Chapter 4

Structural Analysis and Design.

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4.1 Introduction:

Concrete is the only major building material that can be delivered to the job site in a plastic state. This unique quality makes concrete desirable as a building material because it can be molded to virtually any form or shape.

Concrete used in most construction work is reinforced with steel. When concrete structure members must resist extreme tensile stresses, steel supplies the necessary strength. Steel is embedded in the concrete in the form of a mesh, or roughened or twisted bars. A bond forms between the steel and the concrete, and stresses can be transferred between both components.

In This Project, there are 4 types of slabs: solid slabs one way and tow way andr rib slab one way and tow way. They would be analyzed and designed by using finite element method of design, with aid of a computer Program called " ATTIR- Software" to find the internal forces, deflections and moments for ribbed slabs.

The design strength provided by a member, its connections to other members, and its cross-sections in terms of flexure, and load, and shear is taken as the nominal strength calculated in accordance with the requirements and assumptions of ACI-code.

<u>NOTE:</u>

*Concrete B300 ………..{ $f_c^{'}$= 24 MPa for rectangular section}.

*The specified yield strength of the reinforcement {fy = 420 MPa}.

4.2 Factored Loads:

The structure may be exposed to different loads such as dead and live loads. The value of the load depends on the structure type and the intended use. The factored loads on which the structural analysis and design is based for our project members, is determined as follows:

$q_u = 1.2DL + 1.6L$ *ACI – 318 - 14 (9.2.1)*

4.3 Determination of Slabs Thickness.

4.3.1- Determination of Thickness of The Two-Way solid Slab. Take the critical panel shown below:

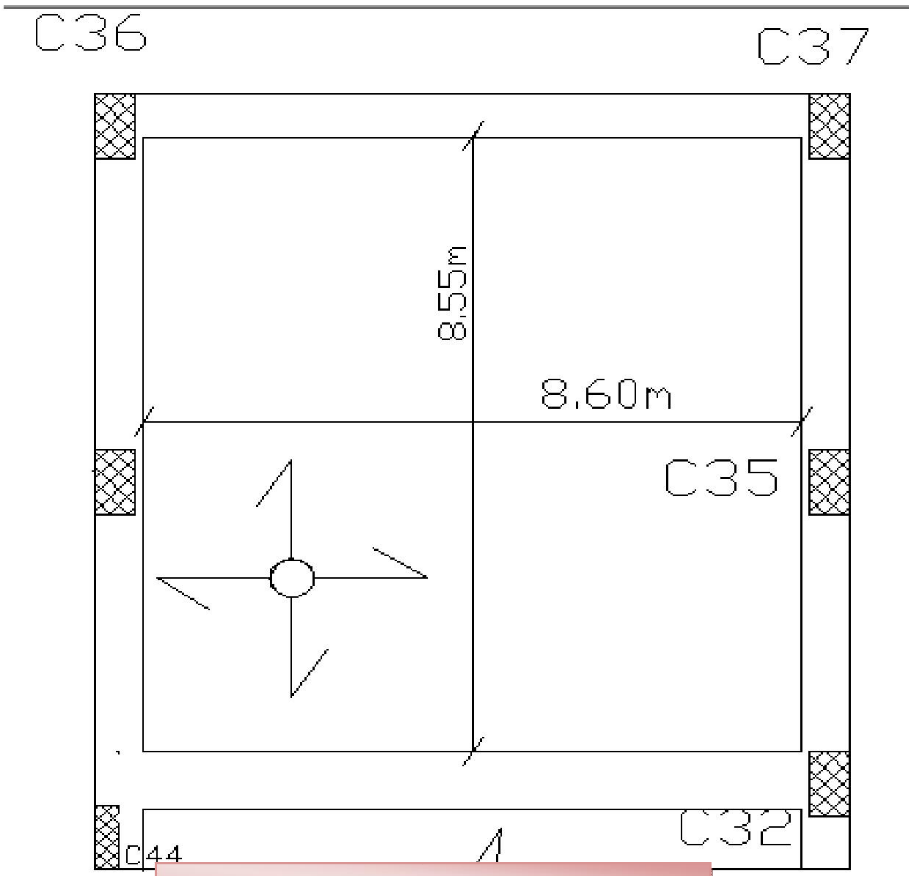

Figure (4-1) Two way solid Slab.

Let h for beam= 60 cm

$$h_{min} = \frac{(2*8.6)+(2*8.55)}{180} = 0.19 \text{ m}$$

take h= 20 cm.

<u>For exterior beam (L section beam)</u>

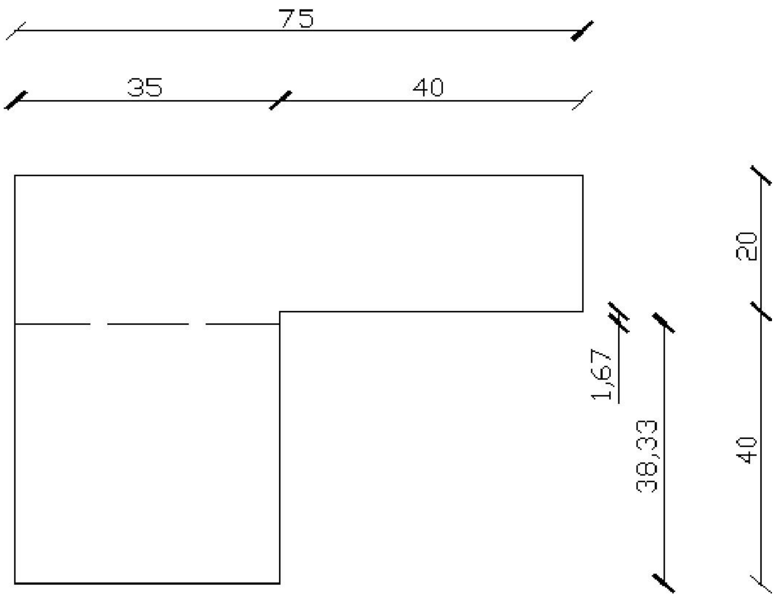

Figure (4-2) exterior beam (L-section).

$$\overline{Y_{beam}} = \frac{\Sigma AY}{\Sigma A} = \frac{(35 \times 60 \times 30) + (75 \times 20 \times 50)}{(35 \times 60) + (75 \times 20)} = 38.33 \ cm$$

$$I_{beam} = \frac{35(38.33)^3}{3} - \frac{40 \times (1.67)^3}{3} + \frac{75 \times (21.67)^3}{3}$$

$$I_{beam} = 911334.4 \ cm^4/b$$

<u>For interior beam (T section beam)</u>

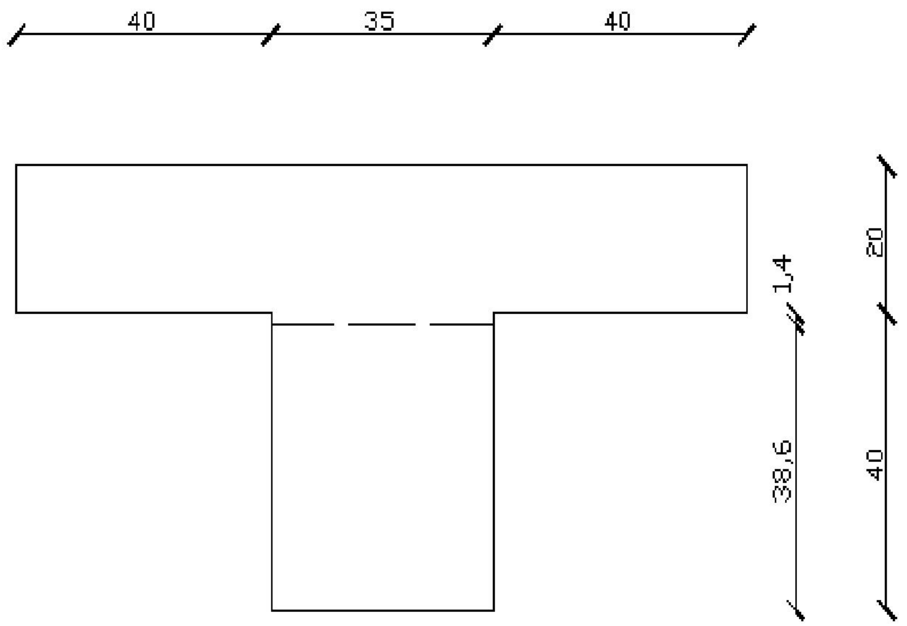

Figure (4-3) interior beam (T-section).

$$\overline{Y_{beam}} = \frac{\Sigma AY}{\Sigma A} = \frac{(35 \times 40 \times 20) + (115 \times 20 \times 50)}{(35 \times 40) + (115 \times 20)} = 38.6 \ cm$$

$$I_{beam} = \frac{35(38.6)^3}{3} - \frac{2 \times 40 \times (1.4)^3}{3} + \frac{115 \times (21.4)^3}{3}$$

$$I_{beam} = 1046585.33 \ cm^4/b$$

For the slab :

$$I_s = \frac{1}{12} bh^3$$

$$b_1 = 35 + 430 = 465 \ cm$$

$$I_{s1} = \frac{465 \times 20^3}{12} = 310000\ cm^4$$

$$\alpha_1 = \frac{I_{b1}}{I_{s1}} = \frac{911334.4}{310000} = 2.93$$

$$b_2 = 35 + 427.5 = 462.5\ cm$$

$$I_{s2} = \frac{462.5 \times 20^3}{12} = 308333.3\ cm^4$$

$$\alpha_2 = \frac{I_{b2}}{I_{s2}} = \frac{911334.4}{308333.3} = 2.95$$

$$b_3 = 35 + 430 = 465\ cm$$

$$I_{s3} = \frac{465 \times 20^3}{12} = 310000\ cm^4$$

$$\alpha_3 = \frac{I_{b3}}{I_{s3}} = \frac{911334.4}{310000} = 2.93$$

$$b_4 = 35 + 210 + 427.5 = 672.5\ cm$$

$$I_{s4} = \frac{672.5 \times 20^3}{12} = 310000\ cm^4$$

$$\alpha_4 = \frac{I_{b4}}{I_{s4}} = \frac{1046585.33}{448333.3} = 2.33$$

$$\alpha_{fm} = \frac{\Sigma\alpha_n}{4} = 2.785 \quad > 2$$

$$\beta = \frac{large\ clear\ span}{short\ clear\ span}$$

$$\beta = \frac{8.6}{8.55} = 1.005$$

$h = \frac{l_n(0.8+ fy \backslash 1400)}{36+9\beta}$ & not less than 90 mm.

$h_m = \frac{8600(0.8+420\backslash 1400)}{36+9(1.005)}$ = 210 mm $> 90\ mm$.

Take h = 25 cm > $h_m = 21$ cm.

4.3.2- Determination of thickness for The One-Way solid slab.

Group A :

$h_{m\ both\ end\ continous}$

$h > \frac{L}{28} = \frac{640}{28} = 22.8\ c\ m - control$.

Group B :

$h_{m\ one\ end\ continous}$

$$h > \frac{L}{24} = \frac{470}{24} = 19.5\ cm$$

$h_{m\ both\ end\ continous}$

$$h > \frac{L}{28} = \frac{470}{28} = 16.8\ cm$$

$h_{m\ cantaliver}$

$$h > \frac{L}{10} = \frac{190}{10} = 19\ cm$$

Take h = 25 cm > $h_m = 22.8$ cm.

4.3.3- Determination of Thickness of The Two-Way ribbed Slab.

Take the critical panel shown below:

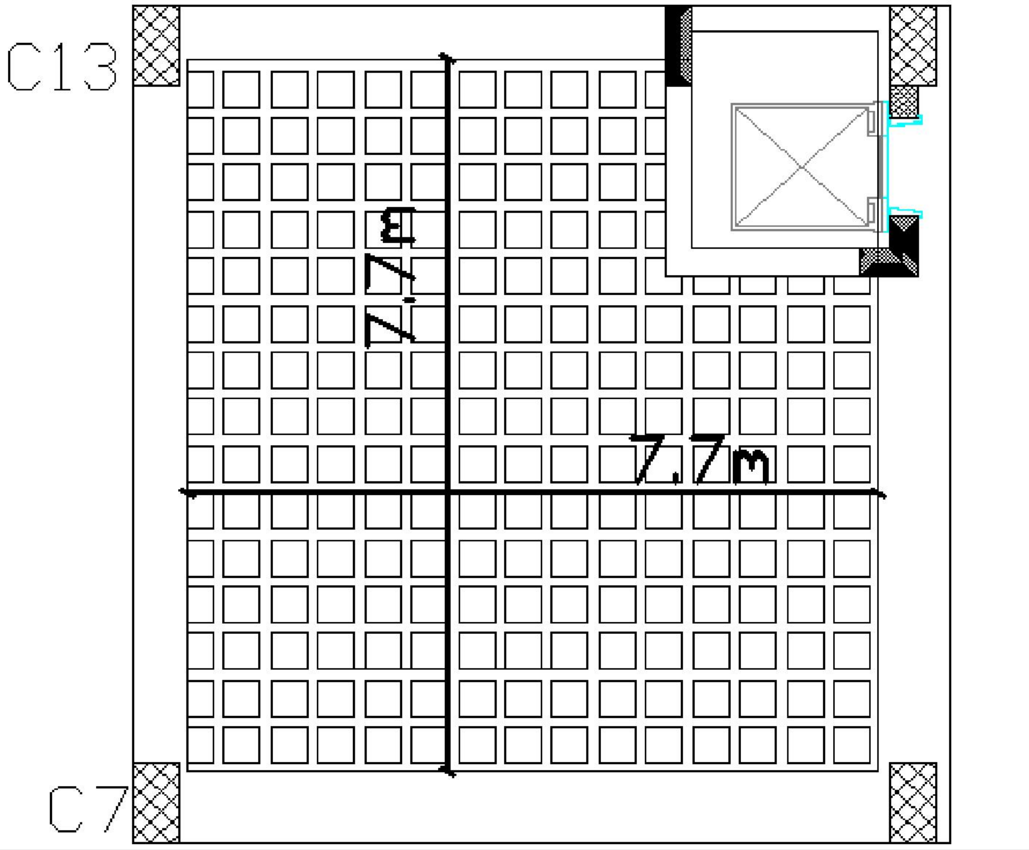

Figure (4-4) two way ribbed slab .

For beams

$$I_{b1,3} = \frac{1}{12} bh^3 = \frac{1}{12} \times 60 \times 60^3 = 1080000\ cm^4$$

$$I_{b2,4} = \frac{1}{12}bh^3 = \frac{1}{12} \times 80 \times 60^3 = 1440000\ cm^4$$

<u>For ribbes</u>

Let h for slab = 32 cm

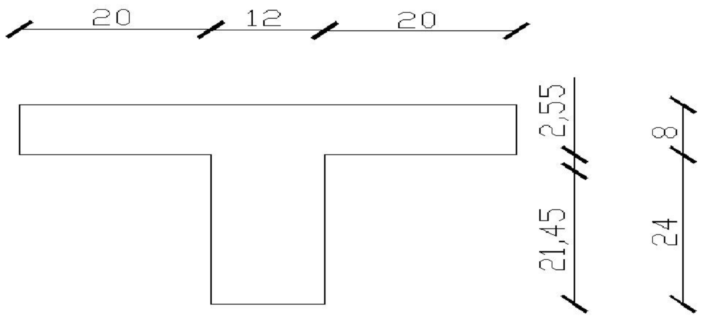

Figure (4-5) section of ribb.

$$\overline{Y_{ribb}} = \frac{\Sigma AY}{\Sigma A} = \frac{(40 \times 8 \times 4) + (32 \times 12 \times 16)}{(40 \times 8) + (32 \times 12)} = 10.55\ cm$$

$$I_{ribb} = \frac{52(10.55)^3}{3} - \frac{2 \times 20 \times (2.55)^3}{3} + \frac{12 \times (21.45)^3}{3}$$

$$I_{ribb} = 59609\ \ cm^4/ribb$$

$$b_1 = 60 + 385 = 455\ cm$$

$$I_{s1} = 59609 \times \frac{455}{52} = 510115.48\ cm^4$$

$$\alpha_1 = \frac{I_{b1}}{I_{s1}} = \frac{1080000}{510115.48} = 2.11$$

$$b_2 = \left(\frac{770}{2}\right) + 80 + \left(\frac{770}{2}\right) = 850\ cm$$

$$I_{s2} = 59609 \times \frac{850}{52} = 974377.88\ cm^4$$

$$\alpha_2 = \frac{I_{b2}}{I_{s2}} = \frac{1440000}{974377.88} = 1.47$$

$$b_3 = 60 + 385 = 445\ cm$$

$$I_{s3} = 59609 \times \frac{445}{52} = 510115.48\ cm^4$$

$$\alpha_3 = \frac{I_{b3}}{I_{s3}} = \frac{1080000}{510115.45} = 2.11$$

$$b_4 = \left(\frac{770}{2}\right) + 80 + \left(\frac{380}{2}\right) = 630\ cm$$

$$I_{s4} = 59609 \times \frac{630}{52} = 750844.13\ cm^4$$

$$\alpha_4 = \frac{I_{b4}}{I_{s4}} = \frac{1440000}{750844.13} = 1.91$$

$$\alpha_{fm} = \frac{\Sigma\alpha_n}{4} = 1.91$$

$$2 > \alpha_{fm} > 0.2$$

$h_m = \frac{l_n(0.8+ fy \backslash 1400)}{36+5\beta(\alpha_{fm}-0.2)}$ & Not Less Than 125 mm.

$$\beta = \frac{large\ clear\ span}{short\ clear\ span}$$

$$\beta = \frac{7.7}{7.7} = 1$$

$h_m = \frac{7700(0.8+420\backslash 1400)}{36+5(1)(1.91-0.2)}$ = 190.3 mm >125mm.

