4

Chapter 4

Structural Analysis And Design

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4.1 Introduction:

Many structures are built of reinforced concrete: bridges, buildings, retaining walls, tunnels, and others. Reinforced concrete is logical union of two materials: plain concrete, which possesses high compressive strength but little tensile strength, and steel bars embedded in the concrete, which can provide the needed strength in tension. Plain concrete is made by mixing cement, fine aggregate, coarse aggregate, water, and frequently admixtures. Understanding of reinforced concrete behavior is still far from complete, building codes and specifications that give design procedures are continually changing to reflect latest knowledge.

Structural concrete can be classified into:

- Lightweight concrete with unit weight from about 1350 to 1850 kg/m^3.
- Normal weight concrete with unit weight from about 1800 to 2400 kg/m^3.
- Heavyweight concrete with unit weight from about 3200 to 5600 kg/m^3.

4.2 Design method and requirements:

The design strength provided by a member is calculated in accordance with the requirements and assumptions of **ACI _ code (318_08)**.

- ✓ **Strength design method:**

In ultimate strength design method, the service loads are increased by factors to obtain the load at which failure is considered to be occurring.

This load called factored load or factored service load. The structure or structural element is then proportioned such that the strength is reached when factored load is acting. The computation of this strength takes into account the nonlinear stress-strain behavior of concrete.

The strength design method is expressed by the following,

Strength provided ≥ strength required to carry factored loads.

NOTE:

The statically calculation and the key plans dependent on the architectural plans.

- ✓ **Code** : ACI 2008

  UBC

- ✓ **Material** :

Concrete: B300…. $Fcu = 30N/mm^2 (MPa)$ For circular section

but for rectangular section ($fc' = 30*0.8 = 24MPa$) .

Reinforcement steel: The specified yield strength of the reinforcement{fy =420 N/mm² (MPa)}

- ✓ **Factored loads:**

The factored loads for members in our project are determined by:

$W_u = 1.2\ D_L + 1.6\ L_L$ ACI-code-318-08(9.2.1).

4.3 Check of Minimum Thickness of Structural Member:

Table (4. 1) MINIMUM THICKNESS OF NONPRESTRESSED BEAMSOR ONE-WAY SLABS UNLESS DEFLECTIONS ARE CALCULATED. (ACI 318M-11)

Minimum thickness (h)				
Member	Simply Supported	One end continuous	Both end continuous	**Cantilever**
solid one way slabs	L/20	L/24	L/28	**L/10**
Beams or ribbed one way slabs	**L/16**	**L/18.5**	**L/21**	**L/8**

- **For Rib :**

  $h_{min}$for (one end continuous)=L/18.5=625/18.5=33.8cm

  $h_{min}$for (both end continuous)=L/18.5=703/21=33.5cm

- **ForBeam :**

  h_{min} for(one end continuous)=L/18.5=507/18.5= 27.4cm

  $h_{min}$for(both end continuous)=L/21=600/21=28.6cm

The minimum thickness will be h_{min}=35cm select 35cm for rib slab with hidden beam

h=35cm (27 cm Hollow Block+8 cm Topping)

4.4 Design of topping:

✓ Statically system for topping :

Consider the topping as strip of (1m) width, and span of mold length with both end fixed in the ribs.

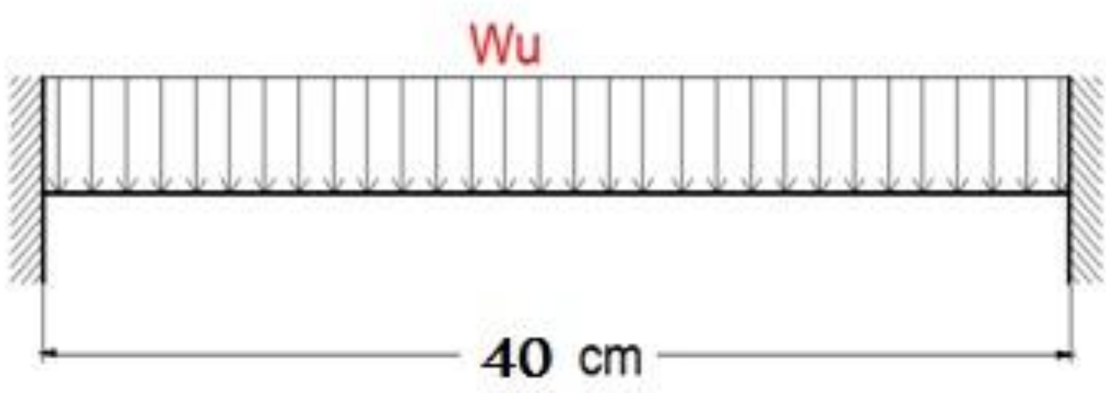


Figure 4. 1 topping load.

✓ Load calculations:

Dead load calculations:

Table (4. 2) Dead load calculation Topping

Dead load from:	δ×γ×1	KN/m
Tiles	0.03×23×1	0.69
Mortar	0.02×22×1	0.44
Coarse sand	0.07×17×1	1.19
Topping	0.08×25×1	2
Interior partitions	2.3×1	2.3
	∑	6.62KN/m

- **Live Load :**

  L_L =4 KN/m^2

L_L =4 KN/m^2×1m=4KN/m

- **Factored Load :**

W_U = 1.2 ×6.62 + 1.6×4 =14.344 KN/m

Check the strength condition for plain concrete, $øM_n \geq M_u$, where ø = 0.55

$M_n = 0.42\ \lambda \sqrt{f_c'}\ S_m$ **(ACI 22.5.1, equation 22-2)**

$$S_m = \frac{b.h^2}{6} = \frac{1000.80^2}{6} = 1066666.67\ mm^2$$

$øM_n$ =0.55×0.42×1×$\sqrt{24}$ ×1066666.67 ×10^{-6} =1.21KN.m

$M_u = \frac{W_u L^2}{12} = 0.191$ KN.m (negative moment)

$M_u = \frac{W_u L^2}{24} = 0.0956$KN.m (positive moment)

$øM_n >> M_u =$ 0.191 KN.m

No reinforcement is required by analysis. **According to ACI 10.5.4**, provide $A_{s,min}$ for slabs as shrinkage and temperature reinforcement.

$\rho_{shrinkage}$= 0.0018 **ACI 7.12.2.1**

$A_s = \rho \times b \times h_{topping}$ =0.0018 ×1000×80 = 144 mm^2/m

Step (s) is the smallest of:

1. 3h = 3×80 =240 mm control by ACI 10.5.4
2. 450mm.
3. S =$380\left(\frac{280}{f_s}\right) - 2.5C_c = 380\left(\frac{280}{\frac{2}{3}420}\right) - 2.5\ .20 = 330mm$ ACI 10.6.4 OR

S ≤$300\left(\frac{280}{f_s}\right)$=300 mm

<u>Take ø 8 @ 200 mm in both direction , S = 200 mm <S_{max} = 240 mm … OK</u>

4.5 Design of One-Way Ribbed Slab(R1) :

Requirements For Ribbed Slab Floor According to ***ACI- (318-08)*** **.**

$b_w \geq 10$cm…………………………………………ACI(8.13.2)

Select b_w=12cm

$h \leq 3.5*b_w$ ……………………………….…….. ACI(8.13.2)

Select h=35cm<3.5*12=42 cm

$t_f \geq L_n/12 \geq 50$mm ……………………………….ACI(8.13.6.1)

Select t_f=8cm

- ✓ **Statically system and Dimensions**

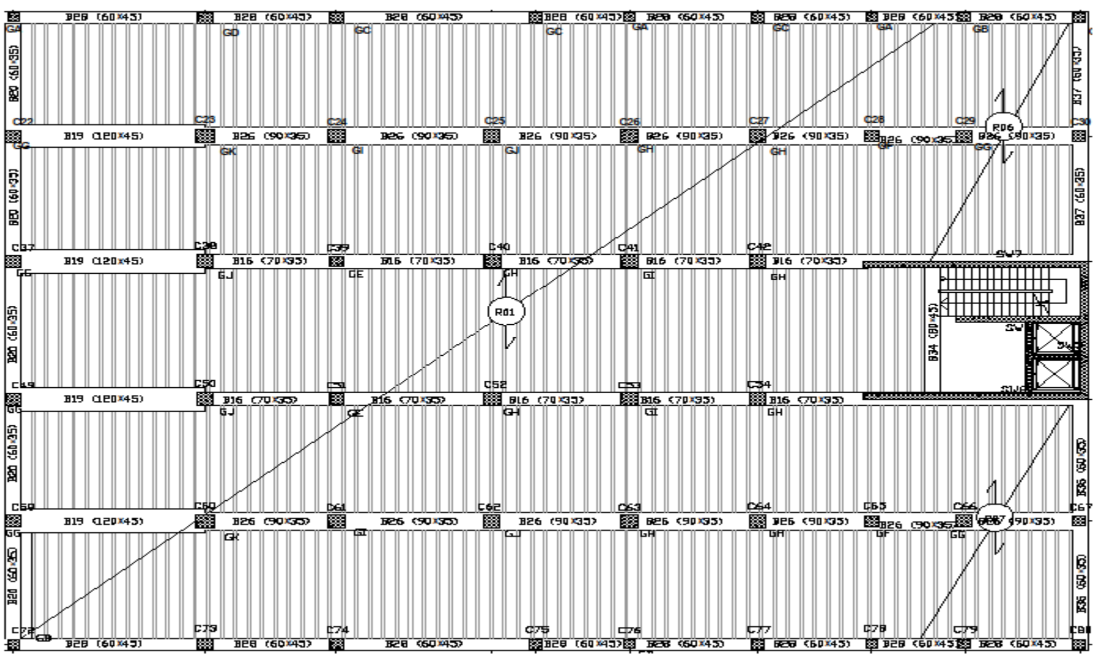


Figure 4. 2 One Way Rib slab (R1)

Load calculations :

- **Dead load:**

Table (4. 3) Dead load calculation Topping of ribS

Dead load from:	h×γ×b	KN/m
Tiles	0.03×23×0.52	0.359
Mortar	0.03×22×0.52	0.343
Coarse sand	0.07×17×0.52	0.619
Topping	0.08×25×0.52	1.04
R.c rib	0.27×25×0.12	0.81
Hollow block	0.27×10×0.4	1.08
Plaster	0.03×22×0.52	0.343
Interior partitions	2.3×0.52	1.196
	∑	5.79 KN/m

Dead load /rib = 5.79 KN/m

- **Live load =4KN/M^2**

Live load /rib = 4KN/m^2 × 0.52m = 2.08 KN/m.

