Chapter 4

Structural Analysis & Design

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✓ 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, all of design calculation for all structural members would be made upon the structural system which was chosen in the previous chapter.

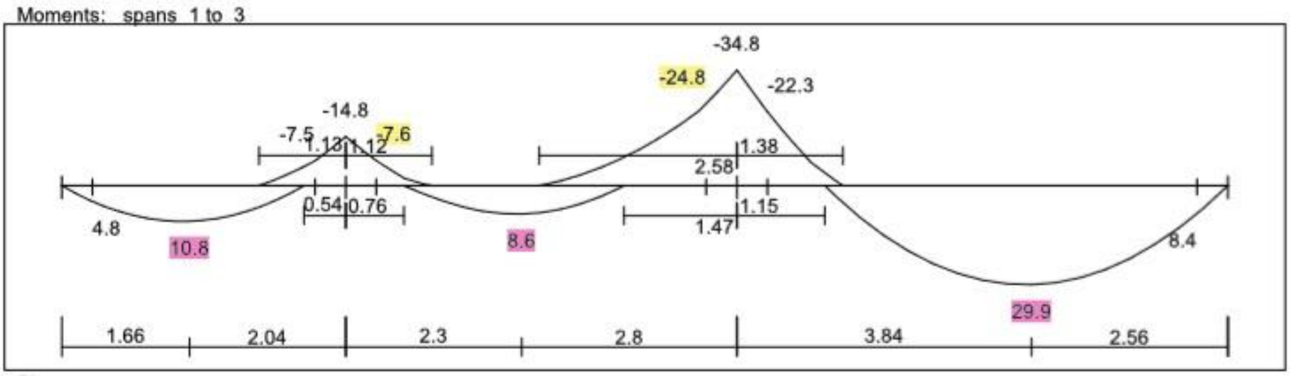
So, in this project, there are five types of slabs: One way solid slab, one way ribbed slab, two ribbed slab, flat slab, and waffle slab. They would be analyzed and designed by using finite element method of design, with aid of a computer program called "ATIR- Soft ware " to find the internal forces, deflections and moments for ribbed slabs and by using the previous program and Etabs, Safe,And programs to find the internal forces.

✓ Factored loads:

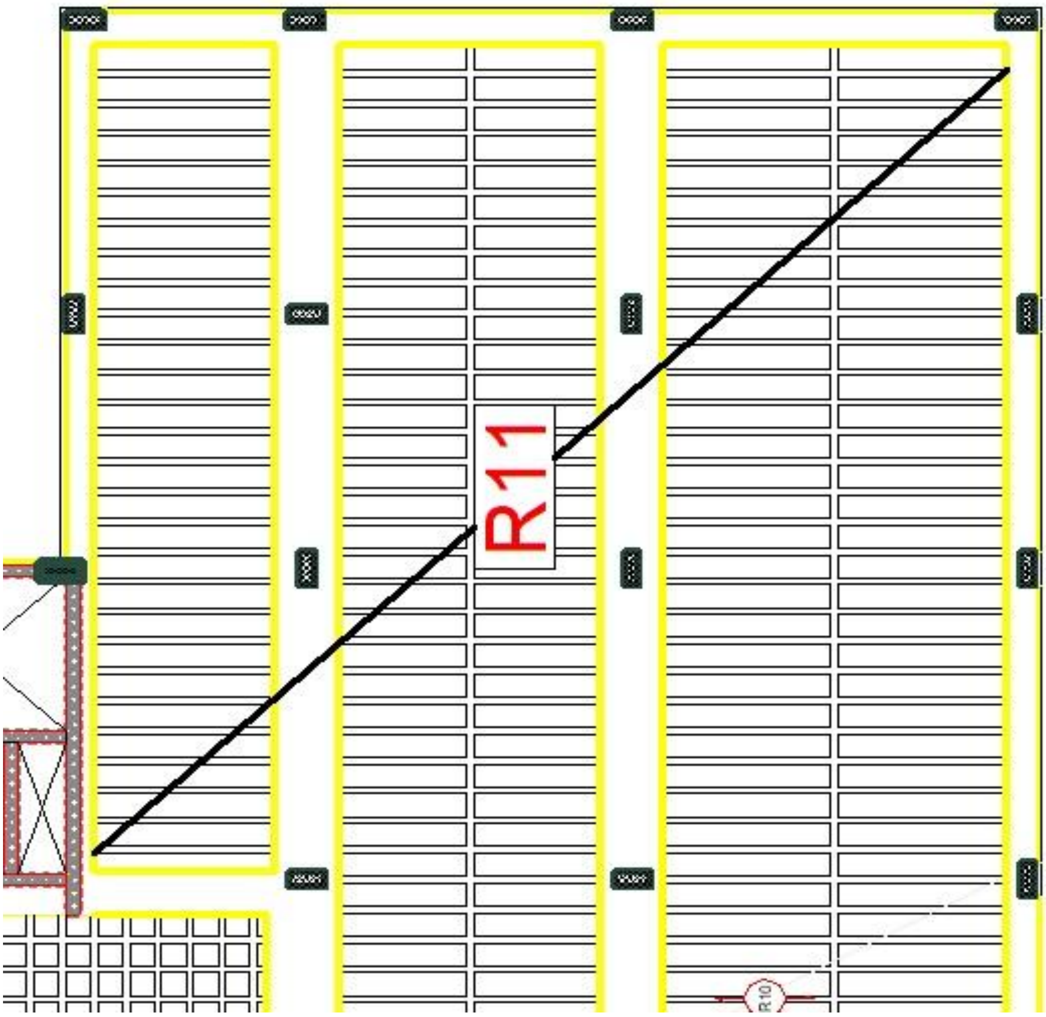
The factored loads on which the structural analysis and design is based for our project members, is determined as follows:

$$qu = 1.2D.L + 1.6L.L\ .$$

4-1 Design of Rib 11:



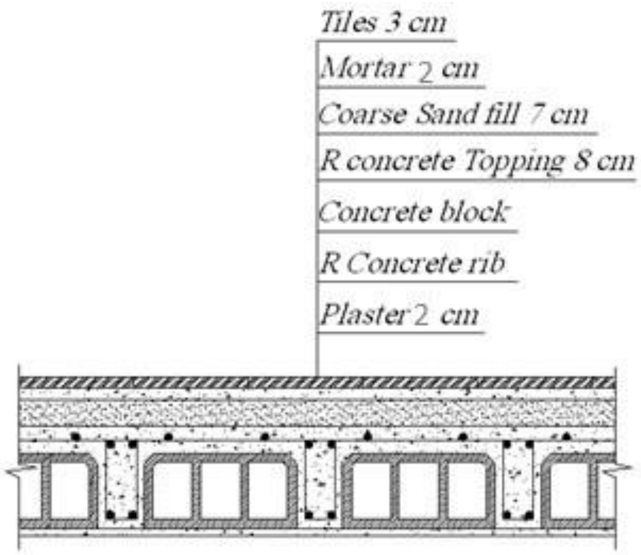
Fig(4-1):Moment Envelope (factored) units:KN.m



Fig(4.2): Rib 11

Table (4.1) : Dead load calculation Topping

Material	Quality Density KN/m³
Tiles	23
mortar	22
Sand	17
Reinforced Concrete	25
Hollow Block	10
Plaster	22
Partitions	1.5KN/m²



TYPICAL SECTION IN RIBBED SLAB

Compressive strength of concrete $fc' = 30\ Mpa$

Yield strength of steel , $fy = 420\ Mpa$

Live Load,LL=3 KN/m^2

✓ **Minimum thickness (deflection requirements):**

There are two groups of ribs and beams

The thickness of the one-way ribbed slab with drop beams can be obtained according to ACI code, table 9.5 (a).

The maximum span length for one-end continuous (for ribs): l =640cm then

$$h_{min} = \frac{l}{18.5} = \frac{640}{18.5} = 34.6\ cm$$

The maximum span length for both-ends continuous (for ribs): l =510cm then

$$h_{min} = \frac{l}{21} = \frac{510}{21} = 24.3cm$$

The minimum ribbed slab thickness will be $h_{min} = 34.6cm$

Take slab thickness $h = 35cm > h_{min} = 34.6cm$

$h = 35cm$ (27cm HOLLOW BLOCK + 8cm TOPPING)

✓ 4-2 Topping Design:

Topping in One way ribbed slab can Be considered as a strip of **1 meter width** and span of hollow block length with both end fixed in the ribs.

