T_{section} \rightarrow I_{sb} = 2,60\,25\,80.99\ cm^4$

$I_s = \frac{(600)(17)^3}{12} = 245,650\ cm^4$

$$\Rightarrow K_{ec} = 23\,468.1764 E = \frac{(2) * 4 E * b^4}{(600)(12} \Rightarrow b = 67.8 \approx \cancel{70cm}\ 68\,cm$$

* Interior beams B5, B6 :



$$C = \left[1 - 0.63\left(\frac{17}{116}\right)\right]\left[\frac{(17)^3\,(116)}{3}\right] + \left[1 - 0.63\left(\frac{30}{43}\right)\right]\left[\frac{(30)^3\,(43)}{3}\right]$$

$\rightarrow C = 3.89 * 10^5 \ cm^4$

OR :

$$C = 2\left(5.29 * 10^4\right) + \left(3.7 * 10^5\right)$$

$\longrightarrow \underline{\underline{C = 4.76 * 10^5 \ cm^4}}$ ✓

- Interior equivelent Column:

$$\frac{1}{K_{ec}} = \frac{1}{\Sigma K_c} + \frac{1}{K_t}$$

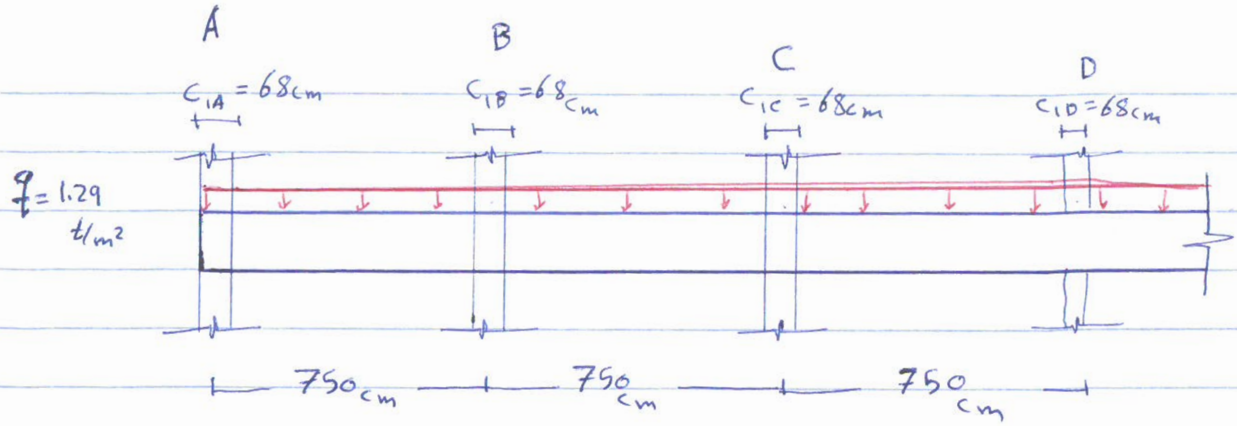
$\Sigma K_c = 26677.8\ E$

$$K_t = \frac{(2)(9)\ E\ C_{b5-66}}{L_2(1-c_2/L_2)^3}\left(\frac{I_{sb}}{I_s}\right)$$

$$= \frac{(2)(9)\ E\ (4.76)10^5}{600\left(1-\frac{70}{600}\right)^3}\left(\frac{2\,602\,580.99}{245\,650}\right)$$

$K_t = 219\ 503.7\ E$

$\Rightarrow K_{ec} = 23\ 786.8\ E \quad \Rightarrow b = 68\ cm$



$$\frac{C_{1A}}{L_1} = \frac{C_{1B}}{L_1} = \frac{68}{750} = 0.09066 \approx 0.10$$

\# From Design Aids Table A.13 A :

