Chapter Four
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/m3.
- Normal weight concrete with unit weight from about 1800 to 2400 kg/m3.
- Heavyweight concrete with unit weight from about 3200 to 5600 kg/m3.

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_011)**.

✓ **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,
<u>Strength provided ≥ strength required to carry factored loads.</u>

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

- ✓ Code: ACI 2011
  UBC 97
- ✓ Material:

Concrete: B300…. ($fc' = 30*0.8 = 24MPa$) .

Reinforcement: 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:
$\mathbf{W_u = 1.2\ D_L + 1.6\ L_L}$ **ACI-code-318-11(9.2.1).**

4.3 Check of minimum thickness of structural member:

TABLE 9.5(a) — MINIMUM THICKNESS OF NONPRESTRESSED BEAMS OR ONE-WAY SLABS UNLESS DEFLECTIONS ARE CALCULATED. (ACI 318M-11)

	Minimum thickness , h			
	Simply supported	One end continuous	Both end continuous	Cantilever
Member	Members not supporting or attached to partitions or other construction likely to be damaged by large deflection			
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

Table (4.1): Check of minimum thickness of structural members

For rib:
h_{min} =L /18.5 = 6.15/18.5=33.24 cm "One end continuous"

h_{min} =L /21 = 5.8/21 = 27.62cm "Both ends continuous"
select: 35 cm thickness with 27cm block and 8cm topping.

For beam:
h_{min} =L/18.5=3.9/18.5=22 cm "One end continuous"
h_{min} =L/21=4.4/21=21.1 cm "Both ends continuous"
Select h=35cm.

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

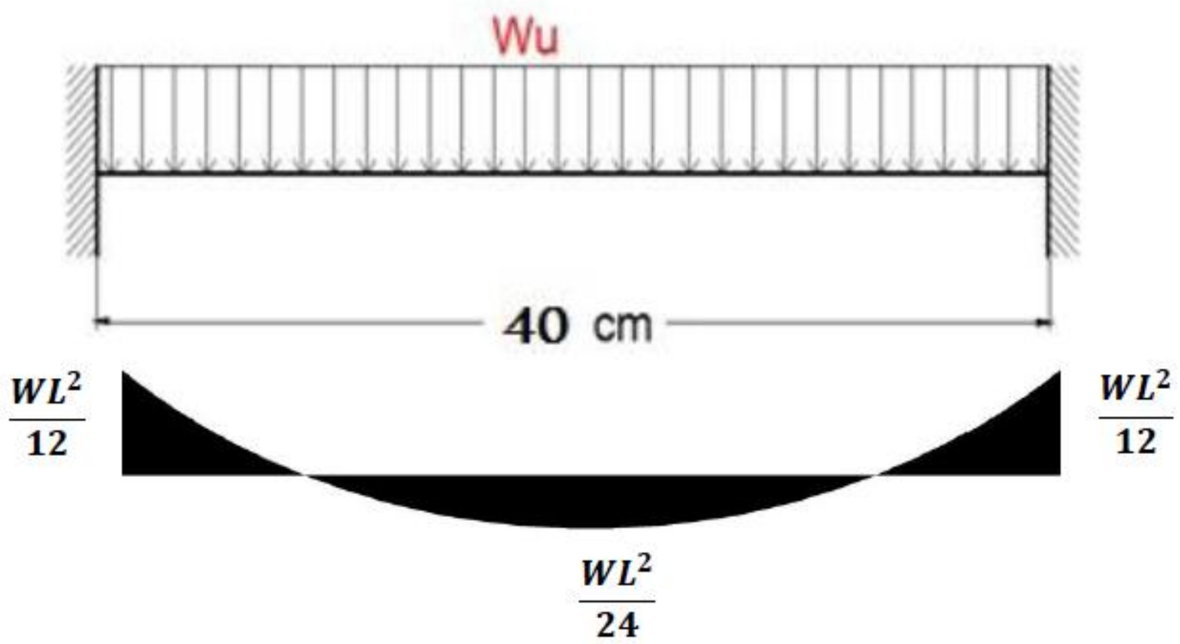


Fig 4.1: topping load and moment diagram.

For the topping, the total dead load to be used in the analysis and design is calculated as follows:

Table (4 – 2) Dead load calculation for topping

No.	Parts of Rib	Calculation
1	**Tiles**	0.03*23= 0.69 KN/m
2	**Mortar**	0.02*22= 0.44 KN/m

3	**Coarse Sand**	0.07*17= 1.19 KN
4	**Topping**	0.08*25= 2 KN/m
5	**Partitions**	1*1.5= 1.5 KN/m/
		Sum = 5.82KN/m

Nominal total dead load = 5.82KN/m^2.
Nominal total live load in room and bathroom = 2KN/m^2
Nominal total live load in corridor = 4KN/m^2.
Nominal total live load in stage = 5KN/m^2.

Design of topping for ribbed slab as a plain concrete section:-

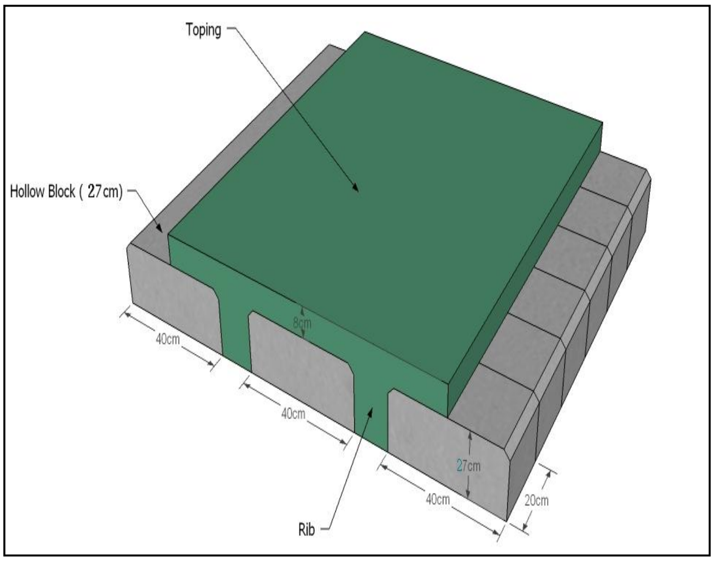


Fig. (4-2): Topping of one way rib slab

W_u = 1.2×D+1.6×L
= 14.984 KN/m. (Total factored load)

$M_u = \frac{W_u * l^2}{12} = 0.199 \text{ KN.m}$

Check the strength condition for plain concrete $\emptyset M_n > M_u$ *where* $\emptyset = 0.55$

$M_n = 0.42\lambda\sqrt{fc}$ Sm (ACI 22.5.1, EQUATION 22-2)

Where Sm=b.h2/6 $\lambda = 1$

$\emptyset M_n = 0.55 * 0.42 * \sqrt{24} * 1000 * 80^2/6 = 1.207 \text{ KN.m}$

$\emptyset M_n = 1.207 \text{ KN.m} > M_u = 0.199 \text{KN.m}$

No structural reinforcement is needed. Therefore, shrinkage and temperature reinforcement must be provided.

For the shrinkage and temperature reinforcement:-

$\rho = 0.0018$ *ACI -318-11(9.6.1.2)*

$A_s = \rho * b * h = 0.0018 * 1000 * 80 = 144 \text{ mm}^2 \text{ /m}$

Step (s) is the smallest of :

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

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

∴ Use Ø8 @ 20 cm in both directions.

Checks shear strength:

$V_u = \frac{q_u * l}{2} = 2.88 \text{ KN.m}$

$$\emptyset * Vc = \frac{0.75}{6} * \sqrt{24} * 1 * 80 = 49 \text{ KN}$$

49 >2.88

∴ No shear reinforcement is required.

4.5 Design of rib:

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

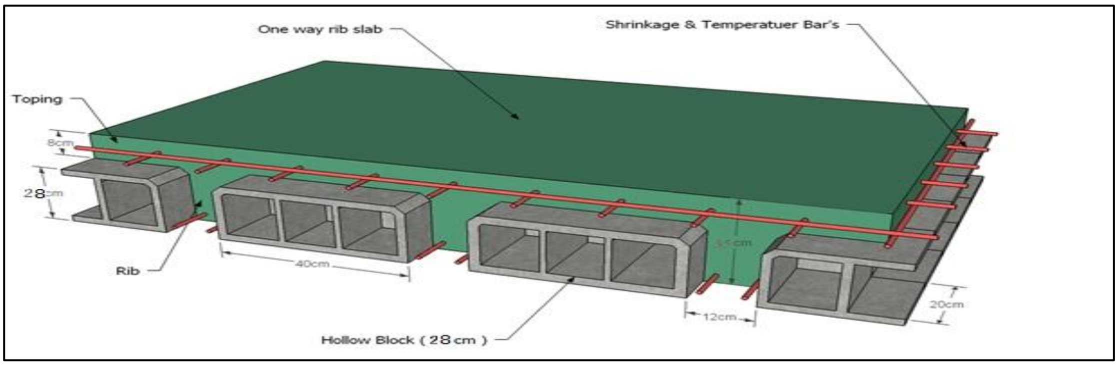


Fig. (4-3): One way rib slab

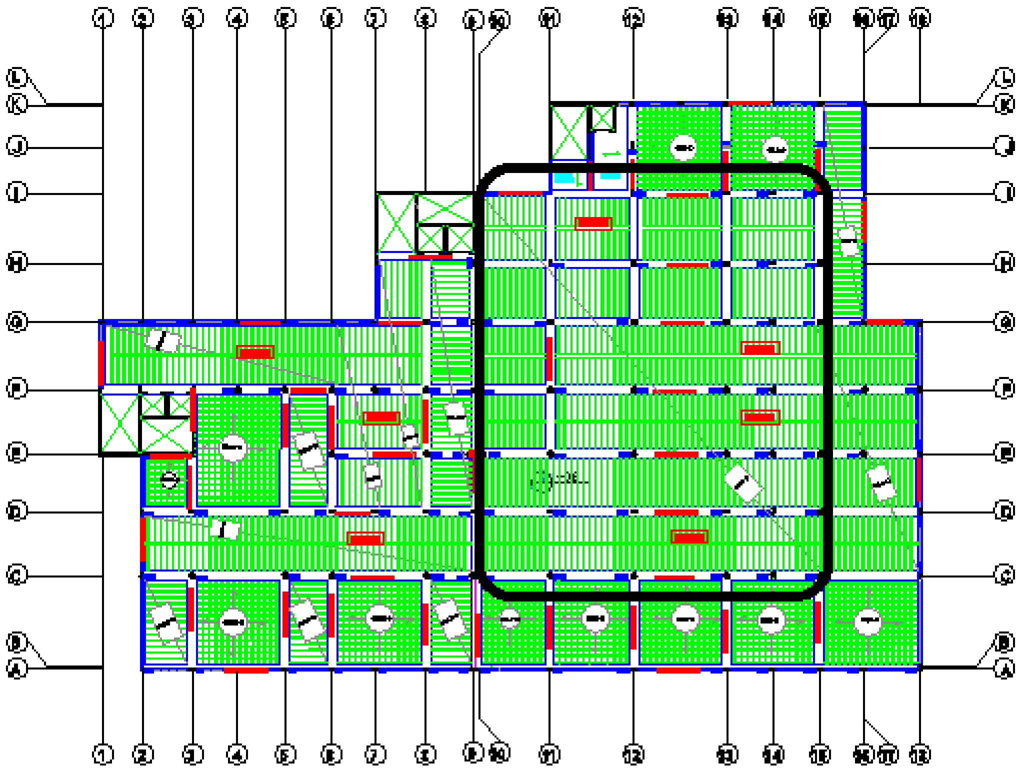


Fig 4.4: Rib 5 in basement floor.

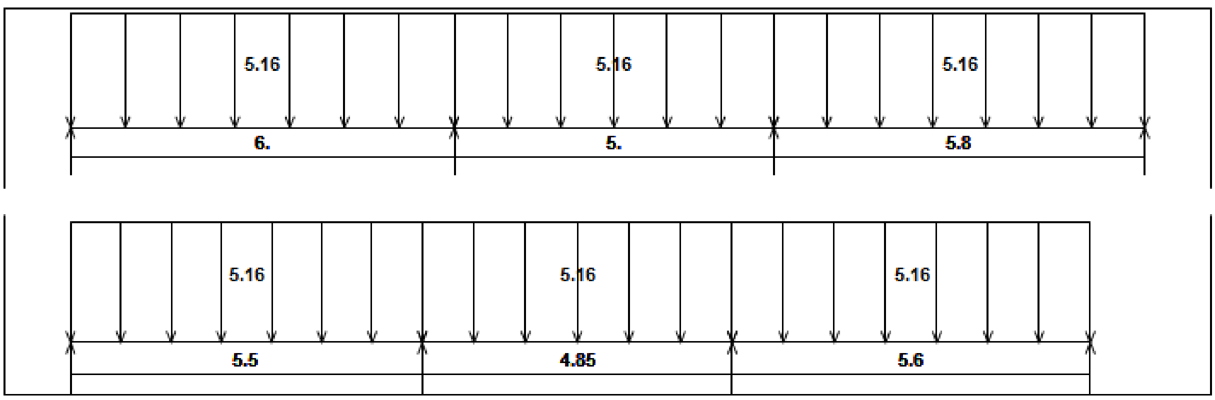


Fig 4.5: Dead load on the rib.