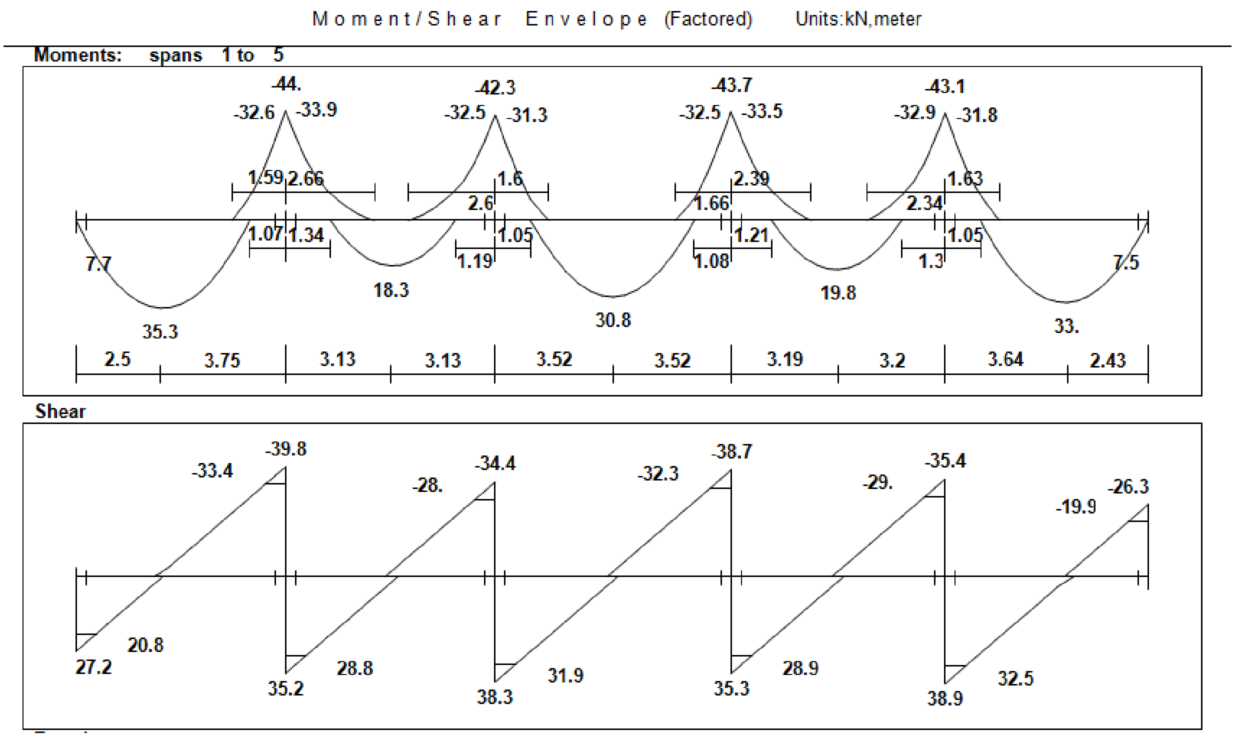
- **<u>The effective flange (be) :</u>**

1) be $\leq \frac{L}{4} = \frac{5400}{4} = 1350$ mm

2) be≤ bw +16hf=120+16*80=1400mm

3) be≤ center to center spacing between adjacent beam = $\frac{400}{2} + \frac{400}{2} + 120 = 520mm$

Take be=520 mm



Reactions

Factored						
DeadR	17.82	49.77	47.05	48.11	49.34	17.15
LiveR	9.4	25.29	25.65	25.87	25.06	9.19
Max R	27.22	75.06	72.71	73.98	74.39	26.34
Min R	16.71	58.9	55.4	57.02	58.82	15.93
Service						
DeadR	14.85	41.47	39.21	40.09	41.11	14.29
LiveR	5.87	15.8	16.03	16.17	15.66	5.74
Max R	20.73	57.28	55.25	56.26	56.77	20.04
Min R	14.16	47.18	44.43	45.66	47.04	13.53

Figure 4. 3 Shear & Moment Envelope Diagram (R1)

✓ Design of positive moment:

1) M_u = 35.3 , 30.8 , 33 KN.m.

Assume bar diameter ø 14 for main positive reinforcement.

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

Check if a>h_f to determine whether the section will act as rectangular or T- section,

$M_{nf} = 0.85. f_c'. b_e. h_f. (d - \frac{h_f}{2})$

$= 0.85 \times 24 \times 520 \times 80 \times \left(315\ - \frac{80}{2}\right) \times 10^{-6} = 233.37\ KN.m$

$M_{nf} \gg \frac{M_u}{\varphi} = \frac{35.3}{0.9} =$ 39.2 KN.m

the section will be designed as rectangular section with b_e =520 mm.

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{35.4 \times 10^6}{0.9 \times 520 \times 315^2} = 0.76\ Mpa.$

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

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.76}{420}}\right) = .001844$

$A_{s,req}$ = ρ.b.d = 0.001844×520×315 = 302.14mm^2

- **Check for $A_{s,min}$.**

$A_{s,min}$ is the maximum of :-

$A_{s,min} = 0.25 \frac{\sqrt{f'_c}}{f_y} b_w. d \geq \frac{1.4}{f_y} b_w. d$

1. $A_{s,min} = 0.25 \frac{\sqrt{24}}{420}\ 120 \times 315 = 110.23\ mm^2$
2. $A_{s,min} = \frac{1.4}{420} 120 \times 315 = 126\ mm^2\ \ Control$

A_s = 302.14 $mm^2 \geq A_{s,min}$ = 126 mm^2

Use 2ø14 ,$A_{s,provided}$= 307.87 $mm^2 > A_{s,required}$= 302.14 mm^2.   Ok

Check for strain:

a $=\frac{A_{s}.f_y}{0.85b\, f_c'} = \frac{307.87\times420}{0.85\times520\times24} = 12.19\ mm$

c$= \frac{a}{\mathcal{B}_1} = \frac{12.19}{0.85} = 14.34\ mm$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{315-14.34}{14.34}\right) = 0.94 > 0.005 \quad 0k$$

2) M_u = 19.8 , 18.3 KN.m.

Assume bar diameter ø 14 for main positive reinforcement.

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

Check if a>h_f to determine whether the section will act as rectangular or T- section,.

M_{nf} =0.85$. f_c'. b_e. h_f. (d - \frac{h_f}{2})$

$= 0.85 \times 24 \times 520 \times 80 \times \left(316\ - \frac{80}{2}\right) \times 10^{-6} = 234.22\ KN.m$

$M_{nf} \gg \frac{M_u}{\varphi} = \frac{19.8}{0.9} =$ 22 KN.m

the section will be designed as rectangular section with b_e =520 mm.

$R_n = \frac{M_u}{\emptyset bd^2} = \frac{19.8\times10^6}{0.9\times520\times316^2} = 0.424\ Mpa.$

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

$\rho= \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times0.424}{420}}\right) = 0.00102$

$A_{s,req}$ = ρ.b.d = 0.00102×520×316 = 167.65mm^2

- **Check for $A_{s,min}$.**

$A_{s,min}$is the maximum of :-

$A_{s,min}$ =$0.25\frac{\sqrt{f'_c}}{f_y}b_w.d \geq \frac{1.4}{f_y}b_w.d$

3. $A_{s,min}$ =$0.25\frac{\sqrt{24}}{420}\ 120 \times 315 = 110.227\ mm^2$
4. $A_{s,min}$ =$\frac{1.4}{420}120 \times 315 = 126mm^2\ \ Control$

A_s = 167.65 mm^2≥$A_{s,min}$ = $126.4mm^2$

Use 2ø12 ,$A_{s,provided}$= 226.19 mm^2>$A_{s,required}$= 167.65mm^2.   Ok

- **Check for strain:**

a =$\frac{A_{s}.f_y}{0.85b\ f'_c} = \frac{226.19\times420}{0.85\times520\times24} = 8.95\ mm$

c= $\frac{a}{\mathcal{B}_1} = \frac{8.95}{0.85} = 10.53\ mm$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{316-10.53}{10.53}\right) = 0.087\ > 0.005\quad 0k$$

✓ **<u>Design of negative moment for face of support :</u>**

1. **M_u = -33.9 , 33.5 KN.m.**

Assume bar diameter ø 16 for main positive reinforcement.

d =h- cover - $d_{stirrups}$$-\frac{d_b}{2} = 350 - 20 - 8 - \frac{16}{2} = 314\ mm.$

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{33.9\times10^6}{0.9\times120\times314^2} = 3.18\ Mpa.$

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

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times3.18}{420}}\right) = 0.00828$

$A_{s,req} = \rho.b.d = 0.00828 \times120\times314 = 311.88mm^2$

- **Check for $A_{s,min}$.**

$A_{s,min}$ is the maximum of :-

$A_{s,min} = 0.25\frac{\sqrt{f'_c}}{f_y} b_w.d \geq \frac{1.4}{f_y} b_w.d$

1. $A_{s,min} = 0.25\frac{\sqrt{24}}{420}\ 120\times314 = 109.8\ mm^2$
2. $A_{s,min} = \frac{1.4}{420} 120\times314 = 125.6\ \ mm^2\ Control$

$A_s = 311.88\ mm^2 \geq A_{s,min} = 125.6 mm^2$

Use 2ø16 , $A_{s,provided}$ = 402 mm^2 > $A_{s,required}$ = 311.88 mm^2. Ok

- **Check for strain:**

$a = \frac{A_s.f_y}{0.85b\ f_c'} = \frac{402\times420}{0.85\times120\times24} = 68.97\ mm$

$c = \frac{a}{\mathcal{B}_1} = \frac{68.97}{0.85} = 81.14\ mm$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{314-81.14}{81.14}\right) = 0.0086 > 0.005 \qquad 0k$$

2. M_u = -32.9 , 32.5 KN.m.

Assume bar diameter ø 14 for main positive reinforcement.

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

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{32.9\times10^6}{0.9\times120\times315^2} = 3.07\ Mpa.$

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

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2\times20.6\times3.07}{420}}\right) = 0.00796$

$A_{s,req}$ = ρ.b.d = 0.00796 ×120×315 = 300.98mm^2

- **Check for $A_{s,min}$.**

$A_{s,min}$ is the maximum of :-

$A_{s,min} = 0.25\frac{\sqrt{f'_c}}{f_y} b_w.d \geq \frac{1.4}{f_y} b_w.d$

3. $A_{s,min} = 0.25\frac{\sqrt{24}}{420}\ 120\times315 = 110.2\ mm^2$
4. $A_{s,min} = \frac{1.4}{420} 120\times315 = 126\ \ mm^2\ Control$

A_s = 300.98 mm^2 ≥ $A_{s,min}$ = 126 mm^2

Use 2ø14 ,$A_{s,provided}$= 307.88 mm^2>$A_{s,required}$= 300.98 mm^2. Ok

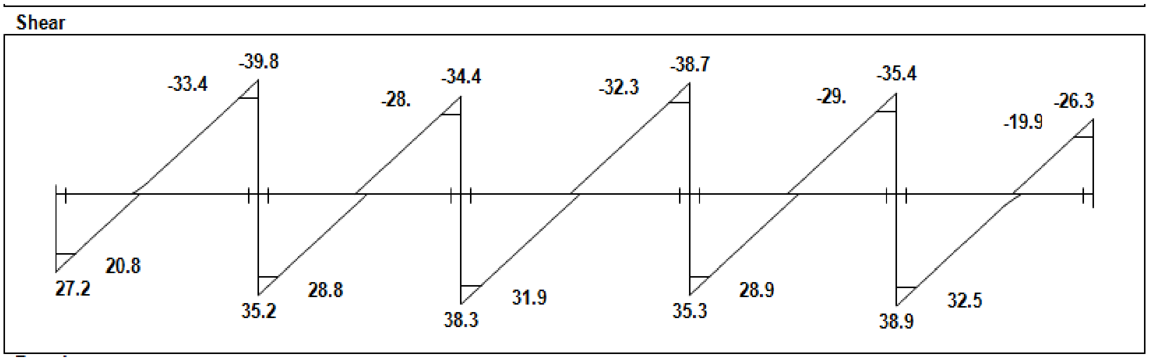
- **Check for strain:**

$a = \frac{A_s.f_y}{0.85 b\ f_c'} = \frac{307.88\times420}{0.85\times120\times24} = 52.82\ mm$

$c = \frac{a}{\mathcal{B}_1} = \frac{52.82}{0.85} = 62.14\ mm$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{315-62.14}{62.14}\right) = 0.0122 > 0.005 \qquad 0k$$

✓ Shear Design for (R1):



V_u at distance d from support= 33.4 KN

Shear strength V_c, provided by concrete for the joists may be taken 10% greater that for beams. This is mainly due to the interaction between the slab and closely spaced ribs. (ACI, 8.13.8).

$V_c = \frac{1.1}{6} \lambda \sqrt{f_c'} b_w d = \frac{1.1}{6}\sqrt{24} \times 120 \times 316 \times 10^{-3} = 34.05 KN$

ϕV_c =0.75×34.05 =25.5 KN.

0.5 ϕ V_c =0.5×25.5=12.77 KN

0.5 ϕ V_c< V_u< ϕ V_c ……………. NOT OK

✓ <u>Shear Design for (R1)</u>:

$V_{u,max}$ = 33.4 KN.

d= $h - cover - d_{stirrup} - \frac{d_b}{2} = 350 - 40 - 10 - \frac{14}{2} = 286\ mm.$

$V_c = \frac{1}{6}\sqrt{f_c'} b.d = \frac{1}{6}\sqrt{24} \times 120 \times 286 \times 10^{-3} = 28.022\ KN$

Check for section dimensions:

$V_s = \frac{V_u}{\phi} - V_c = \frac{33.4}{0.75} - 28.022 = 16.5 KN$

$V_{s,max} = \frac{2}{3}\sqrt{f_c'} b.d = \frac{2}{3}\sqrt{24} \times 120 \times 286 \times 10^{-3} = 112.08\ KN$

$V_s < V_{smax}$ so the section is large enough.