Column dim.		FEM Coff.		Stiffness Factor		Carryover	
$\frac{C_{1A}}{L_1}$	$\frac{C_{1B}}{L_1}$	M_{AB}	M_{BA}	K_{AB}	K_{BA}	COF_{AB}	COF_{BA}
0.10	0.10	0.085	0.085	4.18	4.18	0.513	0.513

$$FEM_{AB} = FEM_{BA} = (0.085)\, q\, L_2 L_1^2 = (0.085)(1.29)(6)(7.5)^2 = 37 \text{ t.m}$$

$$K_{s,AB} = K_{s,BA} = (4.18)\, E \frac{(L_2 h_1^3)}{12 L_1} = (4.18) E \frac{(\cancel{\;} I_{sb})}{L_1} = \frac{(4.18) E (2602580.[illegible])}{750}$$

without beams

$$= 14\,505.05\, E$$

$C.O.f = 0.513$

- Distribution Factor :

→ Ex. Column :

$$D.F_{slab} = \frac{K_s}{K_s + K_{ec}} = \frac{14\,505.05}{14\,505.05 + 23\,468.2} = 0.38$$

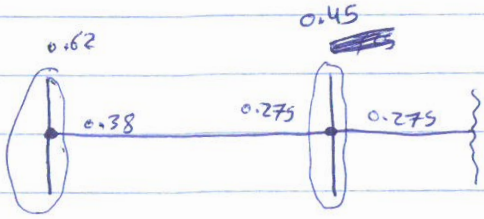
$$D.f_{Column} = 0.62$$

→ Int. Column :

$$D.f_{slab} = \frac{K_s}{2K_s + K_{ec}} = \frac{14\,505.05}{(2)(14505.05) + 23\,786.8} = 0.275$$

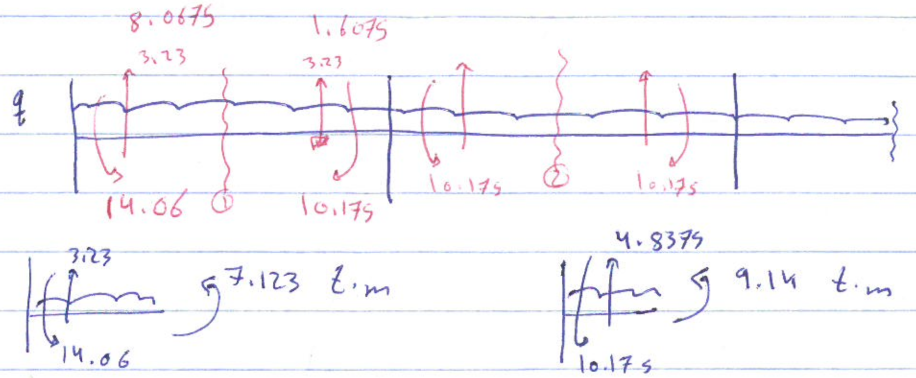
$$D.f_{column} = \cancel{\;} \; 0.45$$

D.f.s :

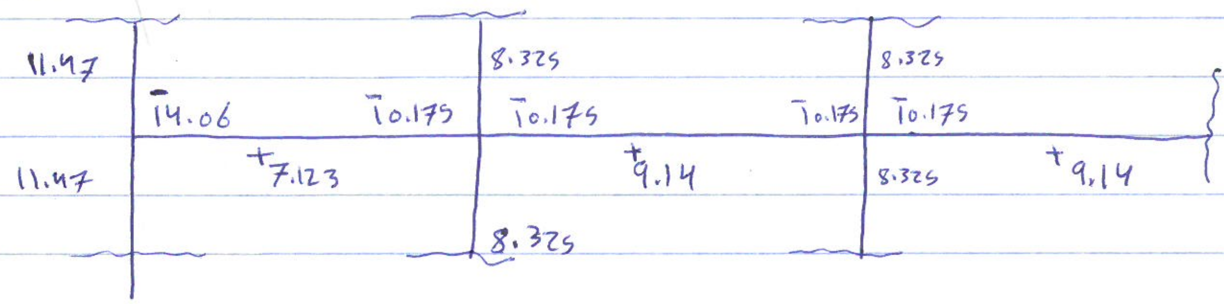


$\Sigma M = 0$ at all joints because all Columns have same equivelant Column width and Same Coefficients.