190.3 mm < 320 mm .

So $\boldsymbol{Select\ h = 32\ cm}$

h =32 cm (24 \cm Hollow block + 8cm Toppin)

4.3.4 - Determination of thickness for The One-Way ribbed slab.

* $h_{m\ one\ end\ continous}$

$$h > \frac{L}{18.5} = \frac{500}{18.5} = 27.2 \quad cm$$

* $h_{m\ both\ end\ continous}$

$$h > \frac{L}{21} = \frac{640}{21} = 30.47\ c\ m - control$$

$$h_m = 30.47\ \boldsymbol{cm} < 32\ cm.$$

So $\boldsymbol{Select\ h = 32\ cm\,.}$

h =32 cm (24 \cm Hollow block + 8cm Toppin)

4.4 Design of Topping:

Table (4 – 1) Calculation of the total dead load on topping:

No.	Material	Calculation
1	Tile	0.03*24.5*1=0.735 KN/m
2	mortar	0.03*22*1= 0.66 KN/m
3	Coarse sand	0.15*17*1= 2.55 KN/m
4	topping	0.08*24.5*1 = 1.96 KN/m
5	Interior partitions	1.25 *1 =1.25 KN/m
Sum		7.155 KN/m

Live load =2.4 *1 KN/m

$W_u = 1.2DL + 1.6L$ =(1.2 * 7.155) + (1.6 * 2.4) = 12.42 KN/m. (Total Factored load)

Assume slab fixed at supported points (ribs):

$$M_u = \frac{Wu \; * \; l^2}{12}$$

$$= \frac{1242 * 0.4^2}{12} = 0.165 \text{ KN.m/m of strip width .}$$

(ΦM)n > Mu [Strength Condition, where Φ= 0.55] for plane concrete

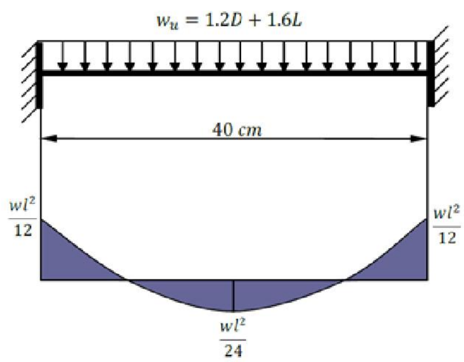

Figure 4.6 Topping System

$Mn = 0.42 \times \sqrt{fc'} Sm$ *ACI-318-14 (22-5.1)*

$S_m = \frac{b*h^2}{6} = \frac{1000*80^2}{6}$ = 1066666.67 mm^3

$Mn = 0.42 \times \sqrt{24} * 1066666.67 =$ 2.194 KN.m

Φ Mn = 0.55 * 2.194= 1.2 KN.m

Φ Mn = 1.2 KN.m > Mu = 0.165 KN.m

<u>No reinforcement required</u> by analysis. According to ACI 10.5.4, provide

$A_{(s,min)}$for slabsas shrinkage and temperature reinforcement.

According to ACI 7.12.2.1,ρshrinkage = 0.0018

As_{min} = ρ * b * h = 0.0018 * 1000 * 80 = 144mm^2 /1m

Try barsΦ 8 with As = 50.27mm^2

No. of Φ8 = $\frac{As_{req}}{A_{bar}} = \frac{144}{50.26}$ = 2.86 →Spacing(S) = $\frac{1}{2.86}$ = 0.348m = 348 mm

$\leq 380\ (\frac{280}{fs}) - 2.5 * Cc \quad \leq 300\ (\frac{280}{fs})$

$S = 380*(\frac{280}{\frac{2}{3}*420}) - 2.5*20 = 330mm$

$S = 300*(\frac{280}{\frac{2}{3}*420}) = 300mm$

Not more than: S_{max} = 450 mm

Use Φ 8 / 20 cm, with As $_{provided}$= 251mm^2/1mboth directions.

4.5 Determination of Loads of Ribs:

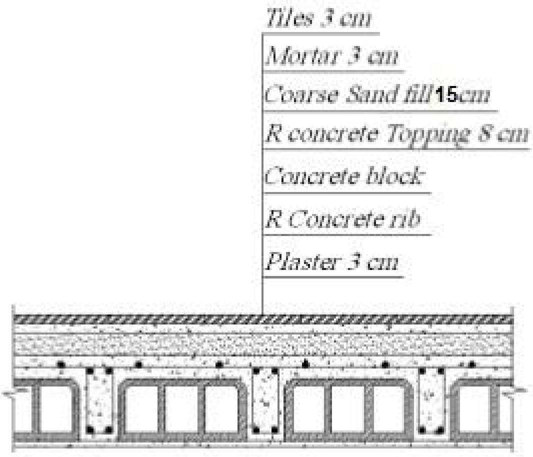

Figure 4.7-Typical Section in Ribbed slab.

For the one-way ribbed slabs, the total dead load to be used in the analysis and design is calculated as follows:

Table (4 – 2) Calculation of the total dead load for one way rib slab.

No.	Parts of Rib	Calculation
1	Tile	24.5*0.03*0.52 = 0.382 KN/m
2	mortar	0.343 KN/m = 22*0.03*0.52
3	Coarse sand	17*0.15*0.52= 1.326 KN/m
4	topping	24.5*0.08*0.52 = 1.02 KN/m
5	RC rib	24.5*0.24*0.12 = 0.705 KN/m
6	Hollow block	9*0.24*0.4 =0.864 KN/m
7	plaster	22*0.02*0.52 =0.228 KN/m
8	Interior partitions	1.25 *0.52 = 0.65 KN/m
Sum		5.52 KN/m

Nominal Total Live Load:

Live load = 2.4 * 0.52 = 1.248 KN/m of rib

Factored dead Load = 1.2* 5.52 = 6.624 KN/m

Factored live Load = 1.6 * 1.248 = 1.99 KN/m

4.6 Design of Rib R(14) :

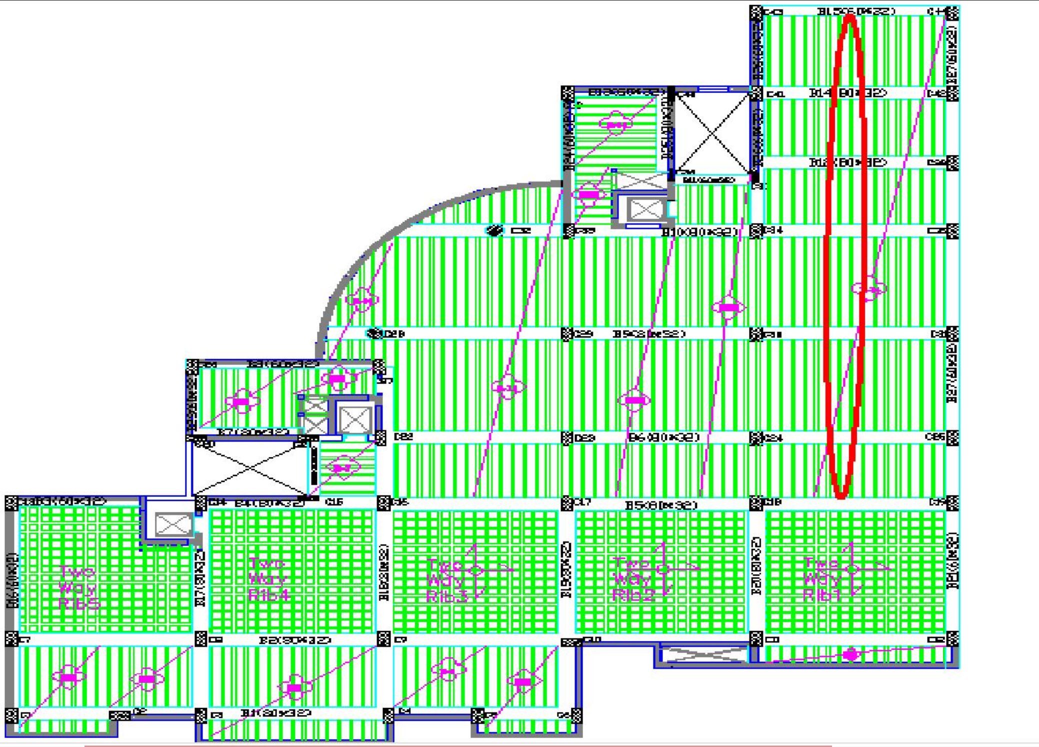

Figure 4.8 Rib 14.

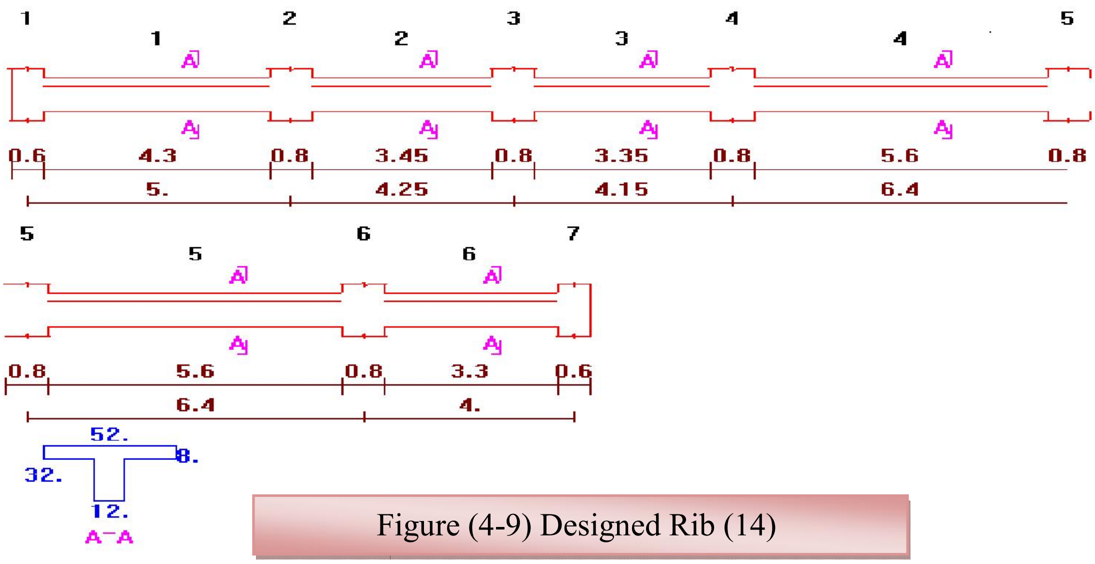

Figure (4-9) Designed Rib (14)

4.6.1 Introduction

The envelope for both shear and moment diagrams drawn using ATIR program as shown below:

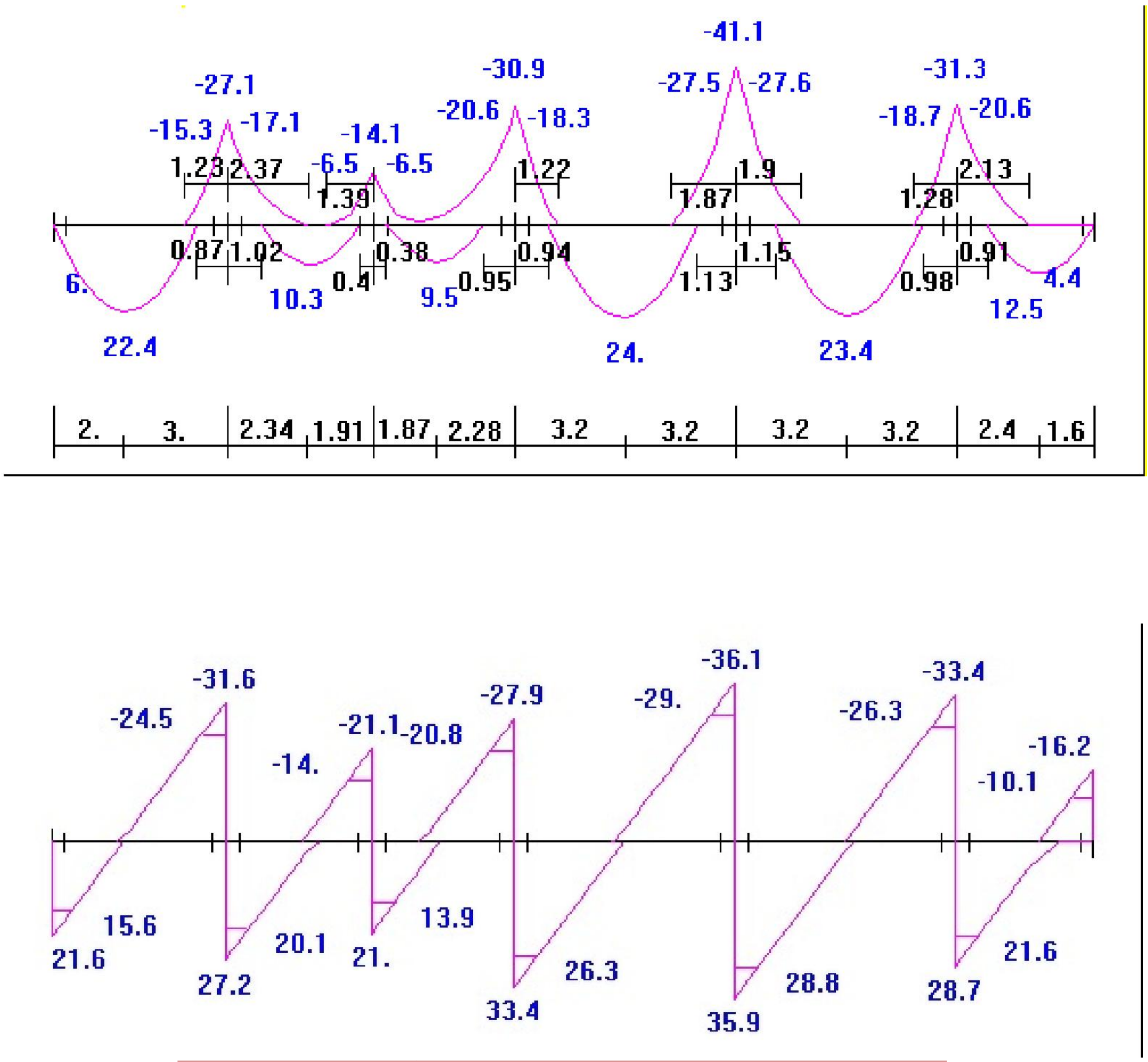

Figure (4-10) shear and moments diagrams for Rib (R14)

4.6.2 Design of Positive Moment of Rib (R14) Mu max = 24 KN.m

$$d = 320 - 20 - 8 - 6 = 286\ mm$$

$$d = 286\ mm$$

$$fc' \leq 28 \dashrightarrow \beta = 0.85$$

Design of $+\boldsymbol{MU_{MAX} = 24\ KN.m}$

$$Rn = \frac{M_u/\Phi}{b \times d^2}$$

$$Rn = \frac{24 \times 10^{-3}/0.9}{0.52 \times (0.286)^2} = 0.6269\ \ MPa$$

$$\rho_{req} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mRn}{fy}}\right)$$

$$m = \frac{fy}{0.85 \times fc'}$$

$$m = \frac{420}{0.85 \times 24} = 20.6$$

$$\rho_{req} = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.6269}{420}}\right) = 0.0015$$

$$As_{req} = \rho \times b \times d$$

$$As_{req} = 0.0015 \times 520 \times 286 = 223.08\ mm^2$$

Check for $\mathbf{As_{min}}$

$$As_{min} = 0.25\frac{\sqrt{fc'} \times b_w \times d}{fy}$$

$$As_{min} = 0.25\frac{\sqrt{24} \times 120 \times 286}{420} = 100.08\ mm^2$$

Not less than

$$As_{min} = \frac{1.4 \times 120 \times 286}{420} = 114.4\, mm^2 \quad \Rightarrow Controls$$

$$As_{req} = 223.08\ mm^2 > \ As_{min} = 114.4\ mm^2$$

selectAs $2\Phi 12 = 226.2\, mm^2 > \ As_{req} = 223.08\, mm^2$

According to Atir Program the limitation of deflection is satisfied and so, no additional reinforcement is required.