Dead Load calculations:

Dead Load from:	$\delta \times \gamma \times 1$	KN/m
Tiles	0.03×23	0.69
Mortar	0.02 ×22	0.44
Coarse Sand	0.07× 17	1.19
Topping	0.08×25	2
Interior Partitions	1.5	1.5
	Σ	5.82

Live Load calculations: $3 \times 1 = 3KN/m$

Total Factored Load: $w_u = \mathbf{1.2 * 5.82 + 1.6 * 3 = 11.78 KN/m}$

$M_u = \frac{wul^2}{12} = \frac{11.78*0.4^2}{12} = \mathbf{0.157} \boldsymbol{KN.m}$/m of strip width

$\boldsymbol{\emptyset Mn \geq Mu}$ Strength condition, where $\emptyset = \mathbf{0.55}$ for plain concrete

$\boldsymbol{Mn} = \mathbf{0.42}\lambda\sqrt{\boldsymbol{fc'}}\, S_m$

where S_m for rectangular section of the slab:

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

$$\boldsymbol{Mn} = \mathbf{0.42}\lambda\sqrt{\boldsymbol{fc'}}\, S_m = 0.42*1*\sqrt{\mathbf{30}} * 1066666.67 \times 10^{-6} = 2.45 \text{ KN.m}$$

$$\boldsymbol{\emptyset Mn} = \mathbf{0.55 * 2.45 = 1.35}\ \boldsymbol{KN.m} \gg \quad \boldsymbol{Mu} = \mathbf{0.157}\boldsymbol{KN.m}$$

NO Reinforcement is required by analysis. According to ACI 10.5.4., provide AS_{min} for slabs as shrinkage and temperature reinforcement.

ρ = 0.0018

AS= ρbt =0.0018 * 1000 *80= 144mm²/m strip

Try bars Ø 8 with AS =50.27mm^2

Bar number $n = \frac{AS}{ASØ\,8} = \frac{144}{50.27} = 2.87$

Take 3 Ø 8/m with AS=150.8 mm^2/m strip or Ø 8 @ 300 mm in both direction

step(s) is smallest of :

1. 3h = 3×80 = 240 mm – control
2. 450 mm
3. S=

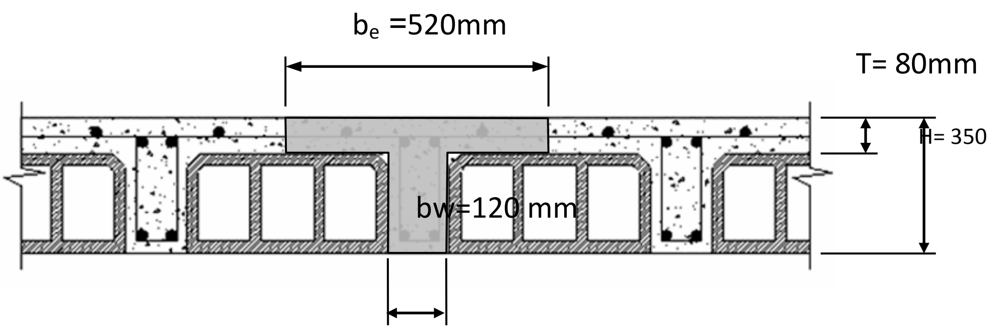
$$380\left(\frac{280}{fs}\right) - \mathbf{2.5Cc} = \mathbf{380}\left(\frac{\mathbf{280}}{\frac{2}{3}\mathbf{420}}\right) - \mathbf{2.5 * 20 =}$$

$\mathbf{330\ \mathit{mm}}$ ***but***

$$S \leq \mathbf{300}\left(\frac{\mathbf{280}}{\mathbf{\mathit{fs}}}\right) = 300\left(\frac{\mathbf{280}}{\frac{2}{3}\mathbf{420}}\right) = \mathbf{300\mathit{mm}}$$

Take Ø 8 @ 200mm in both directions S=200mm < S_{max}=240 mm – ok

From practical consideration, the secondary reinforcement parallel to the ribs shall be placed in the slab and spaced at distances not more than half of the spacings between ribs (usually two bars upon each 40 cm width block).



✓ **Load Calculations for Rib 11:**

T= h_f= 80 mm bw= 120 mm be = 520 mm h= 350 mm

Dead Load calculations

Dead Load from:	δ × γ × b	$KN.m$
Tiles	0.03*23*0.52=	0.359
Mortar	0.02*22*0.52=	0.2288
Coarse Sand	0.07*17*0.52=	0.619
Topping	0.08*25*0.52=	1.04
RC Rib	0.27*25*0.12=	0.81
Hollow Block	0.27*10*0.4=	1.08
Plaster	0.02*22*0.52=	0.2288
Interior Partitions	1.5*0.52=	0.78
	Σ	5.146

Table(4−2):Dead Load Calculation of Rib (R 11)

Live Load calculations:

LL=3 * 0.52 = 1.56 KN/m

Dead Load / rib*: DL= 5.146* $KN.m$

Live Load /rib: LL=1.56 $KN.m$

✓ **Design of Rib 11 for positive moments:**

1.Mu =+29.9KN.m

*Assume bar diameter Ø 14 for main positive reinforcement.

d = h – cover – d_{strip}– $\frac{db}{2}$= 350 – 20 – 10 - $\frac{14}{2}$ = 313 mm

*Check if a > h_f

M_{nf} = 0.85 $fc'b\,hf\left(d-\frac{hf}{2}\right)$

= 0.85 * 30*520 *80 $\left(313-\frac{80}{2}\right)*10-6$ = 289.6 KN.m

M_{nf} =289.6KN.m ≫ $\frac{Mu}{\emptyset}=\frac{29.9}{0.9}=33.2$ $a<hf$

***Design:**

Rn = $\frac{Mu}{bd^2 \emptyset} = \frac{29.9*10^6}{0.9*520*313^2} = \mathbf{0.652\ MPa}$

M= $\frac{fy}{0.85\, fc'} = \frac{420}{0.85*30} = \mathbf{16.47}$

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2Rn\, m}{fy}}\right)$ = $\frac{1}{16.47}\left(1 - \sqrt{1 - \frac{2*0.652*16.47}{420}}\right)$ = 0.00157

$As = \rho\, b\, d$ =0.00157 * 520 * 313 = 255.5mm^2

*Cheak for As$_{min}$:

AS$_{min}$ = 0.25 $\frac{\sqrt{fc'}}{fy}\ bw\ d \geq \frac{1.4}{fy}\ bw\ d$

AS$_{min}$= $\mathbf{0.25}\ \frac{\sqrt{30'}}{420} * 120 * 313 = \mathbf{122.46\ mm^2}$

AS$_{min}$= $\frac{1.4}{420} * 120 * 313 = \mathbf{125.2\ mm^2}$ -----Control

$As = 255.5mm^2 > As\min = 125.2\ mm^2$ ---- OK

$use\ 2\,\emptyset 14\ with\ As = 3.08\ cm^2 > AS\ req = 2.56\ cm^2$ ---- *OK*

**Cheak for strain (Tension-Controlled section- $\varepsilon s \geq 0.005$):*

a= $\frac{As\, fy}{0.85\, fc\, b'} = \frac{3.08*420}{0.85*30*520}$ = *9.76 mm*

C= $\frac{a}{\beta}$ $\beta = 0.836$

C = $\frac{9.76}{0.836}$ = 11.67 mm

$\varepsilon s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{313-11.67}{11.67}\right) = 0.0775 > 0.005$ ---- OK

<u>2.Mu =+10.8KN.m</u>

*Assume bar diameter Ø 12 for main positive reinforcement:

d = h – cover – d$_{strip}$– $\frac{db}{2}$= 350 – 20 – 10 - $\frac{12}{2}$ = 314 mm

*Check if a > h$_f$

M$_{nf}$ = 0.85 fc´b hf $\left(d - \frac{hf}{2}\right)$

= 0.85 * 30*520 *80 $\left(314 - \frac{80}{2}\right) * \mathbf{10 - 6}$ = 290.7 KN.m

M_{nf} =290.7KN.m ≫ $\frac{Mu}{\emptyset} = \frac{10.8}{0.9} = 12 \quad a < hf$

***Design:**

Rn = $\frac{Mu}{bd^2\emptyset} = \frac{10.8*10^6}{0.9*520*314^2} = 0.234\ MPa$

M= $\frac{fy}{0.85\,fc'} = \frac{420}{0.85*30} = 16.47$

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2Rn\,m}{fy}}\right) = \frac{1}{16.47}\left(1-\sqrt{1-\frac{2*0.234*16.47}{420}}\right)$ = 0.00056

$As = \rho\, b\, d$ =0.00056 * 520 * 314 = 91.4mm^2

*Cheak for As_{min}

AS_{min} = 0.25 $\frac{\sqrt{fc'}}{fy}\, bw\, d \geq \frac{1.4}{fy}\, bw\, d$

AS_{min}= $0.25\,\frac{\sqrt{30'}}{420} * 120 * 314 = 122.8\ mm^2$

AS_{min}= $\frac{1.4}{420} * 120 * 314 = 125.6\ mm^2$ -----Control

$As = 91.4mm^2 < As\min = 125.6\ mm^2$

$use\ 2\ \emptyset 10\ with\ As = 1.57\ cm^2 > AS\ min = 1.26\ cm^2$ ---- OK

**Cheak for strain (Tension-Controlled section- $\varepsilon s \geq 0.005$):*

$a = \frac{As\, fy}{0.85\, fc\, b'} = \frac{1.57*420}{0.85*30*520} = 4.97\ mm$

$C = \frac{a}{\beta}$ $\beta = 0.836$

$C = \frac{4.97}{0.836} = 5.95\ mm$

$\varepsilon s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{315-5.95}{5.95}\right) = 0.156 > 0.005$ ---- OK

Usually, no Reinforcement less than $2\emptyset 10$ can be used .So for other span with positive moment equal 8.6 KN.m, use $2\emptyset 10$ for rib span .