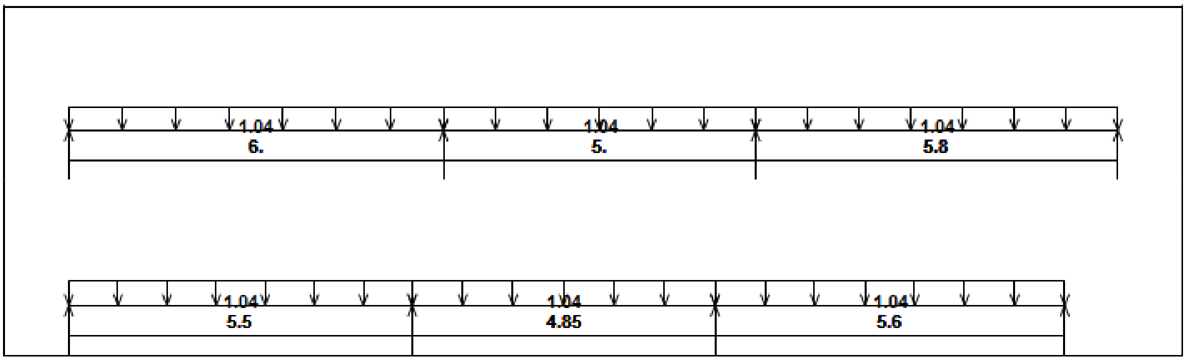


Fig 4.6: Live load on the rib.

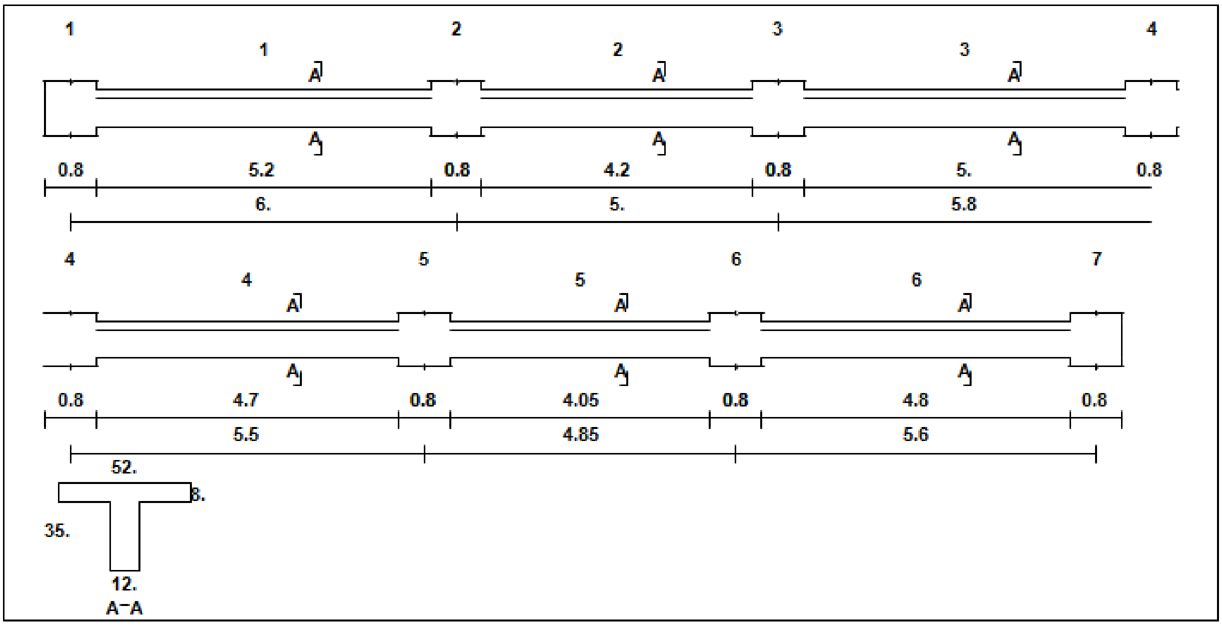


Fig 4.7: Geometry of rib and it's dimension.

Reactions

Factored							
Dead R	15.07	39.14	30.91	37.19	29.32	37.17	13.99
Live R	4.39	11.38	10.37	11.18	9.96	10.8	4.12
Max R	19.47	50.53	41.28	48.36	39.28	47.96	18.11
Min R	14.73	42.66	33.67	41.41	31.95	40.63	13.63
Service							
Dead R	12.56	32.62	25.76	30.99	24.43	30.97	11.66
Live R	2.75	7.11	6.48	6.99	6.23	6.75	2.57
Max R	15.31	39.74	32.24	37.97	30.66	37.72	14.23
Min R	12.35	34.82	27.48	33.63	26.08	33.14	11.43

Fig 4.8: Reactions of rib (live and dead).

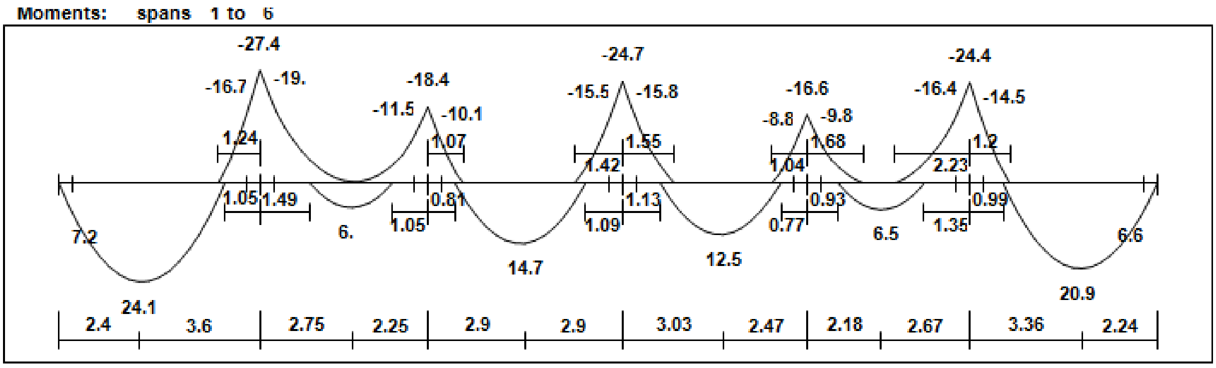


Fig 4.9: Moment diagram of rib.

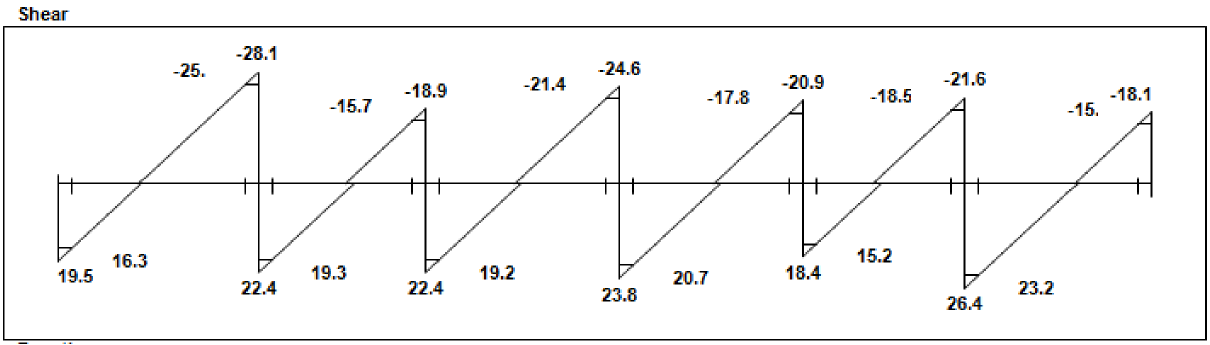


Fig 4.10: Shear diagram of rib.

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

bw ≥ 10cm……………………………………………**ACI(8.13.2)**

Select bw=12 cm

h ≤ 3.5*bw ……………………………..…**ACI(8.13.2)**

Select h=35cm<3.5*12= 49 cm

tf≥ Ln/12≥50mm …………………………………..**ACI(8.13.6.1)**

Select tf=8cm

Calculation of the total dead load for one way rib slab is shown in the following table:

Table (4 – 3) Calculation of the total dead load for one way rib slab.

No.	Material	Quality Density KN/m^3	Calculation		
1	**Topping**	25	0.52×0.08×25 = 1.04		
2	**Rib**	25	0.27×0.12×25 = 0.81		
3	**Sand**	17	0.52×0.07×17 = 0.6188		
4	**Mortar**	22	0.52×0.02×22 =0.2288		
5	**Tile**	23	0.52×0.03×23 =0.3588		
6	**Plaster**	22	0.52×0.02×22 =0.2288		
7	**Block**	10	0.4×0.27×10 = 1.08		
8	**Partitions**	1.5	1.5×0.52 = 0.78		
			∑ =	5.145	KN/m/rib

L = 2.5*0.52 = 1.3KN/m

Qu = 1.2* D = 6.17 KN/m
1.6*L = 4.16 KN/m

❖ Effective Flange Width (b_E):-**ACI-318-11 (8.10.2)**

For T- section is the smallest of the following:- b_E

= L (smallest span) / 4 = 420/ 4 =105 cm b_E

= 12 + 16 t = 12 + 16 (8) = 140 cm b_E

= $b_e \leq$ center to center spacing between adjacent beams = 52 cm ***Control*** b_E

For T-section = 52cm. b_E

Design of Rib (5):-

✓ **Moment Design for (R 5):-**

Design of Positive Moment:-

4.5.1 Design of Positive Moment for (Span1) :-(Mu=24.1 KN.m)

Assume bar diameter ø 12 for main positive reinforcement

d =h- cover - $d_{stirrups} - \frac{d_b}{2} = 350 - 20 - 10 - \frac{12}{2} = 314\ mm$

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

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

$$= 0.85 \times 24 \times 0.52 \times 0.08 \times \left(0.314 - \frac{0.08}{2}\right) \times 10^3 = 233.4\text{KN.m}$$

$M_{nf} \gg \frac{M_u}{\varphi} = \frac{24.1}{0.9}$= 26.77KN.m , the section will be designed as rectangular section with b_e =520 mm.

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{24.1\times10^6}{0.9\times520\times314^2} = 0.522\ Mpa$

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

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2\times20.6\times0.522}{420}}\right) = 0.00126$

$A_{s,req} = \rho.b.d = 0.00126 \times520\times314 = 205.6\ mm^2$

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}= 205.6 mm^2 >As_{min}= 126 mm^2 **OK**

Use 2 ø 12 ,$A_{s,provided}$= 2*113.1=226.2mm^2>$A_{s,required}$= 205. mm^2 …. Ok

$$S = \frac{120-40-20-(2\times12)}{1} = 36\ mm > d_b = 12 > 25\ \ mm \quad \boldsymbol{OK}$$

Check for strain:-

$$a = \frac{A_{s.}f_y}{0.85b\ f_c^{'}} = \frac{226.2\times420}{0.85\times520\times24} = 8.96\ mm$$

$$x = \frac{a}{\square_1} = \frac{8.96}{0.85} = 10.54\ mm$$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{314\ -10.54}{10.54}\right) = 0.0864 > 0.005 \qquad \boldsymbol{0k}$$

4.5.2 Design of Positive Moment for (Span2) :- (Mu=6 KN.m)

d =h- cover - $d_{stirrups}$$-\frac{d_b}{2} = 350 - 20 - 10 - \frac{12}{2} = 314\ mm$

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

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

$$= 0.85\times24\times0.52\times0.08\times\left(0.314 - \frac{0.08}{2}\right)\times10^3 = 233.4\text{KN.m}$$

M_{nf} $\gg\frac{M_u}{\varphi} = \frac{24.1}{0.9}$= 26.77KN.m , the section will be designed as rectangular section with b_e =520 mm.

$$R_n = \frac{M_u}{\emptyset bd^2} = \frac{6\times10^6}{0.9\times520\times314^2} = 0.13\ Mpa$$

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

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2\times20.6\times0.13}{420}}\right) = 0.00031$$

$A_{s,req}$ = ρ.b.d = 0.00031 ×520×314 = 50.78 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**

$As_{req} = 50.78\ mm^2 < As_{min} = 126\ mm^2$ **OK**

Use 2 ø 10 ,$A_{s,provided}$= 2*78.53=157.1mm^2>$A_{s,required}$= 50.78 mm^2 …. Ok

$S = \frac{120-40-20-(2\times10)}{1} = 40\ mm > d_b = 10 > 25\ mm$ $\boldsymbol{OK}$

Check for strain:-

$a = \frac{A_{s.}f_y}{0.85b\ f_c'} = \frac{157.1\times420}{0.85\times520\times24} = 6.22mm$

$x = \frac{a}{\square_1} = \frac{6.22}{0.85} = 7.31\ mm$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{314\ -7.31}{7.31}\right) = 0.125 > 0.005 \qquad \boldsymbol{0k}$$

4.5.3 Design of Positive Moment for (Span3):- (Mu=14.7KN.m)

d =h- cover - $d_{stirrups} - \frac{d_b}{2} = 350 - 20 - 10 - \frac{12}{2} = 314\ mm$

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

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

$$= 0.85 \times 24 \times 0.52 \times 0.08 \times \left(0.314 - \frac{0.08}{2}\right) \times 10^3 = 233.4\mathrm{KN.m}$$

$M_{nf} \gg \frac{M_u}{\varphi} = \frac{24.1}{0.9} = 26.77$KN.m , the section will be designed as rectangular section with b_e =520 mm.