-Check of Strain

Tension=Compression

T=C

$fy \times As = 0.85 \times fc' \times \mathrm{b} \times \mathrm{a}$

$420 \times 226.2 = 0.85 \times 24 \times 520 \times \mathrm{a}$

a= $8.95 mm$

$$c = \frac{a}{0.85} = 10.53\, mm$$

$$\frac{\epsilon_s}{d-c} = \frac{0.003}{c}$$

$$\epsilon_s = \frac{268 - 10.53}{10.53} \times 0.003$$

$$\epsilon_s = 0.073 \ > 0.005$$

Design of other spans :

$select\ 2\phi 12\ with\ As = 226.2\ m$ > As req.

4.6.3 Design of Negative Moment of Rib (R14) $\_MU_{MAX} = 27_{.}6\ KN_{.}m$

The design of the negative moment for the T-section is made as a rectangular section with b = bw.

Design of $-MU_{MAX} = \mathbf{27.6\ KN.m}$

$$Rn = \frac{M_u/\Phi}{b \times d^2}$$

$$Rn = \frac{27.6 \times 10^{-3}/0.9}{0.12 \times (0.286)^2} = 3.124\ MPa$$

$$\rho_{req} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mRn}{fy}}\right)$$

$$m = \frac{fy}{0.85 \times fc'}$$

$$m = \frac{420}{0.85 \times 24} = 20.6$$

$$\rho_{req} = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 3.124}{420}}\right) = 0.0081$$

$$As_{req} = \rho \times b \times d$$

$$As_{req} = 0.0081 \times 120 \times 286 = 278.59\, mm^2$$

Check for As_{min}

$$As_{min} = 0.25 \frac{\sqrt{fc'} \times b_w \times d}{fy}$$

$$As_{min} = 0.25 \frac{\sqrt{24} \times 120 \times 286}{420} = 100.08\, mm^2$$

Not less than

$$As_{min} = \frac{1.4 \times 120 \times 286}{420} = 114.4\ mm^2 \quad \Rightarrow Controls$$

$$As_{req} = 278.59\ mm^2 > \ As_{min} = 114.4\, mm^2$$

selectAs $2\Phi 14 = 307.87\, mm^2 > \ As_{req} = 278.59\ mm^2$

According to Atir Program the limitation of deflection is satisfied and so, no additional reinforcement is required.

-Check of Strain

Tension=Compression

T=C

$fy \times As = 0.85 \times fc' \times \text{b} \times \text{a}$

$420 \times 307.87 = 0.85 \times 24 \times 120 \times \text{a}$

a= $52.82\ mm$

$$c = \frac{a}{0.85} = 62.14\ mm$$

$$\frac{\epsilon_s}{d - c} = \frac{0.003}{c}$$

$$\epsilon_s = \frac{286 - 62.14}{62.14} \times 0.003$$

$$\epsilon_s = 0.01 > 0.005$$

Design of other spans :

$select\ 2\phi 14\ with\ As = 307.78\ mm^2 > As\ \ req.$

4.6.4 Design of Shear of Rib (R14) Vu $_{max}$ = 29 kN.

Design of Shear of Rib for the mid Span :

Vu $_{max}$ = 29 kN.

$$\Phi Vc = \Phi \frac{1}{6} \sqrt{fc'}\ bw \times d$$

$$\Phi Vc = \frac{0.75}{6} \times \sqrt{24} \times 120 \times 286 = 21.01\ kN$$

$$\Phi Vs_{min} = \Phi \frac{1}{3} \times bw \times d$$

$$\Phi Vs_{min} = 0.75 \times \frac{1}{3} \times 120 \times 286 = 7.58 \text{ kN}$$

$$\Phi Vs_{min} = \frac{\Phi}{16}\sqrt{24} \times 120 \times 286 = 7.88 \text{ kN} \Longrightarrow\Longrightarrow Control$$

$$\Phi Vc + \Phi Vs_{min} < \text{Vu} < \Phi Vc + \frac{\Phi}{3}\sqrt{fc'}\, bw \times d$$

$$21.01 + 7.88 < 29 < 21.01 + \frac{0.75}{3}\sqrt{24} \times 120 \times 286$$

$$28.89 < 29 < 63.04$$

$$Category\ No.\ 4\ Is\ Satisfied$$

$$Vs = \frac{Vu}{\Phi} - Vc$$

$$Vs = \frac{29}{0.75} - 28.01 = 10.65\ kN$$

$$\left(\frac{\text{Av}}{s}\right)_{min} = \frac{bw}{3 \times fy} = \frac{120}{3 \times 420} = 0.095 \times 10^{-3}$$

$$\geq \frac{bw \times \sqrt{fc'}}{16 \times fy} = \frac{120 \times \sqrt{24}}{16 \times 420} = 0.087 \times 10^{-3}$$

$$\frac{\text{Av}}{s} = \frac{Vs}{d \times fy} = \frac{0.0106}{420 \times 0.286} = 0.08866 \times 10^{-3}$$

$$\frac{\text{Av}}{s} = \frac{2 \times 50 \times 10^{-6}}{s} = 0.08866 \times 10^{-3}$$

s$= 112\ cm$

s$\leq \frac{d}{2} = \frac{286}{2} = 14.3\ cm$

s$\leq 60\ cm, select\ s = 15cm \Longrightarrow Use\ \Phi 8 -$ Stirrup at 15cm.

4.7 Design of two way ribbed slab (R05)

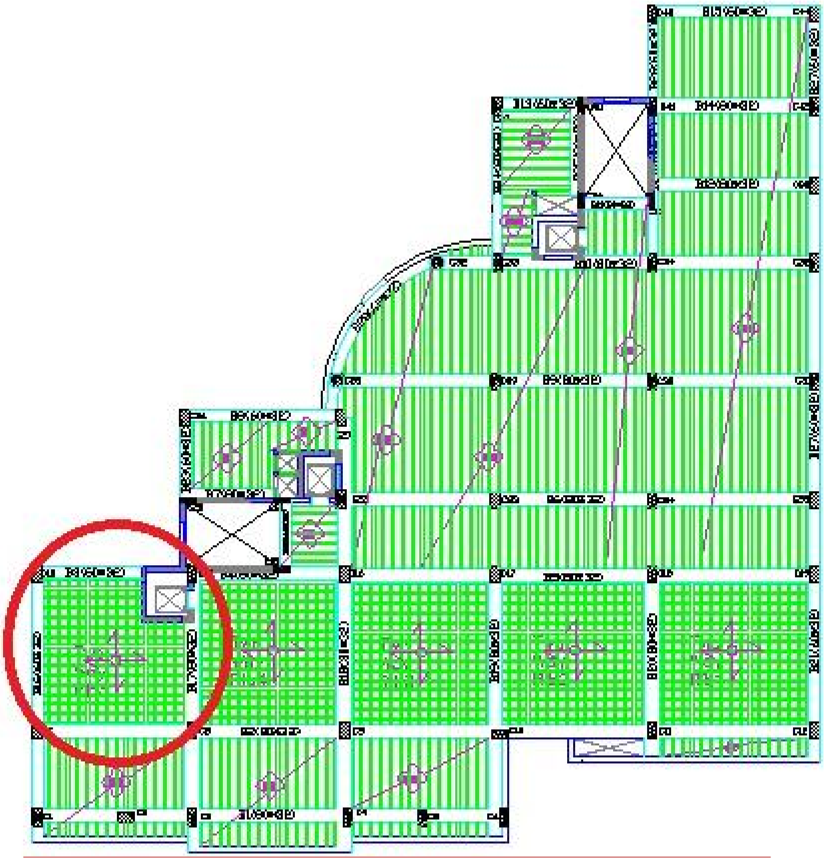

Figure 4.11 Rib 05.

4.7.1 Load calculation:

For the two-way ribbed slabs, the total dead load to be used in the analysis and design is calculated as follows:

Table (4-3) Calculation of the total dead load for two way rib slab (25).

Material	Quality Density (KN/m^3)	$W = \gamma * V$ (KN)
Tiles	24.5	24.5 × 0.03 ×0.52 ×0.52 = 0.199
Mortar	22	22 × 0.03 ×0.52 ×0.52 = 0.179
Sand	17	17 × 0.15 ×0.52 ×0.52 = 0.69
Topping	24.5	24.5 × 0.08 ×0.52 ×0.52 = 0.53
Concrete Rib	24.5	24.5 × 0.24 ×0.12 ×(0.52+0.40)= 0.649
Hollow block	9	9 × 0.24 ×0.4 ×0.4= 0.3456
Plaster	22	22 × 0.02 ×0.52 ×0.52 = 0.119
Partition = 1.5 KN/m^2		1.5 ×0.52 ×0.52 = 0.405
Total Dead load, KN		3.1166

Dead Load of slab:

$$DL = \frac{3.1166}{0.52 * 0.52} = 11.526 \ KN/m^2$$

$$w_D = 1.2 * 11.526 = 13.83 \ KN/m^2$$

$$LL = 2.4 \ KN/m^2$$

$$w_L = 1.6 * 2.4 = 3.84 \ KN/m^2$$

$$w = 13.83 + 3.84 = 17.67 \ KN/m^2$$

4.7.2 Moments calculations:

m = la/lb = 7.7/7.7 = 1 with case 4.

$$Ma = Cawla^2bf \ and \ Mb = Cbwlb^2bf$$

-Negative moment

$C_{a,neg} = 0.05$

$C_{b,neg} = 0.05$

$$\boldsymbol{M_{a,neg} = (0.05 * 17.67 * 7.7^2) * 0.52 = 27.24\ KN.m}$$

$$\boldsymbol{M_{b,neg} = (0.05 * 17.67 * 7.7^2) * 0.52 = 27.24\ KN.m}$$

-Positive moment

$C_{aD,pos} = 0.027$

$C_{bD,pos} = 0.027$

$C_{aL,pos} = 0.032$

$C_{bL,pos} = 0.032$

$$\boldsymbol{M_{a,pos,(dl+ll)} = (0.027 * 13.83 * 7.7^2 + 0.032 * 3.84 * 7.7^2) * 0.52 = 29.425\ KN.m}$$

$$\boldsymbol{M_{b,pos,(dl+ll)} = (0.027 * 13.83 * 7.7^2 + 0.032 * 3.84 * 7.7^2) * 0.52 = 29.425\ KN.m}$$

- **Short direction or long , It is the same:**

- **Mid span ($\boldsymbol{Mu = +\ 29.425\ KN.m}$)**

$$\boldsymbol{bf = 520\ mm}$$

Assume bar diameter ϕ14 for main positive reinforcement.

$$d = h - cover - dstirrups - \frac{d_b}{2} = 320 - 20 - 10 - \frac{14}{2} = 283\ mm.$$

$$R_n = \frac{M_u}{\emptyset b d^2} = \frac{29.425 \times 10^6}{0.9 \times 520 \times 283^2} = 0.785\ MPa$$

$$m = \frac{fy}{0.85\, fc'} = \frac{420}{0.85 * 24} = 20.6$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.785}{420}}\right) = 0.0019$$

$$As\ =\ \rho.b.d\ = 0.0019 \times 520 \times 283 = 280.57\ mm^2$$

- $Check for As, min..$

$$As, min\ = 0.25 \frac{\sqrt{f'_c}}{f_y} b_w * d \geq \frac{1.4}{f_y} b_w * d$$

$$As, min\ = 0.25 * \frac{\sqrt{24}}{420} * 120 \times 283 = 99.03\ mm^2$$

$$As, min\ = \frac{1.4}{420} * 120 \times 283 = 113.2\ mm^2 \ldots\ldots \boldsymbol{Control.}$$

- $As, required = 280.57\ mm^2 > As, min = 113.2\ mm^2$ (OK)

Use $2\emptyset 14$, bottom with As = 308 $mm^2 > As, required = 280.57\ mm^2$

Check for strain: ($\boldsymbol{\varepsilon_s \geq 0.005}$)

Tension = Compression

$$As * fy\ =\ 0.85 * f'_c * b * a$$

$$308 * 420\ =\ 0.85 * 24 * 520 * a$$

$$a\ = 12.19\ mm$$

$$x = \frac{a}{\beta_1} == \frac{12.19}{0.85} = 14.346\ mm$$

$$\varepsilon_s = 0.003 * (\frac{d - x}{x})$$

$= 0.003 * \left(\frac{283-14.346}{14.346}\right) = 0.056 > 0.005 \quad \therefore \phi = 0.9 \ldots OK.$

Design for Discontinuous edge 4.7.3

$$A_s = \frac{1}{3}A_{s,pos} = \frac{1}{3} * \ 308\, mm^2 = 102.67\, mm^2 < As, min$$

$$= 113.2\, mm^2$$

Provide $As, min = 113.2\, mm^2$

$$n = \frac{As}{As\phi 10} = \frac{113.2}{78.54} = 1.44$$

Use 2Ø 10 , Top .. with As = 157.08 mm^2

4.7.4 Design for continuous edge ($\boldsymbol{Mu = -27.24\ KN.m}$)

$$\boldsymbol{bw = 120mm}$$

$$d = h - cover - dstirrups - \frac{d_b}{2} = 320 - 20 - 10 - \frac{14}{2} = 283\, mm.$$

$$R_n = \frac{M_u}{\emptyset b d^2} = \frac{27.24 \times 10^6}{0.9 \times 120 \times 283^2} = 3.15\, MPa$$

$$m = \frac{fy}{0.85\, fc'} = \frac{420}{0.85 * 24} = 20.6$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2\times 20.6\times 3.15}{420}}\right) = 0.00819$$

$$As = \rho.b.d = 0.00819 \times 120 \times 283 = 278.13\, mm^2$$

- $Check for As, min..$

$$As,min\ = 0.25\frac{\sqrt{f'_c}}{f_y}b_w * d \geq \frac{1.4}{f_y}b_w * d$$

$$As,min\ = 0.25 * \frac{\sqrt{24}}{420}\ 120 \times 283 = 99.03\ mm^2$$

$$As,min\ =\ \frac{1.4}{420} * 120 \times 283 = 113.2\ mm^2 \ldots..\boldsymbol{Control}.$$

- $As,required = 278.13\ mm^2 > As,min = 113.2\ mm^2 \quad (OK)$

Use 2Ø14, Top .with As=308 mm^2 $>As,required = 278.13\ mm^2$

Check for strain: ($\boldsymbol{\varepsilon_s \geq 0.005}$)

Tension = Compression

$$As * fy\ =\ 0.85\ *\ f'_c\ *\ b\ *\ a$$

$$308 *\ 420\ =\ 0.85\ *\ 24\ *\ 120 *\ a$$

$a\ = 52.84\ mm$

$$x = \frac{a}{\beta_1} = \frac{52.84}{0.85} = 62.17\ mm$$

$$\varepsilon_s = 0.003 * (\frac{d - x}{x})$$

$$= 0.003 * \left(\frac{283-62.17}{62.17}\right) = 0.01\ > 0.005\ \ \therefore \phi\ = 0.9\ \ldots\ OK.$$

4.7.5 Check shear strength:

$W_a\ = 0.5$

$W_b\ = 0.5$

Short direction or long…the same

$$Au_a =\ 17.67 * 7.7 * 7.7 * 0.5 * 0.5 * \frac{0.52}{7.7} =\ 17.688\ KN$$

$$Vu = Au_a - W * 0.52 * d = 17.688 - 17.67 * 0.52 * 283 = 15.087\ KN$$

$$\emptyset * V_c = 1.1 * \frac{0.75}{6} * \sqrt{fc'} * bw * d = 1.1 * \frac{0.75}{6} * \sqrt{24} * 120 * 283$$
$$= 22.87\ KN$$

$$Case\ 1$$

$$V_u < \frac{1}{2} * \phi * V_c$$

$$V_u = 15.87 > \frac{1}{2} * \phi * V_c = 11.435\ KN\ \ldots\ \boldsymbol{Not\ OK}$$

$$Case\ 2$$

$$\frac{1}{2} * \phi * V_c < V_u < \phi * V_c$$

$$\frac{1}{2} * \phi * V_c = 11.435 < V_u = 15.087\ KN < \phi * V_c = 22.87\ KN - \boldsymbol{OK}$$

∴Case (2) is satisfy minimum shear reinforcement ..

$$A_{V\,min} = \frac{\sqrt{fc'}}{16(fy)}(bw)(S) \geq \frac{1}{3(fy)}(bw)(S)$$

$$157 = \frac{\sqrt{24}}{16(420)}(120)(S)$$

S=1794.6 mm.