- ✓ **Design of Rib 11 for negative moments:**

1.Mu = -24.8KN.m

*Assume bar diameter Ø 14 for main negative reinforcement:

d = h – cover – d_{strip}– $\frac{db}{2}$= 350 – 20 – 10 - $\frac{14}{2}$ = 313 mm

*Design:

Rn = $\frac{Mu}{bd^2 \emptyset} = \frac{24.8*10^6}{0.9*120*313^2} = \mathbf{2.34\ MPa}$

M= $\frac{fy}{0.85\, fc'} = \frac{420}{0.85*30} = \mathbf{16.47}$

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2Rn\, m}{fy}}\right) = \frac{1}{16.47}\left(1 - \sqrt{1 - \frac{2*2.34*16.47}{420}}\right)$ = 0.00585

$As = \rho\, b\, d$ =0.00585 * 120 * 313 = 219.86 mm^2

*Cheak for As$_{min}$

$As = 219.86 mm^2 > As\min = 125.2\ mm^2$ ---- OK

$use\ 2\,\emptyset 14\ with\ As = 3.08\, cm^2 > AS\, req = 2.19\, cm^2 - - -$ ok

Cheak for strain (Tension-Controlled section- $\varepsilon s \geq 0.005$):

$a = \frac{As\, fy}{0.85\, fc\, b'} = \frac{3.08*420}{0.85*30*120} = 42.26\ mm$

$C = \frac{a}{\beta}$ $\beta = 0.836$

$C = \frac{42.26}{0.836} = 50.55mm$

$\varepsilon s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{313-50.55}{50.55}\right) = 0.0156 > 0.005$ ---- OK

<u>2.Mu = -7.6KN.m</u>

*Assume bar diameter Ø 12 for main negative reinforcement:

d = h – cover – d$_{strip}$– $\frac{db}{2}$= 350 – 20 – 10 - $\frac{12}{2}$ = 314 mm

*Design:

Rn = $\frac{Mu}{bd^2 \emptyset} = \frac{7.6*10^6}{0.9*120*314^2} = \mathbf{0.714\ MPa}$

M= $\frac{fy}{0.85\, fc'} = \frac{420}{0.85*30} = \mathbf{16.47}$

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2Rn\, m}{fy}}\right) = \frac{1}{16.47}\left(1 - \sqrt{1 - \frac{2*0.714*16.47}{420}}\right)$ = 0.00172

$As = \rho\, b\, d$ =0.00172 * 120 * 314 = 64.81mm^2

*Cheek for As_{min}

$AS_{min} = 0.25 \frac{\sqrt{fc'}}{fy} bw\, d \geq \frac{1.4}{fy} bw\, d$

$AS_{min} = 0.25 \frac{\sqrt{30'}}{420} * 120 * 314 = 122.8\, mm^2$

$AS_{min} = \frac{1.4}{420} * 120 * 314 = 125.6\, mm^2$ -----Control

$As = 64.81 mm^2 < As\min = 125.6\ mm^2$

$use\ 2\ \emptyset 10\ with\ As = 1.57\ cm^2 > AS\ min = 1.26\ cm^2$ ---- OK

*Cheak for strain (Tension-Controlled section- $\varepsilon s \geq 0.005$):

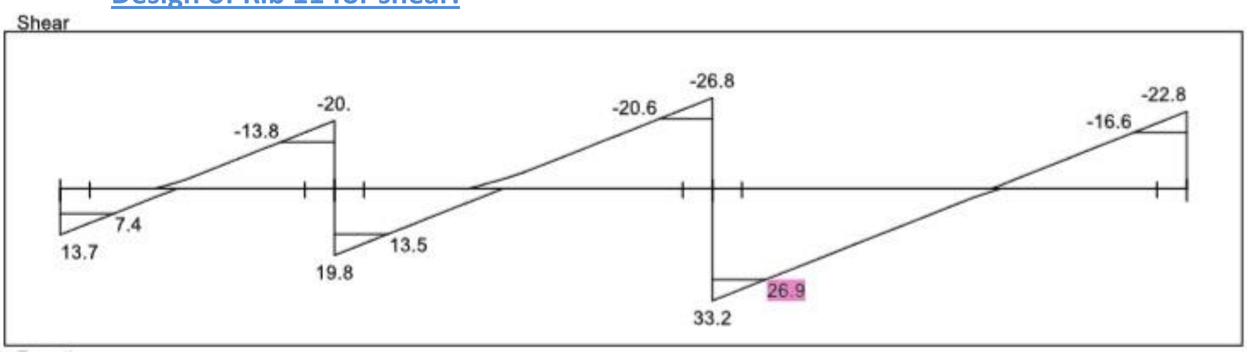
$a = \frac{As\, fy}{0.85\, fc\, b'} = \frac{1.57*420}{0.85*30*120} = 21.55\ mm$

$C = \frac{a}{\beta}$ $\beta = 0.836$

$C = \frac{21.55}{0.836} = 25.78\ mm$

$\varepsilon s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{315-25.78}{25.78}\right) = 0.0337 > 0.005$ ---- OK

✓ Design of Rib 11 for shear:



Fig(4-3) Shear Envelope (factored) units:KN

The maximum shear force at the distance d from the face of support V_u = 26.9 KN .

Shear strength, Vc , provided by concrete for the ribs may be taken 10% greater than that for beams. This is mainly due to the interaction between the slab and the closely spaced ribs.

$Vn=\frac{Vu}{\emptyset}$

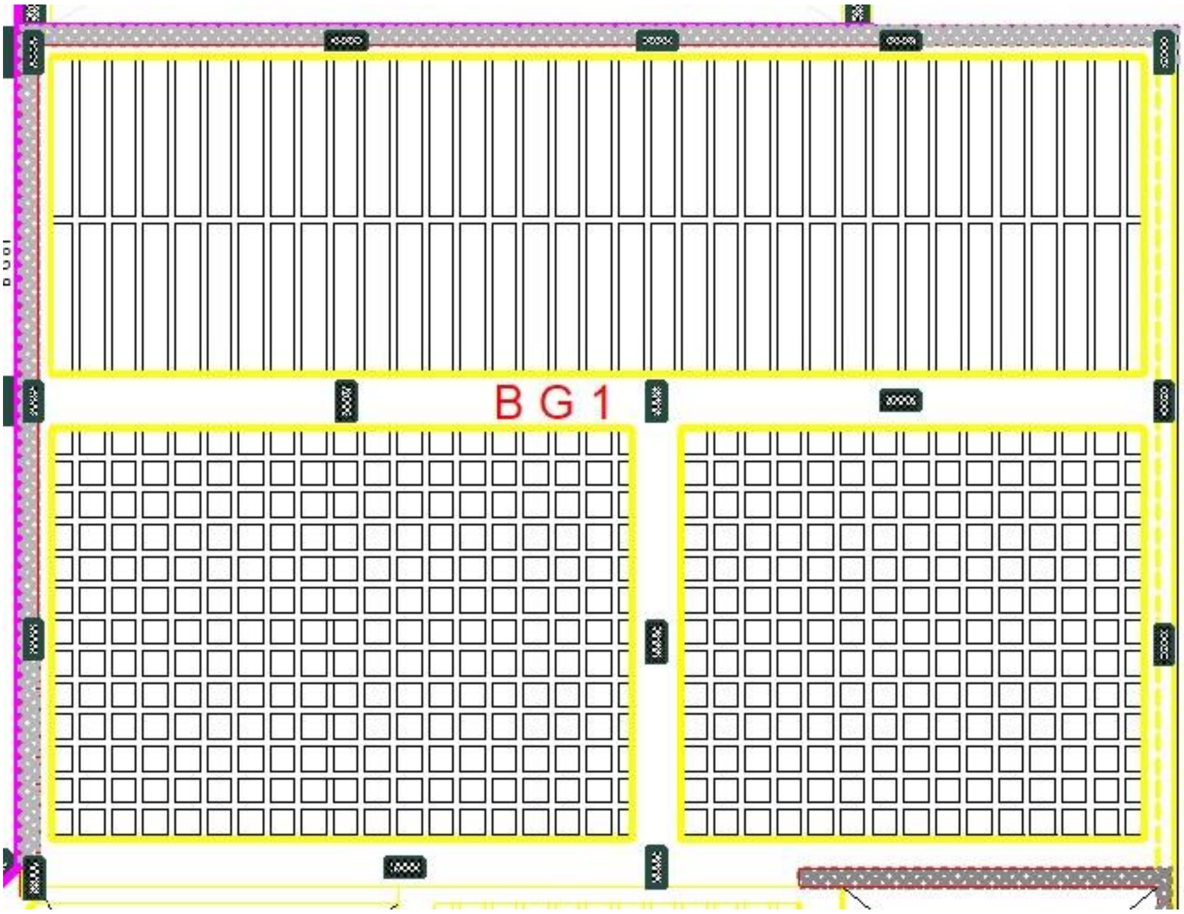
$Vc = 1.1 * \frac{1}{6}\ \lambda\sqrt{fc'}\,bw\ d = 1.1 * \frac{1}{6} * 1\sqrt{30} *120 *314 *10^{-3} = 37.84$ KN

$$\emptyset\, Vc = 0.75 * 37.84 = 28.38\ KN$$

$$Vu\ =\ 26.9\ KN < \ \emptyset\, Vc = 28.38\ KN$$

→Minimum Shear reinforcement is required **expect for concrete joist construction . So, no shear reinforcement is provided .**