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{14.7\times10^6}{0.9\times520\times314^2} = 0.318\ Mpa$

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

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2\times20.6\times0.318}{420}}\right) = 0.000765$

$A_{s,req} = \rho.b.d = 0.000765 \times 520 \times 314 = 124.84\ mm^2$

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}= 124.83 mm^2 < As_{min}= 126 mm^2 **OK**

Use 2 ø 10 ,$A_{s,provided}$= 2*78.5=157mm^2>$A_{s,required}$= 124.83 mm^2 …. Ok

$S = \frac{120-40-20-(2\times12)}{1} = 40\ mm > d_b = 12 > 25\ mm \qquad \boldsymbol{OK}$

Check for strain:-

$a = \frac{A_{s.}f_y}{0.85b\ f_c'} = \frac{157.1\times420}{0.85\times520\times24} = 6.22mm$

$x = \frac{a}{\beta_1} = \frac{6.22}{0.85} = 7.31\ mm$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{314\ -7.31}{7.31}\right) = 0.125 > 0.005 \qquad \boldsymbol{0k}$$

4.5.4 Design of Positive Moment for(Span4):- (Mu=12.5KN.m)

d =h- cover - $d_{stirrups} - \frac{d_b}{2} = 350 - 20 - 10 - \frac{12}{2} = 314\ mm$

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

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

$= 0.85 \times 24 \times 0.52 \times 0.08 \times \left(0.314 - \frac{0.08}{2}\right) \times 10^3 = 233.4\mathrm{KN.m}$

$M_{nf} \gg \frac{M_u}{\varphi} = \frac{24.1}{0.9}$= 26.77KN.m , the section will be designed as rectangular section with b_e =520 mm.

$R_n = \frac{M_u}{\emptyset bd^2} = \frac{12.5\times10^6}{0.9\times520\times314^2} = 0.270\ Mpa$

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

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times0.270}{420}}\right) = 0.00064$

$A_{s,req}$ = ρ.b.d = 0.00064×520×314 = 106 mm^2

Check for As min:-

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

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

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

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

As_{req}= 106 mm^2 < As_{min}= 126 mm^2 **OK**

Use 2 ø 10 ,$A_{s,provided}$= 2*78.5=157mm^2>$A_{s,required}$= 106 mm^2 …. Ok

$$S = \frac{120-40-20-(2\times12)}{1} = 40\ mm > d_b = 12 > 25\ mm \quad \boldsymbol{OK}$$

Check for strain:-

$$a = \frac{A_s.f_y}{0.85 b\ f_c'} = \frac{157.1\times420}{0.85\times520\times24} = 6.22mm$$

$$x = \frac{a}{\square_1} = \frac{6.22}{0.85} = 7.31\ mm$$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{314\ -7.31}{7.31}\right) = 0.125 > 0.005 \quad \boldsymbol{0k}$$

4.5.5 Design of Positive Moment for (Span5) :- (Mu=6.5 KN.m)

d =h- cover - $d_{stirrups}$ $-\frac{d_b}{2} = 350 - 20 - 10 - \frac{12}{2} = 314\ mm$

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

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

$= 0.85 \times 24 \times 0.52 \times 0.08 \times \left(0.314 - \frac{0.08}{2}\right) \times 10^3 = 233.4\text{KN.m}$

$M_{nf} \gg \frac{M_u}{\varphi} = \frac{24.1}{0.9} = 26.77$KN.m , the section will be designed as rectangular section with b_e =520 mm.

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{6.5\times10^6}{0.9\times520\times314^2} = 0.140 Mpa$

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

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2\times20.6\times0.140}{420}}\right) = 0.00033$

$A_{s,req} = \rho.b.d = 0.00033 \times520\times314 = 54.6$ mm^2

Check for As min:-

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

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

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

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

$As_{req} = 54.6$ mm^2 < $As_{min} = 126$ mm^2 **OK**

Use 2 ø 10 ,$A_{s,provided}$= 2*78.5=157mm^2>$A_{s,required}$= 54.6 mm^2 …. Ok

$S = \frac{120-40-20-(2\times12)}{1} = 40\ mm > d_b = 12 > 25\ mm$ $\boldsymbol{OK}$

Check for strain:-

a $=\frac{A_{s}.f_y}{0.85b\ f_c'} = \frac{157.1\times420}{0.85\times520\times24} = 6.22mm$

x$= \frac{a}{\beta_1} = \frac{6.22}{0.85} = 7.31\ mm$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{314\ -7.31}{7.31}\right) = 0.125 > 0.005 \qquad \mathbf{0}\boldsymbol{k}$$

4.5.6 Design of Positive Moment for (Span6) :- (Mu=20.9KN.m)

Assume bar diameter ø 12 for main positive reinforcement
d =h- cover - $d_{stirrups} - \frac{d_b}{2} = 350 - 20 - 10 - \frac{12}{2} = 314\ mm$

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

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

$$= 0.85 \times 24 \times 0.52 \times 0.08 \times \left(0.314 - \frac{0.08}{2}\right) \times 10^3 = 233.4\text{KN.m}$$

$M_{nf} \gg \frac{M_u}{\varphi} = \frac{24.1}{0.9} = 26.77$KN.m , the section will be designed as rectangular section with b_e =520 mm.

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{20.9\times10^6}{0.9\times520\times314^2} = 0.453\ Mpa$

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

ρ$= \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times0.453}{420}}\right) = 0.0011$
$A_{s,req}$ = ρ.b.d = 0.0011 ×520×314 = 178.1mm^2

Check for As min:-

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

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

$A\,s\ \min = \frac{1.4}{(fy)}(bw)(d)$

$A\,s\ \min = \frac{1.4}{420}(120)(315) = 126mm^2$ **controls**

As_{req}= 178.2mm^2 >As_{min}= 126 mm^2 **OK**

Use 2 ø 12 ,$A_{s,provided}$= 2*113.1=226.2mm^2>$A_{s,required}$= 178.1 mm^2 …. Ok

$S = \frac{120-40-20-(2\times12)}{1} = 36\,mm > d_b = 12 > 25\ mm$ $\boldsymbol{OK}$

Check for strain:-

$a = \frac{A_{s.}f_y}{0.85b\ f_c^{'}} = \frac{226.2\times420}{0.85\times520\times24} = 8.96\,mm$

$x = \frac{a}{\beta_1} = \frac{8.96}{0.85} = 10.54\,mm$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{314\ -10.54}{10.54}\right) = 0.0864 > 0.005 \qquad \boldsymbol{0k}$$

Design of Negative Moment:-

4.5.7 Design of Negative Moment for(Support1):- (Mu=-19 KN.m)

Assume bar diameter ø 12 for main positive reinforcement

d =h- cover - $d_{stirrups} - \frac{d_b}{2} = 350 - 20 - 10 - \frac{12}{2} = 314\ mm$

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{19\times10^6}{0.9\times120\times314^2} = 1.78\ Mpa$

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

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times0.453}{420}}\right) = 0.0044$

$A_{s,req} = \rho.b.d = 0.0044 \times120\times314 = 165.8mm^2$

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**

$As_{req} = 165.8mm^2 > As_{min} = 126\ mm^2$ **OK**

Use 2 ø 12 ,$A_{s,provided}$= 2*113.1=226.2mm^2>$A_{s,required}$= 165.8 mm^2 …. Ok

$S = \frac{120-40-20-(2\times12)}{1} = 36\ mm > d_b = 12 > 25\ mm$ $\boldsymbol{OK}$

Check for strain:-

a $=\frac{A_{s}.f_{y}}{0.85b\ f_{c}'}=\frac{226.2\times420}{0.85\times120\times24}=38.8mm$

x$=\frac{a}{\beta_1}=\frac{38.8}{0.85}=45.65\ mm$

$$\varepsilon_s=0.003\left(\frac{d-x}{x}\right)=0.003\left(\frac{314\ -45.65}{45.65}\right)=0.0176>0.005 \qquad \mathbf{0k}$$

4.5.8 Design of Negative Moment for(Support2):- (Mu=-11.5 KN.m)

Assume bar diameter ø 12 for main positive reinforcement
d =h- cover - $d_{stirrups}-\frac{d_b}{2}=350-20-10-\frac{12}{2}=314\ mm$

$R_n=\frac{M_u}{\emptyset bd^2}=\frac{11.5\times10^6}{0.9\times120\times314^2}=1.07Mpa$

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

ρ$=\frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right)=\frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times0.453}{420}}\right)=0.00262$

$A_{s,req}$ = ρ.b.d = 0.0044 ×120×314 = 99.04 mm^2

Check for As min:-

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

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

$A\,s\ \min=\frac{1.4}{(fy)}(bw)(d)$

$A\,s\ \min=\frac{1.4}{420}(120)(315)=126mm^2$ **controls**

As_{req}= 99.04mm^2 < As_{min}= 126 mm^2 **OK**

Use 2 ø 10 ,$A_{s,provided}$= 2*78.5=157mm^2>$A_{s,required}$= 99.04 mm^2 …. Ok

$$S = \frac{120-40-20-(2\times10)}{1} = 40\ mm > d_b = 12 > 25\ mm \qquad \boldsymbol{OK}$$

Check for strain:-

$$a = \frac{A_{s.}f_y}{0.85b\ f_c^{'}} = \frac{157\times420}{0.85\times120\times24} = 26.93mm$$

$$x= \frac{a}{\beta_1} = \frac{26.93}{0.85} = 31.7\ mm$$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{314\ -31.7}{31.7}\right) = 0.0267 > 0.005 \qquad \boldsymbol{0k}$$

4.5.9 Design of Negative Moment for(Support3):- (Mu=-15.8 KN.m)

Assume bar diameter ø 12 for main positive reinforcement

d =h- cover - $d_{stirrups}$$-\frac{d_b}{2} = 350-20-10-\frac{12}{2} = 314\ mm$

$$R_n= \frac{M_u}{\emptyset bd^2} = \frac{15.8\times10^6}{0.9\times120\times314^2} = 1.48Mpa$$

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

$$\rho= \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times1.48}{420}}\right) = 0.0036$$

$A_{s,req}$ = ρ.b.d = 0.0036×120×314 = 135.65 mm^2

Check for As min:-

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

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

$$A\,s\ \min = \frac{1.4}{(fy)}(bw)(d)$$

$$A\,s\ \min = \frac{1.4}{420}(120)(315) = 126mm^2$$ **controls**

As_{req}= 135.65mm^2 > As_{min}= 126 mm^2 **OK**

Use 2 ø 10 ,$A_{s,provided}$= 2*78.5=157mm^2>$A_{s,required}$= 135.6 mm^2 …. Ok

$$S = \frac{120-40-20-(2\times10)}{1} = 40\ mm > d_b = 12 > 25\ \ mm \qquad \boldsymbol{OK}$$

Check for strain:-

$$a = \frac{A_{s.}f_y}{0.85b\ f_c'} = \frac{157\times420}{0.85\times120\times24} = 26.93mm$$

$$x = \frac{a}{□_1} = \frac{26.93}{0.85} = 31.7\ mm$$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{314\ -31.7}{31.7}\right) = 0.0267 > 0.005 \qquad \boldsymbol{0k}$$

4.5.10 Design of Negative Moment for(Support4):- (Mu=-9.8 KN.m)

Assume bar diameter ø 12 for main positive reinforcement

d =h- cover - $d_{stirrups}$$-\frac{d_b}{2} = 350-20-10-\frac{12}{2} = 314\ mm$

$$R_n = \frac{M_u}{\emptyset bd^2} = \frac{15.8\times10^6}{0.9\times120\times314^2} = 0.914Mpa$$