Frame A : [longitudinal moment Dist

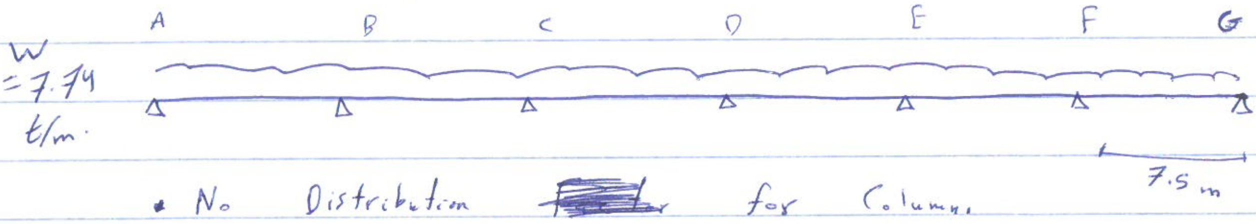


Frame A: (long. moment dist.)



— Equivalent Beam method :

Frame A :



- No Distribution for Column.

$q_f = 1.29\ t/m^2$ (for case load we used)

$W = (1.29)(6) = 7.74\ t/m$ $\quad f = 1.2DL + 1.6LL.$

- Distribution Factor :

$K_{BA} = K_{FG} = \dfrac{3EI}{L}$

$K_{rest} = \dfrac{4EI}{L}$

Same E, I_{sb}, L

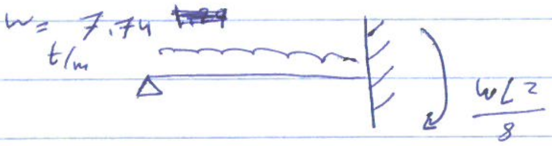
$D.F_{AB} = D.F_{GF} = 1$

$D.F_{BA} = D.F_{FG} = \dfrac{3EI/L}{\frac{3EI}{L} + \frac{4EI}{L}} = 0.43$

$D.F_{BC} = D.F_{FE} = 0.57$

$D.F_{the\ rest} = 0.5 \quad \left(\dfrac{K}{K+K} = 0.5\right).$

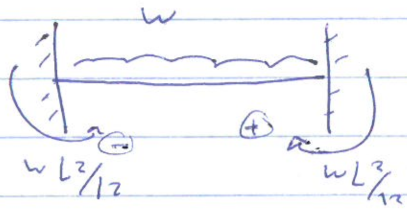
- Fixed END Moments :