Check for the case of shear:

$V_{s,min} = \frac{1}{16}\sqrt{f_c'}b.d \quad OR \; = \; \frac{1}{3}\; b.d \quad which\ is\ larger.$

$V_{s,min} = \frac{1}{16}\sqrt{24} \times 120 \times 286 \times 10^{-3} \; = 10.5\ KN$

$V_{s,min} = \frac{1}{3} \times 120 \times 286 \times 10^{-3} = 11.44 \quad \boldsymbol{cont}.$

$Ø(V_{smin} + V_c) = 0.75(11.44 + 28.022) = 29.6$ KN.

$V_u > Ø(V_{smin} + V_c)$ **case(III) for shear design.**

$S_{max} \le \frac{d}{2} = \frac{286}{2} = 143$ mm OR $S_{max} \le 600mm$

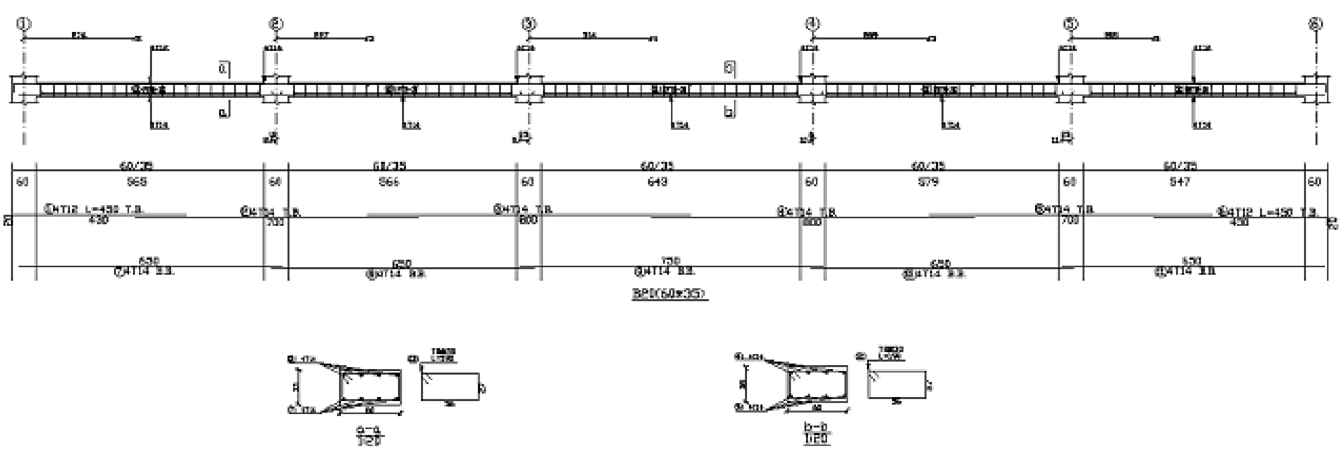
$S_{max} = 143$ mm cont.

By using ϕ 10 double legs stirrups, $A_v = 157.1\ mm^2$

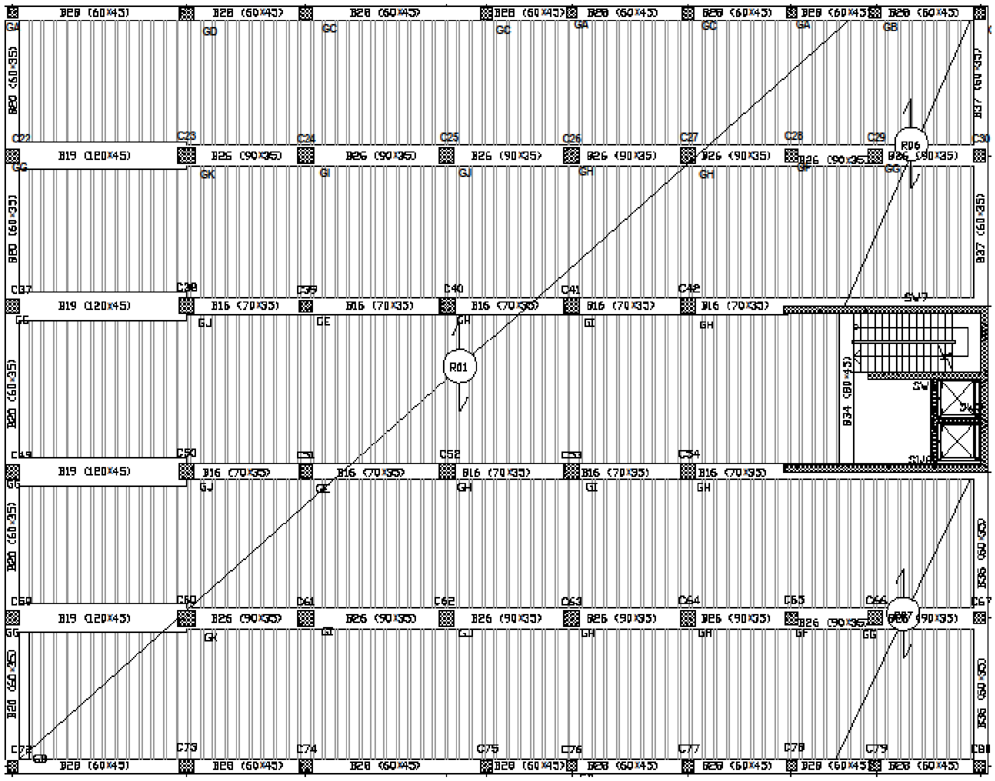
$$s = \frac{A_v f_{yt}}{V_s} d = \frac{157.1 \times 420 \times 286}{112.08 \times 1000} = 168.4\ mm$$

Use 2 leg ϕ10 @150mm

For all spans 2 leg ϕ10 @150mm will be used for stirrups.



4.6 Design of Beam(B16) :



Load calculations:

Load calculations for B16:

Dead Load Calculations for Beam(B16):-

Table (4. 4)Dead Load Calculations for Beam(B16)

Dead load from:	h×γ×1	KN/m
Tiles	0.03×23×1	0.69
Mortar	0.03×22×1	0.66
Coarse sand	0.07×17×1	1.19
Reinforced concrete	0.35×25×1	8.75
Plaster	00.2×22×1	0.44
	∑	11.7 KN/m

The distributed Dead and Live loads acting upon B1 can be defined from the support reactions of the R1

From R1

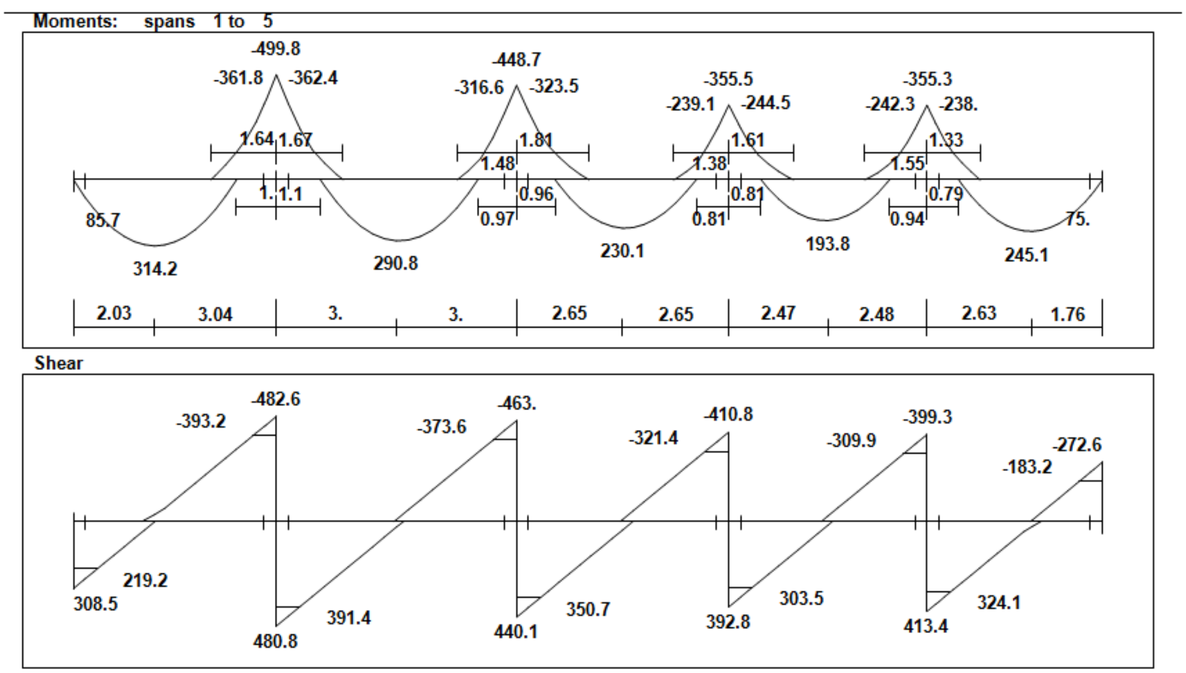
The maximum support reaction (Service) from Dead Loads for R1 upon B1 is 40.09 KN . The distributed Dead Load from the R1,1BF on B1,1BF:

DL =40.09/ 0.52= 78.65 KN/m

Live Load calculations: The maximum support reaction (Service) from Live Loads for R1,1BF upon B1,1BF is 16.17 KN .

The distributed Live Load from the R1,1BF on B1,1BF :

LL =16.17/ 0.52= 31.09KN/m



Reactions

Factored

DeadR	194.15	634.72	579.32	505.85	529.34	172.1
LiveR	114.38	328.69	323.78	297.76	283.42	100.5
Max R	308.53	963.41	903.1	803.61	812.76	272.6
Min R	174.74	774.21	683.67	599.93	641.12	155.77
Service						
DeadR	161.79	528.93	482.77	421.54	441.12	143.41
LiveR	71.49	205.43	202.36	186.1	177.14	62.81
Max R	233.28	734.36	685.13	607.64	618.25	206.23
Min R	149.66	616.11	547.99	480.34	510.98	133.21

Figure 4. 4Loading and Moment /Shear Envelope.

✓ Flexural Design for (B16) :

Determine of $M_{n,max}$:

$$d = 350 - 40 - 8 - \frac{25}{2} = 289.5\ mm$$

$$c = \frac{3}{7} d = \frac{3}{7} \times 289.5 = 124.07\ mm$$

$$a = \mathcal{B}.c = 124.07 \times 0.85 = 105.5mm$$

$Mn_{max} = 0.85f'_c ab(d - \frac{a}{2}) = 0.85 \times 24 \times 105.5 \times 1000 \times (289.5-105.5/2) \times 10^{-6} = 509.53$ KN.m

$\emptyset Mn_{max} = 0.82 \times 509.53 = 417.8$ KN.m

Design as singly reinforcement

<u>Design for positive moment :</u>

1) $\boldsymbol{Mu = 314.2\ , 290.9\ KN.m}$

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{314.2 \times 10^6}{0.9 \times 1000 \times 289.5^2} = 4.16\ Mpa.$$

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

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 4.16}{420}}\right) = 0.0112$$

$A_s = \rho.b.d = 0.0112 \times 1000 \times 289.5 = 3242.4mm^2$

- **Check for $A_{s,min}$.**

$$A_{s,min} = 0.25 \frac{\sqrt{f'_c}}{f_y} b_w.d \geq \frac{1.4}{f_y} b_w.d$$

$$A_{s,min} = 0.25 \frac{\sqrt{24}}{420}\ 1000 \times 289.5 = 844.19\ mm^2$$

$$A_{s,min} = \frac{1.4}{420} 1000 \times 289.5 = 965\ mm^2\ Control.$$

A_s= 3242.4 > $A_{s,min}$=965 mm^2

Use7ø 25 Bottom, $A_{s,provided}$= 3436.1mm^2>$A_{s,required}$= 3242.4 mm^2. Ok

- **Check spacing :**

S_{max}= $380\left(\frac{280}{f_y}\right) - 2.5C_c = 203.33\ control \quad OR \quad S = 300\left(\frac{280}{f_s}\right) = 200$