4-3 Design of Beam B G 1 :



Fig(4.4): Beam B G 1

Dead load from two way ribbed slab

Parts of Flight	Calculation	
Tiles	22*0.03*0.52*0.52 = 0.178 KN	
Mortar	22*0.02*0.52*0.52 = 0.119KN	
Sand	16*0.07*0.52*0.52 = 0.303 KN	
Topping	25*0.08*0.52*0.52 = 0.541 KN	
Rib	25*0.27*0.12*(0.52+0.4) = 0.745 KN	
Block	9*0.27*0.4*0.4 = 0.389 KN	
Plaster	22*0.02*0.52*0.52 = 0.119 KN	
	Sum	2.80 KN

Table (4.3) : Dead load from two way ribbed slab

DL=2.8/(0.52*0.52)=10.36 KN/m^3

LL=3 KN/m^3

Dead Load from Two Way Ribbed Slab on Beam=2×3.37×10.36=69.83 KN/m

Live Load from Two Way Ribbed Slab on Beam=1×3.37×3=69.83 KN/m

Dead Load from One Ribbed Slab on Beam=5.146 KN/m

Live Load from One Ribbed Slab on Beam=1.56 KN/m

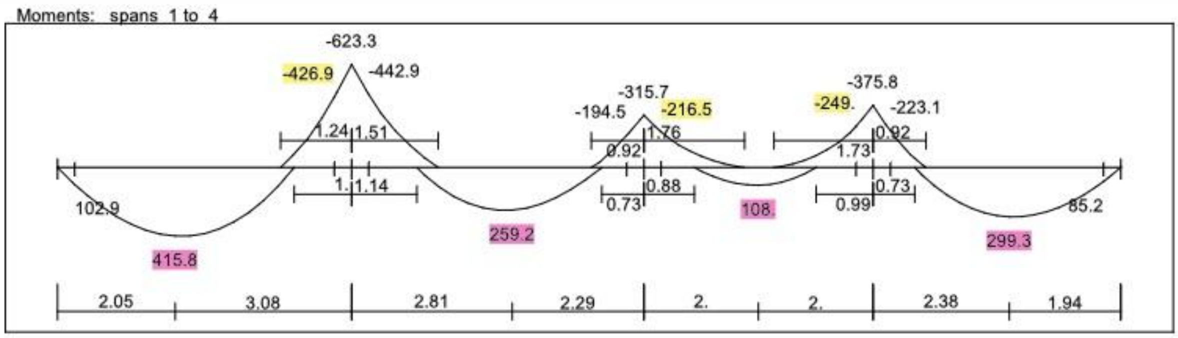
Dead load of Beam:

Material	W(KN/m)
Tiles	22×0.03×0.8=0.528
Mortar	22×0.02×0.8=0352
Sand	16×0.07×0.8=0.896
Plaster	22×0.02×(0.8+0.3)=0.484
Partitions 1.5KN/m^2	1.5×0.8=1.2
Total Dead Load ,KN	3.46

Table (4.4) : Dead load of Beam

Service DD upon the Beam=3.46+0.8×0.5×25=13.46 KN/m

Service LL upon the Beam=3×0.8=2.4 KN/m



Fig(4-5) :Moment Envelope (factored) units:KN.m

✓ Design of Positive Moment :

MAX. Positive Moment=108 KN.m

$bw = 80cm, h = 50cm$

$d = 500 - 40 - 10 - 12.5 = 437.5mm$

Mu = 108 KN .m

$Mn = \frac{Mu}{\Phi} = \frac{108}{0.9}$ = 120 KN .m

$$As_{min} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) \geq \frac{1.4}{fy}(bw)(d).$$

$$As_{min} = \frac{\sqrt{30}}{4(420)}(800)(437.5) \geq \frac{1.4}{420}(800)(437.5)$$

$As_{min} = 1141.1mm^2 < 1166.7mm^2$ …………the larger is control

$As_{min} = 1166.7mm^2$

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

Rn = $\frac{120}{800 * 437.5^2}$ = 0.784 Mpa

$m = \frac{fy}{0.85 * fc'} = \frac{420}{0.85 * 30} = 16.5$

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

$\rho = \frac{1}{16.5}(1 - \sqrt{1 - \frac{2(16.5)(0.784)}{420}}) = 0.001896$

$A_{req} = \rho * b * d = 0.001896*800 * 437.5 = 663.72$ mm²

$663.72 mm^2 < As_{min} = 1166.7mm^2$

$A_{min} =$ As

Use 3 Φ 25>> # of bar = $\frac{1166.7}{491}$ = 3

Then we select 3 bars $\Phi 25$ $A_s\, provided = 3*491 = 1472.62\,mm^2$

Check for yielding

Tension = compression

As * fy = 0.85 * f_c * b * a

$$1472.62*420 = 0.85*30*800*a$$
$$a = 30.32mm$$
$$\beta_1 = 0.85 - 0.007(fc - 28)$$
$$\beta_1 = 0.85 - 0.007(30 - 82) = 0.836$$
$$x = \frac{a}{\beta_1} = \frac{30.32}{0.836} = 36.27mm$$
$$\varepsilon_s = \frac{437.5 - 36.27}{36.27} \times 0.003$$
$$\varepsilon_s = 0.033 > 0.005$$

$$\Phi * Mn = \Phi * As * fy * (d - \frac{a}{2})$$

$\Phi * Mn = 0.9*1166.7*420*(437.5 - 15.16)$ =186.25>108 singly reinforcement

Check for spacing between the bar

S = $\frac{800 - 2*40 - 2*10 - 3*25}{2}$

S = 332.5 mm $\geq 25\,mm$

- ✓ **Design of Positive Moment :**

MAX. Positive Moment=415.8 KN.m

$$bw = 80cm, h = 50cm$$
$$d = 500 - 40 - 10 - 12.5 = 437.5mm$$

Mu= 415.8KN .m

$Mn = \frac{Mu}{\Phi} = \frac{415.8}{0.9}$ = 462 KN .m

$$As_{min} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) \geq \frac{1.4}{fy}(bw)(d).$$

$$As_{min} = \frac{\sqrt{30}}{4(420)}(800)(437.5) \geq \frac{1.4}{420}(800)(437.5)$$

$As_{min} = 1141.1mm^2 < 1166.7mm^2$ …………the larger is control

$$As_{min} = 1166.7mm^2$$

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

Rn = $\frac{462}{800*437.5^2}$ = 3.02Mpa

$m = \frac{fy}{0.85*fc'} = \frac{420}{0.85*30} = 16.5$

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

ρ = $\frac{1}{16.5}$(1 - $\sqrt{1-\frac{2(16.5)(3.02)}{420}}$) = 0.007677

A_{req} = ρ * b * d = 0.007677*800 * 437.5 = 2686.95 mm^2

$2686.95\,mm^2 > As_{min} = 1166.7mm^2$

Use 6 Φ 25>> # of bar = $\frac{2945.22}{491}$ = 6

Then we select 6 bars Φ25 $A_s\,provided = 6*491 = 2945.22mm^2$

Check for yielding

Tension = compression

As * fy = 0.85 * f_c * b * a

$2945.22 * 420 = 0.85 * 30 * 800 * a$

$a = 60.6mm$

$\beta_1 = 0.85 - 0.007(fc - 28)$

$\beta_1 = 0.85 - 0.007(30 - 82) = 0.836$

$x = \frac{a}{\beta_1} = \frac{60.6}{0.836} = 72.5mm$

$\varepsilon_s = \frac{437.5 - 72.5}{72.5} \times 0.003$

$\varepsilon_s = 0.015 > 0.005$

$\Phi * Mn = \Phi * As * fy * (d - \frac{a}{2})$

$\Phi * Mn = 0.9 * 2945.22 * 420 * (437.5 - 30.3)$ =453.33>415.8 ….. singly reinforcement

Check for spacing between the bar

S = $\frac{800 - 2*40 - 2*10 - 6*25}{5}$

S = 110 mm ≥ 25 mm

MAX. Positive Moment=259.2 KN.m

$bw = 80cm, h = 50cm$

$d = 500 - 40 - 10 - 12.5 = 437.5mm$

Mu= 259.2 KN .m

Mn= $\frac{Mu}{\Phi}$ = $\frac{259.2}{0.9}$ = 288 KN .m

$As_{min} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) \geq \frac{1.4}{fy}(bw)(d).$

$As_{min} = \frac{\sqrt{30}}{4(420)}(800)(437.5) \geq \frac{1.4}{420}(800)(437.5)$