$FEM_{BA} = FEM_{FG} = \dfrac{(7.74)(7.5)^2}{8}$

$FEM_{BA} = {}^{+}54.42\ t.m$

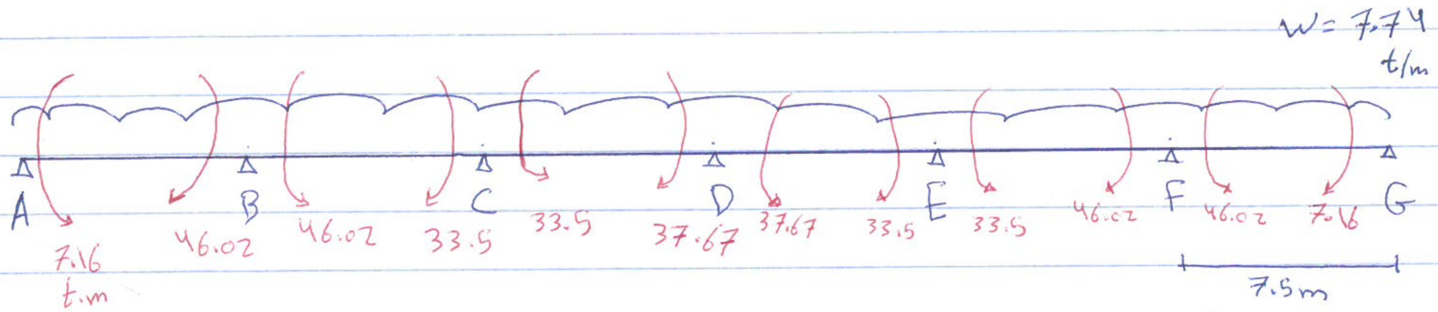
$FEM_{FG} = {}^{-}54.42\ t.m$



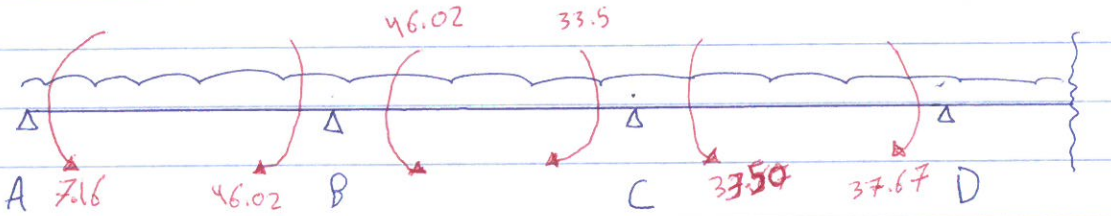
$wL^2/12 = 36.28\ t.m.$

Moment Distribution Method :

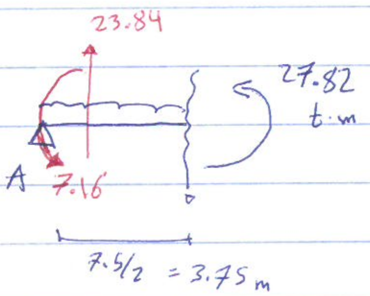
joint	A	B		C		D		E		F		G
	AB	BA	BC	CB	CD	DC	DE	ED	EF	FE	FG	GF
DF	1	0.43	0.57	0.5	0.5	0.5	0.5	0.5	0.5	0.57	0.43	1
FEM	-7.16	54.42	-36.28	36.28	-36.28	36.28	-36.28	36.28	-36.28	36.28	-54.42	-7.16
DIS.		-7.8	-10.34	0	0	0	0	0	0	10.34	7.8	
CO.		0	0	-5.17	0	0	0	0	5.17	0		
DIS.		0	0	2.585	2.585	0	0	-2.585	-2.585	0	0	
CO.			1.292	0	0	1.292	-1.292	0	0	-1.292		
DIS.		-0.556	-0.737	0	0	0	0	0	0	0.737	0.556	
CO.			0	-0.368	0	0	0	0	0.368	0		
DIS.		0	0	0.1842	0.1842	0	0	-0.1842	-0.1842	0	0	
CO.			0.092	0	0	0.092	-0.092	0	0	-0.092		
DIS.		-0.0396	-0.0525	0	0	0	0	0	0	0.0525	0.0396	
CO.			0	-0.02625	0	0	0	0	0.026245	0		
DIS.		0	0	0.01312	0.01312	0	0	-0.013	-0.013	0	0	
CO.			0.00656	0	0	0.00656	-0.00656	0	0	-0.00656		
DIS.		-0.00282	-0.0037	0	0	0	0	0	0	0.0037	0.00282	
M	-7.16	46.02	-46.02	33.5	-33.5	37.67	-37.67	33.5	-33.5	46.02	-46.02	-7.16