S = $\frac{1000-40\times2-10\times2-(25\times7)}{6} = 120.8\ mm > 25 > 20.33 \ldots\ OK$

- **Check for strain:**

a =$\frac{A_s.f_y}{0.85b\ f_c'} = \frac{3436.1\times420}{0.85\times1000\times24} = 70.7mm$

c= $\frac{a}{\mathcal{B}_1} = \frac{70.7}{0.85} = 83.22mm$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{289.5-83.22}{83.22}\right) = 0.007 > 0.005 \qquad 0k$$

2) $\boldsymbol{Mu = 230.1\ , 245.1\ KN.m}$

$Rn = \frac{M_u}{\emptyset bd^2} = \frac{245.1\times10^6}{0.9\times1000\times289.5^2} = 3.25\ Mpa.$

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

ρ= $\frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times3.25}{420}}\right) = 0.00847$

A_s = ρ.b.d = 0.00847×1000×289.5 =2452.07 mm^2

- **Check for $A_{s,min}$.**

$A_{s,min}$ =$0.25\frac{\sqrt{f'_c}}{f_y}b_w.d \geq \frac{1.4}{f_y}b_w.d$

$A_{s,min}$ =$0.25\frac{\sqrt{24}}{420}\ 1000\times289.5 = 844.19\ mm^2$

$A_{s,min}$ =$\frac{1.4}{420}1000\times289.5 = 965\ mm^2\ Control.$

A_s= 2452.07 > $A_{s,min}$=965 mm^2

Use 5ø 25 Bottom, $A_{s,provided}$= 2453.12mm^2>$A_{s,required}$= 2452.07 mm^2. Ok

- **Check spacing :**

S_{max}= $380\left(\frac{280}{f_y}\right) - 2.5C_c = 203.33\ control \quad OR \quad S = 300\left(\frac{280}{f_s}\right) = 200$

S = $\frac{1000-40\times2-10\times2-(25\times5)}{4} = 193.75\ mm > 25 > 20.33 \ldots\ OK$

- **Check for strain:**

a =$\frac{A_s.f_y}{0.85b\ f_c'} = \frac{2453.12\times420}{0.85\times1000\times24} = 50.5mm$

c= $\frac{a}{\mathcal{B}_1} = \frac{50.5}{0.85} = 59.4mm$

$$\boldsymbol{\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{289.5-59.4}{59.4}\right) = 0.0116 > 0.005 \qquad 0k}$$

3) $\boldsymbol{Mu\ =\ 193.8\ KN.m}$

$Rn = \frac{M_u}{\emptyset bd^2} = \frac{193.8\times10^6}{0.9\times1000\times289.5^2} = 2.57\ Mpa.$

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

ρ= $\frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times2.57}{420}}\right) = 0.00656$

A_s = ρ.b.d = 0.00656×1000×289.5 =1899.9 mm^2

- **Check for $A_{s,min}$.**

$A_{s,min}$ =$0.25\frac{\sqrt{f'_c}}{f_y}b_w.d \geq \frac{1.4}{f_y}b_w.d$

$A_{s,min}$ =$0.25\frac{\sqrt{24}}{420}\ 1000\times289.5 = 844.19\ mm^2$

$A_{s,min}$ =$\frac{1.4}{420}1000\times289.5 = 965\ mm^2\ Control.$

A_s= 1899.9 > $A_{s,min}$=965 mm^2

Use 4ø25 Bottom, $A_{s,provided}$= 1962.5 mm^2>$A_{s,required}$= 1899.9 mm^2. Ok

- **Check spacing :**

S_{max}= $380\left(\frac{280}{f_y}\right) - 2.5C_c = 203.33\ control \quad OR \quad S = 300\left(\frac{280}{f_s}\right) = 200$

S = $\frac{1000-40\times2-10\times2-(4\times25)}{3} = 266.6\ mm > 25 > 20.33 \ldots\ OK$

- **Check for strain:**

a =$\frac{A_{s}.f_y}{0.85b\ f'_c} = \frac{1962.5\times420}{0.85\times1000\times24} = 40.4\ mm$

c= $\frac{a}{\mathcal{B}_1} = \frac{40.4}{0.85} = 47.53mm$

$$\boldsymbol{\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{289.5-47.53}{47.53}\right) = 0.0153 > 0.005 \qquad 0k}$$

Design for Negative moment :

1) **M_u = 362.4 KN.m**

R_n= $\frac{M_u}{\emptyset b d^2} = \frac{362.4\times10^6}{0.9\times1000\times289.5^2} = 4.8Mpa.$

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

ρ= $\frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times4.8}{420}}\right) = 0.0132$

A_s = ρ.b.d = 0.0132×1000×289.5= 3821.4mm^2.

- **Check for $A_{s,min}$.**

$A_{s,min}$ =$0.25\frac{\sqrt{f'_c}}{f_y}b_w.d \geq \frac{1.4}{f_y}b_w.d$

$A_{s,min}$ =$0.25\frac{\sqrt{24}}{420}\ 1000\times289.5 = 844.19\ mm^2$

$A_{s,min}$ =$\frac{1.4}{420}1000\times289.5 = 965\ mm^2\ Control.$

A_s= 3821.4 mm^2> $A_{s,min}$ = 965mm^2

Use 8ø 25 Top ,$A_{s,provided}$= 3925 mm^2>$A_{s,required}$= 3821.4 mm^2.   Ok

- **Check spacing :**

$$S = \frac{1000-40*2-2*10-(8\times25)}{7} = 100\ mm > 25 > S_{max}\quad OK$$

- **Check for strain:**

$$a = \frac{A_{s}.f_y}{0.85b\ f_c'} = \frac{3925\times420}{0.85\times1000\times24} = 80.8mm$$

$$c = \frac{a}{\mathcal{B}_1} = \frac{80.8}{0.85} = 95.06mm$$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{289.5-95.06}{95.06}\right) = 0.006 > 0.005 \qquad 0k$$

2) M_u = -323.5 KN.m

$$R_n = \frac{M_u}{\emptyset bd^2} = \frac{323.5\times10^6}{0.9\times1000\times289.5^2} = 4.3Mpa.$$

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

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times4.3}{420}}\right) = 0.0116$$

A_s = ρ.b.d = 0.0116×1000×289.5= 3358.2 mm^2.

- **Check for $A_{s,min}$.**

$$A_{s,min} = 0.25\frac{\sqrt{f'_c}}{f_y}b_w.d \geq \frac{1.4}{f_y}b_w.d$$

$$A_{s,min} = 0.25\frac{\sqrt{24}}{420}\ 1000\times289.5 = 844.19\ mm^2$$

$$A_{s,min} = \frac{1.4}{420}1000\times289.5 = 965\ mm^2\ Control.$$

A_s= 3358.2 mm^2> $A_{s,min}$ = 965 mm^2

Use7 ø 25Top ,$A_{s,provided}$= 3434.37 mm^2>$A_{s,required}$= 3358.2 mm^2.   Ok

- **Check spacing :**

S $= \frac{1000-40*2-2*10-(25\times7)}{6} = 120.8\,mm > 25 > S_{max} \quad OK$

- **Check for strain:**

a $= \frac{A_s.f_y}{0.85b\,f_c'} = \frac{3434.37\times420}{0.85\times1000\times24} = 70.7mm$

c$= \frac{a}{\mathcal{B}_1} = \frac{70.7}{0.85} = 83.18mm$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{289.5-83.18}{83.18}\right) = 0.007 > 0.005 \qquad OK$$

3) M_u = -244.5 , -242.3 KN.m

$R_n$$= \frac{M_u}{\emptyset bd^2} = \frac{244.5\times10^6}{0.9\times1000\times289.5^2} = 3.4\,Mpa.$

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

ρ$= \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times3.4}{420}}\right) = 0.0089$

A_s = ρ.b.d = 0.0089×1000×289.5= 2576.5 mm^2.

- **Check for $A_{s,min}$.**

$A_{s,min}$ $=0.25\frac{\sqrt{f'_c}}{f_y}b_w.d \geq \frac{1.4}{f_y}b_w.d$

$A_{s,min}$ $=0.25\frac{\sqrt{24}}{420}\;1000\times289.5 = 844.19mm^2$

$A_{s,min}$ $=\frac{1.4}{420}1000\times289.5 = 965\,mm^2\;Control.$

A_s=2576.5mm^2 >$A_{s,min}$ = 965mm^2

Use 6 ø 25Top ,$A_{s,provided}$= 2943.75mm^2>$A_{s,required}$= 2576.5 mm^2. Ok

- **Check spacing :**

$$S = \frac{1000-40*2-2*10-(25\times6)}{5} = 150\ mm > 25 > S_{max}\quad OK$$

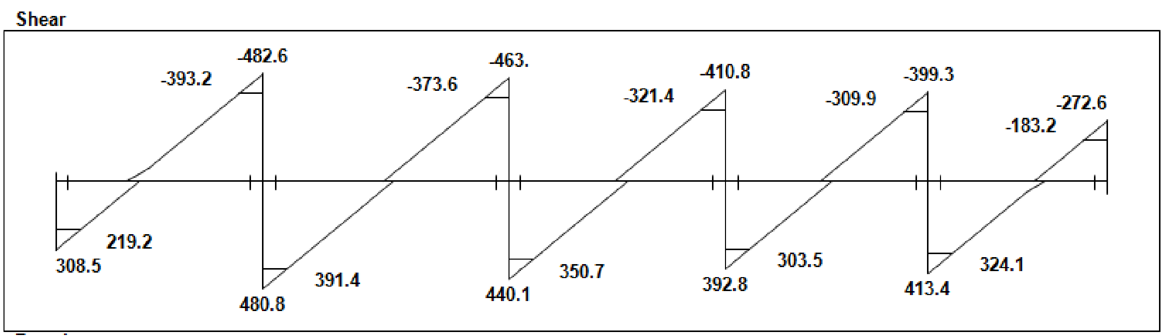
- **Check for strain:**

$$a = \frac{A_{s}.f_y}{0.85b\ f_c'} = \frac{2943.75\times420}{0.85\times1000\times24} = 60.6mm$$

$$c = \frac{a}{\mathcal{B}_1} = \frac{60.6}{0.85} = 71.3\ mm$$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{289.5-71.3}{71.3}\right) = 0.009 > 0.005$$

✓ **<u>Shear Design for (B16):</u>**



1. Vu = 393.2 KN

$$V_c = \frac{1}{6}\sqrt{fc'}b\ d == \frac{1}{6}\sqrt{24}*700*289.5*10^{-3} = 165.46KN$$

Φ V_c= 0.75*165.46 =124.1KN

$$V_{s,min} = \frac{1}{3}b\ d == \frac{1}{3}\ 700*\ 289.5*10^{-3} = 67.48\ KN\text{control}$$

$$V_{\ s,min} = \frac{1}{16}\sqrt{fc'}b_w\ d = \frac{1}{16}*\sqrt{24}*700*289.5*10^{-3} = 62.04KN$$

$$V_{s'} = \frac{1}{3}\sqrt{fc'}b_w\ d = \frac{1}{3}\sqrt{24}*700*289.5*10^{-3} = 330.9\text{ KN}$$

$$\emptyset V_c < V_u \leq \emptyset(V_c + V_{s,\min})$$

$$124.1 < 393.2 < 0.75(165.46 + 62.04)$$

$124.1 < 393.2 < 170.67$ … Not ok

$V_{u,max}$ =393.2KN.

d= $h - cover - d_{stirrup} - \frac{d_b}{2} = 350 - 40 - 10 - \frac{25}{2} = 287.5\ mm.$

$V_c = \frac{1}{6}\sqrt{f_c'}b.d = \frac{1}{6}\sqrt{24} \times 700 \times 287.5 \times 10^{-3} = 164.3\ KN$

Check for section dimensions:

$V_s = \frac{V_u}{\emptyset} - V_c = \frac{393.2}{0.75} - 164.3 = 359.9\ KN$

$V_{s,max} = \frac{2}{3}\sqrt{f_c'}b.d = \frac{2}{3}\sqrt{24} \times 700 \times 289.5 \times 10^{-3} = 657.27\ KN$

$V_s < V_{smax}$ so the section is not large enough.