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

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times0.914}{420}}\right) = 0.0022$$

$A_{s,req} = \rho.b.d = 0.0022\times120\times314 = 84.2\ mm^2$

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**

$As_{req} = 84.2mm^2 < As_{min} = 126\ mm^2$ **OK**

Use 2 ø 10 ,$A_{s,provided}$= 2*78.5=157mm^2>$A_{s,required}$= 84.2 mm^2 …. Ok

$$S = \frac{120-40-20-(2\times10)}{1} = 40\ mm > d_b = 12 > 25\ \ mm \qquad \boldsymbol{OK}$$

Check for strain:-

$$a = \frac{A_{s}.f_y}{0.85b\ f_c'} = \frac{157\times420}{0.85\times120\times24} = 26.93mm$$

$$x = \frac{a}{\beta_1} = \frac{26.93}{0.85} = 31.7\ mm$$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{314\ -31.7}{31.7}\right) = 0.0267 > 0.005 \qquad \boldsymbol{0k}$$

4.5.11 Design of Negative Moment for(Support5):- (Mu=-16.4 KN.m)

Assume bar diameter ø 12 for main positive reinforcement

d =h- cover - $d_{stirrups} - \frac{d_b}{2} = 350 - 20 - 10 - \frac{12}{2} = 314\ mm$

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{16.4 \times 10^6}{0.9 \times 120 \times 314^2} = 1.53 Mpa$

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

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 1.53}{420}}\right) = 0.0038$

$A_{s,req} = \rho.b.d = 0.0038 \times 120 \times 314 = 143.65\ mm^2$

Check for As min:-

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

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

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

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

$As_{req} = 143.65 mm^2 > As_{min} = 126\ mm^2$ **OK**

Use 2 ø 12 ,$A_{s,provided}$= 2*113=226mm^2>$A_{s,required}$= 143.65 mm^2 …. Ok

$S = \frac{120 - 40 - 20 - (2 \times 12)}{1} = 36\ mm > d_b = 10 > 25\ mm$ $\boldsymbol{OK}$

Check for strain:-

a $= \frac{A_{s.}f_y}{0.85b\ f_c'} = \frac{226\times420}{0.85\times120\times24} = 38.77mm$

x= $\frac{a}{\beta_1} = \frac{38.77}{0.85} = 45.6\ mm$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{314\ -45.6}{45.6}\right) = 0.0176 > 0.005 \qquad \mathbf{0k}$$

✓ **Shear Design for (R 5):-**

V_u at distance d from support= 25 KN (for Span1)
Shear strength V_c, provided by concrete for the joists may be taken 10% greater than for beams. This is mainly due to the interaction between the slab and closely spaced ribs.**(ACI, 8.13.8).**

$V_c = \frac{1.1}{6}\sqrt{f_c'}b_w d = \frac{1.1}{6}\sqrt{24}\ \times 120\times 315\times 10^{-3} = 40\ KN$

$øV_c$ =0.75×40 =30 KN

0.5 ø V_c =0.5×30 =15 KN

0.5 ø V_c< V_u< ø V_c
V_u> ø V_c

for shear design, shear reinforcement is required ($A_{v,}$),

$Vs_{min} = \frac{1}{16}\sqrt{f_c'}bw\ d \geq \frac{1}{3}bw\ d$

Vs min$=\frac{1}{16}\sqrt{24} * 120 * 315 * 10^{-3} = 11.57kn$

$Vs_{min} = \frac{1}{3}\ bw\ d = \frac{1}{3} * 120 * 315 * 10^{-3} = 12.6kn$

ø(V_c+Vs_{min})= 0.75(40+12.6)=39.45kn

øV_c<Vu <ø (V_c+Vs_{min})

22.96 < 23 <31.4775

for shear design, minimum shear reinforcement is required ($A_{v,min}$), Reinforcement.

50.24 = 100.5 mm^2 ×Use stirrups (2 leg stirrups) ø 8 @ 150 mm , A_v = 2

$Av_{min} = \frac{1}{16}\sqrt{f_c'}\frac{b_w s}{fyt} \geq \frac{1}{3}\frac{b_w s}{fyt}$

Av_{min}=100.5 =$\frac{1}{16}\sqrt{24}\,\frac{120s}{420} \rightarrow s = 1.145m$

100.5 =$\frac{1}{3}\frac{120s}{420} \rightarrow s = 1.055m$

S max$\rightarrow \frac{d}{2} = 142mm$

S max →≤600mm

Take (2 leg stirrups) ø 8 @ 150 mm

$A_v = \frac{2*50.3}{0.15} = 670.67$ mm^2/m_{strip}

4.6 Design Beam (149) at the Electrical wiring Floor Slab:

Material :-

concrete B300 Fc' = 24 N/mm^2
Reinforcement Steel fy = 420 N/mm^2

By using **ATIR** program we get the envelope moment and shear force diagram as the follows:-

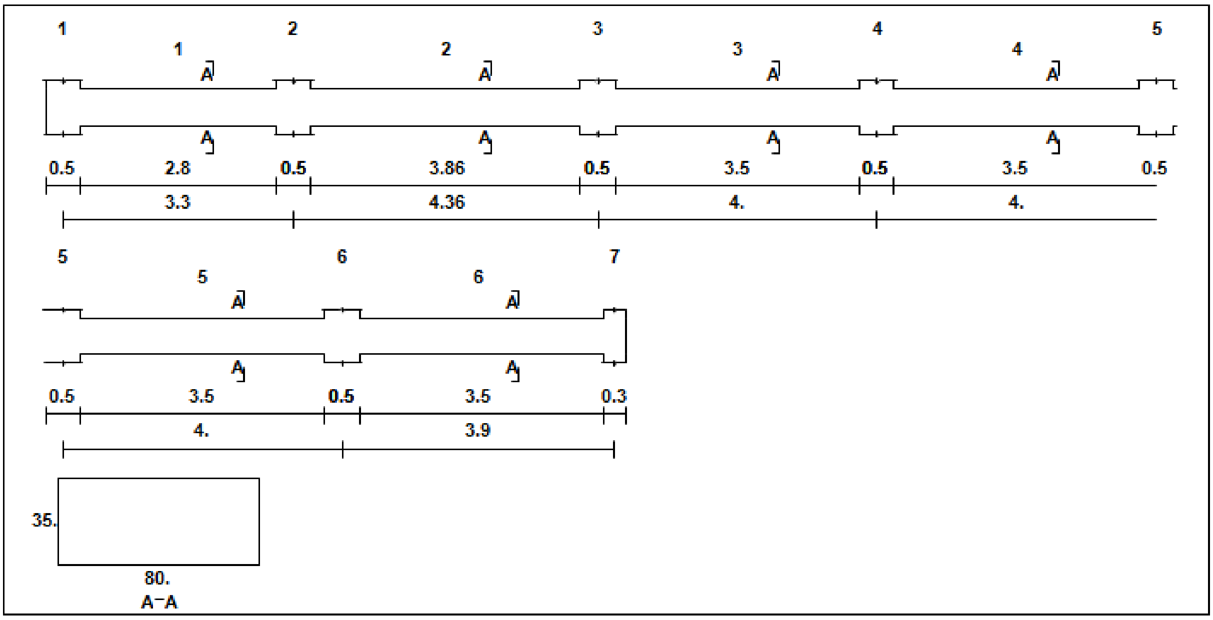


Fig. (4-11) : Beam geometry.

Load of beam :-

Load of this beam come from reaction of Rib57 as following :

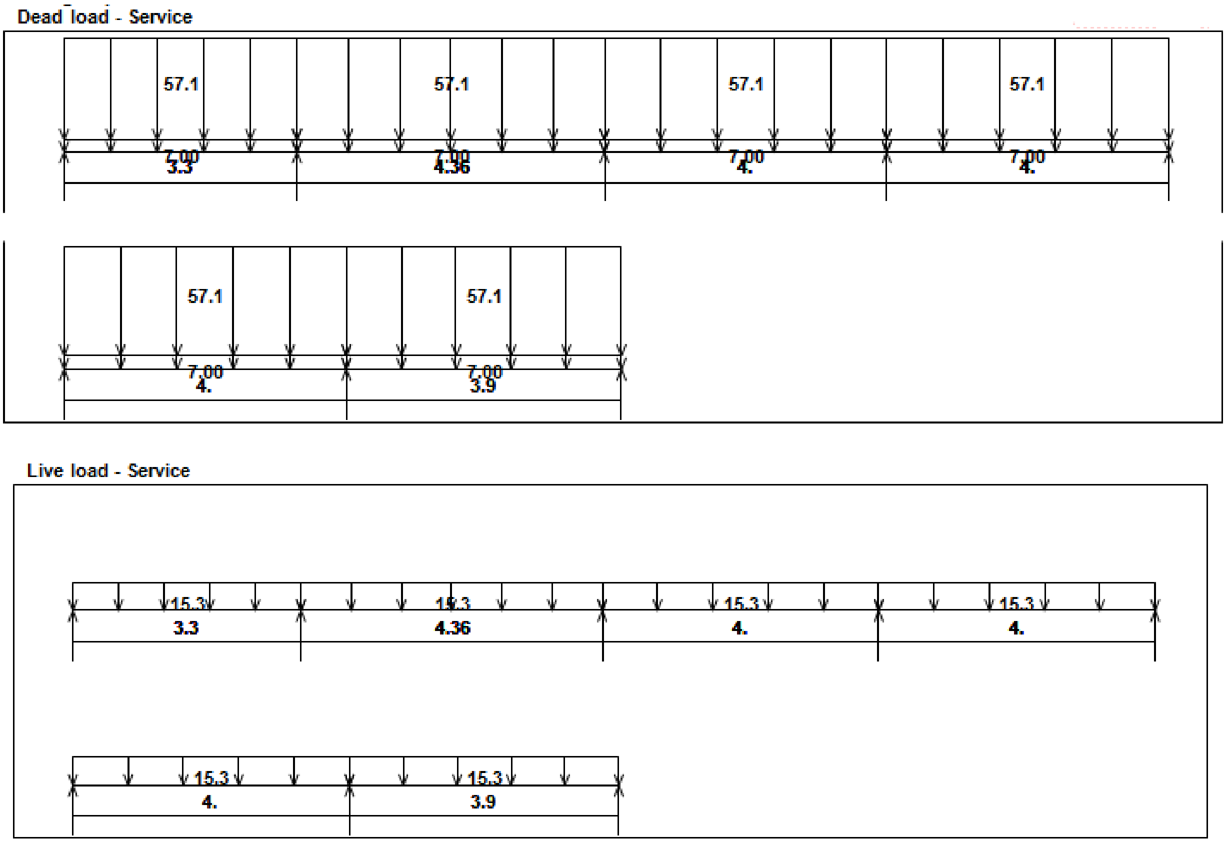


Fig. (4-12) : Load on the beam.

»Self weight of beam = (0.35*0.8) *25 = 7 KN/m

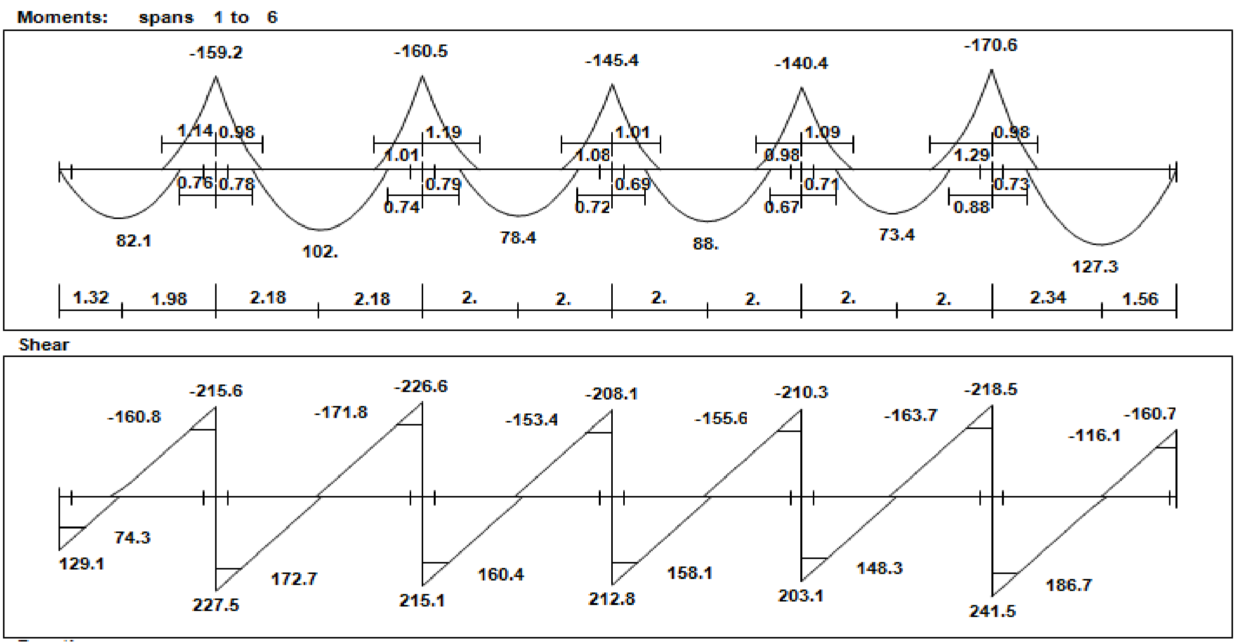


Figure (4-13) :Moment & Shear Diagram in beam

- ✓ **Load Calculations:-**

Dead Load Calculations for Beam(B 149):-

The distributed Dead and Live loads acting upon B149 can be defined from the support reactions R57 in Electrical floor.