Use 2 Legϕ10 for stirrups with Av = 157 mm2.

$S_{max} \leq \frac{d}{2} = \frac{283}{2} = 141.5$ mm.

≤ 600 mm.

Select 2 leg **ϕ**10 @12cm.

4.8 Design of one-way solid slab:

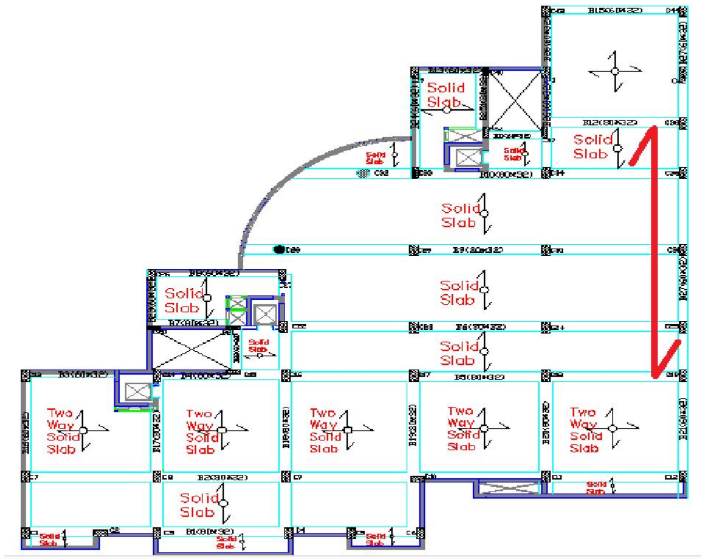

Figure 4.12 one way solid slab.

4.8.1 Load calculations:

Table (4.4) calculation loads for one way solid slab.

Material	γ **×h×1**	**Load** (KN/m)
Tile	**0.03×24.5×1**	**0.735**
mortar	**0.03×22×1**	**0.66**
sand	**0.15×17×1**	**2.55**
plaster	**0.02×22×1**	**0.44**
Partition	**1.25×1**	**1.25**
Self weight	**0.25×24.5×1**	**6.125**
$\sum$		**11.76**

Dead Load = 11.76 KN/m.
Live load = 5×1= 5 KN/m.

By using ATIR program use b = 100 cm.

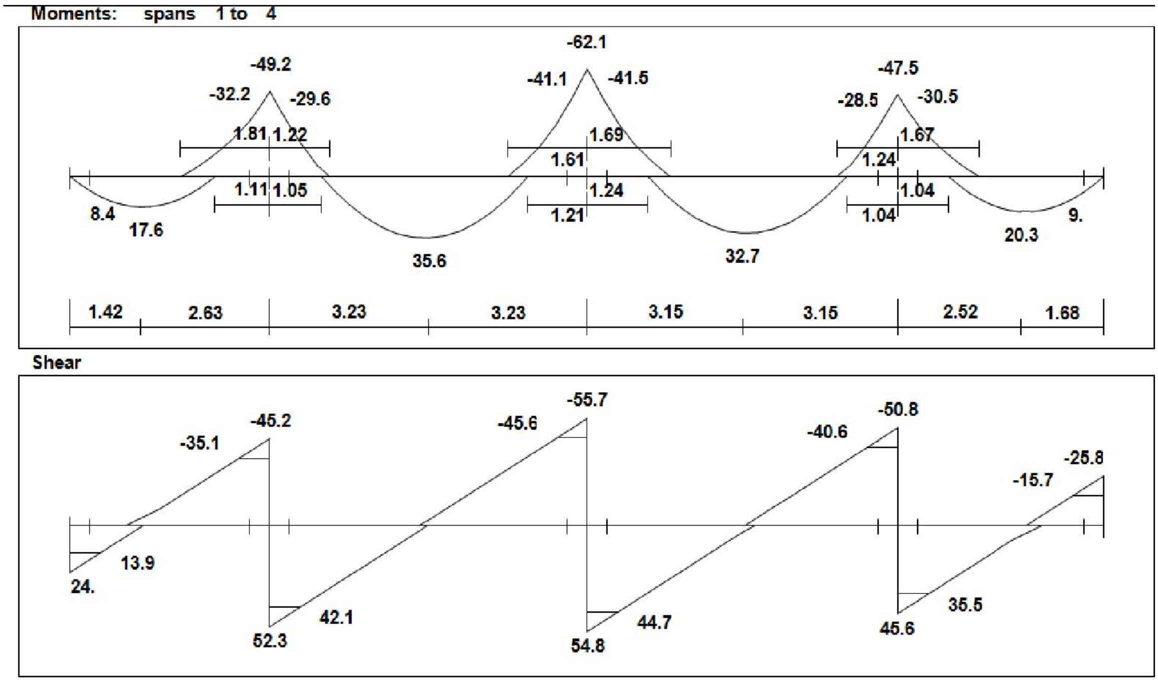

Figure (4.13) envelop moment and shear diagrams of solid slab.

4.8.2 Check whether the thickness of the slab is adequate for shear:

d = 250-20-(16/2) = 222 mm.

$V_{u,max}$ = 45.6 KN.

$V_c = \frac{1}{6}\lambda\sqrt{f_c'}b_w\, d = \frac{1}{6}\sqrt{24} \times 1000 \times 222 \times 10^{-3} = 181.26\ \frac{KN}{1m\ strip}$

$ø\ V_c = 0.75 \times 181.26 = 135.9 \frac{KN}{1m\ strip}$

$V_{u,max} < ø\ V_c$

No need to increase the slab thickness, its adequate enough.

4.8.3 Design for positive moment:

For M_u =+17.6 KN.m

$R_n = \frac{M_u}{\varnothing b d^2}$

$Rn = \frac{17.6 \times 10^6}{0.9 \times 1000 \times 222^2} = 0.397\ Mpa$

$m = \frac{f_y}{0.85 f_c'}$

$m = \frac{420}{0.85 \times 24} = 20.59$

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right)$

$\rho = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2\times 20.59\times 0.397}{420}}\right) = 0.00095$

$A_s = \rho.b.d = 0.00095 \times 1000 \times 222 = 212\ mm2$

$A_{s,min} = 0.0018\times b \times h = 0.0018\times 1000 \times 250 = 450\ mm^2$ …cont.

$n_{\phi 16} = \frac{450}{201} = \frac{3\phi 16}{m\ strip}$, ϕ16@300.

Step (S) is the smallest of:

- $3\times h = 3\times 250 = 750mm$.
- 450mm.
- $S = 380\left(\frac{280}{\frac{2}{3}f_y}\right) - 2.5C_c = 330mm$

  $S \leq 300\left(\frac{280}{\frac{2}{3}f_y}\right) = 300mm$ $\qquad cont.$

S=300mm Ok

For M_u =+35.6 KN.m

$R_n = \frac{M_u}{\emptyset b d^2}$

$Rn = \frac{35.6\times 10^6}{0.9\times 1000\times 222^2} = 0.8\ Mpa$

$m = \frac{f_y}{0.85 f_c'}$

$m = \frac{420}{0.85\times 24} = 20.59$

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right)$

$\rho = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2\times 20.59\times 0.8}{420}}\right) = 0.00195$

$A_s = \rho.b.d = 0.00195 \times 1000 \times 222 = 432.92\ mm2$

$A_{s,min} = 0.0018\times b \times h = 0.0018\times 1000 \times 250 = 450\ mm^2$ …cont.

$n_{\phi 16} = \frac{450}{201} = \frac{3\phi 16}{m\ strip}$, ϕ16@300.

Step (S) is the smallest of:

- $3\times h = 3\times 250 = 750mm$.
- 450mm.

- S= $380\left(\frac{280}{\frac{2}{3}f_y}\right) - 2.5C_c$ = 330mm

  S≤$300\left(\frac{280}{\frac{2}{3}f_y}\right) = 300mm \qquad cont.$

S=300mm

Ok

The other spans has a smaller values of moment , so let the reinforcement be as the previous spans: ϕ16@300.

4.8.4 Design for negative moment:

For M_u = -32.2 $\boldsymbol{KN.m}$

It will be smaller than the minimum As , so take ϕ16@300.

For M_u = - 41.5$\boldsymbol{KN.m}$

$R_n = \frac{M_u}{\emptyset b d^2}$

$$Rn = \frac{41.5\times10^6}{0.9\times1000\times222^2} = 0.935\ Mpa$$

m= $\frac{f_y}{0.85f_c'}$

m= $\frac{420}{0.85\times24} = 20.59$

ρ= $\frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right)$

ρ= $\frac{1}{20.59}\left(1-\sqrt{1-\frac{2\times20.59\times0.935}{420}}\right) = 0.00227$

A_s = ρ.b.d =$0.00227 \times 1000 \times 222 = 506.1\ mm2$ …cont.

$A_{s,min}$ = 0.0018$\times\ b \times h$ =0.0018$\times\ 1000 \times 250 = 450$ mm^2 .

$n_{\phi16} = \frac{506.1}{201} = \frac{3\phi16}{m\ strip}$, ϕ16@300.

Step (S) is the smallest of:

- 3×h =3×250 =750mm.
- 450mm.
- S= $380\left(\frac{280}{\frac{2}{3}f_y}\right) - 2.5C_c$ = 330mm

$$S \le 300\left(\frac{280}{\frac{2}{3}f_y}\right) = 300mm \qquad cont.$$

S=300mm

For M_u = - 30.5$KN.m$
It will be smaller than the minimum As , so take ϕ16@300.

4.8.5 Temperature and shrinkage reinforcement:

$A_{s,min} = 450mm^2$

$n_{\phi12} = \frac{450}{113.1} = \frac{4\phi12}{m\ strip}$, ϕ12@250mm.

Step (S) is the smallest of:

- 5×h =5×250 =1250mm.
- 450 mm…..cont.

S = 250mm < 450mm. ok.

4.9 Design of two way solid slab (R05)

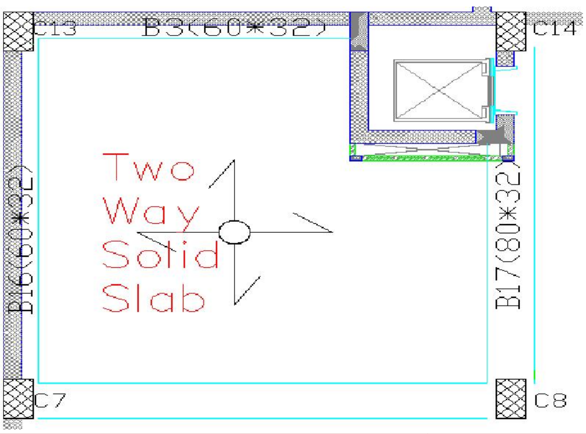

Figure (4.14) two way solid slab (R05).

4.9.1 Load calculation:

For the two-way solid slabs, the total dead load to be used in the analysis and design is calculated as follows:

Table (4.5) calculation loads for two way solid slab.

Material	**γ ×h×1**	**Load** (KN/m)
Tile	**0.03×24.5×1**	**0.735**
mortar	**0.03×22×1**	**0.66**
sand	**0.15×17×1**	**2.55**
plaster	**0.02×22×1**	**0.44**
Partition	**1.25×1**	**1.25**
Self weight	**0.25×24.5×1**	**6.125**
∑		**11.76**

Dead Load = 11.76 KN/m.
Live load = 2.4×1= 2.4 KN/m.

$$w_D = 1.2 * 11.76 = 14.1\ KN/m^2$$

$$LL = 5\ KN/m^2$$

$$w_L = 1.6 * 5 = 8\ KN/m^2$$

$$w = 14.1 + 8 = 22.1\ KN/m^2$$

4.9.2 Moments calculations:

m = la/lb = 7.7/7.7 = 1 with case 4.

$Ma = Cawla^2bf \ and \ Mb = Cbwlb^2bf$

-Negative moment

$C_{a,neg} = 0.05$

$C_{b,neg} = 0.05$

$$\boldsymbol{M_{a,neg} = (0.05 * 22.1 * 7.7^2) = 65.5\ KN.m}$$

$$\boldsymbol{M_{b,neg} = (0.05 * 22.1 * 7.7^2) = 65.5\ KN.m}$$

-Positive moment

$C_{aD,pos} = 0.027$

$C_{bD,pos} = 0.027$

$C_{aL,pos} = 0.032$

$C_{bL,pos} = 0.032$

$$\boldsymbol{M_{a,pos,(dl+ll)} = (0.027 * 14.1 * 7.7^2 + 0.032 * 8 * 7.7^2) = 37.8\ KN.m}$$

$$\boldsymbol{M_{b,pos,(dl+ll)} = (0.027 * 14.1 * 7.7^2 + 0.032 * 8 * 7.7^2) = 37.8\ KN.m}$$

Design of positive moment

- **Short direction ($\boldsymbol{Mu = 37.8\ \ KN.m}$)**

$$\boldsymbol{b = 1000\ mm}$$

Assume bar diameter ϕ20 for main positive reinforcement.

$d \ = h - cover - db/2 = 250 - 20 - 8 = 222\ mm.$

$$R_n = \frac{M_u}{\emptyset b d^2} = \frac{37.8 \times 10^6}{0.9 \times 1000 \times 222^2} = 0.87\ MPa$$

$$m = \frac{fy}{0.85\, fc'} = \frac{420}{0.85 * 24} = 20.6$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.87}{420}}\right) = 0.0024$$

$$As\ = \rho.b.d\ = 0.0024 \times 1000 \times 222 = 532.8\ mm^2$$

$$As, min\ = 0.0018*1000*250 = 450\ mm^2$$

- $As, required > As, min \quad (OK)$

Use 3Ø16, with As = 602.88 $mm^2 > As, required.$

Check for strain: ($\varepsilon_s \geq 0.005$)

Tension = Compression

$$As * fy = 0.85 * f_c' * b * a$$

$$602.88 * 420 = 0.85 * 24 * 1000 * a$$

$$a = 12.19\ mm$$

$$x = \frac{a}{\beta_1} == \frac{12.19}{0.85} = 14.346\ mm$$

$$\varepsilon_s = 0.003 * (\frac{d - x}{x})$$

$$= 0.003 * \left(\frac{222 - 14.346}{14.346}\right) = 0.043 > 0.005 \quad \therefore \phi = 0.9\ \ldots\ OK.$$

Design for Discontinuous edge

$$A_s = \frac{1}{3} A_{s,pos} = \frac{1}{3} * \ 602.88 = 200.96\ mm^2 < As, min.$$

Provide $As, min = 450\ mm^2$

$$n = \frac{As}{As\phi 14} = \frac{450}{153.86} = 3$$

Use $3Ø\ 14$, Top .. with $As = 461.58\ mm^2$

- **Long direction ($Mu = 37.8 \ KN.m$)**

$$b = 1000 \, mm$$

It will be the same as the previous direction :

Use $3Ø16$, for bottom.