Check for the case of shear:

$V_{s,min} = \frac{1}{16}\sqrt{f_c'}b.d \quad OR = \frac{1}{3}\ b.d \quad which\ is\ larger.$

$V_{s,min} = \frac{1}{16}\sqrt{24} \times 700 \times 289.5 \times 10^{-3} = 62.04KN$

$V_{s,min} = \frac{1}{3} \times 700 \times 289.5 \times 10^{-3} = 67.55 \quad \boldsymbol{cont}.$

$\emptyset(V_{smin} + V_c) = 0.75(67.55+164.3) = 173.6$ KN.

$V_u > \emptyset(V_{smin} + V_c)$ ok

case(III) for shear design.

$S_{max} \leq \frac{d}{2} = \frac{289.5}{2} = 144.75$ mm OR $S_{max} \leq 600mm$

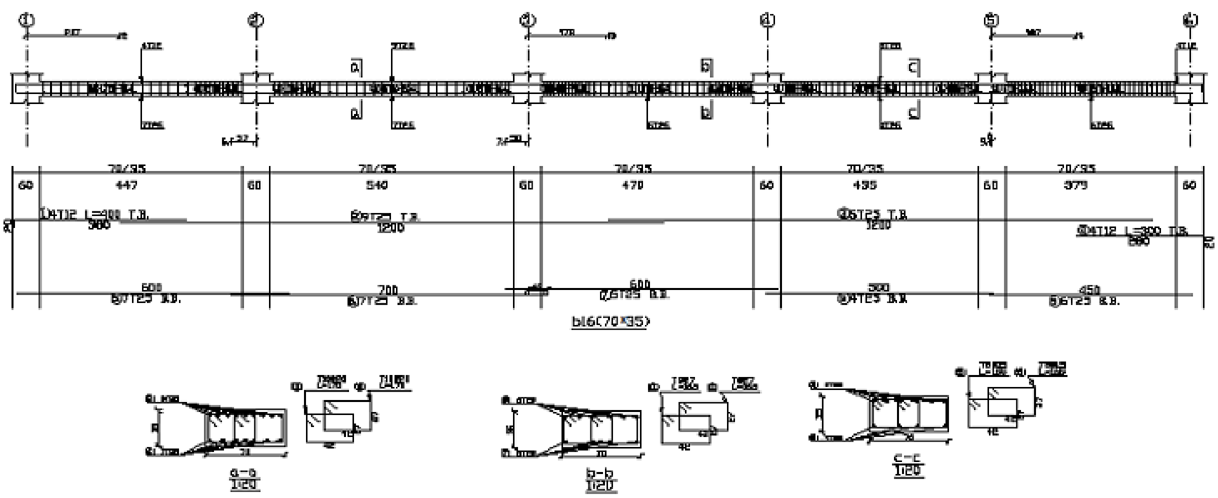
S_{max} =144.75 mm cont.

By using ϕ 10 double legs stirrups, $A_v = 157.1\ mm^2$

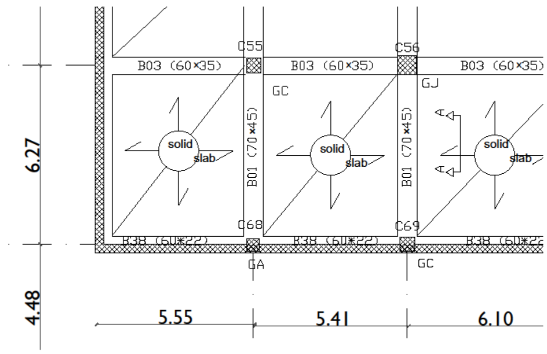
$$s = \frac{A_v f_{yt}}{V_s} d = \frac{157.1 \times 420 \times 289.5}{67.55 \times 700} = 403.97\ mm$$

Use 2 leg ϕ10 @200mm

For all spans 2 leg ϕ10 @200mm will be used for stirrups.



4.7 Design of Two Way Solid Slab:



- ✓ **Calculate the minimum thickness slab :**

$$hmin = 22\ cm$$

$$Y = \frac{\sum AY^3}{\sum A}$$

$$y(B38) = \frac{22 * (60 + 38) * (22\backslash 2 + 38) + 60 * 38 * (38\backslash 2)}{60 * 38 + (60 + 38) * 22} = 33.58cm$$

$$Ib = \sum I + \sum Ay^2$$

$$Ib(B38) = \frac{98 * 22^3}{12} + 98 * 22 * 15.42^2 + \frac{60 * 38^3}{12} + 60 * 38 * 14.58^2$$
$$= 1358638.78cm^4$$

$$Y = \frac{\sum AY^3}{\sum A}$$

$$y(B38) = \frac{22 * 136 * (22\backslash 2 + 38) + 60 * 38 * (38\backslash 2)}{136 * 22 + 38 * 60} = 36cm$$

$$Ib = \sum I + \sum Ay^2$$

$$Ib(B38) = \frac{136 * 22^3}{12} + 136 * 22 * 13^2 + \frac{60 * 38^3}{12} + 38 * 60 * 17^2$$
$$= 1296037.33cm^4$$

$$Is1 = \frac{(481\backslash 2 + 60) * 22^{\wedge}3}{12} = 266643.67cm^4$$

$$Is2 = \frac{(567\backslash 2 + 60) * 22^{\wedge}3}{12} = 304799cm^4$$

$$Is3 = \frac{(283.5 + 282 + 60) * 22^{\wedge}3}{12} = 555027cm^4$$

$$Is4 = \frac{(240.5 + 275 + 60) * 22^{\wedge}3}{12} = 510660.33cm^4$$

$$\propto f1 = \propto f2 = \frac{Ib01}{Is3} = \frac{1296037.33}{555027} = 2.34$$

$$\propto f3 = \frac{Ib38}{Is4} = \frac{1296037.33}{510660.33} = 2.5$$

$$\propto f4 = \frac{Ib03}{Is2} = \frac{1358638.78}{266643.67} = 5.09$$

$$\propto fm = \frac{\sum \propto}{4} = \frac{2 * 2.34 + 2.5 + 5.09}{4} = 3.067 > 2$$

$$for \propto fm > 2$$

$$hmin = \frac{ln * \left(0.8 + \frac{Fy}{1400}\right)}{36 + 9B} = \frac{5670 * \left(0.8 + \frac{420}{1400}\right)}{36 + 9 * 1.18} = 133.7cm > 90mm$$

h=13.37<hmin=22 ok

take hslab=22cm

✓ Dead load calculations:

Table(4.7) calculation of the two way solid Dead load

Dead load from:	δ×γ	KN/m
Tiles	0.03×23×1	0.69
Mortar	0.02×22×1	0.44

Coarse sand	**0.07×16×1**	**1.12**
Slab	**0.22×25×1**	**5.5**
Plaster	**0.02×22×1**	**0.44**
Partitions	**2.3*1**	**2.3**
		10.49

Dead load =10.49 KN/m^2.

Live load = 5 KN/m^2.

WuD = 1.2*Dead load = 1.2*10.49= 12.59 KN/m^2.

WuL = 1.6*live load = 1.6*5 = 8 KN/m^2.

Wu = 12.59+8 = 20.59 KN/m^2

✓ **Shear Design :**

l_a/l_b=0.85

W_a = 0.49

Wb = 0.51

- **The total load on the panel being (4.81*5.67*20.59) = 561.5 KN\m**
- **The load at face of the long beam is (0.70×561.5/(2*5.67))=34.7 KN\m**
- **The load at face of the long beam is (0.30×561.5/(2*4.81))=17.5 KN\m**

Assume the Φ 14

d=220-20-14\2=193mm

- **V_c =(√24 *1000*193*10^-3)\6 =157.6KN**

ϕV_c = 0.75*157.6 = 118.18KN

$V_u < \phi V_c$.

The thickness of the slab is adequate enough

- ✓ **Flexural Design:**

(l_a/l_b=0.85)

Positive moments :

Dead load :

Ca=0.029

Cb=0.017

Live load :

Ca=0.040

Cb=0.022

$M_{a+ve,Dl}=C_a*W*L_a^2=0.029*12.59*4.81^2=8.45KN.m\backslash m$

$M_{a+ve,Ll}=C_a*W*L_a^2=0.040*8*4.81^2=7.4KN.m\backslash m$

$\underline{M_{a+ve}=M_{a+ve,L}+M_{a+ve,D}=15.85KN.m\backslash m}$

$M_{b+ve,D}=C_b*W*L_b^2=0.017*12.59*5.67^2=6.88KN.m\backslash m$

$M_{b+ve,L}=C_b*W*L_b^2*b=0.022*8*5.67^2=5.66KN.m\backslash m$

$\underline{M_{b+ve}=M_{b+ve,L}+M_{b+ve,D}=12.54KN.m/m}$

- ✓ **Positive Moment:**

***Mua= 15.85KN.m/m**

Assume the d_{Bar} =16 mm

d =h- cover - (d_{Bar}\2) =220-20-14\2=193mm

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{15.85 \times 10^6}{0.9 \times 1000 \times 193^2} = 0.47 Mpa.$

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

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.47}{420}}\right) = 0.001132$

$A_s = \rho.b.d = 0.001132 \times 1000 \times 193 = 218.5\ mm^2.$

$As_{min} = 0.0018 * b * h = 0.0018*1000*193 = 347.4$

$A_s = 218.5 mm^2 < A_{s,min} = 347.4 mm^2$

Use3 ø 14 ,$A_{s,provided} = 461.8\ mm^2 > A_{s,required} = 347.4\ mm^2$. Ok

***Mub= 12.54KN.m/m**

Assume the d_{Bar} =14 mm

d =h- cover - (d_{Bar}\2) =220-20-14\2=193mm

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{12.54 \times 10^6}{0.9 \times 1000 \times 193^2} = 0.37 Mpa.$

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

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.37}{420}}\right) = 0.000899$

$A_s = \rho.b.d = 0.000899 \times 1000 \times 193 = 173.5 mm^2.$

$As_{min} = 0.0018 * b * h = 0.0018*1000*193 = 347.4$

$A_s = 173.5 mm^2 < A_{s,min} = 347.4 mm^2$

Use3 ø 14 ,$A_{s,provided} = 461.8\ mm^2 > A_{s,required} = 347.4\ mm^2$. Ok

✓ **Negative Moment:**

Ca = 0.049

Cb = 0.046

Ma = Ca * W * la^2 = 0.049*20.59*4.81^2=23.3

Mb = Cb * W * lb^2 = 0.046*20.59*5.67^2=30.45

Mua= 23.3N.m/m

Assume the d_{Bar} =16 mm

d =h- cover - (d_{Bar}\2) =220-20-16\2=192mm

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{23.3 \times 10^6}{0.9 \times 1000 \times 192^2} = 0.702 Mpa.$

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

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.702}{420}}\right) = 0.0017$

A_s = ρ.b.d = 0.0017×1000×192= 326.77mm^2.

As_{min}=0.0018 * b * h = 0.0018*1000*192=345.6

A_s=326.77mm^2 <$A_{s,min}$ = 345.6mm^2

Use2 ø 16 ,$A_{s,provided}$= 402.12 mm^2>$A_{s,required}$= 345.6 mm^2. Ok

Mua= 30.45N.m/m

Assume the d_{Bar} =16 mm

d =h- cover - (d_{Bar}\2) =220-20-16\2=192mm

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{30.45 \times 10^6}{0.9 \times 1000 \times 192^2} = 0.92 Mpa.$

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

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2\times20.6\times0.92}{420}}\right) = 0.00224$$

$A_s = \rho.b.d = 0.00224\times1000\times192 = 430.5mm^2$.