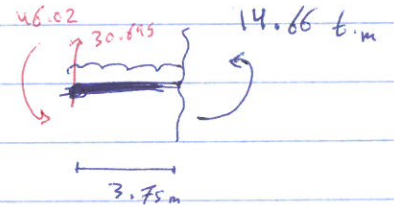


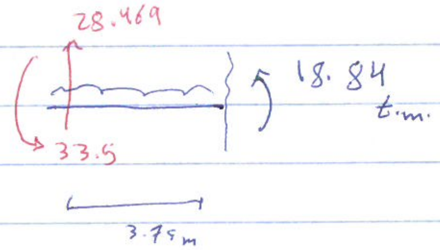
* Symmetric at D



↑ 29.025	↑ 29.025	↑ 29.025	↑ 29.025	↑ 29.025	↑ 29.025
↓ 5.18	↑ 5.18	↑ 1.67	↓ 1.67	↓ 0.556	↑ 0.556
↑ 23.84 t	↑ 34.2 t	↑ 30.695 t	↑ 27.355 t	↑ 28.469	↑ 29.581







$\Sigma M_o = 0 \Rightarrow M = {}^{+}27.82$ t.m

Longitudinal Moment Distribution: Frame A

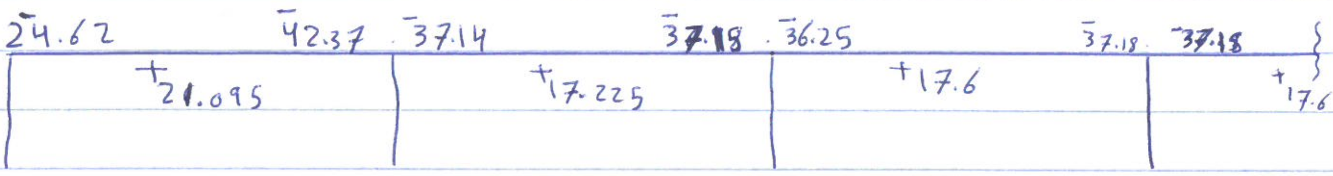
M (t.m):

	A–B	B–C	C–D	D–
Support	$^{-}$7.16 / $^{-}$46.02	$^{-}$46.02 / $^{-}$33.5	$^{-}$33.5 / $^{-}$37.67	$^{-}$37.67
Span	$^{+}$27.82	$^{+}$14.66	$^{+}$18.84	$^{+}$18.84

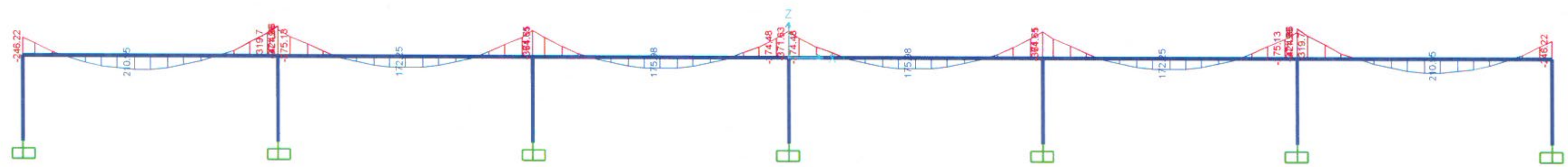
The values Symmetric at D support.

- Analysis using Computer Software (SAP2000)

For Frame A:



Frame A.sdb



SAP2000 22.0.0

Moment 3-3 Diagram (1.2D+1.6L)

KN, m, C

- Longitudinal moments in each method:
- For Frame A:

1 Direct Design method.

Exterior span			Interior span	
Ex. -ve	+ve	int. -ve	-ve	+ve
-7.16	+25.5	-31.32	-29.08	+15.66

2 Equivalent Frame method.

Exterior span			Interior span	
Ex. -ve	+ve	int -ve	-ve	+ve
-14.06	+7.123	-10.175	-10.175	+9.14

3 Equivalent Beam method.

Exterior span			First interior span			Second interior span		
EX. -ve	+ve	int -ve	EX. -ve	+ve	int -ve	EX -ve	+ve	int. -ve
-7.16	+27.82	-46.02	-46.02	+14.66	-33.5	-33.5	+18.84	-37.67