Design for Discontinuous edge

Use 3Ø 14 , Top

Design of negative moment ($Mu = 65.5 \ KN.m$)

$$b = 1000 \, mm$$

Assume bar diameter ф20 for main positive reinforcement.

$$d \ = h - cover - \frac{d_b}{2} = 250 - 20 - \frac{20}{2} = 220 \, mm.$$

$$R_n = \frac{M_u}{Øbd^2} = \frac{65.5 \times 10^6}{0.9 \times 1000 \times 220^2} = 1.5 \, MPa$$

$$m = \frac{fy}{0.85 \, fc'} = \frac{420}{0.85 * 24} = 20.6$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2\times20.6\times1.5}{420}}\right) = 0.0037$$

$$As \ = \ \rho.b.d \ = 0.0037 \times 1000 \times 220 = 814 \, mm^2 > As,min$$

..

Use 3Ø20,with As = $942 \ mm^2$ $>As, required = 814 \, mm^2$

Check for strain: ($\varepsilon_s \geq 0.005$)

Tension = Compression

$$As \ * \ fy \ = \ 0.85 \ * \ f_c' \ * \ b \ * \ a$$

$$942 * 420 = 0.85 * 24 * 1000 * a$$

$a = 19.4\ mm$

$$x = \frac{a}{\beta_1} = \frac{19.4}{0.85} = 22.8\ mm$$

$\varepsilon_s = 0.003 * (\frac{d - x}{x})$

$= 0.003 * \left(\frac{220-22.8}{22.8}\right) = 0.026 > 0.005 \quad \therefore \phi = 0.9 \ \dots\ OK.$

4.10 Design of stairs.

4.10.1 thickness :

For Flight:

L = 5.55 m.

h_{req} = L/ 20.

h_{req} = 5.55 / 20 = 0.2775 m = 27.75 cm.

take h=32 cm.

For Landing:

L = 3.7 m.

h_{req} = L/ 20.

h_{req} = 3.7 / 20 = 0.185 cm.

Use h= 32 cm.

4.10.2 Load Calculations:

The stair slope by θ = $\tan^{-1}\left(\frac{166}{300}\right) = 28.96$

For Flight :

Dead Load for flight:

$$Tiles = 24.5\left(\frac{0.166 + 0.35}{0.3}\right) * 0.03 * 1 = 1.26\ KN/m$$

$$Mortar = 22\left(\frac{0.166 + 0.3}{0.3}\right) * 0.03 * 1 = 1.025\ KN/m$$

$$stair\ stips = \frac{24.5}{0.3}\left(\frac{0.166 * 0.30}{2}\right) * 1 = 2.03\ KN/m$$
$$slab = \left(\frac{24.5 * 0.32 * 1}{\cos 28.96}\right) = 8.96\ KN/m$$
$$Plaster = 22\left(\frac{0.02 * 1}{\cos 28.96}\right) = 0.502\ KN/m$$

Total Dead Load = 13.777 KN/m.
Live Load = 2.4× 1 = 2.4 KN/m.
Total Load For Flight = 1.2× 13.777 + 1.6× 2.4 = 20.37 KN/m.

For Landing :

Dead Load For Landing:
Tiles = 24.5× 0.03× 1 = 0.735 KN/m
Mortar = 22× 0.03× 1 =0.66 KN/m
Slab = 24.5× 0.32× 1= 7.84 KN/m
Plaster = 22*0.02*1= 0.44 KN/m

Total dead load for landing = 9.675 KN/m.
Live Load = 2.4× 1 = 5 KN /m.
Total Dead Load For landing = 1.2× 9.675 + 1.6× 2.4= 14.49 KN/m.

4.10.3 Design for flight:

By using ATIR program use b = 100 cm.

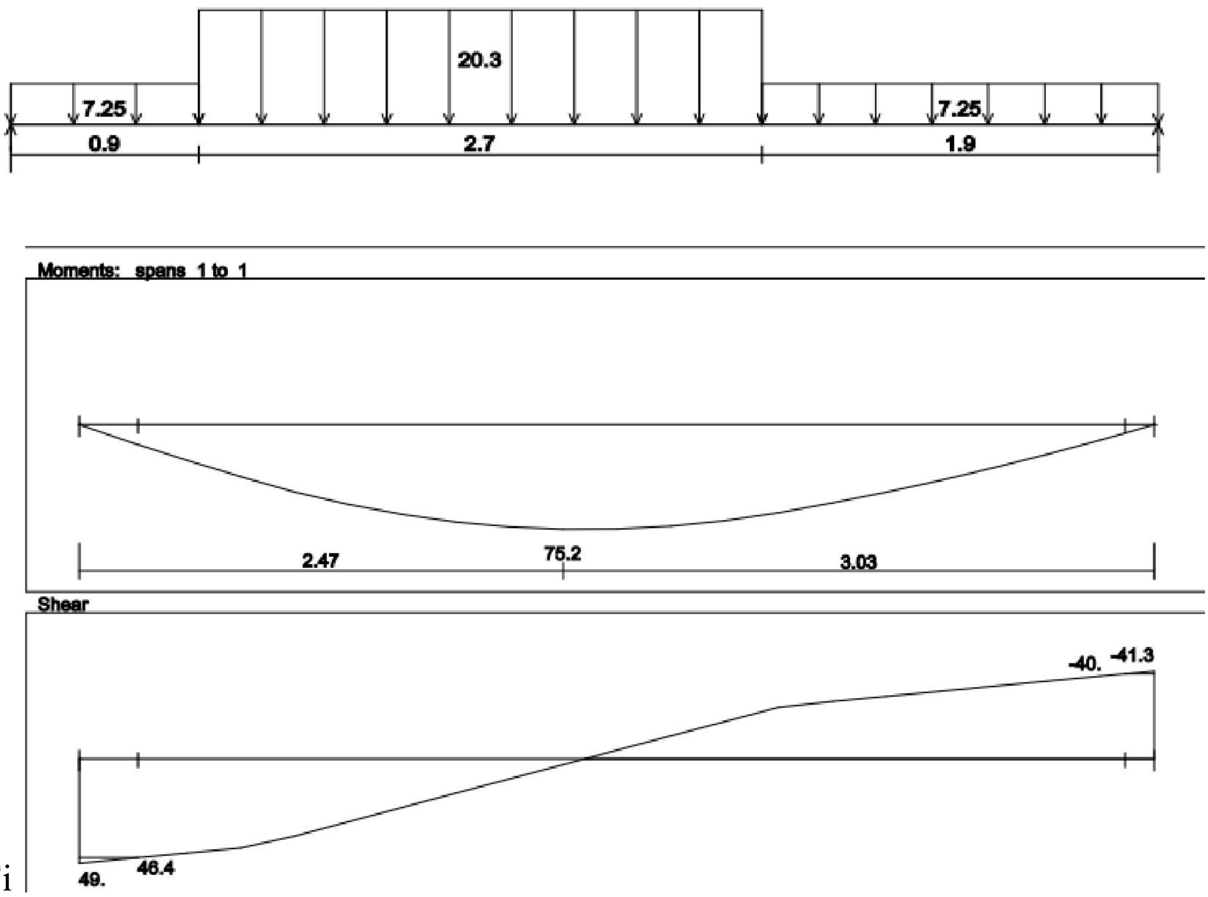

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Figure (4.15) loads on flight with shear and moment .

Design of Shear for flight:

- Assume Ø 14 for main reinforcement:

So, d =320 – 20 - (14/2) = 293 mm..

- Vu = 49 KN .

- $\phi Vc = \frac{\phi\sqrt{f_c'} * b_w * d}{6}$
- $\phi Vc = \frac{0.75 * \sqrt{24} * 1000 * 293 * 10^{-3}}{6} = 179.4KN$
- Vu = 46.4 KN < Ø.Vc = 179.4 KN. .

Depth of flight is ok. Since, there is no shear Reinforcement .

Design of Bending Moment for Flight:

Mu =75.2 KN.m.
Mn req = Mu / 0.9 = 75.2/ 0.9 = 83.56 KN.m .
Assume bar diameter 14 for main reinforcement.
d = 320 – 20 – (14/2) = 293 mm.

$$R_n = \frac{Mn}{b \cdot d^2}$$

$$R_n = \frac{83.56 * 10^6}{1000 * (293)^2} = 0.98MPa \text{ .}$$

$$m = \frac{fy}{0.85 \times fc'}$$

$$m = \frac{420}{0.85 \times 24} = 20.588$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mR_n}{f_y}}\right) = \frac{1}{20.588}\left(1 - \sqrt{1 - \frac{2 * 20.588 * 0.98}{420}}\right) = 0.0024$$

As req = 0.0024 $\times$ 1000 $\times$ 293= 700.92 mm^2……….control
$A_s$min=0.0018 $\times$ 1000 $\times$ 320= 576 mm^2
S=153.9/700.92 =0.219m.
Use 5 Φ 14 /m OR use Φ14 @ 20 cm.

- **Check for spacing:**

3h=3 $\times$ 320=960 mm.
S=450.

$$s = 380\left(\frac{280}{0.667 * 420}\right) - 2.5 * 20 = 330mm$$

$s = 300\left(\frac{280}{0.667*420}\right) = 300mm$…… control.
Use 1Φ 14@ 20 cm.

- **Secondary Reinforcement:**

For shrinkage & Tempreture As provide equal :
As min = 0.0018 × B × h = 0.0018× 1000× 320 = 576 mm^2
Use 4 Φ14 /m or 1 Φ 14 @ 25 cm.

- **Check for strain:**

Tension = Compression

$$A_s * fy = 0.85 * fc^{\backslash} * b * a$$

$$769.5 * 420 = 0.85 * 24 * 1000 * a$$

$$a = 15.84\,mm$$

$$c = \frac{a}{\beta_1} = \frac{15.84}{0.85} = 18.63\,mm$$

$$\varepsilon_s = \frac{293 - 18.63}{18.63} * 0.003$$

$$\varepsilon_s = 0.044 > 0.005 \longrightarrow ok$$

4.10.4 Design of landing:
same thickness =32 cm

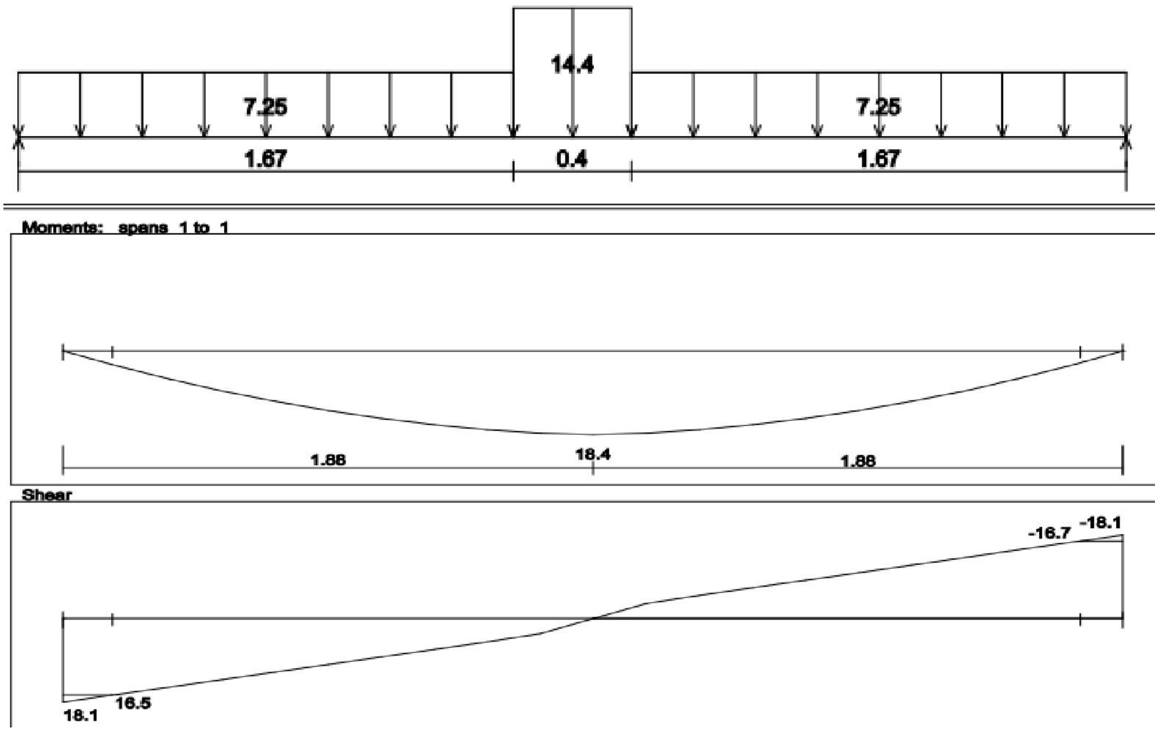


Figure (4.16) loads on landing with shear and moment.

Design of Shear for Landing:

- Vu = 16.7 KN.
- $\phi Vc = \frac{\phi\sqrt{f_c'} * b_w * d}{6}$
- $\phi Vc = \frac{0.75 * \sqrt{24} * 1000 * 293 * 10^{-3}}{6} = 179.42$ KN
- Vu = 16.7 KN < Ø.Vc = 179.4 KN.

Depth of flight is ok. Since, there is no shear Reinforcement.

4.10.5 Design of bending moment for landing:

Mu = 18.4 KN.m.
Mn req = 18.4 / 0.9 = 20.44 KN.m.
Assume diameter bar 14 for main reinforcement . because the bar in the landing will be placed on the top of the main reinforcement .
d= 320 – 20 – 14 – (14/2) = 279 mm.

$$R_n = \frac{Mn}{b \cdot d^2}$$

$$Rn = \frac{20.44 * 10^6}{1000 * (279)^2} = 0.262 MPa \ .$$

$$m = \frac{fy}{0.85 \times fc'}$$

$$m = \frac{420}{0.85 \times 24} = 20.588$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mR_n}{f_y}}\right) = \frac{1}{20.588}\left(1 - \sqrt{1 - \frac{2 * 20.588 * 0.262}{420}}\right) = 0.00063$$

As req = 0.00063 × 1000 × 279= 175.77 mm^2.
As min= 0.0018 × 1000 × 320 =576 mm^2… control
As < As min ..

Use 4 Φ14 /m or 1 Φ 14 @ 25 cm with As = 615.6 mm^2.

- **Check for spacing:**

3h=3*320=960mm.
S=450

$$s = 380\left(\frac{280}{0.667 * 420}\right) - 2.5 * 20 = 330mm$$

$s = 300\left(\frac{280}{0.667*420}\right) = 300mm$…… control .

Use 1Φ 14@ 25 cm.

- **Secondary Reinforcement :**

For shrinkage & Tempreture As provide equal :

As min = 0.0018 × B × h = 0.0018×1000×320 = 576 mm^2

Use 4 Φ14 /m or 1 Φ 14 @ 25 cm.