From Rib 57
The maximum support reaction from Dead Loads for R44 upon B149 is 57.1KN.
Self-weight = 0.8*0.35*25= 7 KN
DL =57.1/ 0.52= 109.8 +7 = 116.8 KN/m

Live Load calculations for Beam (B 149):-

From Rib 57

The maximum support reaction from Live Loads for CE120 upon B149 is 15.3KN.
The distributed Live Load from the Rib 57 on B149.
LL =15.3/ 0.52= 29.4 KN/m.

- ✓ **Moment Design for (B 149):-**

Design of Positive Moment

4.6.1 Flexural Design of Positive Moment for(Span1):-(Mu=82.1 KN.m)

Assume bars of Ø 16

Determine of $M_{n,max}$

d =350 – 40 -10 – 16\2 = 292mm

$x = \frac{3}{7} d = \frac{3}{7}.292 = 125.14mm$

$a = \mathcal{B}.x = 125.14 * 0.85 = 106.37\ mm$

Mn_{max}= 0.85* f_c' *a *b (d - $\frac{a}{2}$) = 0.85*24*106.37*800*(292-106.37/2) *10^{-6}= 414.57 KN.m

Ø Mn_{max} = 0.9* 414.57= 373.12KN.m >82.8KN.m.

Design as singly reinforcement

$$Rn = \frac{M_u}{\emptyset b d^2} = \frac{82.1\times10^6}{0.9\times800\times292^2} = 1.37\ Mpa$$

$$\text{m}= \frac{f_y}{0.85 f_c^{\prime}} = \frac{420}{0.85\times24} = 20.6$$

$$\rho= \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times4.7}{420}}\right) = 0.00338$$

A_s = ρ.b.d = 0.00338 ×800×292 =786.5 mm^2

Check for $A_{s,min}$:-

$$As_{min} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) = \frac{\sqrt{24}}{4*420}*800*292 = 681.19mm^2$$

$$As_{min} = \frac{1.4}{(fy)}(bw)(d) = \frac{1.4}{420}*800*292 = 778.77mm^2$$

A_s = 778.77mm^2 **Controls**

Use 4ø 16,$A_{s,provided}$= 804 mm^2>$A_{s,required}$= 778.77mm^2… Ok

Check spacing :-

$$S = \frac{800-40*2-20-(4*16)}{3} = 212\ mm > d_b = 16 > 25 \qquad \boldsymbol{OK}$$

Check for strain:-

$$a = \frac{A_s.f_y}{0.85 b\ f_c^{\prime}} = \frac{804\ \times420}{0.85\times800\times24} = 20.96\ mm$$

$$x= \frac{a}{\beta_1} = \frac{20.96}{0.85} = 24.35 mm$$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{292-24.35}{84.75}\right) = 0.0329 > 0.005 \qquad \boldsymbol{0k}$$

4.6.2 Flexural Design of Positive Moment for(Span2):-(Mu=102 KN.m)

$Rn = \frac{M_u}{\emptyset bd^2} = \frac{102\times10^6}{0.9\times800\times292^2} = 1.6Mpa$

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

ρ= $\frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times1.6}{420}}\right) = 0.0041$

A_s = ρ.b.d = 0.0041 ×800×292 =975.7 mm^2

Check for $A_{s,min}$:-

$As_{min} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) = \frac{\sqrt{24}}{4*420}*800*292 = 681.19mm^2$

$As_{min} = \frac{1.4}{(fy)}(bw)(d) = \frac{1.4}{420}*800*292 = 778.77mm^2$

A_s = 975.7mm^2 **Controls**

Use 4ø 18,$A_{s,provided}$= 1016 mm^2>$A_{s,required}$= 975.7mm^2… Ok

Check spacing :-

S = $\frac{800-40*2-20-(4*18)}{3} = 210\ mm > d_b = 18 > 25$ ***OK***

Check for strain:-

a =$\frac{A_s.f_y}{0.85b\ f_c'} = \frac{1016\times420}{0.85\times800\times24} = 26.15\ mm$

x= $\frac{a}{\mathcal{B}_1} = \frac{26.15}{0.85} = 30.8mm$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{292-30.8}{30.8}\right) = 0.0254 > 0.005 \qquad \mathbf{0}\boldsymbol{k}$$

4.6.3 Flexural Design of Positive Moment for(Span3):-(Mu=78.4 KN.m)

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{78.4\times10^6}{0.9\times800\times291^2} = 1.27Mpa$$

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

ρ= $\frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times1.27}{420}}\right) = 0.00304$

A_s = ρ.b.d = 0.00304 ×800×291 =710 mm^2

Check for $A_{s,min}$:-

$As_{min} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) = \frac{\sqrt{24}}{4*420}*800*292 = 681.19mm^2$

$As_{min} = \frac{1.4}{(fy)}(bw)(d) = \frac{1.4}{420}*800*292 = 778.77mm^2$

A_s = 778.77mm^2**Controls**

Use 4ø 16,$A_{s,provided}$= 804 mm^2>$A_{s,required}$= 778.77mm^2… Ok

Check spacing :-

S = $\frac{800-40*2-20-(4*16)}{3} = 212\ mm > d_b = 16 > 25$ ***OK***

Check for strain:-

a =$\frac{A_s.f_y}{0.85b\ f_c'} = \frac{804\times420}{0.85\times800\times24} = 20.96\ mm$

x= $\frac{a}{B_1} = \frac{20.96}{0.85} = 24.43mm$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{292-24.43}{24.43}\right) = 0.0329 > 0.005 \qquad \mathbf{0k}$$

4.6.4 Flexural Design of Positive Moment for(Span4):-(Mu=88 KN.m)

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{88\times10^6}{0.9\times800\times291^2} = 1.44Mpa$$

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

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times1.27}{420}}\right) = 0.00356$$

$A_s = \rho.b.d = 0.00356 \times800\times291 = 831.6\ mm^2$

Check for $A_{s,min}$:-

$$As_{min} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) = \frac{\sqrt{24}}{4*420}*800*292 = 681.19mm^2$$

$$As_{min} = \frac{1.4}{(fy)}(bw)(d) = \frac{1.4}{420}*800*292 = 778.77mm^2$$

$A_s = 831.6mm^2$ **Controls**

Use 4ø 18, $A_{s,provided}$= 1016 mm^2 > $A_{s,required}$= 831.6mm^2… Ok

Check spacing :-

$$S = \frac{800-40*2-20-(4*18)}{3} = 210\ mm > d_b = 16 > 25 \qquad \boldsymbol{OK}$$

Check for strain:-

$$a = \frac{A_{s.}f_y}{0.85b\ f_c'} = \frac{1016\times420}{0.85\times800\times24} = 26.15\ mm$$

$$x = \frac{a}{\mathcal{B}_1} = \frac{26.15}{0.85} = 30.77mm$$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{292-30.77}{30.77}\right) = 0.0254 > 0.005 \qquad \boldsymbol{0k}$$

4.6.5 Flexural Design of Positive Moment for(Span5):-(Mu=73.4 KN.m)

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{73.4\times10^6}{0.9\times800\times291^2} = 1.20Mpa$$

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

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times1.20}{420}}\right) = 0.00295$$

$A_s = \rho.b.d = 0.00295 \times800\times292 = 688.3\ mm^2$

Check for $A_{s,min}$:-

$$As_{min} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) = \frac{\sqrt{24}}{4*420}*800*292 = 681.19mm^2$$

$$As_{min} = \frac{1.4}{(fy)}(bw)(d) = \frac{1.4}{420}*800*292 = 778.77mm^2$$

$A_s = 778.77mm^2$ **Controls**

Use 4ø 16,$A_{s,provided}$= 804 mm^2>$A_{s,required}$= 778.77mm^2… Ok

Check spacing :-

$$S = \frac{800-40*2-20-(4*16)}{3} = 212\ mm > d_b = 16 > 25 \qquad \boldsymbol{OK}$$

Check for strain:-

$$a = \frac{A_{s.}f_y}{0.85b\ f_c'} = \frac{804\times420}{0.85\times800\times24} = 20.96\ mm$$

$$x = \frac{a}{\beta_1} = \frac{20.96}{0.85} = 24.43mm$$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{292-24.43}{24.43}\right) = 0.0329 > 0.005 \qquad \mathbf{0}\boldsymbol{k}$$

4.6.6 Flexural Design of Positive Moment for(Span6):-(Mu=127.3 KN.m)

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{127.3\times10^6}{0.9\times800\times291^2} = 2.07Mpa$$

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

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times2.07}{420}}\right) = 0.0052$$

$A_s = \rho.b.d = 0.0052 \times800\times292 = 1218.8\ mm^2$

Check for $A_{s,min}$:-

$$As_{min} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) = \frac{\sqrt{24}}{4*420}*800*292 = 681.19\text{mm}^2$$

$$As_{min} = \frac{1.4}{(fy)}(bw)(d) = \frac{1.4}{420}*800*292 = 778.77\text{mm}^2$$

A_s = 1218.8mm^2**Controls**

Use 5ø 18,$A_{s,provided}$= 1270 mm^2>$A_{s,required}$= 1218.8mm^2… Ok

Check spacing :-

$$S = \frac{800-40*2-20-(5*18)}{3} = 152.5\,mm > d_b = 18 > 25 \qquad \boldsymbol{OK}$$

Check for strain:-

$$a = \frac{A_{s.}f_y}{0.85b\,f_c'} = \frac{1270\times420}{0.85\times800\times24} = 32.68\,mm$$

$$x = \frac{a}{\beta_1} = \frac{30.68}{0.85} = 38.45mm$$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{292-38.45}{38.45}\right) = 0.02 > 0.005 \qquad \boldsymbol{0k}$$

4.6.7 Flexural Design of Negative Moment for(Support1):-(Mu=-107.6 KN.m)

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{107.6\times10^6}{0.9\times800\times292^2} = 1.75Mpa$$

$$M = \frac{f_y}{0.85f_c'} = \frac{420}{0.85\times24} = 20.59$$

$$P = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.59}\left(1-\sqrt{1-\frac{2\times20.59\times1.75}{420}}\right) = 0.0044MPa$$

A_s = ρ.b.d = 0.0044×800×292= 1027.84 mm^2

Check for $A_{s,min}$:-

$As_{min} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) = \frac{\sqrt{24}}{4*420}*800*292 = 681.19mm^2$

$As_{min} = \frac{1.4}{(fy)}(bw)(d) = \frac{1.4}{420}*800*292 = 778.77\ mm^2$Controls

A_s= 1027.84mm^2

Use 4ø 20 Top, $A_{s,provided}$= 1256.6mm^2>$A_{s,required}$= 1027.84mm^2… Ok

Check spacing :-

$S = \frac{800-40*2-20-(20\times4)}{3} = 206.67\ mm > d_b = 20 > 25\ mmOK$

Check for strain:-

$a = \frac{As.fy}{0.85bf_c'} = \frac{1256.6\times420}{0.85\times800\times24} = 32.34mm$

$x = \frac{a}{\beta_1} = \frac{50.53}{0.85} = 38\ mm$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{292-38}{38}\right) = 0.02 > 0.005$$

4.6.8 Flexural Design of Negative Moment for(Support2):-(Mu=-107.9 KN.m)

$Rn = \frac{M_u}{\emptyset bd^2} = \frac{107.9\times10^6}{0.9\times800\times292^2} = 1.75Mpa$

$M = \frac{f_y}{0.85f_c'} = \frac{420}{0.85\times24} = 20.59$

$P = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.59}\left(1-\sqrt{1-\frac{2\times20.59\times1.75}{420}}\right) = 0.0044MPa$

$A_s = \rho.b.d = 0.0044\times800\times292$= 1027.84 mm^2

Check for $A_{s,min}$:-

$As_{min} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) = \frac{\sqrt{24}}{4*420}*800*292 = 681.19mm^2$

$As_{min} = \frac{1.4}{(fy)}(bw)(d) = \frac{1.4}{420}*800*292 = 778.77\ mm^2$Controls