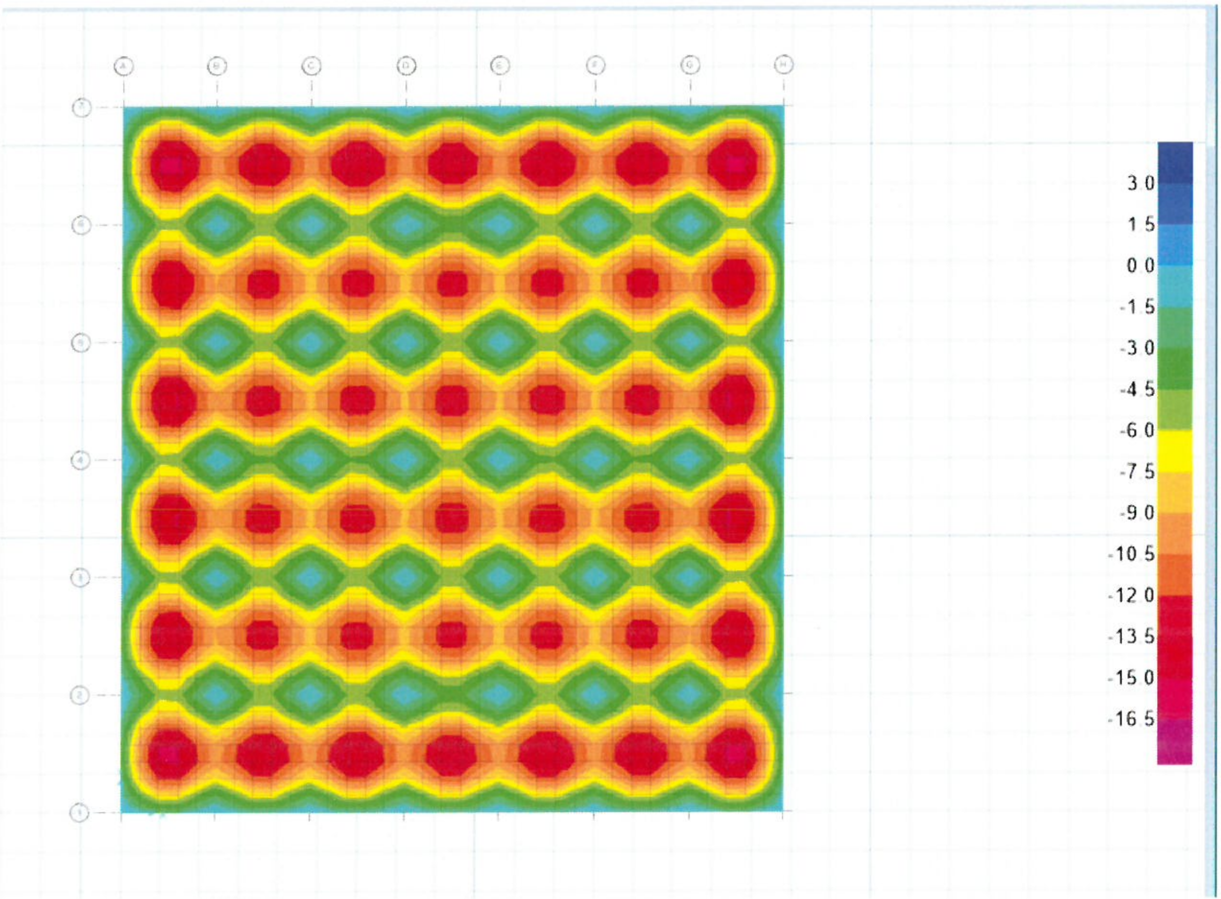
4 Computer Software analysis.

Exterior span			First interior span			Second interior span		
EX -ve	+ve	int. -ve	EX. -ve	+ve	int. -ve	EX -ve	+ve	int. -ve
-24.62	+21.095	-42.37	-37.14	+17.225	-37.18	-36.25	+17.6	-37.18

Deflection Check (using Etabs) :

Deflection limit = L/240 = 600/240 = 2.5 cm

Max. deflection : 2 cm < def. limit (acceptable)



Moment distribution (for load case 1.2 DL + 1.6 LL)