$As_{min} = 1141.1mm^2 < 1166.7mm^2$ …………the larger is control

$As_{min} = 1166.7 mm^2$

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

Rn = $\frac{288}{800*437.5^2}$ = 1.88 Mpa

$m = \frac{fy}{0.85*fc'} = \frac{420}{0.85*30} = 16.5$

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

ρ = $\frac{1}{16.5}(1 - \sqrt{1-\frac{2(16.5)(1.88)}{420}})$ = 0.00465

A_{req} = ρ * b * d = 0.00465*800 * 437.5 = 1629.23 mm^2

$1629.23\, mm^2 > As_{min} = 1166.7 mm^2$

Use 4 Φ 25>> # of bar = $\frac{1929.23}{491}$ = 4

Then we select 4 bars Φ25 $A_s\, provided = 4*491 = 1963.5 mm^2$

Check for yielding

Tension = compression

As * fy = 0.85 * f_c * b * a

$1963.5*420 = 0.85*30*800*a$

$a = 40.42mm$

$\beta_1 = 0.85 - 0.007(fc - 28)$

$\beta_1 = 0.85 - 0.007(30 - 82) = 0.836$

$x = \frac{a}{\beta_1} = \frac{40.42}{0.836} = 48.36mm$

$\varepsilon_s = \frac{437.5 - 48.36}{48.36} \times 0.003$

$\varepsilon_s = 0.024 > 0.005$

$\Phi * Mn = \Phi * As * fy * (d - \frac{a}{2})$

$\Phi * Mn = 0.9*1963.5*420*(437.5-20.21)$ =309.72 >259.2 singly reinforcement

Check for spacing between the bar

S = $\frac{800-2*40-2*10-4*25}{3}$

S = $200\,mm \quad \geq 25\,mm$

MAX. Positive Moment=299.3 KN.m

$bw = 80cm, h = 50cm$

$d = 500-40-10-12.5 = 437.5mm$

Mu= 299.3KN .m

Mn= $\frac{Mu}{\Phi}$ = $\frac{299.3}{0.9}$ = 332.56 KN .m

$As_{\min} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) \geq \frac{1.4}{fy}(bw)(d).$

$As_{\min} = \frac{\sqrt{30}}{4(420)}(800)(437.5) \geq \frac{1.4}{420}(800)(437.5)$

$As_{\min} = 1141.1mm^2 < 1166.7mm^2$ …………the larger is control

$As_{\min} = 1166.7mm^2$

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

Rn = $\frac{332,56}{800*437.5^2}$ = 2.17 Mpa

$m = \frac{fy}{0.85*fc'} = \frac{420}{0.85*30}$ = 16.5

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

ρ = $\frac{1}{16.5}$ (1 - $\sqrt{1-\frac{2(16.5)(2.17)}{420}}$) = 0.00541

$A_{req} = \rho * b * d = 0.00541*800 * 437.5 = 1894.42\ mm^2$

$1894.42\, mm^2 > As_{min} = 1166.7 mm^2$

Use 4 Φ 25>> # of bar = $\frac{1894.41}{491} = 4$

Then we select 4 bars Φ25 $A_s\, provided = 4*491 = 1963.5 mm^2$

Check for yielding

Tension = compression

As * fy = 0.85 * f_c * b * a

$$1963.5*420 = 0.85*30*800*a$$
$$a = 40.42mm$$
$$\beta_1 = 0.85 - 0.007(fc - 28)$$
$$\beta_1 = 0.85 - 0.007(30 - 82) = 0.836$$
$$x = \frac{a}{\beta_1} = \frac{40.42}{0.836} = 48.36mm$$
$$\varepsilon_s = \frac{437.5 - 48.36}{48.36} \times 0.003$$
$$\varepsilon_s = 0.024 > 0.005$$

$$\Phi * Mn = \Phi * As * fy * (d - \frac{a}{2})$$

$\Phi * Mn = 0.9*1963.5*420*(437.5 - 20.21)$ =309.72 >259.2 singly reinforcement

Check for spacing between the bar

S = $\frac{800 - 2*40 - 2*10 - 4*25}{3}$

S = 200 mm $\geq 25\, mm$

- ✓ **Design of Negative Moment :**

MAX. Negative Moment=442.9 KN.m

$bw = 80cm, h = 50cm$
$d = 437.5mm$

$Mu = 442.9KN.m$

$Mn = \frac{Mu}{\Phi} = \frac{442.9}{0.9} = 492.11 \text{ KN.m}$

$As_{\min} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) \geq \frac{1.4}{fy}(bw)(d).$

$As_{\min} = \frac{\sqrt{30}}{4(420)}(800)(437.5) \geq \frac{1.4}{420}(800)(437.5)$

$As_{\min} = 1141.1mm^2 < 1166.7mm^2$ ....the larger is control

$As_{\min} = 1166.7mm^2$

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

$Rn = \frac{492.11}{800*437.5^2} = 3.21Mpa$

$m = \frac{fy}{0.85*fc'} = \frac{420}{0.85*30} = 16.5$

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

$\rho = \frac{1}{16.5}(1 - \sqrt{1 - \frac{2(16.5)(3.21)}{420}}) = 0.008197$

$A_{req} = \rho * b * d = 0.008197*800*437.5 = 2868.95 \text{ mm}^2$

$2868.95 mm^2 > As_{\min} = 1166.7mm^2$

Use 6 Φ 25>> # of bar = $\frac{2945.22}{491}$ = 6

Then we select 6 bars $\Phi 2, A_s\, provided = 6*491 = 2945.22mm^2$

Check for yielding

Tension = compression

As * fy = 0.85 * f_c * b * a

$$2945.22*420 = 0.85*30*800*a$$
$$a = 60.6mm$$
$$\beta_1 = 0.85 - 0.007(fc - 28)$$
$$\beta_1 = 0.85 - 0.007(30 - 82) = 0.836$$
$$x = \frac{a}{\beta_1} = \frac{60.6}{0.836} = 72.5mm$$
$$\varepsilon_s = \frac{437.5 - 72.5}{72.5} \times 0.003$$
$$\varepsilon_s = 0.015 > 0.005$$

$$\Phi * Mn = \Phi * As * fy * (d - \frac{a}{2})$$

$\Phi * Mn = 0.9*2945.22*420*(437.5 - 30.3)$ =453.33>442.9 singly reinforcement

Check for spacing between the bar

S = $\frac{800 - 2*40 - 2*10 - 6*25}{5}$

S = 110 mm $\geq 25\, mm$

MAX. Negative Moment=249 KN.m

$$bw = 80cm, h = 50cm$$
$$d = 437.5mm$$

Mu= 249 KN .m

Mn= $\frac{Mu}{\Phi}$ = $\frac{249}{0.9}$ = 276.67 KN .m

$$As_{\min} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) \geq \frac{1.4}{fy}(bw)(d).$$

$$As_{\min} = \frac{\sqrt{30}}{4(420)}(800)(437.5) \geq \frac{1.4}{420}(800)(437.5)$$