A_s= 1027.84mm^2

Use 4ø 20 Top, $A_{s,provided}$= 1256.6mm^2>$A_{s,required}$= 1027.84mm^2… Ok

Check spacing :-

$S = \frac{800-40*2-20-(20\times4)}{3} = 206.67\ mm > d_b = 20 > 25\ mmOK$

Check for strain:-

a $=\frac{A_{s}.f_{y}}{0.85bf'_c}=\frac{1256.6\times420}{0.85\times800\times24}=32.34mm$

x$=\frac{a}{\mathcal{B}_1}=\frac{50.53}{0.85}=38\ mm$

$$\varepsilon_s=0.003\left(\frac{d-x}{x}\right)=0.003\left(\frac{292-38}{38}\right)=0.02>0.005$$

4.6.9 Flexural Design of Negative Moment for(Support3):-(Mu=-96.9 KN.m)

$$Rn=\frac{M_u}{\emptyset bd^2}=\frac{96.9\times10^6}{0.9\times800\times292^2}=1.58Mpa$$

m$=\frac{f_y}{0.85f'_c}=\frac{420}{0.85\times24}=20.59$

ρ$=\frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right)=\frac{1}{20.59}\left(1-\sqrt{1-\frac{2\times20.59\times1.58}{420}}\right)=0.0039MPa$

A_s = ρ.b.d = 0.0039×800×292= 911 mm^2

Check for $A_{s,min}$:-

$As_{min}=\frac{\sqrt{fc'}}{4(fy)}(bw)(d)=\frac{\sqrt{24}}{4*420}*800*292=681.19mm^2$

$As_{min}=\frac{1.4}{(fy)}(bw)(d)=\frac{1.4}{420}*800*292=778.77\ mm^2$Controls

A_s= 911mm^2

Use 4ø 20 Top, $A_{s,provided}$= 1256.6mm^2>$A_{s,required}$= 911 mm^2… Ok

Check spacing :-

S $=\frac{800-40*2-20-(20\times6)}{3}=206.7mm>d_b=20>25\ mmOK$

Check for strain:-

a $=\frac{A_{s}.f_{y}}{0.85bf'_c}=\frac{1256.6\times420}{0.85\times800\times24}=32.34mm$

x$=\frac{a}{\mathcal{B}_1}=\frac{32.34}{0.85}=38\ mm$

$$\varepsilon_s=0.003\left(\frac{d-x}{x}\right)=0.003\left(\frac{292-38}{38}\right)=0.02>0.005$$

4.6.10 Flexural Design of Negative Moment for(Support 4):-(Mu=-91.6 KN.m)

$$Rn = \frac{M_u}{\emptyset b d^2} = \frac{91.6 \times 10^6}{0.9 \times 800 \times 292^2} = 1.49 Mpa$$

$$\text{M} = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 \times 24} = 20.59$$

$$\text{P} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2 \times 20.59 \times 1.49}{420}}\right) = 0.00368 MPa$$

$A_s = \rho.b.d = 0.00368 \times 800 \times 292 = 859.65\ mm^2$

Check for $A_{s,min}$:-

$$As_{min} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) = \frac{\sqrt{24}}{4 * 420} * 800 * 292 = 681.19 mm^2$$

$$As_{min} = \frac{1.4}{(fy)}(bw)(d) = \frac{1.4}{420} * 800 * 292 = 778.77\ mm^2$$ Controls

$A_s = 859.65 mm^2$

Use 4ø 20 Top, $A_{s,provided}$= 1256.6mm^2>$A_{s,required}$= 859.65 mm^2… Ok

Check spacing :-

$$S = \frac{800-40*2-20-(20\times4)}{3} = 206.7\ mm > d_b = 20 > 25\ mm OK$$

Check for strain:-

$$a = \frac{A_{s.}f_y}{0.85 b f_c'} = \frac{1256.6 \times 420}{0.85 \times 800 \times 24} = 32.34\ mm$$

$$x = \frac{a}{\mathcal{B}_1} = \frac{64.98}{0.85} = 38\ mm$$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{292-38}{38}\right) = 0.02 > 0.005$$

4.6.11 Flexural Design of Negative Moment for(Support 5):-(Mu=-123.9 KN.m)

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{123.9 \times 10^6}{0.9 \times 800 \times 292^2} = 2.02Mpa$$

M= $\frac{f_y}{0.85f_c'} = \frac{420}{0.85\times24} = 20.59$

P= $\frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.59}\left(1-\sqrt{1-\frac{2\times20.59\times2.02}{420}}\right) = 0.0051MPa$

A_s = ρ.b.d = 0.0051×800×292= 1191.36 mm^2

Check for $A_{s,min}$:-

$As_{min} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) = \frac{\sqrt{24}}{4*420}*800*292 = 681.19mm^2$

$As_{min} = \frac{1.4}{(fy)}(bw)(d) = \frac{1.4}{420}*800*292 = 778.77$ mm^{2}Controls

A_s= 1191.36mm^2

Use 4ø 20 Top, $A_{s,provided}$= 1256.6mm^2>$A_{s,required}$= 1191.36 mm^2… Ok

Check spacing :-

S = $\frac{800-40*2-20-(20\times4)}{3} = 206.7\ mm > d_b = 20 > 25\ mmOK$

Check for strain:-

a =$\frac{A_{s.}f_y}{0.85bf_c'} = \frac{1256.6\times420}{0.85\times800\times24} = 32.34\ mm$

x= $\frac{a}{\beta_1} = \frac{64.98}{0.85} = 38\ mm$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{292-38}{38}\right) = 0.02 > 0.005$$

4.6.12 Shear Design for (B 149):-

V_u max = 186.7 KN

$V_c = \frac{1}{6}\sqrt{fc'}b_wd == \frac{1}{6}\sqrt{24}*800*\frac{292}{1000} = 190.73$KN

Φ V_c= 0.75*190.73 =143 KN

Φ Vsmin$\geq$ 0.75 ($\frac{1}{3}$) * bw * d =0 .75* ($\frac{1}{3}$)*800*292*10^{-3} = 58.4 KN Controls

Φ Vsmin $\geq$ 0.75 ($\frac{\sqrt{fc'}}{16}$) * bw * d = 0.75*($\frac{\sqrt{24}}{16}$) * 800 * 292*10^{-3} = 53.64KN

Φ Vc<Vu $\leq$ Φ Vc + Φ Vsmin
143<198.8 $\leq$ 196.64…… not satisfied
Cases 1&2&3 is not suitable

Case 4 :-

$v_{s'} = \frac{1}{3}\sqrt{fc'}b_w d = \frac{1}{3}\sqrt{24} * 800 * 293/1000$ = 382.77 KN
$\emptyset(v_c + v_{s,\min)} < v_u \leq \emptyset(v_c + v_{s'})$
0.75(190.73+ 77.87)< 309.5< 0.75(190.73 + 382.77)
201.45 < 309.5<430.125

shear reinforcement are required

Use 4 leg Φ 8 for b = 80 cm

As =201.06 mm^2

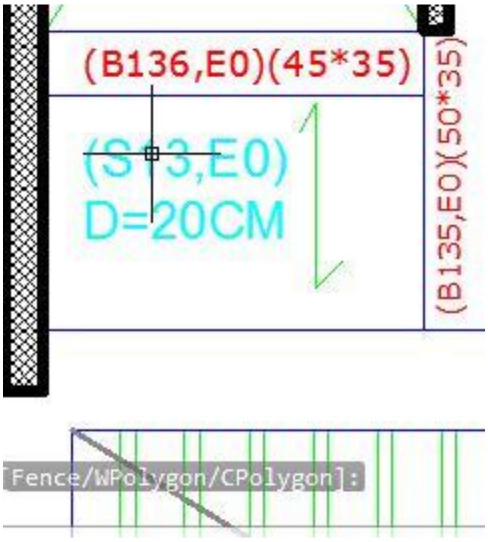
$V_s = V_n - V_c = \frac{198.8}{0.75} - 190.73 = 74.3$KN

$S = \frac{A_v f_{yt} d}{v_s} = \frac{201.06*420*292}{74.3*1000} = 332mm\ control$
$s_{max} \leq \frac{d}{2} = \frac{292}{2} = 146mm or\ s_{max} \leq 600\ mm$

Use 4 leg Φ 8 @300mm

4. 7 Design of One Way Solid Slab:

Simply supported one –way solid slab(S3) :



Fig(4-14):Plan Of one way Solid Slab

$hmin = l\backslash 20 = \frac{377}{20} = 18.85cm$

Take h=20

Assume the d_{Bar} =12 mm

d =h- cover - (d_{Bar}\2) =200-20-6=174mm

***Load Calclation :**

Total dead load = 5 KN/m.
Wu = 1.2*Dead load+ 1.6*live load = 1.2*5+1.6*2 = 9.2 KN/m^2.
M= $\frac{wl^2}{8} = 9.2 * \frac{3.77^2}{8} = 16.34$

$$R_n = \frac{Mu}{b \cdot d^2} = \frac{16.34*10^6}{0.9*1000*174^2} = 0.599MPa \ .$$

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

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

$$= \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2*20.59*0.593}{420}}\right) = 0.00143 \ \rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mR_n}{f_y}}\right)$$

As_{req} = = 0.00143*1000*175 =248.82mm^2

As_{min} = $0.0018*b*h = 0.0018*1000*200 = 360mm2$

As_{req} = 248.82mm^2 < As_{min} = 360mm^2 provide As_{min} = 360 mm^2

Use Φ 12 then :

$n = \frac{As}{As\,\emptyset\,12} = \frac{360}{113.1} = 3.2$

Take 5 $\emptyset \frac{10}{m}$ or $\emptyset\ 12@200mm.$

Steps (s) is the smallest of :

1. 3h= 3 * 200 = 600 mm .

2. 450 mm .

3. S = 380($\frac{280}{fs}$) – 2.5Cc = 380($\frac{280}{\frac{2}{3}*420}$) – 2.5*20 = 330 mm

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

S = 200 mm < Smax = 300 mm ok

As (Temperature and shrinkage) = .0018bh = .0018*1000*200 = 360mm ^{2}

Take 5 $\emptyset \frac{10}{m}$ or $\emptyset\ 10@200mm.$

Steps (s- for temperature and shrinkage reinforcement) is the smallest of :

1. 5h = 5 * 200 = 1000 mm

2. 450 mm

s = 200 mm < s max = 450 mm ok

Check for strain

$$\frac{A_{s.f_y}}{0.85bf_c'} = \frac{360 \times 420}{0.85 \times 1000 \times 24} = 4.23\ mm$$

$$= \frac{a}{\mathcal{B}_1} = \frac{4.23}{0.85} = 5\ mm$$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{293-5}{5}\right) > 0.005$$

4.8 Design of Two Way Solid Slab:

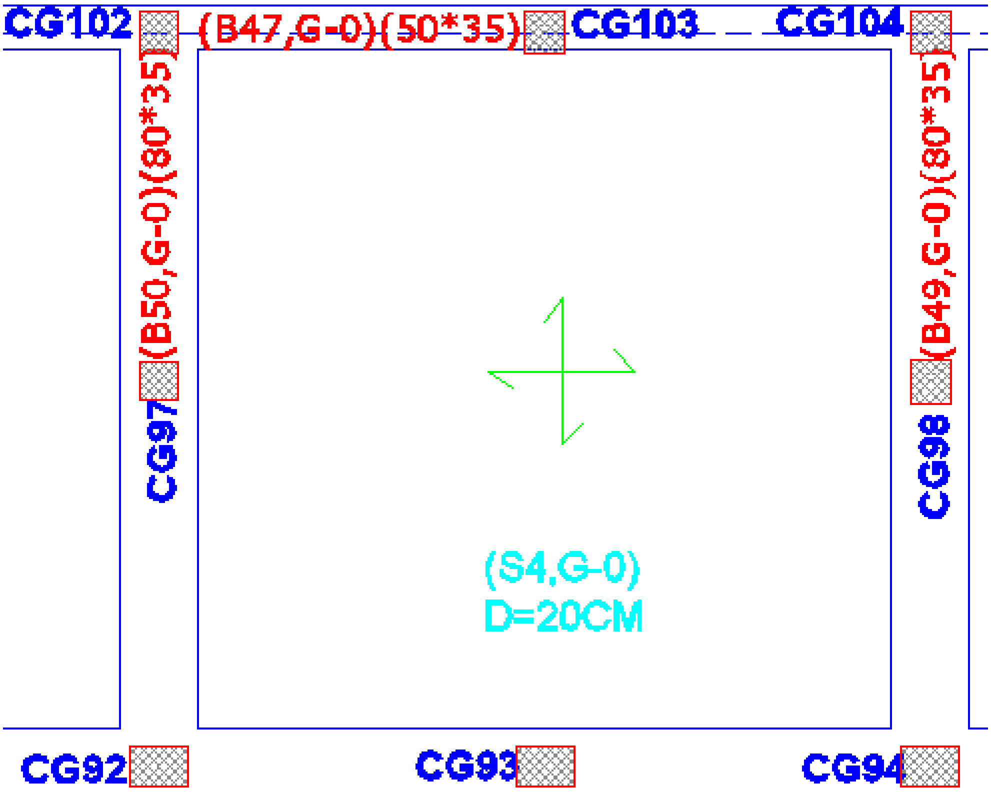


Fig (4-15): Plan Of two way Solid Slab

(4.8.1) Calculate the minimum thickness slab :

$$hmin = 20\ cm$$

$$Ib(BG49,50\ ) = \frac{80*35^3}{12} = 285833.333$$

$$Ib(BG47\ ) = \frac{50*35^3}{12} = 178645.8333$$

$$y(BG54) = \frac{35*80*\left(10+\frac{35}{2}\right)+50*10*\frac{10}{2}}{35*80+50*10} = 24.1cm$$

$$Ib(BG54\ ) = \frac{(50+30)*20.9^3}{3} - \frac{(2*15)*20.9^3}{3} + \frac{50*24.1^3}{3} + \frac{80*14.1^3}{3} = 227309cm$$