- **Check for strain:**

Tension = Compression

$$A_s * fy = 0.85 * fc^{\backslash} * b * a$$

$$615.6 * 420 = 0.85 * 24 * 1000 * a$$

$$a = 12.67\,mm$$

$$c = \frac{a}{\beta_1} = \frac{12.67}{0.85} = 14.91\,mm$$

$$\varepsilon_s = \frac{259 - 14.91}{14.91}) * 0.003$$

$$\varepsilon_s = 0.049 > 0.005 \longrightarrow ok$$

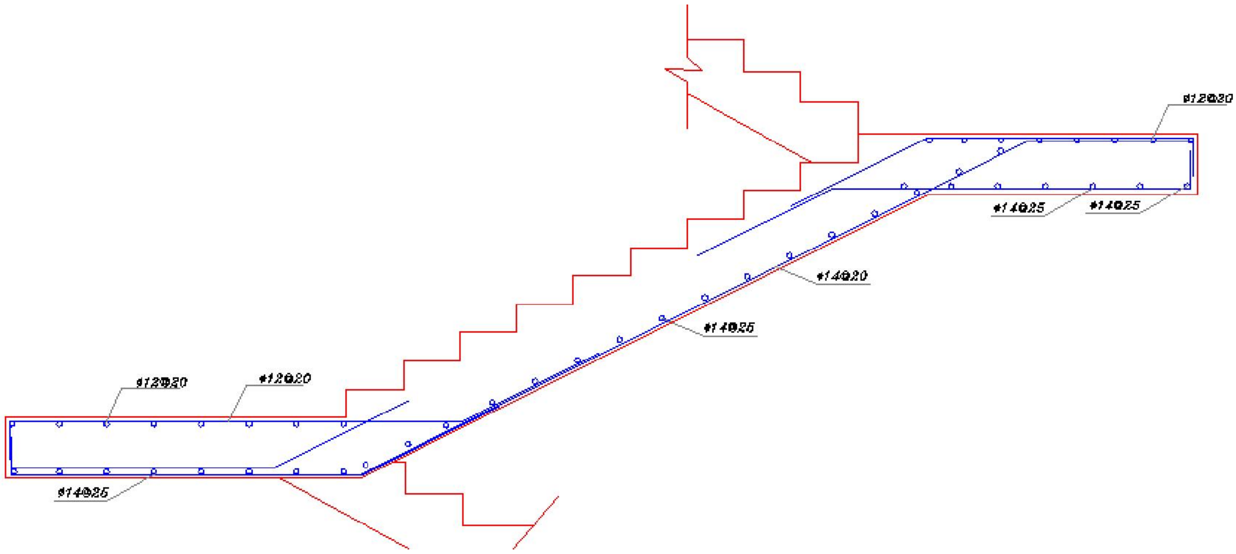


Figure (4.17) Reinforcement of stair.

4.11 Design of Beam (B6) in the ground floor slab:

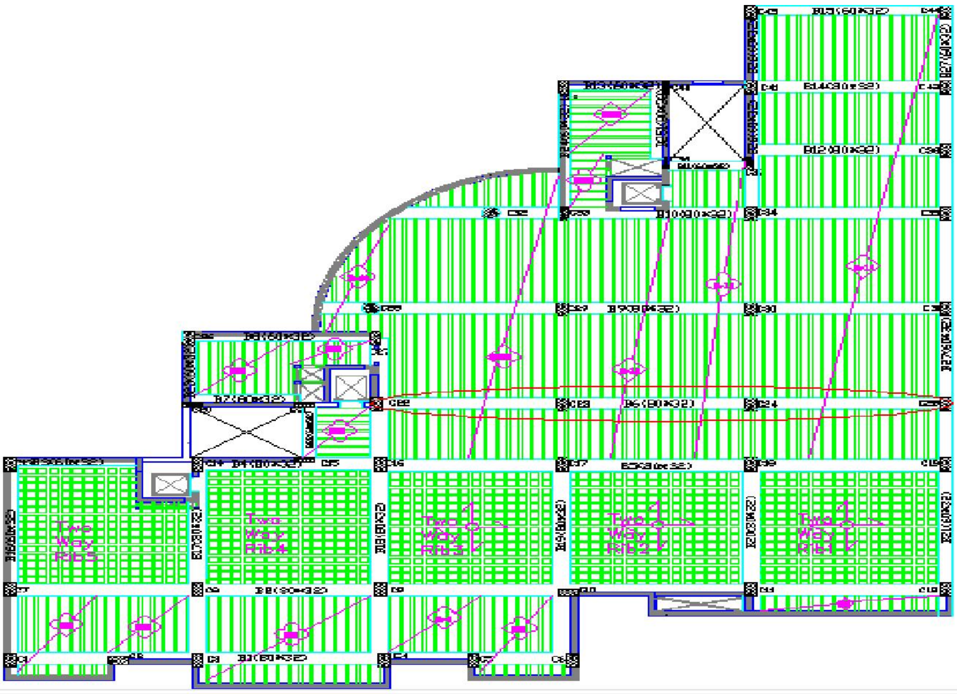
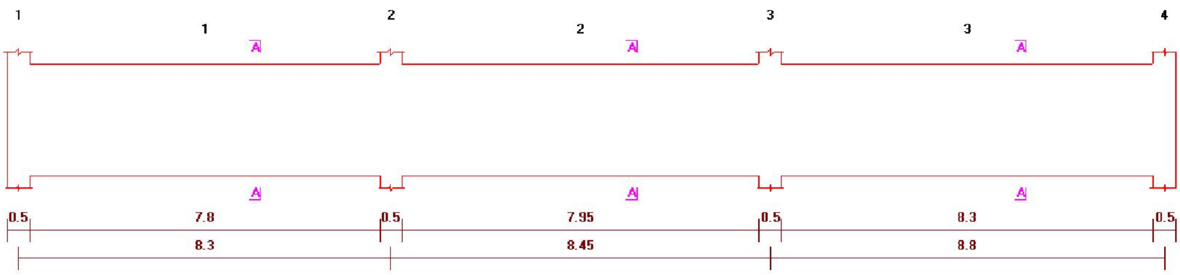


Figure (4.18) location of beam (B6).



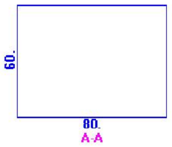


Figure (4-19) Designed Beam (B6).

4.11.1 Introduction

The envelope for both shear and moment diagrams drawn using ATIR program as shown below:

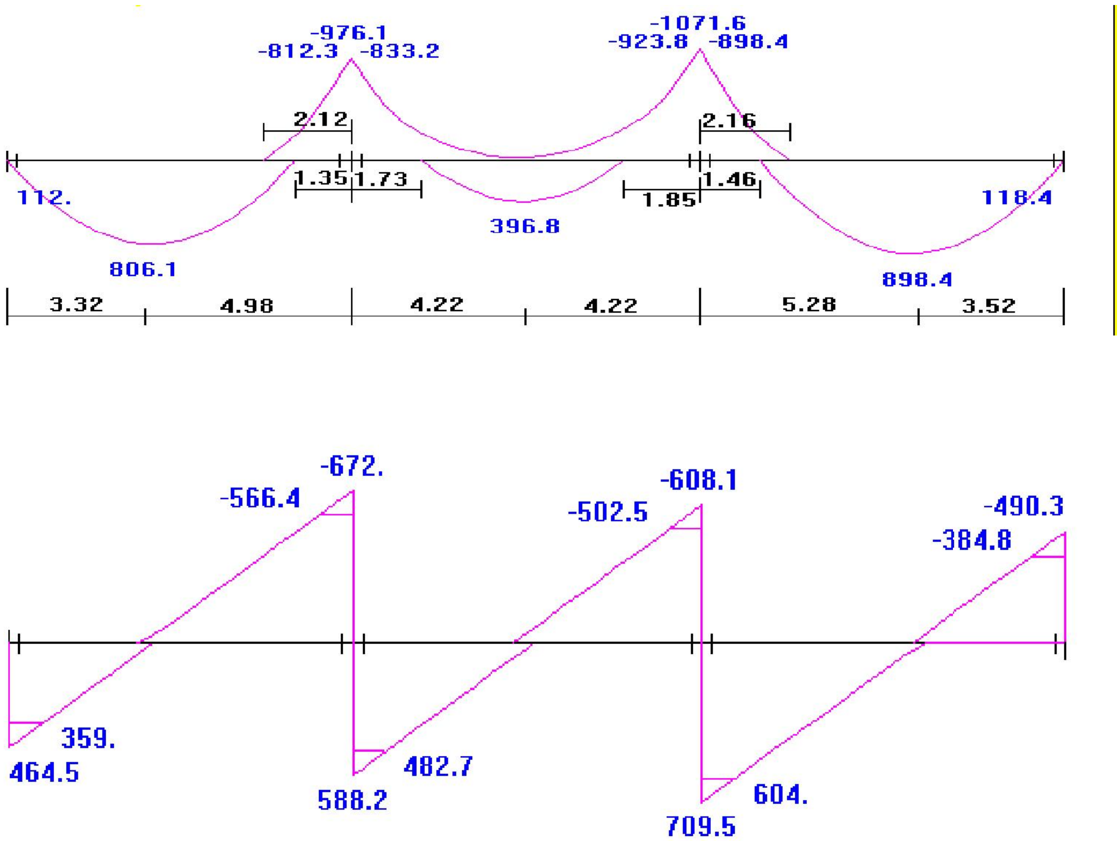


Figure (4-20) shear and moments diagrams for Beam (B6).

$d = 600 - 40 - 10 - \frac{25}{2}$

$d = 537.5\ mm$

$C_{MAX} = \frac{3}{7}$ d

$C_{MAX} = 230.35\ mm$

a = 0.85 * 230.35 = 195.79

$fc' \leq 28 \rightarrow \beta = 0.85$

4.11.2 Design of Positive moment of Beam (6):

Design of $+MU_{MAX} = \mathbf{89\ 84\ KN.m}$

Check if the section can be designed as a singly reinforced section or must be as a doubly reinforced section

$Mn_{MAX} = 0.85 \times fc' \times b \times a \times (d - a/2)$

$Mn_{MAX} = 0.85 \times 24 \times 0.8 \times 0.196 \times (0.5375 - 0.196/2)$

$Mn_{MAX} = 1405.84$ kN.m

$\Phi = 0.65 + (\epsilon_s - 0.002)\frac{250}{3}$

$\Phi = 0.65 + (0.004 - 0.002)\frac{250}{3} = 0.82$

$Mn_{MAX} = \frac{M_u}{\Phi}$

$\Phi Mn_{MAX} = 0.82 \times 1405.84 = 1152.78\ \text{kN.m} > \boldsymbol{MU}$

Design the section as a singly reinforced section

$Rn = \frac{M_u/\Phi}{b \times d^2}$

$Rn = \frac{898.4 \times 10^{-3}/0.9}{0.8 \times (0.5375)^2} = 4.32\ MPa$

$$\rho_{req} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mRn}{fy}}\right)$$

$$m = \frac{fy}{0.85 \times fc'}$$

$$m = \frac{420}{0.85 \times 24} = 20.6$$

$$\rho_{req} = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 4.32}{420}}\right) = 0.0117$$

$$As_{req} = \rho \times b_E \times d$$

$$As_{req} = 0.0117 \times 800 \times 537.5 = 5038\, mm^2$$

Check for As_{min}

$$As_{min} = 0.25\frac{\sqrt{fc'} \times b_w \times d}{fy}$$

$$As_{min} = 0.25\frac{\sqrt{24} \times 800 \times 537.5}{420} = 1253.9\, mm^2$$

Not less than

$$As_{min} = \frac{1.4 \times 800 \times 537.5}{420} = 1433.3\, mm^2 \quad \Rightarrow Controls$$

$$As_{req} = 5038\, mm^2 > \ As_{min} = 1433.3\, mm^2$$

selectAs $11\Phi25 = 5399.6\ mm^2 > \ As_{req} = 5038\, mm^2$

According to Atir Program the limitation of deflection is satisfied and so, no additional reinforcement is required.

-Check of Strain

Tension=Compression

T=C

$fy \times As = 0.85 \times fc' \times \mathrm{b} \times \mathrm{a}$

$420 \times 5399.6 = 0.85 \times 24 \times 800 \times \mathrm{a}$

a= $138.9\ mm$

$$c = \frac{a}{0.85} = 163.48mm$$

$$\frac{\epsilon_s}{d - c} = \frac{0.003}{c}$$

$$\epsilon_s = \frac{537.5 - 163.48}{163.48} \times 0.003$$

$$\epsilon_s = 0.0068 > 0.005$$

Design of + Mu = 396.8 kN.m

$select\ 5\phi25\ with\ As = 2454.36\ \ mm^2 > As = 2058.1\ \ mm^2$

Design of + Mu =145.5 kN.m

$select\ 10\phi25\ with\ As = 4908.7\ \ mm^2 > As = 4449\ mm^2$

4.11.3 Design of Negative moment of Beam (6):

Design of $\_MU_{MAX} = 9\ 23\d{3}8$ KN.m

Check if the section can be designed as a singly reinforced section or must be as a doubly reinforced section

$$Mn_{MAX} = 0.85 \times fc' \times b \times a \times (d - a/2)$$

$$Mn_{MAX} = 0.85 \times 24 \times 0.8 \times 0.196 \times (0.5375 - 0.196/2)$$

$$Mn_{MAX} = 1405.83\ \ \text{kN.m}$$

$$\Phi = 0.65 + (\epsilon_s - 0.002)\frac{250}{3}$$

$$\Phi = 0.65 + (0.004 - 0.002)\frac{250}{3} = 0.82$$

$$Mn_{MAX} = \frac{M_u}{\Phi}$$

$$\Phi Mn_{MAX} = 0.82 \times 1405.83 = 1152.78 \text{ kN.m} > \boldsymbol{MU}$$

Design the section as a singly reinforced section:

$$Rn = \frac{M_u/\Phi}{b \times d^2}$$

$$Rn = \frac{923.8 \times 10^{-3}/0.9}{0.8 \times (0.5375)^2} = 4.45 \;\; MPa$$

$$\rho_{req} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mRn}{fy}}\right)$$

$$m = \frac{fy}{0.85 \times \; fc'}$$

$$m = \frac{420}{0.85 \times 24} = 20.6$$

$$\rho_{req} = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 4.45}{420}}\right) = 0.012$$

$$As_{req} = \rho \times b_E \times d$$

$$As_{req} = 0.012 \times 800 \times 537.5 = 5203.9 \, mm^2$$

Check for $\boldsymbol{As_{min}}$

$$As_{min} = 0.25\frac{\sqrt{fc'} \times b_w \times d}{fy}$$

$$As_{min} = 0.25\frac{\sqrt{24} \times 800 \times 537.5}{420} = 1253.9 \;\; mm^2$$

Not less than

$$As_{min} = \frac{1.4 \times 800 \times 537.5}{420} = 1433.3\, mm^2 \quad \Rightarrow Controls$$

$As_{req} = 5203.9\ mm^2 > \ As_{min} = 1433.3\, mm^2$

selectAs $11\Phi25 = 5399.6\ mm^2$

According to Atir Program the limitation of deflection is satisfied and so, no additional reinforcement is required.