$As_{\min} = 1141.1 mm^2 < 1166.7 mm^2$ …………the larger is control

$$As_{\min} = 1166.7 mm^2$$

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

Rn = $\frac{276.67}{800 * 437.5^2}$ = 1.81Mpa

$m = \frac{fy}{0.85 * fc'} = \frac{420}{0.85 * 30} = 16.5$

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

ρ = $\frac{1}{16.5}(1 - \sqrt{1 - \frac{2(16.5)(1.81)}{420}})$ = 0.00447

A_{req} = ρ * b * d = 0.00447*800 * 437.5 = 1563.27 mm²

$1563.27\, mm^2 > As_{\min} = 1166.7 mm^2$

Use 4 Φ 25>> # of bar = $\frac{1563.27}{491}$ = 4

Then we select 4 bars Φ25 $A_s\, provided = 4 * 491 = 1963.5 mm^2$

Check for yielding

Tension = compression

As * fy = 0.85 * f_c * b * a

$1963.5*420=0.85*30*800*a$

$a=40.42mm$

$\beta_1=0.85-0.007(fc-28)$

$\beta_1=0.85-0.007(30-82)=0.836$

$x=\frac{a}{\beta_1}=\frac{40.42}{0.836}=48.36mm$

$\varepsilon_s=\frac{437.5-48.36}{48.36}\times 0.003$

$\varepsilon_s=0.024>0.005$

$\Phi*Mn=\Phi*As*fy*(d-\frac{a}{2})$

$\Phi*Mn=0.9*1963.5*420*(437.5-20.21)$ =309.71>249 singly reinforcement

Check for spacing between the bar

S = $\frac{800-2*40-2*10-4*25}{3}$

S =200 mm $\geq 25\,mm$

MAX. Negative Moment=216.5 KN.m

$bw=80cm, h=50cm$

$d=437.5mm$

Mu= 216.5 KN .m

Mn= $\frac{Mu}{\Phi}=\frac{216.5}{0.9}$ = 240.56 KN .m

$As_{min}=\frac{\sqrt{fc'}}{4(fy)}(bw)(d)\geq\frac{1.4}{fy}(bw)(d).$

$As_{min}=\frac{\sqrt{30}}{4(420)}(800)(437.5)\geq\frac{1.4}{420}(800)(437.5)$

$As_{min}=1141.1mm^2<1166.7mm^2$ …………the larger is control

$As_{min}=1166.7mm^2$

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

Rn = $\frac{240.56}{800*437.5^2}$ = 1.57Mpa

$m = \frac{fy}{0.85*fc'} = \frac{420}{0.85*30} = 16.5$

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

ρ = $\frac{1}{16.5}(1 - \sqrt{1-\frac{2(16.5)(1.57)}{420}})$ = 0.00386

A_{req} = ρ * b * d = 0.00386*800 * 437.5 = 1352.25 mm²

$1352.25\,mm^2 > As_{min} = 1166.7 mm^2$

Use 3 Φ 25>> # of bar = $\frac{1352.25}{491}$ = 3

Then we select 3 bars Φ25 $A_s\,provided = 3*491 = 1472.62\,mm^2$

Check for yielding

Tension = compression

As * fy = 0.85 * f_c * b * a

$1472.62*420 = 0.85*30*800*a$

$a = 30.32mm$

$\beta_1 = 0.85 - 0.007(fc - 28)$

$\beta_1 = 0.85 - 0.007(30 - 82) = 0.836$

$x = \frac{a}{\beta_1} = \frac{30.32}{0.836} = 36.27mm$

$\varepsilon_s = \frac{437.5 - 36.27}{36.27} \times 0.003$

$\varepsilon_s = 0.033 > 0.005$

$\Phi * Mn = \Phi * As * fy * (d - \frac{a}{2})$

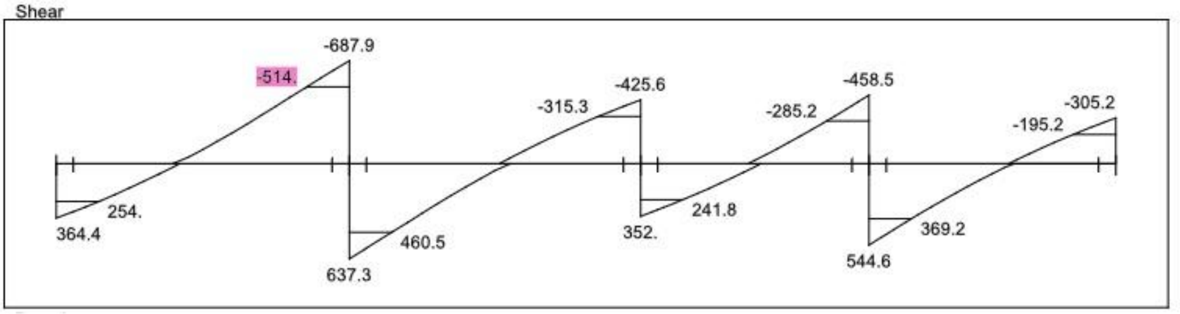
$\Phi * Mn = 0.9*1472.62*420*(437.5 - 15.16)$ =235.095>216.5 singly reinforcement

Check for spacing between the bar

$$S = \frac{800-2*40-2*10-3*25}{2}$$

$S = 312.5\,mm \quad \geq 25\,mm$

✓ **Design of shear**



Shear Envelope (factored) units:KN Fig(4-6)

$V_{u,max}$=514 KN

d= 500-40-10-12.5=437.5mm

$$Vc = \frac{\sqrt{fc'}}{6} b_w * d$$

$$Vc = \frac{\sqrt{30}}{6} *800 *437.5*10^{-3} = 319.5 \text{ KN}$$

= 0.75 * 319.5 = 239.63KN < $V_{u,max}$=514 KN

$\Phi V_{s,min}=0.75\,(\frac{1}{16})\sqrt{fc'}\ * b_w * d = 0.75\,(\frac{1}{16})\sqrt{30}\ * 800 * 437.5 = 89.\ 9KN$ -control-

$\Phi V_{s,min} = 0.75\,(\frac{1}{3}) * b_w * d = 0.75\,(\frac{1}{3}) * 800 * 437.5 = 87.5kN$

$\Phi\ Vc + \Phi V_{s,min} = 329.53\ KN < V_{u,max}$

$\Phi \tilde{V}_s = 0.75(\frac{1}{3}) * b_w * d = 0.75\,(\frac{1}{3}) * \sqrt{30} * 800 * 437.5 = 479.26kN$

$\Phi\ Vc + \Phi \tilde{V}_s = 239.63 + 479.26 = 718.89\ KN > V_{u,max} = 514\ KN$

-Stirrups are required-

Use 4 leg φ10

Av= $314.2\ mm^2$

$\tilde{V}_s = \frac{479.26}{0.75} = 639.01\ KN$

$$\frac{Av}{s} = \frac{Vs}{fy * d}$$

$S_{req} = \frac{314.2*420*437.5}{639.01*10^3} = 90.35\ mm < \frac{d}{2} = 218.75\ mm$,, S= 7.5 cm

4.7 Design Two Way Ribbed Slab

- ✓ **Determination of Thickness for Two Way Ribbed Slab:**

Assume H =35cm

$I_{b1}= \frac{100*35^3}{12}$ =357291.66 cm^4

$I_{b2}= \frac{80*35^3}{12}$ =285833.3 cm^4

$I_{b3}= \frac{100*35^3}{12}$=337291.66 cm^4

$I_{b4}= \frac{80*35^3}{12}$ =285833.3 cm^4

$Y_c= \frac{40*8*4+35*12*16}{40*8+35*12}$ =10.81 cm

$I_r= \frac{52*10.81^3}{3} - \frac{40*2.81^3}{3} + \frac{21*24.19^3}{3}$ =78219.6 cm^4

$I_{s1}= \frac{78219.6*(\frac{600}{2}+80)}{52}$ = 571604.77 cm^4

$I_{s2}= \frac{78219.6*(\frac{589}{2}+80)}{52}$ = 563331.54 cm^4

$I_{s3}= \frac{78219.6*(\frac{600}{2}+100)}{52}$ = 601689.23 cm^4

$I_{s4}= \frac{78219.6*(\frac{589}{2}+100)}{52}$ = 593416 cm^4

$\alpha_1= \frac{285833.3}{563331.54}$ = 0.51

$\alpha_2 = \frac{357291.66}{601689.23} = 0.59$

$\alpha_3 = \frac{337291.66}{593416}\ 0.57$

$\alpha_4 = \frac{285833.3}{601689.23} = 0.48$

$\alpha_{fm} = \frac{0.51+0.59+0.57+0.48}{4} = 0.538 > 0.2$

$\beta = \frac{6}{5.89} = 1.02$

$h_{min} = \frac{6000*(0.8+(\frac{420}{1400}))}{36+1.02*5*(0.538-0.2)} = 174.9\ mm$

h= 35 cm > h_{min}=17.49cm

✓ Load Calculation:

Parts of Flight	Calculation	
Tiles	22*0.03*0.52*0.52 = 0.178 KN	
Mortar	22*0.02*0.52*0.52 = 0.119KN	
Sand	16*0.07*0.52*0.52 = 0.303 KN	
Topping	25*0.08*0.52*0.52 = 0.541 KN	
Rib	25*0.27*0.12*(0.52+0.4) = 0.745 KN	
Block	9*0.27*0.4*0.4 = 0.389 KN	
Plaster	22*0.02*0.52*0.52 = 0.119 KN	
	Sum	2.80 KN

Table (4.3) : Dead load from two way ribbed slab

Dead Load of slab:

DL = $\frac{2.80}{0.52*0.52}$ =10.36 KN/m^2

W_D =1.2*10.36 =12.432 KN/m^2

LL=3 KN/m^2

W_L =1.6*3=4.8 KN/m^2

W = 12.432+4.8 =17.232 KN/ m^2

- ✓ **Moments Calculations:**

From tables use Case (9):

$C_{a.\,neg}$ = 0.0626 , , $C_{b.\,neg}$ = 0.0314

$M_{a,neg} = C_{a,neg} \times W_u \times (L_a)^2 = 0.0626 \times 17.232 \times (5.89)^2 \times 0.52 = 20.1$ KN .m / rib.

$M_{b,neg} = C_{b,neg} \times W_u \times (L_b)^2 = 0.0314 \times 17.232 \times (6)^2 \times 0.52 = 10.13$ KN .m / rib.

$C_a^+{}_{,dL}$ =0.0234 , , $C_b^+{}_{,dL}$ =0.019

$M^+_{a,d} = C_{a,dl} \times W_{ud} \times (L_a)^2 = 0.0234 \times 12.432 \times (5.89)^2 \times 0.52 = 5.25$ KN .m / rib.

$M^+_{b,d} = C_{b,dl} \times W_{ud} \times (L_b)^2 = 0.019 \times 12.432 \times (6)^2 \times 0.52 = 4.42$ KN .m / rib.

$C_a^+{}_{,LL}$ =0.031 , , $C_b^+{}_{,LL}$ =0.027

$M^+_{a,L} = C_{a,ll} \times W_{ul} \times (L_a)^2 = 0.031 \times 4.8 \times (5.89)^2 \times 0.52 = 2.68$ KN .m / rib.

$M^{+}b,L$ = Cb,ll × Wul ×(Lb)^2 = 0.027 × 4.8 × (6)^2 × 0.52= 2.43 KN .m / rib.

Ma positive = 5.25 + 2.68= 7.93KN .m / rib.

M**b** positive = 4.42+2.43 = 6.85KN .m / rib.

Negative moment at discontinuous edges = 1/3

positive.