Slab section for exterior beam:

$Long\ direction\ l = 8m = 800\ cm$

$$I_{s1} = \frac{\left(\frac{l}{2}+bw\right)\boldsymbol{h^3}}{12} = \frac{\left(\frac{800}{2}+50\right)\boldsymbol{35^3}}{12} = 160.78 * 10^{-4}$$

Slab section for interior beam:

$Short\ direction\ l = 7.7\ m = 770cm$

$$I_{s1} = \frac{\left(\frac{l}{2}+\frac{l}{2}+w\right)\boldsymbol{h^3}}{12} = \frac{\left(\frac{770}{2}+50\right)\boldsymbol{35^3}}{12} = 155.42 * 10^{-4}$$

$Long\ direction\ l = 8\ m = 800\ cm$

$$I_{s1} == \frac{\left(\frac{l}{2}+\frac{l}{2}+w\right)\boldsymbol{h^3}}{12} = \frac{\left(\frac{800}{2}+\frac{800}{2}+50\right)\boldsymbol{35^3}}{12} = 303.69 * 10^{-4}$$

$$\alpha_{f1} = \frac{I_b}{I_s} = \frac{17.89}{160.78} = 0.11$$

$$\alpha_{f2} = \frac{I_b}{I_s} = \frac{28.58}{303.69} = 0.94$$

$$\alpha_{f3} = \frac{I_b}{I_s} = \frac{28.58}{303.69} = 0.94$$

$$\alpha_{f4} = \frac{I_b}{I_s} = \frac{22.7309}{155.42} = 0.15$$

$$\alpha_m = \frac{(0.15+0.94+0.94+0.11)}{4} = 0.535 < 2.0$$

The minimum slab thickness will be:

$$h = \frac{L_n(0.8+\frac{f_y}{1400})}{36+5\beta(\alpha_m-0.2)} = \frac{8*(0.8+\frac{420}{1400})}{36+5*\frac{8}{7.7}*(0.535-0.2)} = 0.23\ m$$

$h = 35\ cm > 23\ cm\ -OK$

(4. 8.2) Dead load calculations:

Table(4-4) calculation of the Dead load on solid slab

Dead load from:	$\delta \times \gamma$	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	**0.20×25×1**	**5**
Plaster	**0.02×22×1**	**0.44**
Partitions	**1.5*1**	**1.5**
		9.17

Dead load =9.17 KN/m^2.
Live load = 2KN/m^2.
WuD = 1.2*Dead load = 1.2*9.67 = 11.34 KN/m^2.
WuL = 1.6*live load = 1.6*2 = 3.2KN/m^2.
Wu = 11.34+3.2= 14.54 KN/m^2

(4.8.3) Shear Design :

l_a/l_b=0.95
W_b=0.71
Wa=0.29

- The total load on the panel being (7.7*8*19.604) = 1207.606 KN
- The load at face of the long beam is (0.71×1207/(2*8))=53.58 KN

Assume the Φ 12
d=200-20-12\2=174mm

- V_c =(√28 *1000*174*10^-3)\6 =153.45KN

$øV_{c = 0.75}$ × 181.26= 115.1 KN
V_u< ø V_c.

The thickness of the slab is adequate enough

(4.8.4) Flexural Design:

(l_a/l_b=0.95)
Positive moments :
Cda=.022

Cla=.031
Cdb=.021
Clb=.027

$M_{a+ve,Dl}$=C_a*W* L_a^2 =0.022*11.604*7.7 ^{2}= 15.14 KN.m\m
$M_{a+ve,Ll}$=C_a*W* L_a^2 =.031*8*7.7 ^{2}= 14.7KN.m\m
M_{a+ve} = $M_{a+ve,L}$ + $M_{a+ve,D}$ = 29.84 KN.m\m

$M_{b+ve,D}$=C_b*W* L_b^2 =0.021*11.604*8^2= 15.59 KN.m\m
$M_{b+ve,L}$=C_b*W* L_b^2*b =0.027*8*8 ^{2}= 13.82KN.m\m
M_{b+ve} = $M_{b+ve,L}$ + $M_{b+ve,D}$ = 29.41KN.m/m

(4.8.5) Positive Moment:

*Mua = 29.84 KN.m/m

Assume the d_{Bar} =12 mm

d =h- cover - (d_{Bar}\2) =200-20-6=174mm

$$R_n = \frac{Mu}{b \cdot d^2} = \frac{29.84*10^6}{0.9*1000*174^2} = 1.09MPa \ .$$

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

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

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2mR_n}{f_y}}\right) = \frac{1}{20.59}\left(1-\sqrt{1-\frac{2*20.59*1.09}{420}}\right) = 0.003$$

As_{req} = 0 .003*1000*174 =522mm^2/m

As_{min} = $0.0018*b*h = 0.0018*1000*200 = 360mm2/m$

As_{req} =522mm2≥ As_{min} = 360mm2/m

Use Φ 12 \ 20cm
S=200 < 2h =2*200=400mm < 450 mm -ok

*Mub = 29.41KN.m/m

Assume the d_{Bar} =12 mm As_{req} = = 0.00143*1000*175 =248.82mm^2

d =h- cover - (d_{Bar}\2) =200-20-6=174mm

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{29.41*10^6}{0.9*1000*174^2} = 1.08MPa\ .$$

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

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

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2mR_n}{f_y}}\right) = \frac{1}{20.59}\left(1-\sqrt{1-\frac{2*20.59*1.08}{420}}\right) = 0.003$$

As_{req} = 0 .003*1000*174 =522mm^2/m

As_{min} = $0.0018*b*h = 0.0018*1000*200 = 360mm2/m$

As_{req} =522mm2≥ As_{min} = 360mm2/m

Use Φ 12 \ 20cm
S=20 < 2h =2*200=400mm < 450 mm –ok

(4.8.5) Negative Moment:

Ca=.038
Cb=.056
$M_{a-ve,}$ =C_a*W* L_a^2 =0.038*14.54*7.7 ^{2}= 32.76 KN.m
$M_{b-ve,}$ =C_{ab}*W* L_a^2 =0.056*14.54*8 ^{2}= 52.11 KN.m

Mua = 32.76KN.m/m

Assume the d_{Bar} =12 mm

d =h- cover - (d_{Bar}\2) =200-20-6=174mm

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{32.76*10^6}{0.9*1000*174^2} = 1.20MPa\ .$$

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

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

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2mR_n}{f_y}}\right) = \frac{1}{20.59}\left(1-\sqrt{1-\frac{2*20.59*1.20}{420}}\right) = 0.003$$

As_{req} = 0 .003*1000*174 =522mm^2/m

$As_{min} = 0.0018 * b * h = 0.0018 * 1000 * 200 = 360 mm2 / m$

As_{req} =522mm2≥ As_{min} = 360mm2/m

Use Φ 12 \ 20cm
S=200 < 2h =2*200=400mm < 450 mm -ok

Use Φ 12 \ 20cm with As=904mm^2/m

Mub = 52.11 KN.m/m

Assume the d_{Bar} =12 mm

d =h- cover - (d_{Bar}\2) =200-20-6=174mm

$$Rn = \frac{M_u}{\emptyset b d^2} = \frac{52.11*10^6}{0.9*1000*174^2} = 1..9 MPa \ .$$

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

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

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mR_n}{f_y}}\right) = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2*20.59*1.90}{420}}\right) = 0.003$$

As_{req} = 0 .003*1000*174 =522mm^2/m

$As_{min} = 0.0018 * b * h = 0.0018 * 1000 * 200 = 360 mm2 / m$

As_{req} =522mm2≥ As_{min} = 360mm2/m

Use Φ 10 \ 15cm
S=150 < 2h =2*200=400mm < 450 mm -ok

Use Φ 12/ 15cm with As=904mm^2/m

Note: other moments requires areinforcement less than minimum, Use Φ 8 \ 12.5cm with As=400 mm^2/m

4.9 Design of two way ribbed slab (R6)

4.9.1 Minimum thickness for ribbed slab h= 35 cm

Check for the minimum thickness of the slab:

Interior beams have a T- section : bw=350mm h=450 t=hf=350

$$y_c = \frac{50 * 35 * 27.5 + 35 * 10 * 5}{50 * 35 + 35 * 15} = 22\ cm$$

$$I_b = \frac{b * h^3}{3} = \frac{15 * 23^3}{3} + \frac{35 * 22^3}{3} + \frac{15 * 12^3}{3} = 19.37 * 10^{-4}\ m^4$$

Exterior beams have a T- section : bw=500cm h=55cm t=hf=350

$$y_c = \frac{35(50 + 2 * 15) * (20 + 17.5) + 50 * 35 * 17.5}{35 * 80 + 50 * 20} = 35.69\ cm$$

$$I_b = \frac{b * h^3}{3} = \frac{80 * 19.31^3}{3} + \frac{80 * 15.69^3}{3} + \frac{50 * 35.69^3}{3} = 105.26 * 10^{-4}\ m^4$$

-The moment of inertia for the ribbed slab:

$$y_c = \frac{40 * 8 * 4 + 35 * 12 * 17.5}{40 * 8 + 35 * 12} = 11.66\ cm$$

$$I_{rib} = 52 * \frac{11.66^3}{3} - 40 * \frac{3.66^3}{3} + 12 * \frac{23.34^3}{3} = 78989.6 cm^4 = 7.8989 * 10^{-4}\ m^4$$

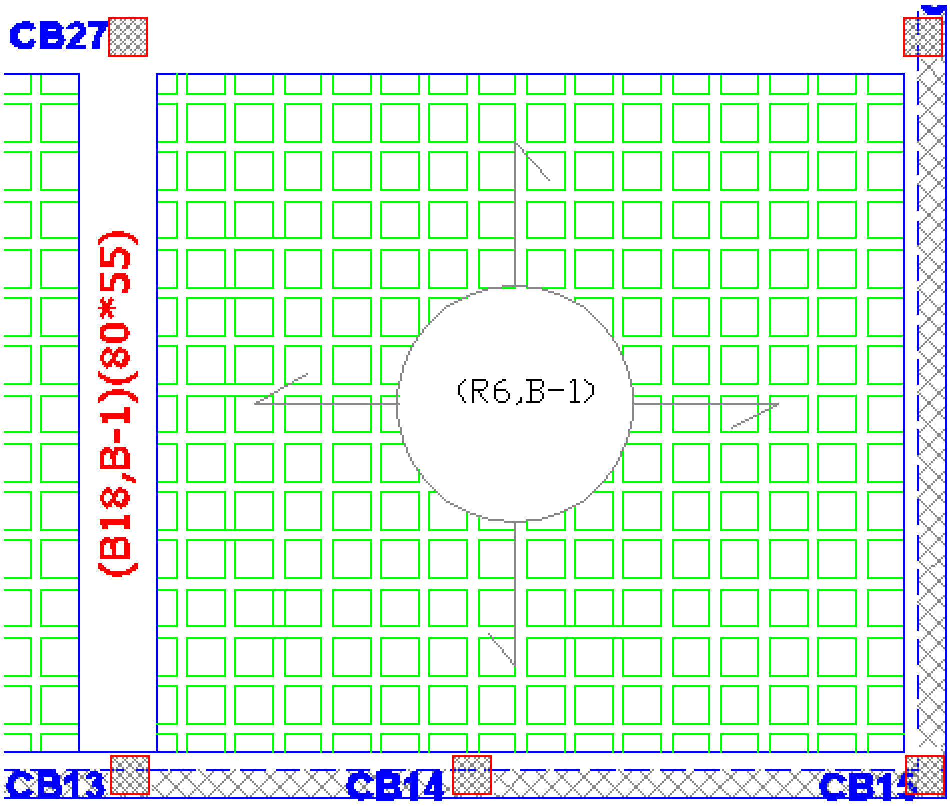


fig.(4.16): Two way Ribbed slab.