$As_{min} = 0.0018 * b * h = 0.0018*1000*192 = 345.6$

$A_s = 430.5mm^2 > A_{s,min} = 345.6mm^2$

Use3 ø 16 ,$A_{s,provided} = 603.18\ mm^2 > A_{s,required} = 430.5\ mm^2$.   Ok

4.8 Design of Stair:

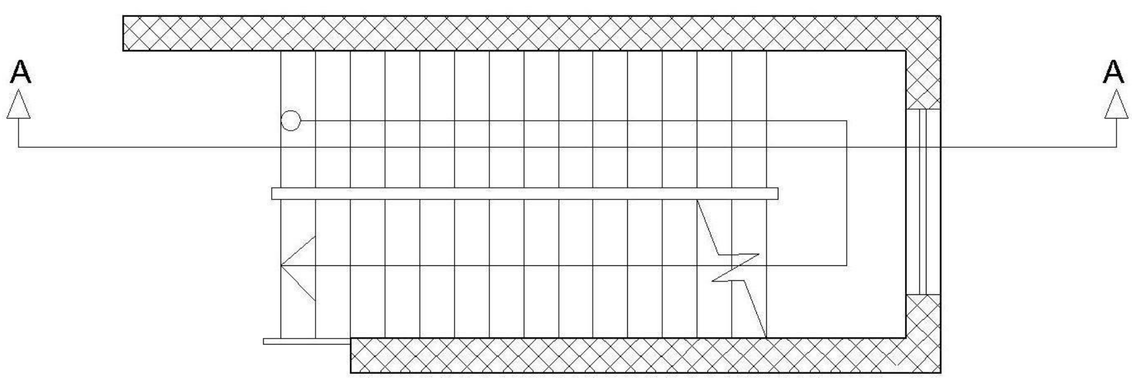


Figure4.6: Stair Plan.

✓ Material :-

⇒ concrete B300 Fc' = 24 N/mm^2

⇒ Reinforcement Steel Fy = 420 N/mm^2

✓ Design of Flight :-

✓ Determination of Thickness:-

h_{min} = L/20

h_{min} = 5.83/20 =29.15 cm

Take h = 30cm

The Stair Slope by $\theta = \tan^{-1}(170/300) = 29.54^o$

✓ Load Calculation:-

Dead Load For Flight For 1m Strip:-

Table1-8:Dead Load Calculation of Flight.

No.	Parts of Flight	Calculation	
1	Tiles	27*0.03*1*(0.3+0.170)/0.3 = 1.269 KN/m	
2	Mortar	22*0.02*1*(0.3+0.170)/0.3 = 0.689 KN/m	
3	Stair	25*1*(0.3*0.170*0.5)/0.3 = 2.125 KN/m	
4	Slab	25*0.30*1 / COS 29.54 = 8.62 KN/m	
5	Plaster	22*0.03*1 / COS 29.54= 0.759KN/m	
		Sum	13.462 KN/m

Live Load For Landing For 1m Strip = 5*1 = 5 KN/m

Factored Load For Flight :-

W_U = 1.2 ×13.462+ 1.6×5 =24.15KN/m

✓ Design of Landing :

✓ **Load Calculation:-**

No.	Parts of Flight	Calculation	
1	Tiles	22*0.03*1= 0.66 KN/m	
2	Mortar	22*0.02*1= 0.44 KN/m	
4	concreat	25*0.25*1= 6.25 KN/m	
5	Plaster	22*0.03*1= 0.66 KN/m	
		Sum	8.01 KN/m

Dead Load For Landing For 1m Strip:-
Live Load For Landing For 1m Strip = 5*1 =5 KN/m

Factored Load For Landing :-

W_U = 1.2 ×8.01+ 1.6×5=17.6KN/m

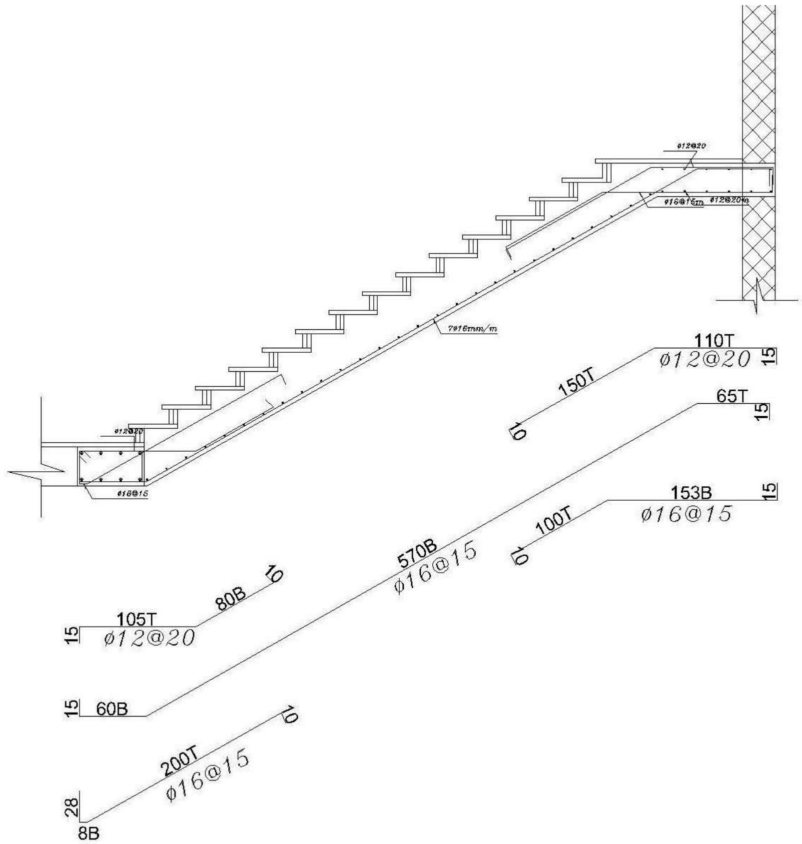
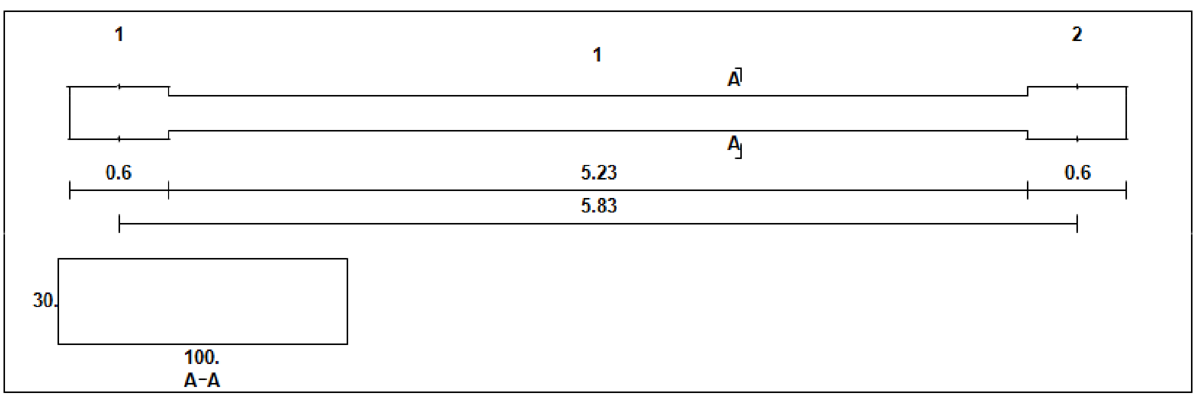


Fig 4.7: Stair Section.

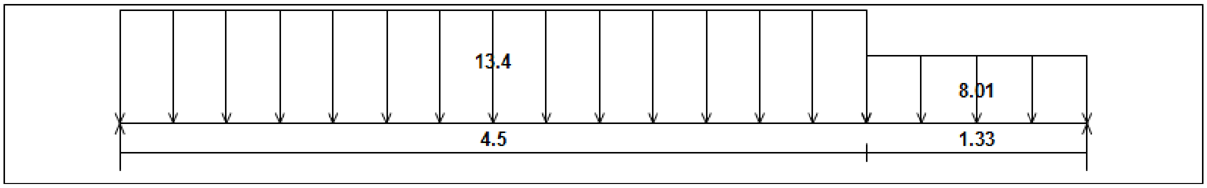
Geometry Units:meter,cm



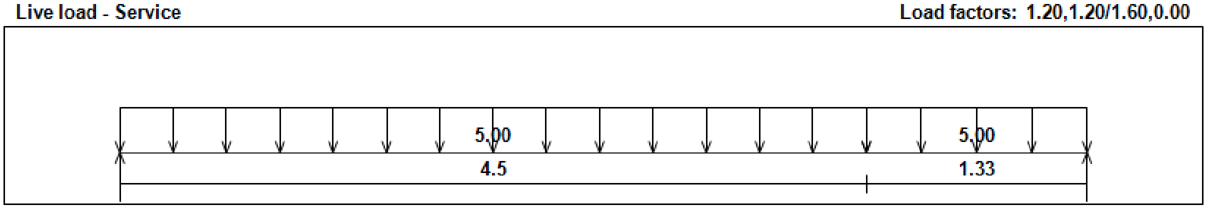
Loading

load group no. 1

Dead load - Service Units:kN,meter

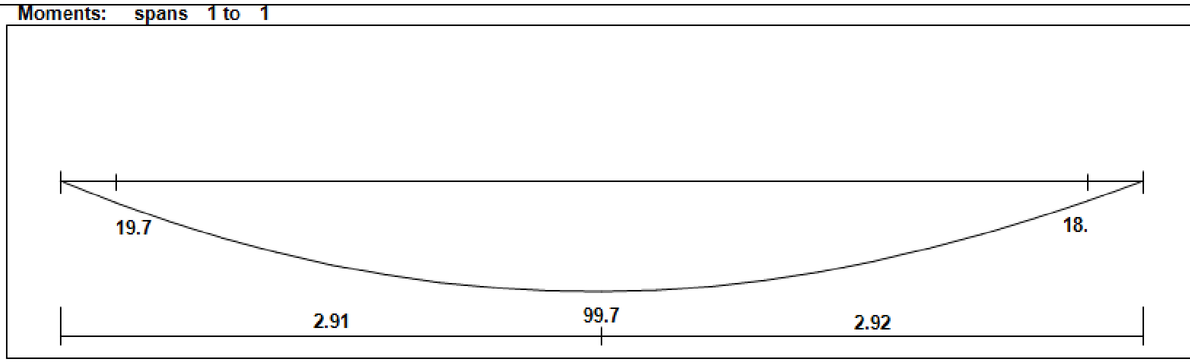


Live load - Service Load factors: 1.20,1.20/1.60,0.00

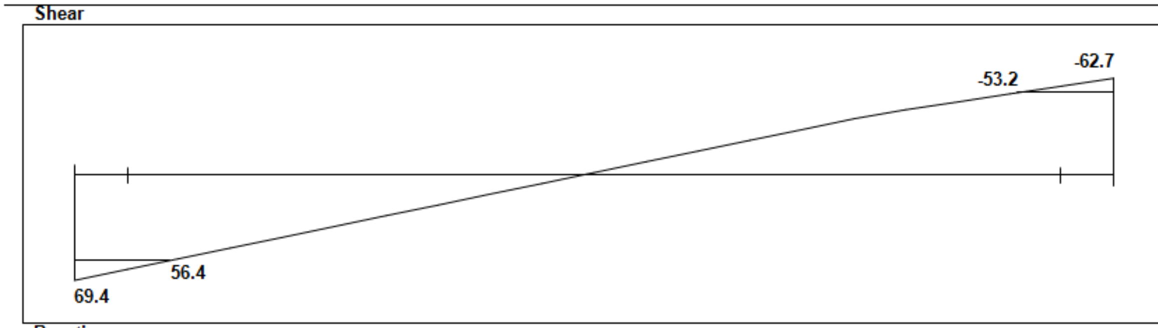


Moment/Shear Envelope (Factored) Units:kN,meter

Moments: spans 1 to 1



Shear



Reactions

Factored

DeadR	46.1	39.38
LiveR	23.32	23.32
Max R	69.42	62.7
Min R	69.42	62.7

Service

DeadR	38.41	32.82
LiveR	14.58	14.57
Max R	52.99	47.39
Min R	52.99	47.39

1- **Design of Shear for Flight :- (Vu=56.4 KN)**

Assume bar diameter ø 12 for main reinforcement

d =h- cover $-\frac{d_b}{2} = 300 - 20 - \frac{12}{2} = 274mm$

Vu = 56.4KN

$V_c = \frac{1}{6}\sqrt{fc'}b_w\, d = \frac{1}{6}\sqrt{24} * 1000 * 274 = 223.7\text{KN}/m$

$\Phi\ V_{c\,=}$ 0.75* 223.7 = 167.7 KN /m

Vu = 56.4 <$\Phi\ V_{C\,=}$ 167.7 KN /m

2- **Design of Bending Moment for Flight :- (Mu=99.7KN.m)**

d =h- cover $-\frac{d_b}{2} = 300 - 20 - \frac{16}{2} = 272mm$

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{99.7\times10^6}{0.9\times1000\times272^2} = 1.49\ \ Mpa$

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

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2\times20.6\times1.49}{420}}\right) = 0.00368$

$A_{s,req}$ = ρ.b.d = 0.00368 ×1000×272 = 1003.1mm^2

$A_{s,min} = 0.25\frac{\sqrt{f'_c}}{f_y}b_w.d \geq \frac{1.4}{f_y}b_w.d$

$A_{s,min} = 0.25\frac{\sqrt{24}}{420}\ 1000\times272 = 793.17mm^2$

$A_{s,min} = \frac{1.4}{420}1000\times272 = 906.67\ mm^2\ Control.$

A_s=1003.1mm^2 >$A_{s,min}$ = 906.67mm^2 ok .