Mb neg = (1/3) × 6.85 = 2.28 KN.m / rib.

- ✓ **Design of positive moments:**

Design of Positive Moment :- (Ma=7.39KN.m)

Assume bar diameter ø 14 for main positive reinforcement

d =h- cover - $d_{stirrups} - \frac{d_b}{2} = 350 - 20 - 8 - \frac{14}{2} = 315mm$

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{7.39\times10^6}{0.9\times520\times315^2} = 0.16Mpa$

$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85\times24} = 20.6$

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times0.16}{420}}\right) = 0.000382$

$A_{s,\ req}$ = ρ.b.d = 0.000382×520×315= 62.65 mm^2

Check for As min:-

$A_s\ \min = \frac{\sqrt{fc'}}{4(fy)}(bw)(d)$ **ACI-318 (10.5.1)**

$A_s\ \min = \frac{\sqrt{24}}{4(420)}(120)(315) = 110.22mm^2$

$A_s\ min = \frac{1.4}{(fy)}(bw)(d)$

$A_s\ min = \frac{1.4}{420}(120)(315) = 126mm^2$ .....controls

As_{req} = 62.65mm^2 <As_{min}= 126 mm^2....OK

<u>Use 2 ø10, As, provided= 157.08 mm^2>$A_{s,\ required}$= 126mm^2... Ok</u>

Check for strain:-

$$a = \frac{A_{s}.f_y}{0.85b\ f_c'} = \frac{157.08 \times 420}{0.85 \times 520 \times 24} = 6.22\ mm$$

$$x = \frac{a}{\beta_1} = \frac{6.22}{0.85} = 7.32mm$$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{315-7.32}{7.32}\right) = 0.126 > 0.005 \qquad 0k$$

Design of Positive Moment:- (Mb=6.85KN.m)

$$d = h - cover - d_{stirrups} - \frac{d_b}{2} = 350 - 20 - 8 - \frac{14}{2} = 315\ mm$$

$$R_n = \frac{M_u}{\emptyset b d^2} = \frac{6.85 \times 10^6}{0.9 \times 520 \times 315^2} = 0.148Mpa$$

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

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.148}{420}}\right) = 0.000354$$

$A_{s,\ req}$ = ρ.b.d = 0.000354 ×520×315 = 57.99 mm^2

Check for As min:-

$A_s \min = \frac{\sqrt{fc'}}{4(fy)}(bw)(d)$ **ACI-318 (10.5.1)**

$A_s \min = \frac{\sqrt{24}}{4(420)}(120)(315) = 110.22 mm^2$

$A_s \min = \frac{1.4}{(fy)}(bw)(d)$

$A_s \min = \frac{1.4}{420}(120)(315) = 126 mm^2$ .....controls

As_{req} = 57.99mm^2 <As_{min}= 126 mm^2 .....OK

Use 2 ø10, As, provided= 157.08 mm^2>$A_{s,\ required}$= 126mm^2... Ok

Check for strain:-

$a = \frac{A_{s}.f_y}{0.85 b\ f_c'} = \frac{157.08 \times 420}{0.85 \times 520 \times 24} = 6.22\ mm$

$x = \frac{a}{\beta_1} = \frac{6.22}{0.85} = 7.32 mm$

$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{315-7.32}{7.32}\right) = 0.126 > 0.005 \qquad 0k$

- ✓ **Design of negative moments:**

Design of Negative Moment :- (Ma=-20.1KN.m)

Assume bar diameter ø 12 for negative reinforcement

d =h- cover - $d_{stirrups} - \frac{d_b}{2} = 350 - 20 - 8 - \frac{12}{2} = 316\ mm$

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{20.1\times10^6}{0.9\times120\times316^2} = 1.86\ Mpa$

$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85\times24} = 20.6$

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times1.86}{420}}\right) = 0.0047$

$A_{s,\ req}$ = ρ.b.d = 0.0047 ×120×316 =178.22 mm^2

Check for As min:-

$As\ \min = \frac{\sqrt{fc'}}{4(fy)}(bw)(d)$ **ACI-318 (10.5.1)**

$As\ \min = \frac{\sqrt{24}}{4(420)}(120)(316) = 110.5 mm^2$

$As\ \min = \frac{1.4}{(fy)}(bw)(d)$

$As\ \min = \frac{1.4}{420}(120)(316) = 126.4 mm^2$ ….controls

As_{req} = 178.22 > As_{min}= 126.4mm^2 ….OK

<u>Use 2 ø12, $As_{,\ provided}$= 226.2 mm^2>$A_{s,\ required}$= 178.22mm^2… Ok</u>

Check for strain:-

$a = \frac{A_s.f_y}{0.85 b\ f_c'} = \frac{226.2\times420}{0.85\times120\times24} = 38.81\ mm$

$x = \frac{a}{\beta_1} = \frac{38.81}{0.85} = 45.66\ mm$

$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{316-45.66}{45.66}\right) = 0.0178 > 0.005$ 0k

✓ Design of Negative Moment :-

(Mb=-10.13KN.m)

Assume bar diameter ø 12 for negative reinforcement

d =h- cover - $d_{stirrups} - \frac{d_b}{2} = 350 - 20 - 8 - \frac{12}{2} = 316\ mm$

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{10.13\times10^6}{0.9\times120\times316^2} = 0.94\ Mpa$

$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85\times24} = 20.6$

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2\times20.6\times0.94}{420}}\right) = 0.00229$

$A_{s,\ req}$ = ρ.b.d = 0.00229 ×120×316 = 86.92 mm^2

Check for As min:-

$As\ min = \frac{\sqrt{fc'}}{4(fy)}(bw)(d)$ **ACI-318 (10.5.1)**

$As\ min = \frac{\sqrt{24}}{4(420)}(120)(316) = 110.57 mm^2$

$As\ min = \frac{1.4}{(fy)}(bw)(d)$

$As\ min = \frac{1.4}{420}(120)(316) = 126.4 mm^2$ ....controls

As_{req} = 86.92 < As_{min}= 126.4mm^2 .....OK

Use 2 ø12, $As_{,\ provided}$= 226.2 mm^2>$A_{s,\ required}$= 126.4mm^2... Ok

Check for strain:-

a $=\frac{A_{s}.f_{y}}{0.85b\ f_{c}'}=\frac{226.2\times420}{0.85\times120\times24}=38.81\ mm$

x$=\frac{a}{\beta_1}=\frac{38.81}{0.85}=45.66\ mm$

$\varepsilon_s=0.003\left(\frac{d-x}{x}\right)=0.003\left(\frac{316-45.66}{45.66}\right)=0.0178>0.005$ 0k

- ✓ **Design of shear for rib:**

Maximum shear coefficient in short direction as in case (9) W_a @ m =0.98

W_a = 0.686

The total load on the panel =5.89*6*17.232= 608.98 KN

The load per rib at the face of long beam =0.686*608.98*0.52/(2*6)= 18.1 KN

W_b = 0.306

The total load on the panel =5.89*6*17.232= 608.98 KN

The load / rib at the face of long beam =0.306*608.98*0.52/(2*5.89) = 8.23KN

V_{ud} = V_{uface} - Wu*b_f*d = 18.1 – 17.232*0.52*0.316= 15.27 KN

The shear strength of one rib:

$V_c=\frac{1.1}{6}\sqrt{f_c'}b_w d=\frac{1.1}{6}\sqrt{24}\times120\times316\times10^{-3}=34.05\ KN$

$øV_c$ =0.75×34.05 =25.54KN

0.5 ø V_c =0.5×25.54=12.77KN

0.5 ø V_c< V_{ud}< ø V_c

12.77<15.27<25.54

Minimum shear reinforcement is required except for joist construction.

V_{ud} = 17.232*0.52*(3.8 - 0.316) = 31.3 KN

øV_c =25.54 < Vud =31.3KN

Φ Vsmin≥ 0.75 ($\frac{1}{3}$) * bw * d =0 .75* ($\frac{1}{3}$)*120*316*10^{-3} = 9.48 KN ….Controls

Φ Vsmin ≥ 0.75 ($\frac{\sqrt{fc'}}{16}$) * bw * d = 0.75*($\frac{\sqrt{24}}{16}$) * 120* 316*10^{-3} = 8.7KN

Φ (Vc+ Vsmin)= 35.021>Vu= 31.3 > ΦVc = 25.54

$\mathrm{Vs}' = \frac{1}{3}\sqrt{fc'} * \mathrm{bw} * \mathrm{d}$ = 61.9 KN

$\mathrm{Vs} = \frac{\mathrm{Vu}}{\Phi} - \mathrm{Vc}$ = 12.92 KN

Vsmin =11.4 <Vs =12.92 < Vs' =55.85 ………**Case IV**

Use 2 leg Φ 8

$A_{v\,min}$ =100 mm^2

$$\frac{A_{v\,min}}{s} = \frac{v_s}{df_{yt}}$$

$$s = \frac{A_{v\,\min}}{v_s}\,df_{yt}$$

S=926.5mm

$$s_{max} \le \frac{d}{2} = \frac{285}{2} = 142.5\,mm \qquad or\ s_{max} \le 600\,mm$$