-Check of Strain

Tension=Compression

T=C

$fy \times As = 0.85 \times fc' \times \text{b} \times \text{a}$

$420 \times 5399.6 = 0.85 \times 24 \times 800 \times \text{a}$

a= $138.9\, mm$

$$c = \frac{a}{0.85} = 163.4\, mm$$

$$\frac{\epsilon_s}{d - c} = \frac{0.003}{c}$$

$$\epsilon_s = \frac{537.5 - 163.4}{163.4} \times 0.003$$

$\epsilon_s = 0.0082\ > 0.005$

Design of - Mu = 833.2 kN.m

$select\ 10\phi25\ with\ As = 4908.7\, mm^2 > As = 4619.7\ mm^2$

4.11.4 Design of Shear of Beam (6):

4.11.4.1 Design of Shear of Beam(6) for the right Span :

$Vu_{max} = 604$ kN.

$$\Phi Vc = \frac{\Phi}{6} \times \sqrt{fc'} \times bw \times d$$

$$\Phi Vc = \frac{0.75}{6} \times \sqrt{24} \times 800 \times 537.5 = 263.32 \ kN$$

$$\Phi Vs_{min} = \Phi \frac{1}{3} \times bw \times d$$

$$\Phi Vs_{min} = 0.75 \times \frac{1}{3} \times 800 \times 537.5 = 107.5 \text{ kN} \Longrightarrow\Longrightarrow Control$$

$$\Phi Vs_{min} = \Phi \frac{1}{16} \sqrt{fc'} \times bw \times d$$

$$\Phi Vs_{min} = \frac{\Phi}{16} \sqrt{24} \times 800 \times 537.5 = 98.74 \text{ kN}$$

$$\Phi Vc + \Phi Vs_{min} < \text{Vu} < \Phi Vc + \frac{\Phi}{3} \sqrt{fc'} \ bw \times d$$

$$263.32 + 107.5 < 604 < 263.32 + \frac{0.75}{3} \sqrt{24} \times 800 \times 537.5$$

$$370.82 < 604 < 626.9$$

$Category\ No.\ 4\ Is\ Satisfied$

$$Vs = \frac{Vu}{\Phi} - Vc$$

$$Vs = \frac{604}{0.75} - 351.09 = 454.24\ kN$$

$$\left(\frac{\text{Av}}{s}\right)_{min} = \frac{bw}{3 \times fy} = \frac{800}{3 \times 420} = 0.63 \times 10^{-3} \Longrightarrow\Longrightarrow Control$$

$$\geq \frac{bw \times \sqrt{fc'}}{16 \times fy} = \frac{800 \times \sqrt{24}}{16 \times 420} = 10 \times 10^{-3}$$

$$\frac{\text{Av}}{s} = \frac{Vs}{d \times fy} = \frac{0.454}{420 \times 0{,}5375} = 0.002$$

$$\frac{\text{Av}}{s} = \frac{4 \times 79 \times 10^{-6}}{s} = 0.63 \times 10^{-3}$$

$$s = 50\ cm$$

$$\text{s} \leq \frac{d}{2} = \frac{537.5}{2} = 27cm$$

$\text{s} \leq 60\ cm\ , select\ s = 20cm\ \Longrightarrow Use\ \Phi 10 - 4\ \text{Leg} - \text{Stirrup at } 20\text{cm}.$

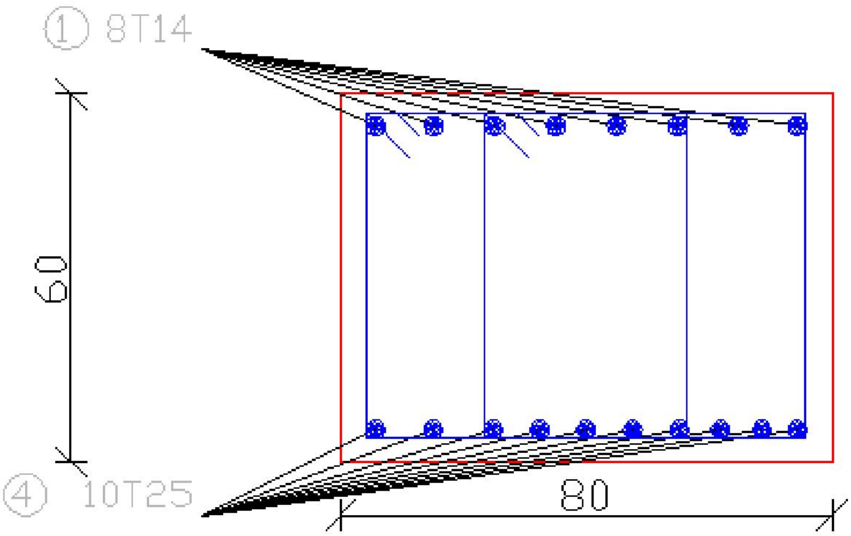


Figure (4.21) Reinforcement of beam (B6).

4.12 Design of Short Column (C3):

C3 in the fourth basement of the building.

4.12.1 Load Calculation:

$P_u = 6880$ KN .

$\phi\ P_n = P_u$

4.12.2 Check Slenderness Effect:

In both direction:

$$\frac{Klu}{r} \leq 34 - 12 \times \frac{M1}{M2} \qquad \boldsymbol{ACI}(\mathbf{10.12.2})$$

$\mathbf{L_u}$: Actual Unsupported Length .

r: Radius Of Gyration = 0.3×h = $\sqrt{\frac{I}{A}}$

K = 1.0, According to **ACI 318-08 (10.12.2)** the effective length factor (K) shall be permitted to be taken as 1.0

L_u = 3.58 m.

$\frac{M_1}{M_2}$= 1.0

r =$0.3 \times 0.6 = 0.18$

$\frac{1.0\times3.58}{0.18} \leq 34 - 12 \times 1.0$

$19.8 < 22$, short column in both directions .

4.12.3 Select the longitudinal bars:

ϕ $P_{n,max}$ =$0.65 \times 0.8[0.85 \times 24(600000 - A_s) + 420A_s]$

$A_{s,req}$ = 2479 mm^2 , try ϕ20 with A_s =314 mm^2

Use 8 ϕ20with A_s =2512 mm^2 > $A_{s,req}$ ok

ρ = $\frac{\text{As}}{\text{Ag}} = \frac{2512}{600000} = 0.0.0042 < 0.01(min)$

assume ρ=0.015 > min

ρ = $\frac{\text{As}}{\text{Ag}} = \frac{\text{As}}{600000}$

As=9000 mm^2.

Use 16 ϕ25 with A_s > $A_{s,req}$ ok

4.12.4 Check for code requirements:

- clear spacing between longitudinal bars = $\frac{1000-40\times2-10\times2-5*25}{4} = 193.7mm$

  193.7 mm > 40mm

  >1.5 d_b = 30 mm . ok

- gross reinforcement ratio = 0.015 , 0.01 < 0.015 <0.08 ok
- NO of bars = 16 > 4 bars for square columns.
- min ties diameter : ϕ8 for ϕ32 longitudinal bars and smaller.

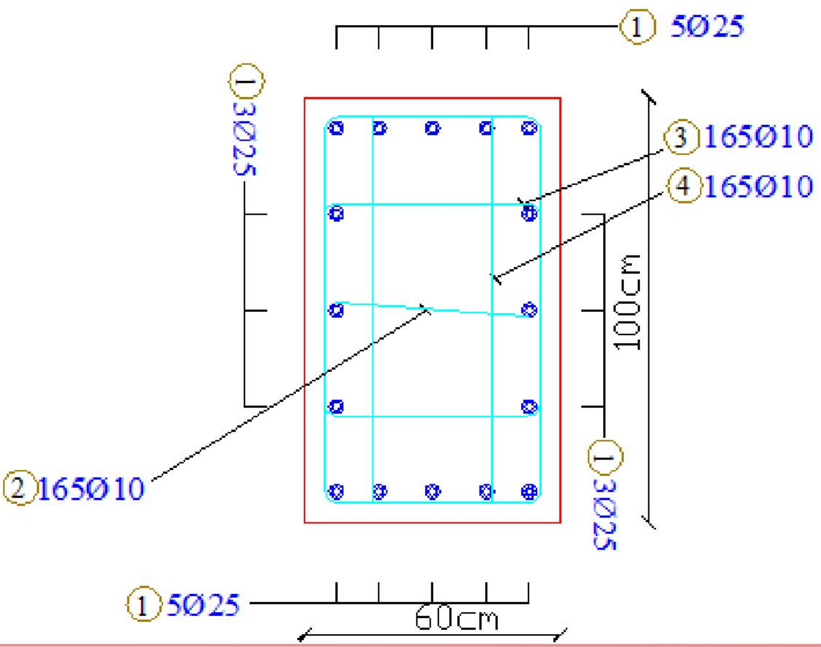


Figure (4.23) Reinforcement of column (C3).

4.13 Design of long column (C 20) :

4.13.1 Check the slenderness effect:

(non sway system)

$$\frac{kL_u}{r} < 34 - 12\left(\frac{M_1}{M_2}\right) \qquad ACI(10.12.2)$$

$$r = \sqrt{\frac{I}{A}} \approx 0.3h = 0.3 \times 0.4 = 0.12$$

$L_u = 3.9 - 0.32 = 3.58m$

$$\frac{kL_u}{r} = \frac{1 \times 3.58}{0.12} = 29.8 > 34 - 12 = 22$$

So the column is long at y direction .

r $=\sqrt{\frac{I}{A}} \approx 0.3h = 0.3 \times 0.6 = 0.18$

$L_u = 3.9 - 0.32 = 3.58$m

$\frac{kL_u}{r} = \frac{1\times3.58}{0.18} = 19.88 < 34 - 12 = 22$

So the column is short at x direction .

4.13.2 Calculate e_{min} ,M_{min}:

$e_{min} = 15 + 0.03h = 15 + 0.03\times400 = 27$mm.

$M_{min} = P_u \times e_{min} = 1056\times0.027 = 29$ KN.m

$E_c = 4700\sqrt{f_c'} = 4700\sqrt{24} = 23025.2$ Mpa.

$I_g = \frac{b.h^3}{12} = 3.2 \times 10^9$ mm^4.

$\beta_{dns} = \frac{D_u}{P_u} = \frac{840}{1056} = 0.79 < 1$.

E.I $= \frac{0.4E_c\, I_g}{1+\beta dns} = \frac{0.4\times23025.2\times32}{1.79} = 164647.47\ KN.m^2$

4.13.3 Determine of Euler buckling load:

$P_c = \frac{\pi^2 EI}{(Kl_u)^2} = \frac{\pi^2\times164647.47}{(3.58)^2} = 126664.1\ KN$

4.13.4 Calculate the moment magnifier factor:

$C_m = 0.6 + 0.4\left(\frac{M_1}{M_2}\right) = 1$

$\delta_{ns} = \frac{C_m}{1-\frac{P_u}{0.75P_c}} = \frac{1}{1-\frac{1056}{0.75\times126664.1}} = 1.1 > 1$

The magnified (e) and (M):

$e = \delta_{ns}\, e_{min} = 1.1 \times 27 = 29.7 \approx 30$mm

$M = \delta_{ns}\, M_{min} = 1.1 \times 29 = 32$ KN.m

From the interaction diagram constructed in **PCA _ COLUMN program**:

$\rho = 0.012 > 0.01$ (min)

4.13.5 Select the longitudinal bars:

$A_s = \rho \times A_g = 0.012 \times (600*400) = 2880$ mm^2

$n_{\phi20} = \frac{2880}{314} = 12\phi20$

Use $12\phi20$

4.13.6 Design the stirrups:

The spacing of ties shall not exceed the smallest of:

- $16\times d_b = 16\times20 = 320$ mm   cont.
- $48\times d_s = 48\times8 = 384$ mm
- Least diminution of the column = 400 mm

Use ϕ8@300mm.

4.13.7 Check for code requirements:

- clear spacing between longitudinal bars = $\frac{600-40\times2-10\times2-4\times20}{3} = 140mm$

  140 mm > 40mm

  $>1.5\ d_b = 30$ mm .   ok
- gross reinforcement ratio = 0.012 , $0.01 < 0.012 < 0.08$ ok
- NO of bars = 8 > 4 bars for square columns.
- min ties diameter : ϕ8 for ϕ32 longitudinal bars and smaller.

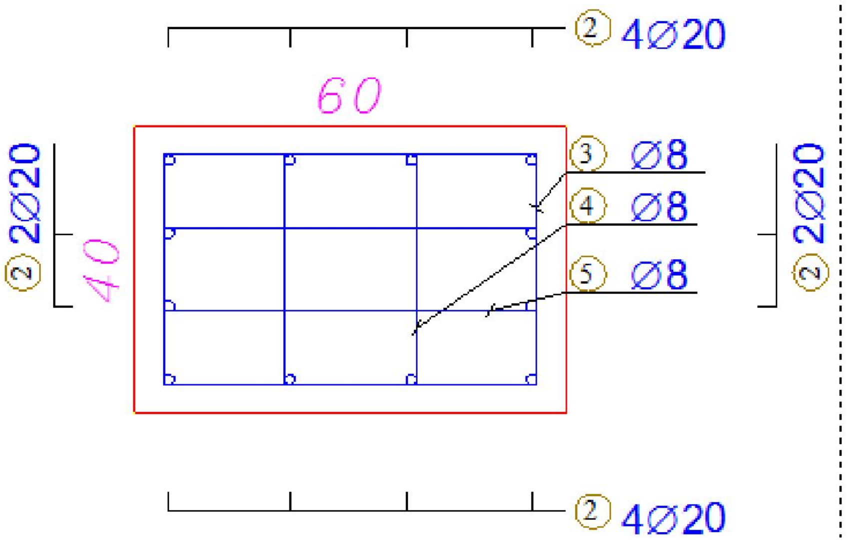


Figure (4.23) Reinforcement of column (C20)

4.14 Shear wall (SW13) design:

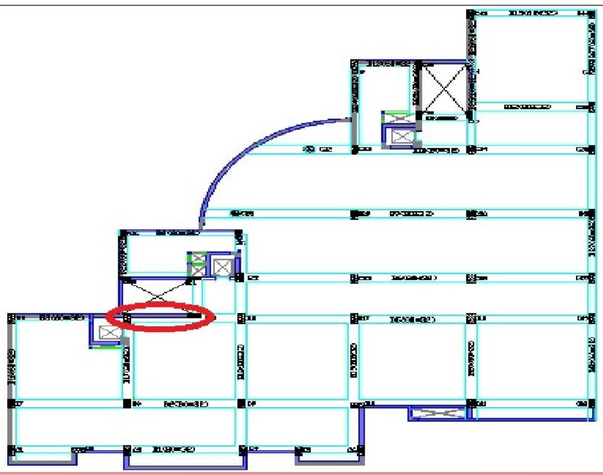


Figure (4.24) location of shear wall (SW13).

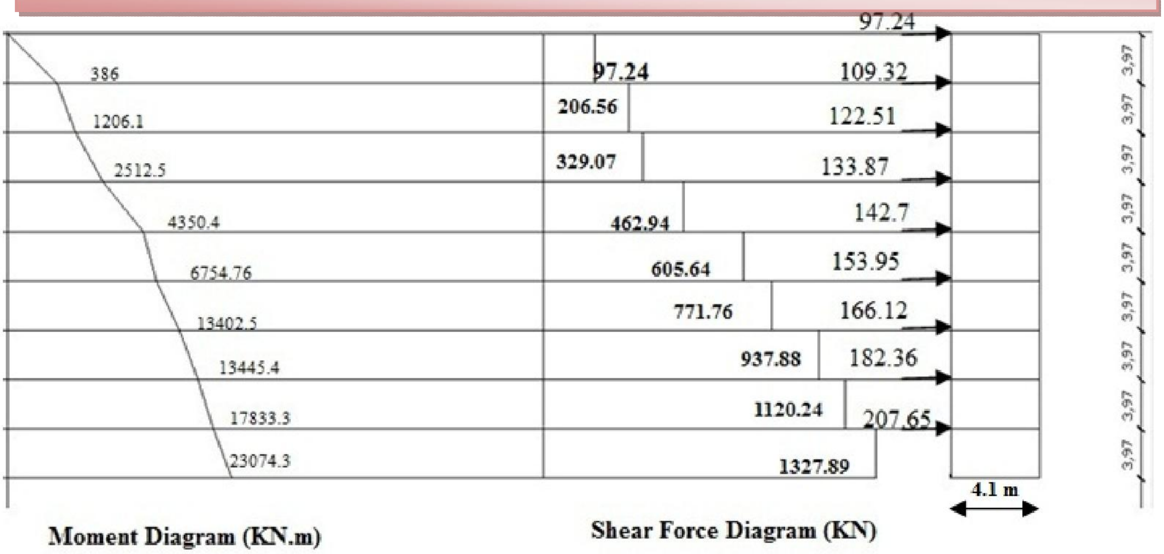


Figure (4.25) shear and moment for the shear wall (SW13).

Shear wall thickness, h = 30 cm.
Story height, h_w = 3.97 m

4.14.1 Check max shear strength permitted:

$\o V_{n,max} = 0.75 \times 0.83 \times \sqrt{f_c'} \times h \times d$
let that d= 0.8 L_w =0.8 ×4.1 =3.28 m
$\o V_{n,max} = 0.75 \times 0.83 \times \sqrt{24} \times 300 \times 3.28 = 3000.8$ KN > V_u =1327.89 KN ok
.

4.14.2 Calculate shear strength provided by concrete:

Critical section for concrete is the smallest of:

- $\frac{L_w}{2} = \frac{4.1}{2} = 2.05\ m \qquad cont.$
- $\frac{\sum h_w}{2} = \frac{4\times 3.97}{2} = 7.94\ m$
- Story height = 3.97 m.