Use 5ø16 ,$A_{s,provided}$= 1005.3 mm^2>$A_{s,required}$= 1003.1mm^2.   Ok

Use 1ø16@20cm

Check for Spacing :-

1) **S = 3h = 3*300 =900 mm**
2) **S = 380*(280/(2/3 * 420)) – 2.5*20 = 330 ≤ S = 300*(280/(2/3 * 420)) = 250mm**
3) **S = 450 mm**

S = 250 mm **……… is control**

Use ø16@ 200 mm

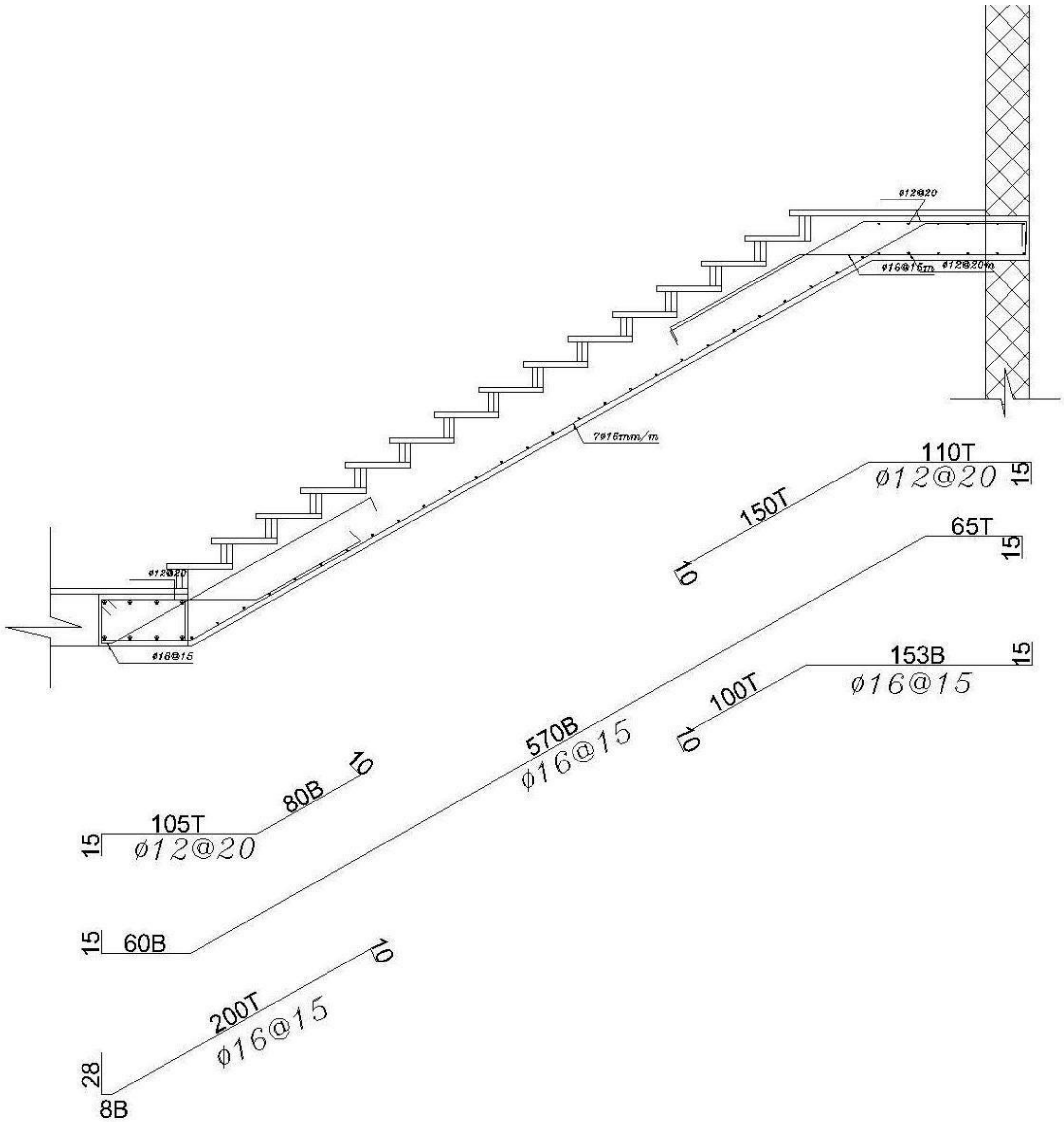


Fig 4.7: Stair Reinforcement.

4.9 Design of Column

✓ **Material :-**

⇒ concrete B350 $Fc' = 24\ N/mm^2$

⇒ Reinforcement Steel $Fy = 420\ N/mm^2$

✓ **Load Calculation:-**

Service Load:-

Dead Load =3000 KN
Live Load =810 KN

Factored Load:-

$P_U = 1.2 \times 3000 + 1.6 \times 810 = 4896$ KN

✓ **Dimensions of Column:-**

$Assume\rho g = 0.01$

$\phi * Pn = 0.65 \text{ x } 0.8 \times Ag\{0.85\ fc'(1-\rho g) + \rho g * Fy\}$

4896*1000 = 0.65*0.8*Ag(28.39)

Ag= 331621.04 mm2

Assume square tied Section

Ag = a*a

a = 550mm

Ag = 550*550 =302500mm2

Selecting Longitudinal Bars:

4896*1000 = .65*.8{.85*24(302500-Ast)+Ast*420}

Ast=8119.08 mm2

Use20 ϕ 25 ,Ast,prov=9812.5 mm2>Ast=8119.08 mm2

ρg =Ast/Ag= 0.03

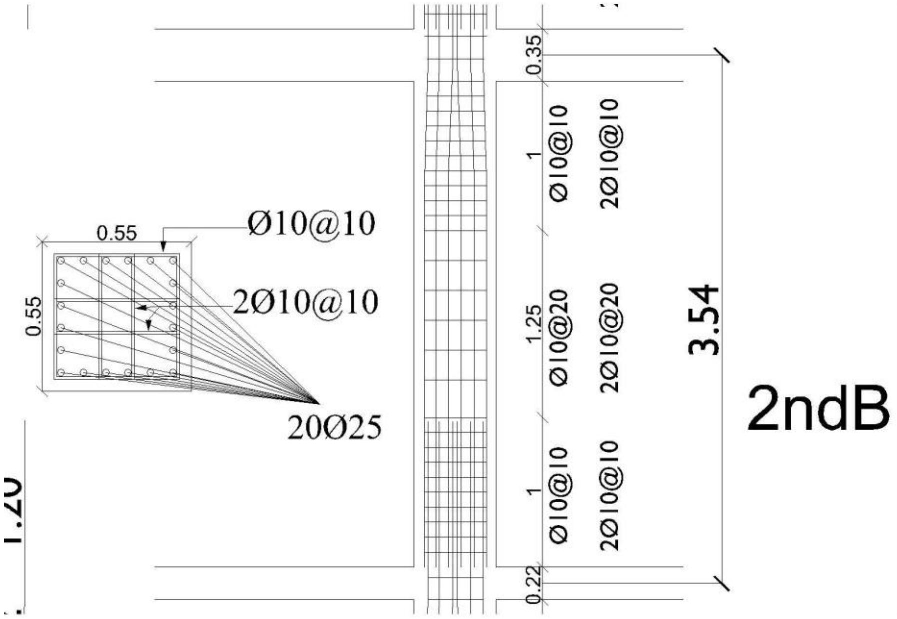


Fig 4.9:Column section and reinforcement.

✓ Design of the tie reinforcement :

S ≤16 db (longitudinal bar diameter)

S≤ 48dt (tie bar diameter).

S ≤ Least dimension.

spacing $\leq 16\times d_b = 16\times 25 = 400$mmcontrol….

spacing $\leq 48\times dt = 48\times 10 = 480$mm

spacing ≤least.dim=550 mm

Use2ϕ10@200 mm

4.10 Design of Basement wall

4.10.1 Load Calculation:-

$\gamma = soildensity = 18KN/m^3$.

$\emptyset = angleofinternalfriction = 30°$.

LL=5 KN/m^2 .

Thickness = 30cm, cover = 4cm .

The design will be for 1m width .

Neglect the axial load, since its low value

$q1 = soilpressure = Ko * \gamma * h$.

$q2 = surchargepressure = Ko * LL$.

$Ko = soilpressurecoefficientatrest = 1 - \sin \emptyset$.

So ,

$Ko = 1 - \sin \emptyset = 0.5$.

$q1 = 0.5 * 18 * 3.50 = 31.5 \frac{KN}{m^2}$.

$q2 = 0.5 * 5 = 2.5 \frac{KN}{m^2}$.

Factored Load :-

$q1u = 31.5 * 1.6 = 50.4 \ KN/m^2$

$q2u = 2.5 * 1.2 = 3 \ KN/m^2$

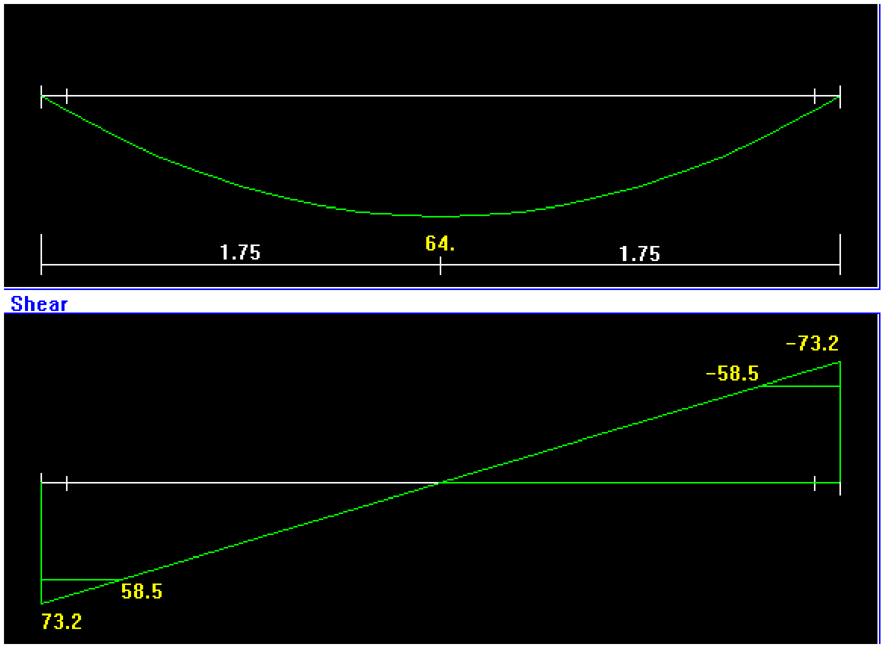


Figure 4. 10 Moment /Shear Envelope

4.10.2 Design of bending moment of wall :-

Design for positive moment Mu =64 KN.m .

$$d = 300 - 40 - \frac{16}{2} = 252\ mm.$$

$$Mn = \frac{Mu}{0.9} = \frac{64}{0.9} = 71.11\ KN.m$$

$$Rn = \frac{Mn * 10^6}{b * d^2} = \frac{45.56 * 10^6}{1000 * 252^2} = 1.12\ Mpa.$$

$$m = \frac{Fy}{0.85 * fc'} = \frac{420}{0.85 * 25} = 19.76$$

$$\rho = \frac{1}{m} * \left(1 - \sqrt{1 - \frac{2 * Rn * m}{Fy}}\right) = \frac{1}{19.76} * \left(1 - \sqrt{1 - \frac{2 * 1.12 * 19.76}{420}}\right)$$

$$= 1.21 * 10^{-3}$$

$$Asreq\ = \rho * b * d = 1.74\ * 10^{-3} * 1000 * 252 = 304.92\ mm^2/m.$$

$$Asminv = 0.0012 * b * h = 0.0012 * 1000 * 300 = 360\ mm^2/m. \dots.control.$$

$$Asminforflexture \; = 0.25 * \frac{\sqrt{fc'}}{fy} * bw * d = 0.25 * \frac{\sqrt{25}}{420} * 1000 * 252$$

$$= 750 \; mm^2/m.$$

$$Asminforflexture \; = \frac{1.4}{fy} * bw * d = \frac{1.4}{420} * 1000 * 252 = 840mm^2/m \; \ldots .control.$$

For inside wall Select $\emptyset 12@25cm = 452.4 \; mm^2 > 437.71mm^2$.