Use 4 leg Φ 8 @125 mm

4.7 Design of column (C36)

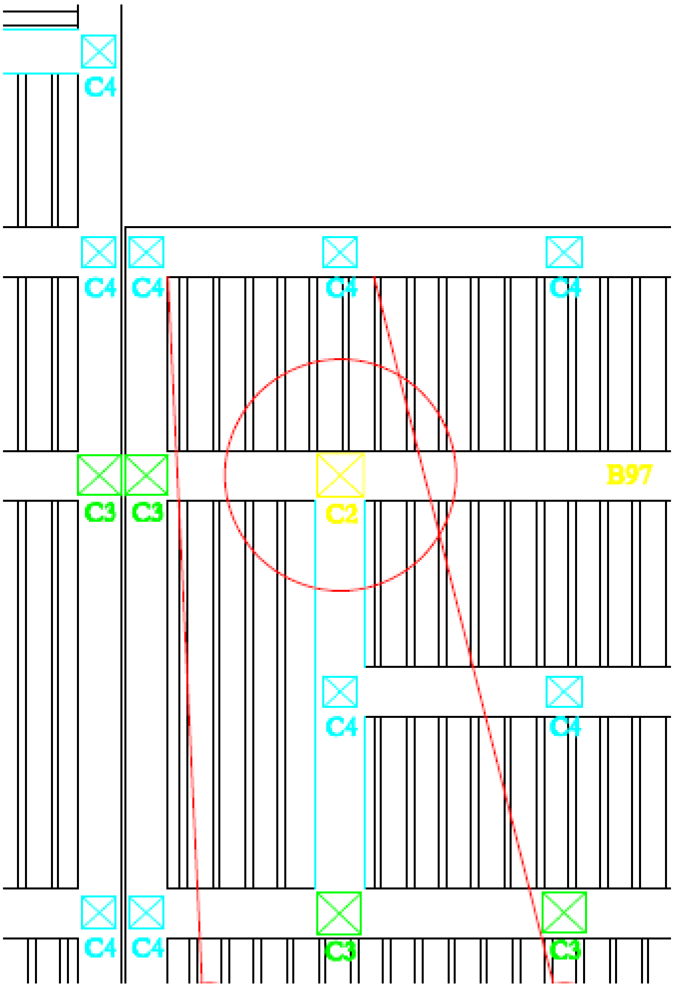


Fig.(37) :Place Of Column (C36) within the second basement floor.

Load Calculation for Column

Column	Column Dimensions	fc'	fy
Col. C36	60cm* 45cm	24Mpa	420Mpa

✓ Load Calculation:

$$Pu = 1941.6KN$$
$$Use \rightarrow \rho = \rho g = 2\%$$
$$Pu = 0.65 * 0.8 * \{0.85 * fc'(Ag - Ast) + Ast(fy)\}$$
$$1941.6 * 10^3 = 0.65 * 0.8 * [0.85 * 24 * (Ag - 0.02Ag) + 0.02Ag * 420]$$
$$Ag = 133389.76.mm^2$$
$$Ag = 600 * a$$
$$133389.76 / 600 = a$$
$$a = 222.3mm$$

Use $600 \times 250mm$ with Ag = 150000mm²

Pu(KN)	ρg	Ag, provided	a(mm)	Ag ,required
1941.6	0.02	150000 mm^2	222.3	133389.76 mm^2

✓ Selecting longitudinal bars:

$$Pu = 0.65 * 0.8 * \{0.85 * fc'(Ag - Ast) + Ast(fy)\}$$
$$1941.6 * 10^3 = 0.65 * 0.8 * [0.85 * 24 * (150000 - Ast) + Ast * 400]$$
$$Ast = 8737.6mm^2$$

Take 18Φ 25 As,provided = 8834.4 mm^2 **> As,req = 8737.6** mm^2

$$\rho g = \frac{Ast}{Ag} = \frac{8834.4}{525000} = 0.017$$

Φ	Ast, required	ρg
0.65	8737.6 mm^2	0.017

✓ Design of Ties:

- Use ties Φ10 with spacing of ties shall not exceed the smallest of

1. 48 * ds = 48 * 10 = 480mm
2. 16 * db = 16 * 25 = 400 mm - control
3. the least dimension of the column = 700 mm

Use ties Φ10 @ 200mm

ds(mm)	db(mm)	S(mm)
Φ10	Φ25	200

✓ Check for code requirements:

1. Clear Spacing $= \frac{750-40*2-10*2-6*25}{5}$ =100 mm > 40mm>1.5db=1.5*25=37.5mm - OK

2. 0.01< ρg = 0.017 <0.08 - OK

3. Number of bars 18>4 for rectangular section – OK

4. Minimum tie diameter ds =Φ10 for db = Φ25 bars – OK

5. Arrangement of ties 100 mm<150mm – OK

Clear Spacing	No. of bars	ρg	ds (mm)	db (mm)
100 mm	18	0.017	Φ10	Φ25

✓ Check Slenderness Effect:

$$\frac{klu}{r} < 34 - 12\frac{M1}{M2} \quad \ldots\ldots\ldots\ldots ACI-(10.12.2)$$

Lu: Actual unsupported (un braced) length.

K: effective length factor (K= 1 for braced frame).

R: radius of gyration = 0.3 h = $\sqrt{\frac{I}{A}}$

Lu = 3.5 m

M1/M2 =1 (Braced frame with M,min)

K=1 , According to ACI 318-02 The effective length factor, k, shall be permitted to be taken as 1.0.

$$\frac{klu}{r} < 34 - 12\frac{M1}{M2} = 22 < 40 \quad .... \; ..... ACI - (10.12.2)$$

$$\frac{klu}{r} = \frac{1*3.5}{0.3*0.750} = 15.55 < 22 < 40 ......$$

…..short column.

Short column in both direction

Lu (m)	M1/M2	K	$\frac{klu}{r}$
3.5	1.0	1.0	15.55

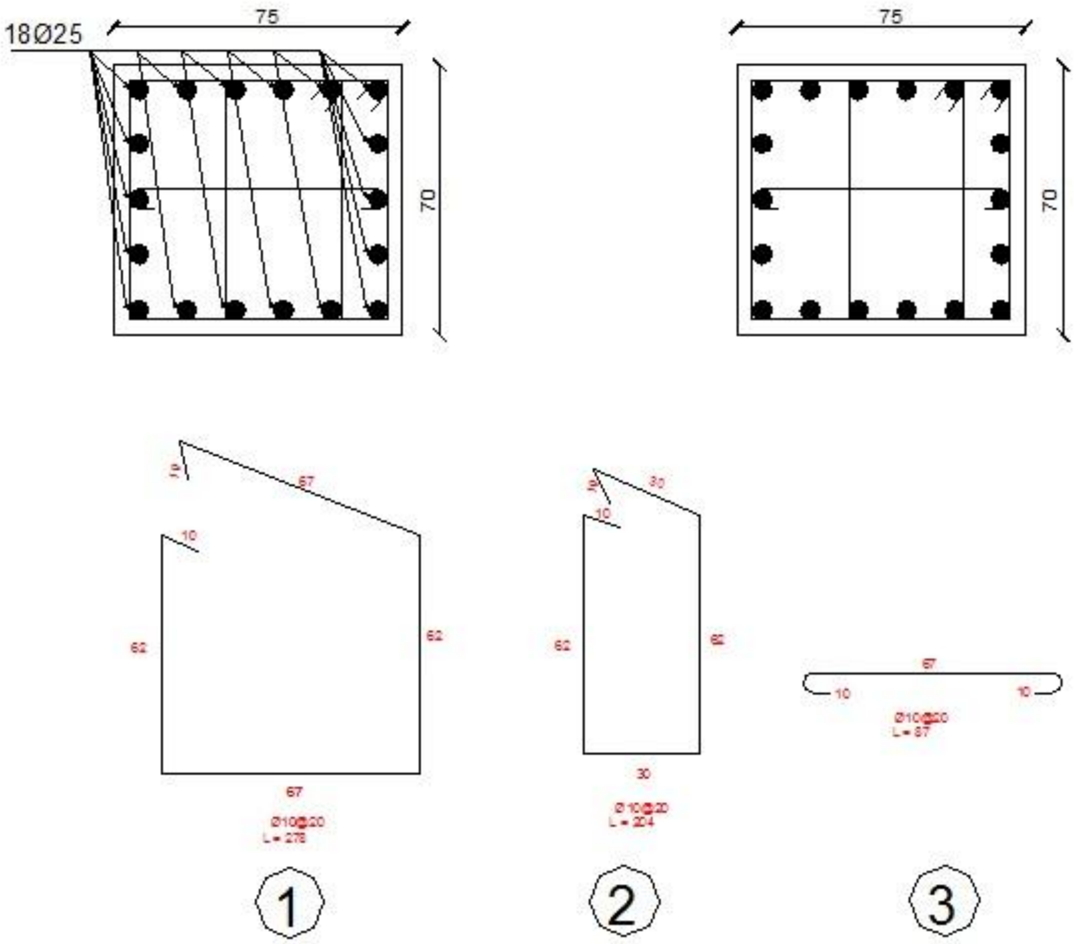


Fig. (38):Section of Column (C2)