V_c is the smallest of :

- $V_c = \frac{1}{6}\sqrt{f_c'}\ h\ d = \frac{1}{6}\sqrt{24} \times 300 \times 3.28 = 803.43\ KN \qquad ,cont.$
- $V_c = 0.27\sqrt{f_c'}\ h\ d + \frac{N_u . d}{4 l_w} = 0.27\sqrt{24} \times 300 \times 3.28 + 0.0 = 1301.56\ KN$
- $V_c = \left[0.05 \lambda \sqrt{f_c'} + \frac{L_w\left(0.1\lambda\sqrt{f_c'} + 0.2\frac{N_u}{h l_w}\right)}{\frac{M_u}{V_u} - \frac{L_w}{2}}\right] hd$

  M_u at critical section =17833.3 + 207.65*(3.97-2.05) = 18231.988 KN.m

  $\frac{18231.988}{1327.89} - \frac{4.1}{2} = 11.68 > 0.0 \quad ok.$

  $V_c = \left[0.05\sqrt{24} + \frac{4.1\left(0.1\sqrt{24}+0.0\right)}{11.68}\right]\ 300 \times 3.28 = 410.25\ KN \qquad ,cont.$

4.14.3 Determine required horizontal reinforcement:

V_u =1327.89 KN.

$0.5øV_c = 0.5\times 0.75\times 410.25 = 153.84$ KN.

$V_u > 0.5øV_c$ need reinforcement.

$V_s = V_n - V_c = \frac{V_u}{ø} - V_c = \frac{1327.89}{0.75} - 410.25 = 1360.27$ KN.

$\rho = \frac{A_s}{s.h} \quad , \frac{A_s}{s} = \frac{A_s}{f_y.d} = \frac{1360\times 10^3}{420\times 3280} = 0.987\ \frac{mm^2}{mm}$

$\rho = \frac{0.987}{200} = 0.00493 > 0.0025$

by using ϕ12

$\rho = \frac{2\times 113}{s\times 200} = 0.00493 \longrightarrow s = 229.2$ mm.

max. spacing is the smallest of:

- $\frac{L_w}{5} = \frac{4.1}{5} = 0.82m$
- 3h = 0.9m.
- 450mm …. cont.

For horizontal reinforcement use *ϕ12@200mm.*

4.14.4 Determine required vertical reinforcement:

$\frac{\Sigma h_w}{L_w} = \frac{9\times3.97}{4.1} = 8.71\ m.$

$\rho_{v,min} > 0.0025 + 0.5\left(2.5 - \frac{\Sigma h_w}{L_w}\right)(\rho_t - 0.0025) \geq 0.0025.$

take ρ_v= 0.0025.

max spacing is the least of,

- $\frac{L_w}{3} = \frac{4.1}{3} = 1.366\ m$
- 3h = 0.9m.
- 450mm cont.

Use *ϕ10@300mm.*

4.14.5 Design for flexure (uniformly distributed flexure reinforcement):

Check moment strength based on required vertical reinforcement for shear,
The uniformly distributed vertical reinforcement ϕ10@300mm.

$A_{st}=\frac{4100}{300} \times 2 \times 78.5 = 2145.66\ mm^2$

$\omega = \left(\frac{\mathrm{Ast}}{L_w h}\right)\frac{f_y}{f_c'} = \left(\frac{2145.66}{4100\times200}\right)\frac{420}{24} = 0.046$

$\alpha = \frac{\mathrm{P_u}}{L_w f_c' h}$ =0.0

$\frac{\mathrm{c}}{L_w} = \frac{\omega+\alpha}{2\omega+0.85\beta_1} = \frac{0.046+0.0}{2\times0.046+0.85\times0.85} = 0.056$

$\text{ø}M_n = \text{ø}\left[0.5A_{st}f_y L_w \left(1+\frac{P_u}{A_s f_y}\right)\left(1-\frac{c}{L_w}\right)\right]$

$=0.9\ [0.5 \times 2145.66 \times 420 \times 4100(1-0.056)] \times 10^{-6}$ =1569.56 KN.m
$øM_n < M_u$.

the uniformly distributed vertical reinforcement ϕ10@300, is not adequate for flexure therefore the amount of vertical reinforcement must be increased.
Try ϕ16@150.

$A_{st}=\frac{4100}{150} \times 2 \times 201 = 10988\ mm^2$

$\omega = \left(\frac{10988}{4100\times150}\right)\frac{420}{24} = 0.3126$

$\frac{c}{L_w} = \frac{0.3126}{2\times0.3126+0.85\times0.85} = 0.232$

$øM_n = 0.9\ [0.5 \times 10988 \times 420 \times 4100(1-0.232)] \times 10^{-6}$ =6539.63 KN.m
$>M_u$

Use ϕ16@150 for vertical reinforcement.

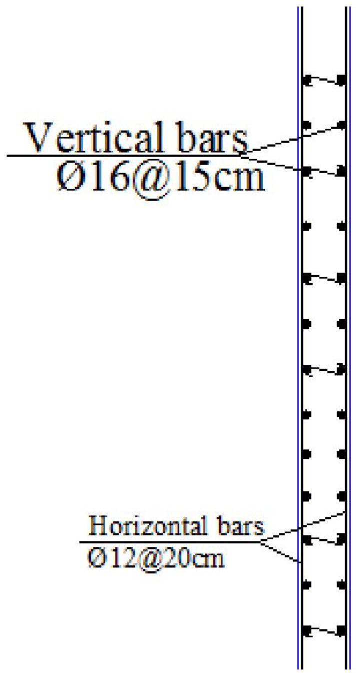


Figure (4.26)Reinforcement of the shear wall (SW13).

4.15 Design of Basement wall:

- Loads on basement wall :

q1 = Earth pressure soil

q1= γ * h * k0

K_0 = 1 – sin 30 = 0.5

q1= 18 * 3.97 * 0.5 = 35.73 KN/m2

factored load (q_u) =1.6 * q1 = 1.6 * 35.73 = 57.2

h wall = 30 cm

- Design of shear force :

From Attir Vu = 75.9 KN

d=300 -20 – 16/2 = 272 mm

$\Phi * Vc = 0.75 * \frac{\sqrt{fc'}}{6} bw * d = 0.75 * \frac{\sqrt{24}}{6} * 272 * 1000 = 166.56 \text{ KN} > Vu$

(h =30 is enough)

- **Design of the Vertical reinforcement** :

- In tension side:

Max Mu from Attir = 122.3 KN.m

$Rn = \frac{Mn}{b * d^2}$

$Rn = \frac{162.3 * 10^6 / 0.9}{1000 * (272)^2} = 1.84 \text{ MPa}$

$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{2mRn}{fy}})$

$\rho = \frac{1}{20.6}(1 - \sqrt{1 - \frac{2(20.6)(1.84)}{420}}) = 0.0046$

As req = ρ * b * d = 0.0046* 100 *27.2 = 12.512 cm²/m.

As min = 0.0012 * b * h = 0.0012*100 * 30 = 3.6 cm²/m

As req> As min

select Φ16@ 15 cm $A_s\,provided\ = 13.4cm^2\,/\,m$

- <u>In compression side :</u>

As min = 0.0012 * b * h = 0.0012*100 * 30 = 3.6 cm²/m

select Φ12@ 20 cm $A_s\,provided\ = 5.65cm^2\,/\,m$

<u>Design of the Horizontal reinforcement :</u>

For One layer :

As min = 0.001 * b * h = 0.001*100 * 30 = 3 cm²/m

select Φ12@ 20 cm $A_s\,provided\ = 5.65cm^2\,/\,m$

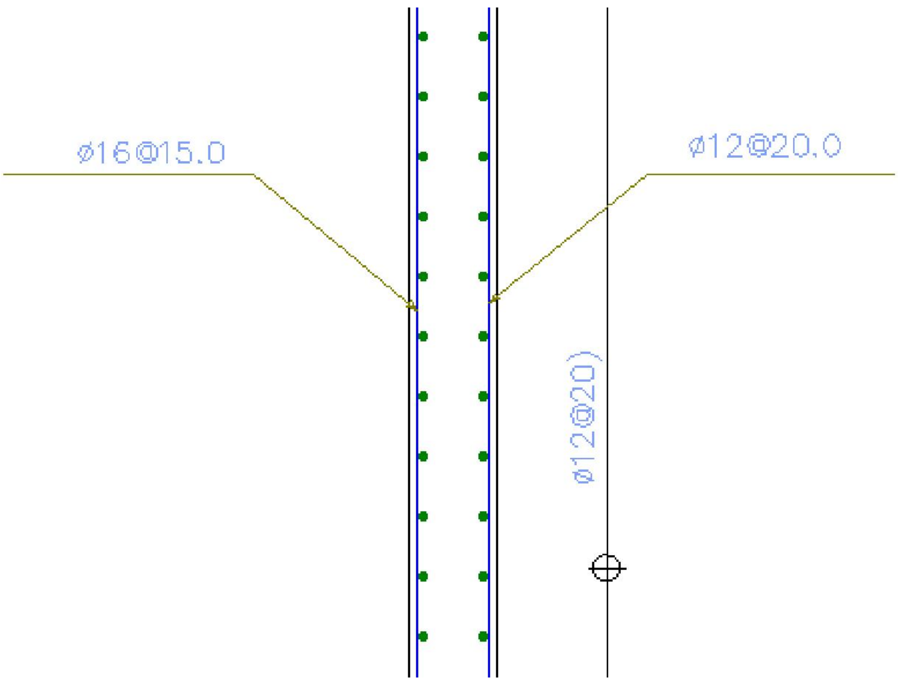


Figure (4.26)Reinforcement of the basement wall.

4.16 Design of Isolated Footing (under column 13):

P_D = 1100 KN (service).
P_L = 409 KN (service).
P_U =1.2*1100+1.6*409= 1974.4 KN (factored).
Column Dimensions D = 80*50 cm.
Allowable bearing capacity q_{all}= 450 KN/m^2.

4.16.1 Area of Footing:

Soil Density = 18 KN/ m^3
live load = 5 KN/ m^2.
assume h = 60 cm.
$q_{all \cdot net} = 450 - 5 - (0.6\times 25) - (0.6\times 18) = 419.2$ KN/m^2

Area $A = \dfrac{PD+PL}{qal\ln et} = \dfrac{1974.4}{419.2} = 14.7$

Use L = 2 m, B = 2.4 m, A = 4.8 m^2

4.16.2 Depth of footing:

Assume h = 60 cm.

- **Check one-way shear:**

$$q_{ult} = \frac{P_u}{Area} = \frac{1974.4}{4.8} = 411.33 \text{ KN/ m}^2.$$

d =600 – 75 – 20-9 = 496 mm

$$\Phi V_c = \Phi \frac{1}{6}\sqrt{f_c'}\, b_w d = \frac{0.75}{6} * \sqrt{24} * 2.4 * 0.496 * 1000 = 729KN$$

$$Vu = q_{ult} \times \left(\frac{B-a}{2} - d\right) \times L$$

$$Vud = 411.33 \times \left(\frac{2.4-0.5}{2} - 0.505\right) \times 2 = 366.1KN$$

$øV_c = 729$ KN > $V_{ud} = 366.1 \rightarrow\rightarrow\rightarrow\rightarrow ok$

- **Check two-way shear:**

.

$\frac{d}{2} = \frac{496}{2} = 248\ mm.$

$V_u = q_u$ (outer area – inner area) = 411.33 ×(4.8 – 1.3) =1439.6 KN

According to ACI , Vc shall be the smallest of :

$$V_c = \frac{1}{6}\left(1+\frac{2}{\beta_c}\right)\sqrt{f_c'}\,b_o d = 0.5\sqrt{f_c'}\,b_o d$$

$$V_c = \frac{1}{12}\left(\frac{\alpha_s}{b_o/d}+2\right)\sqrt{f_c'}\,b_o d = 0.585\sqrt{f_c'}\,b_o d$$

$$V_c = \frac{1}{3}\sqrt{f_c'}\,b_o d \quad \text{.....Control}$$

Where:

β_c= a / b = 80 / 50 = 1.6

b_o = 4584 mm.

α_s = 40 for interior column.

$\phi V_c = 0.75\times0.33\sqrt{24}\times4584\times496\times1000 = 27568KN$

$$\varphi Vc = 2756.8\ KN > \ Vu\ = 1439.6\ KN$$

SO h = 60 cm Is OK.

4.16.3 Design of flexural reinforcement:

$$\text{Mu} = \left(q_{ult}\times W\times\left(\frac{L}{2}-\frac{a}{2}\right)\right)\times0.5\left(\frac{L}{2}-\frac{a}{2}\right)$$

$$= \left(411.33\times2.4\times\left(\frac{2.4}{2}-\frac{0.5}{2}\right)\right)\times0.5\left(\frac{2.4}{2}-\frac{0.5}{2}\right) = 445.5\ \text{KN.m}$$

Mn = 445.5/0.9 = 494.96 KN.m.

Rn = $\frac{Mn}{b*d^2}$

Rn = $\frac{494.9*10^{-3}}{2.4*(0.496)^2}$ =0.83 Mpa

$m = \frac{fy}{0.85*fc'} = \frac{420}{0.85*24} = 20.59$

ρ = $\frac{1}{m}(1 - \sqrt{1-\frac{2mRn}{fy}})$

ρ = $\frac{1}{20.59}(1 - \sqrt{1-\frac{2(20.59)(0.83)}{420}}) = 0.002$

A req = ρ× b × d =0.002 × 2400× 496= 2380.8 mm²

$A_{s\,min} = 0.0018\times2400\times600 = 2592\ mm^2$….. control

So , Use 10 Φ 18 with As = 2543.4mm² > As req = 2380.8 mm²… in both directions .

4.16.4 Development length of flexural reinforcement:

Ld for Φ 18:

$$L_d = \frac{9}{10}\times\frac{fy}{\sqrt{fc'}}\times\frac{\alpha\times\beta\times\gamma\times\lambda}{\left(\frac{k_{tr}+c}{db}\right)}\times db = \frac{9}{10}\times\frac{420}{\sqrt{24}}\times\frac{1\times1\times0.8\times1}{2.5}\times18 = 444.5mm$$

Available length = ((2400-800)\2)-75=725mm

725mm > 444.5 mm ……………ok

4.16.5 Development length of column reinforcement:

Ld for Φ 22 :

$$L_d = \frac{fy}{4\sqrt{fc'}}db = \frac{420}{4\sqrt{24}}\times18 = 286.7mm$$ ……cont.

$L_d = 0.043\times db\times fy = 0.043\times18\times420 = 245.3$ mm

Available embedment =700 – 75 – (2 x18) = 589 mm > 286.7 mm

∴ OK.

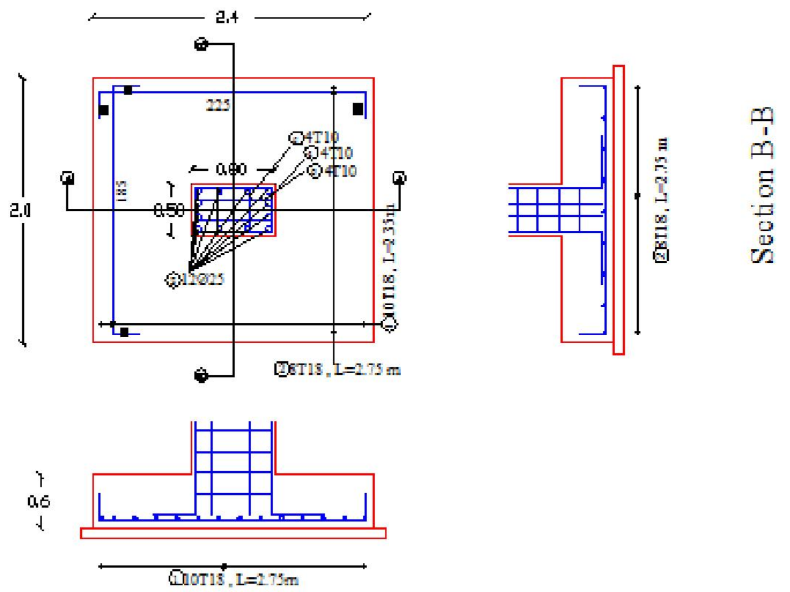


Figure (4.27)Reinforcement of isolated footing under column (C13).