For outside wall Select $\emptyset 12@12.5cm = 904mm^2 > 840mm^2$.

4.10.3 Design of shear force :-

$d = 300 - 40 - 8 = 252mm$

$$\emptyset Vc = 0.75 * \frac{1}{6} * \sqrt{fc'} * b * d = 0.75 * \frac{1}{6} * \sqrt{25} * 1000 * 252 * 10^{-3} = 157.5 \; KN.$$

$(\emptyset Vc = 157.5) > (Vu = 58.8)$.

No shear Reinforcement is required and thickness of wall is adequate enough.

But horizontal Reinforcement due to Cracking:

$Asreqh = 0.002 * b * h = 0.002 * 1000 * 300 = 600mm^2/m$.

For one side $As = 300 \; mm^2/m$.

Select for one side horizontal reinforcement $\emptyset 10@25cm = 314.16 \; mm^2 > 300 \; mm^2$

4.11 Design of Footing: Design of Group (2) – 6232 KN Factored Load

✓ **Material :**

⇒ concrete B350 Fc' = 24 N/mm^2

⇒ Reinforcement Steel Fy = 420 N/mm^2

✓ **Load Calculations :(From Column Group 4)**

Dead Load = 3450KN , Live Load = 1100KN

Total services load = 3450+ 1100 = 4550 KN

Total Factored load = 1.2*3450 + 1.6*1100= 5900KN

Column Dimensions (a*b) = 65*65 cm

Soil density = 18 Kg/cm^3

Allowable Bearing Capacity$Q_{allawable}$ = 350 KN/m2

Assume h = 90cm

= 350 – 25*0.9 – 18*0.5= 318.5kn/m^2 $q_{net-allow}$

✓ Area of Footing :

$$A = \frac{Pt}{q_{net-allow}} = \frac{4550}{318.5} = 14.07\ m^2$$

Assume Square Footing

B = $\sqrt{14.07}$ =3.73 m

Select B =3.75

L= B = 3.75 m

✓ Bearing Pressure :

q_u = 5900/(3.75*3.75) = 419.6 KN/m^2

✓ Design of Footing :

1- Design of One Way Shear Strength :

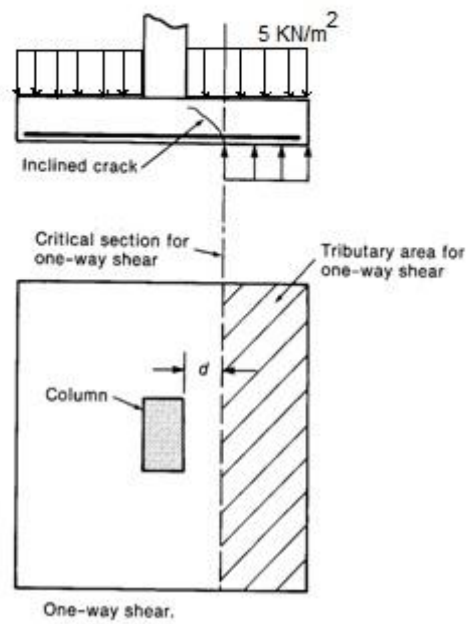
Critical Section at Distance (d) From The Face of Column

Assume h = 90cm, bar diameter ø 14 for main reinforcement and 7.5 cm Cover

d = 900 – 75 – 14 = 811 mm

$$Vu = q_u * \left(\frac{B}{2} - \frac{a}{2} - d\right) * L$$

$$Vu = 419.6 * \left(\frac{3.75}{2} - \frac{0.65}{2} - 0.811\right) * 3.75 = 1165\ KN$$



$$\phi.Vc = \phi.\frac{1}{6} * \sqrt{fc'} * b_w * d$$

$$\phi.Vc = 0.75 * \frac{1}{6} * \sqrt{28} * 3750 * 811 = 2010Kn$$

$$\phi.Vc = 2010KN > Vu = 1165$$

2- Design of Two Way Shear Strength :

$Vu = Pu - FR_b$

$FR_b = q_u * area\ \ of\ \ critical\ \ \text{sec}tion$

$Vu = 5900 - 419.6 * [(0.65 + 0.811) * (0.65 + 0.811)] = 5000Kn$

The punching shear strength is the smallest value of the following equations:

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

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

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

Where:

$$\beta_C = \frac{Column\ Length\ (a)}{Column\ Width\ (b)} = \frac{65}{65} = 1.0$$

b_o = Perimeter of critical section taken at (d/2) from the loaded area

$b_o = 4 * (811 + 650) = 5844\,mm$

α_s = 40 for interior column

$$\phi.V_C = \phi.\frac{1}{6}\left(1 + \frac{2}{\beta_c}\right)\sqrt{f_c'}\,b_o d = \frac{0.75}{6} * \left(1 + \frac{2}{1}\right) * \sqrt{28} * 5844 * 811 = 9400Kn$$

$$\phi.V_C = \phi.\frac{1}{12}\left(\frac{\alpha_s}{b_o / d} + 2\right)\sqrt{f_c'}\,b_o d = \frac{0.75}{12} * \left(\frac{40 * 811}{5844} + 2\right) * \sqrt{28} * 5844 * 811 = 11835Kn$$

$$\phi.V_C = \phi.\frac{1}{3}\sqrt{f_c'}\,b_o d = \frac{0.75}{3} * \sqrt{28} * 5844 * 811 = 6270Kn$$

Vu=5000 KN>ΦVc =6270 KN

3- **Design of Bending Moment :**

Critical Section at the Face of Column

FR = $q_u * \left(\frac{B-a}{2}\right) * L$ = 419.6 * $\left(\frac{3.75-0.65}{2}\right)$ *3.75 =2439 KN

= 1891 KN.m/2M_u = 2439* $\left(\frac{3.75}{2} - \frac{0.65}{2}\right)$

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{1891\times10^6}{0.9\times3750\times811^2} = 0.85\ Mpa$

$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85\times28} = 17.65$

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{17.6}\left(1-\sqrt{1-\frac{2\times17.65\times0.85}{420}}\right) = 0.00206$

$A_{s,req}$ = ρ.b.d = 0.00206×3750×811 = 6265 mm^2

$A_{s,min}$ = 0.0018*3750*900 = 6075 mm^2

$A_{s,req}$ =6265 mm^2 **……… is control**

Check for Spacing:

S = 45 cm

S = 3h = 3*90 = 270cm

S = 380*$\left(\frac{280}{\frac{2}{3}*420}\right)$ – 2.5*75 = 192.5 cm

S = 45 cm **……… is control**

Use 32ø16 in Both Direction, $A_{s,provided}$= 6432mm^2>$A_{s,required}$= 6265 mm^2… Ok

Check for strain:

$a = \frac{A_s.f_y}{0.85 b\ f_c'} = \frac{6432\times420}{0.85\times3750\times28} = 30.26\ mm$

$c = \frac{a}{\mathcal{B}_1} = \frac{30.26}{0.85} = 35.61\ mm$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{811-35.61}{35.61}\right) = 0.065 > 0.005 \;\; \ldots\ldots \mathbf{0}\boldsymbol{k}$$

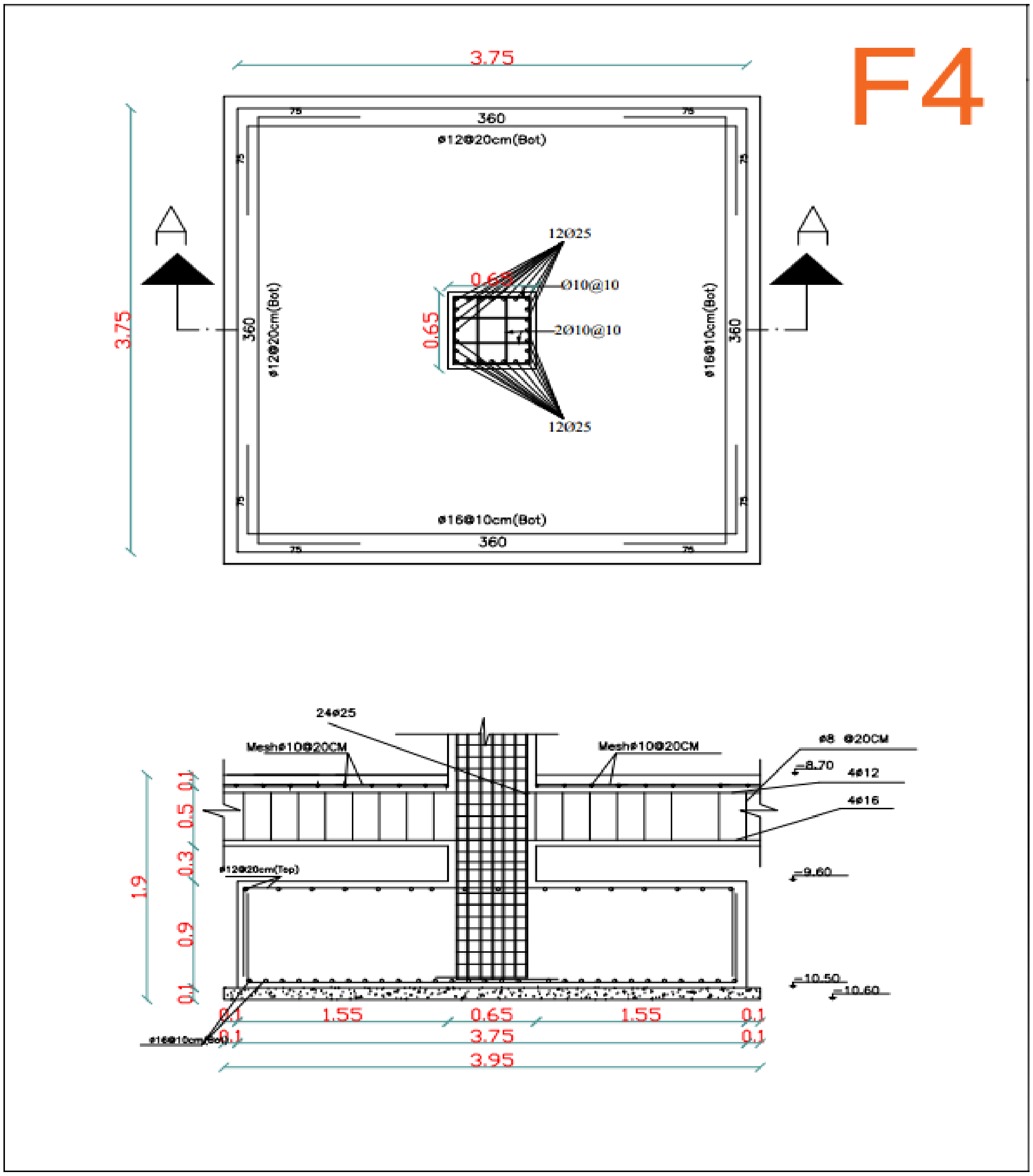


Fig 4.19:Foot Reinforcement Details.