Slab section for exterior beam:

$Short\ direction\ l = 790\ m = 790\ cm$

$$I_{s1} = \frac{I_{rib} * (\frac{l}{2} + bw)}{b_f} = \frac{7.8989 * 10^{-4} * 4.5}{0.52} = 68.35 * 10^{-4}$$

$Long\ direction\ l = 8.6m = 860\ cm$

$$I_{s1} = \frac{I_{rib} * (\frac{l}{2} + bw)}{b_f} = \frac{7.8989 * 10^{-4} * 4.8}{0.52} = 75.95 * 10^{-4}$$

Slab section for interior beam:

$Short\ direction\ l = 7.9\ m = 790cm$

$$I_{s1} = \frac{I_{rib} * (\frac{l}{2} + \frac{l2}{2})}{b_f} = \frac{7.8989 * 10^{-4} * 7.9}{0.52} = 121.52 * 10^{-4}$$

$Long\ direction\ l = 8.6\ m = 860\ cm$

$$I_{s1} = \frac{I_{rib} * (\frac{l}{2} + \frac{l2}{2})}{b_f} = \frac{7.8989 * 10^{-4} * 8.6}{0.52} = 130.6 * 10^{-4}$$

$$\alpha_{f1} = \frac{I_b}{I_s} = \frac{19.37}{121.52} = 0.16$$

$$\alpha_{f2} = \frac{I_b}{I_s} = \frac{105.26}{75.95} = 1.4$$

$$\alpha_{f3} = \frac{I_b}{I_s} = \frac{105.26}{68.35} = 1.54$$

$$\alpha_{f4} = \frac{I_b}{I_s} = \frac{19.37}{130.60} = 0.14$$

$$\alpha_m = \frac{(0.16 + 1.4 + 1.54 + 0.14)}{4} = 0.81 < 2.0$$

The minimum slab thickness will be:

$$h = \frac{L_n(0.8 + \frac{f_y}{1400})}{36 + 5\beta(\alpha_m - 0.2)} = \frac{8 * (0.8 + \frac{420}{1400})}{36 + 5 * \frac{9}{8} * (0.81 - 0.2)} = 0.22\ m$$

$h = 35\ cm > 22\ cm\ - OK$

Take slab thickness 35 cm

$b_{eff.}$= 520 mm b_w = 120 mm h_f= 80 mm

h = 350 mm h etolit block = 270 mm

4.9.2 Load calculation:

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

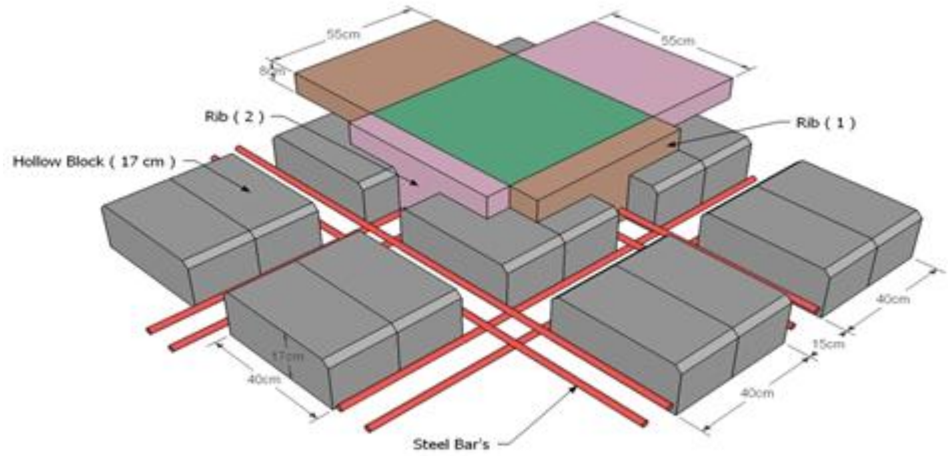


Fig.(4.17): Two way ribbed slab

Table (4-5) Calculation of the total dead load for two way rib slab (6).

No.	Material	Quality Density KN/m^3	Calculation
1	**Topping**	25	**0.52×0.52×0.08×25 = 0.54**
2	**Rib**	25	**(0.4+0.52)0.27×0.12×25 = 0.526**
3	**Sand**	17	**0.52×0.52×0.07×17 = 0.321**
4	**Mortar**	22	**0.52×0.52×0.02×22 =0.119**
5	**Tile**	23	**0.52×0.52×0.03×23 =0.186**
6	**Plaster**	22	**0.52×0.52×0.02×22 =0.119**
7	**Block**	10	**4×0.2×0.2×0.27×10 = 0.432**
8	**Partitio**	1.5	**1.5×0.52 = .78**
			3.02 **KN/unit**

Dead Load of slab:

$$DL = \frac{3.02}{0.52 * 0.52} = 10.81 KN/m^2$$

$$w_D = 1.2 * 10.81 = 12.97 KN/m^2$$

$LL = 5\ KN/m^2$

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

$w = 12.97 + 8 = 20.97\ KN/m^2$

4.9.3 Moments calculations:

Ratio=7.9/8.6=0.90

$Ma\ =\ Ca\, wla^2 bf\quad and\quad Mb\ =\ Cb\, wlb^2 bf$

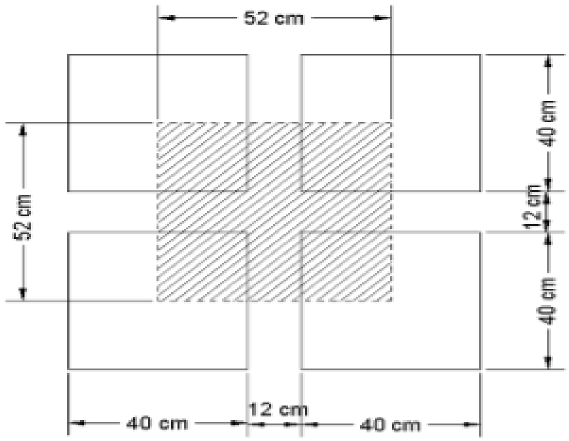


Fig.(4.18): Two way ribbed slab

-Negative moment

$C_{a,neg} = 0.06$

$C_{b,neg} = 0.04$

$M_{a,neg} = (0.06 * 20.97 * 7.9^2) * 0.52 = 40.8\ KN.m$

$M_{b,neg} = (0.04 * 20.97 * 8.6^2) * 0.52 = 32.25 KN.m$

-Positive moment

$C_{aD,pos} = 0.033$

$C_{bD,pos} = 0.022$

$C_{aL,pos} = 0.039$

$C_{bL,pos} = 0.022$

$M_{a,pos,(dl+ll)} = (0.033 * 12.97 * 7.9^2 + 0.039 * 8 * 7.9^2) * 0.52 = 24.02 KN.m$

$M_{b,pos,(dl+ll)} = (0.022 * 12.97 * 8.6^2 + 0.022 * 8 * 8.6^2) * 0.52 = 17.74 KN.m$

Design of positive moment

- Short direction ($Mu = 24.02\ KN.m$)

  $bf = 520\ mm$

Assume bar diameter $\phi 12$ for main positive reinforcement.

$$d = h - cover - dstirrups - \frac{d_b}{2} = 350 - 20 - 10 - \frac{12}{2} = 314\ mm.$$

$$R_n = \frac{M_u}{\emptyset b d^2} = \frac{24.02 \times 10^6}{0.9 \times 120 \times 314^2} = 2.27\ MPa$$

$$m = \frac{fy}{0.85\ fc'} = \frac{420}{0.85 * 24} = 20.58$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.58}\left(1 - \sqrt{1 - \frac{2 \times 20.58 \times 2.27}{420}}\right) = 0.00574$$

$$As = \rho.b.d = 0.00574 \times 120 \times 314 = 216.3 mm^2$$

- $Check\ for\ As, min..$

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

$$As, min = 0.25 * \frac{\sqrt{24}}{420}\ 120 \times 313 = 109.5\ mm^2$$

$$As, min = \frac{1.4}{420} * 120 \times 313 = 125.2\ mm^2 \ldots..\boldsymbol{Control.}$$

- $As, required = 216.5\ mm^2 > As, min = 125.2\ mm^2 \quad (OK)$

Use 2Ø12, with As = 226 $mm^2 > As, required = 215.75\ mm^2$

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

Tension = Compression

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

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

$a = 52.84\,mm$

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

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

$= 0.003 * \left(\frac{313 - 62.17}{62.17}\right) = 0.0121 > 0.005 \quad \therefore \phi = 0.9 \ \ldots\ OK.$

4.9.4 Check shear strength:

$W_a = 0.75$

$W_b = 0.25$

Short direction

$Au_a = 20.97 * 7.32 * 8 * 0.75 * 0.5 * \frac{0.52}{8} = 30\,KN$

$Vu = Au_a - W * 0.52 * Wa = 30 - 20.97 * 0.52 * 0.85 = 20.73\,KN$

$\emptyset * V_c = .75 * \frac{1.1}{6} * \sqrt{fc'} * bw * d = .75 * \frac{1.1}{6} * \sqrt{24} * 120 * 313 = 25.3\,KN$

$Case\ 1$

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

$V_u = 20.73KN > \frac{1}{2} * \phi * V_c = 12.65 \ \ldots\ \boldsymbol{Not\ OK}$

$Case\ 2$

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

$$\frac{1}{2} * \phi * V_c = 12.65\,KN < V_u = 20.73\,KN < \phi * V_c = 25.3\,KN - \boldsymbol{OK}$$

$\boldsymbol{OK}$

Provide minimum shear reinforcement

$Vs_{min} \geq \frac{1}{16} * \sqrt{f_c'} * bw*d = \frac{1}{16} * \sqrt{24} * 120*313*10^{-3} = 11.5$ KN.

$\phi Vs\ min = 8.63$

$\leq \frac{1}{3} * b_w * d = \frac{1}{3} * 0.12 * 0.313*10^3 = 12.52$ KN

$\phi Vs,min = 8.63$ ………….. control

$\phi Vc = 25.3$ KN $<$ $Vu = 12.65$ KN $\leq \phi(Vc+Vs_{min}) = 33.93$KN …… satisfy

∴Case (3) is satisfy shear reinforcement is required.

Use 2 Legϕ8 for stirrups with Av = 100.53mm2

$$Vsmin = \frac{\phi Vsmin}{\phi} = \frac{8.63}{0.75} = 11.5$$

$$s = \frac{Av * fy * d}{Vsmin} = \frac{100.53 * 420 * 313}{11.5} * 10^{-3} = 1149mm$$

$S_{max} \leq \frac{d}{2} = \frac{313}{2} = 157$ mm.

$\leq$ 600 mm.

Select 2 leg ϕ8 @ 15cm

4.10 Design of Column (C9):

❖ **Material :-**

⇒ concrete B300 Fc' = 24N/mm^2

⇒ Reinforcement Steel Fy = 420 N/mm^2

✓ **Load Calculation:- (From Column Group D)**

Service Load:-

Dead Load = 1800 KN
Live Load = 560 KN

Factored Load:-

P_U = 1.2 ×1800+ 1.6× 560=3100 KN

✓ Dimensions of Column:-

$Assume \rho g = 0.01$

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

$3100 * 1000 = 0.65 \text{ x } 0.8 \times \text{Ag}\{0.85 * 24\,(1-0.02) + 0.02 * 420\}$

Ag = 209972 mm2

Ag = a *b

Take a = 400mm ,

b = Ag/400 = 610.91

Take b = 600 mm

Ag = 400 * 600 = 240000

* selection longitudinal bars :

3100* 1000 =0 .65*0.8 { 0.85 *24 *(24000 – Ast) + Ast * 300) }

Ast = 2666.5 mm^2

Use 14 Φ 16 with Ast = 2814.87 > 2666.5

$Assume \rho g = Ast / Ag = 0.0117 > .01$

✓ Design of Ties :-

Use Φ 10 for ties

The spacing of ties shall not exceed the smallest of :-

1- 48 times the tie diameter , 48 ds = 48 * 10 = 480 mm

2. 16 the times the longitudinal bar aiameter , 16 dp = 16 *16 = 256 mm – control

3. The least dimenstion of the colume = 400 mm

$Use \phi 10 @ 20\ cm$

*** check for code requirement :**

1. clear spacing between longitudinal bars :

Clear space = 400 – 40*2 -10*2 – 5*16 / 4 = 55 mm > 40mm

> 1.5dp = 24 ok

2. Gross reinforcement ratio :

.01 < $\rho g = .0117$ < .08 ok

3. Nmber of bars 14 > 4 – ok

4. Minimum tie diameter : Φ 10 for Φ 28 – ok

5. spacing of ties : s = 400 mm - ok

6. arrangement of ties : 44 < 150 – 150

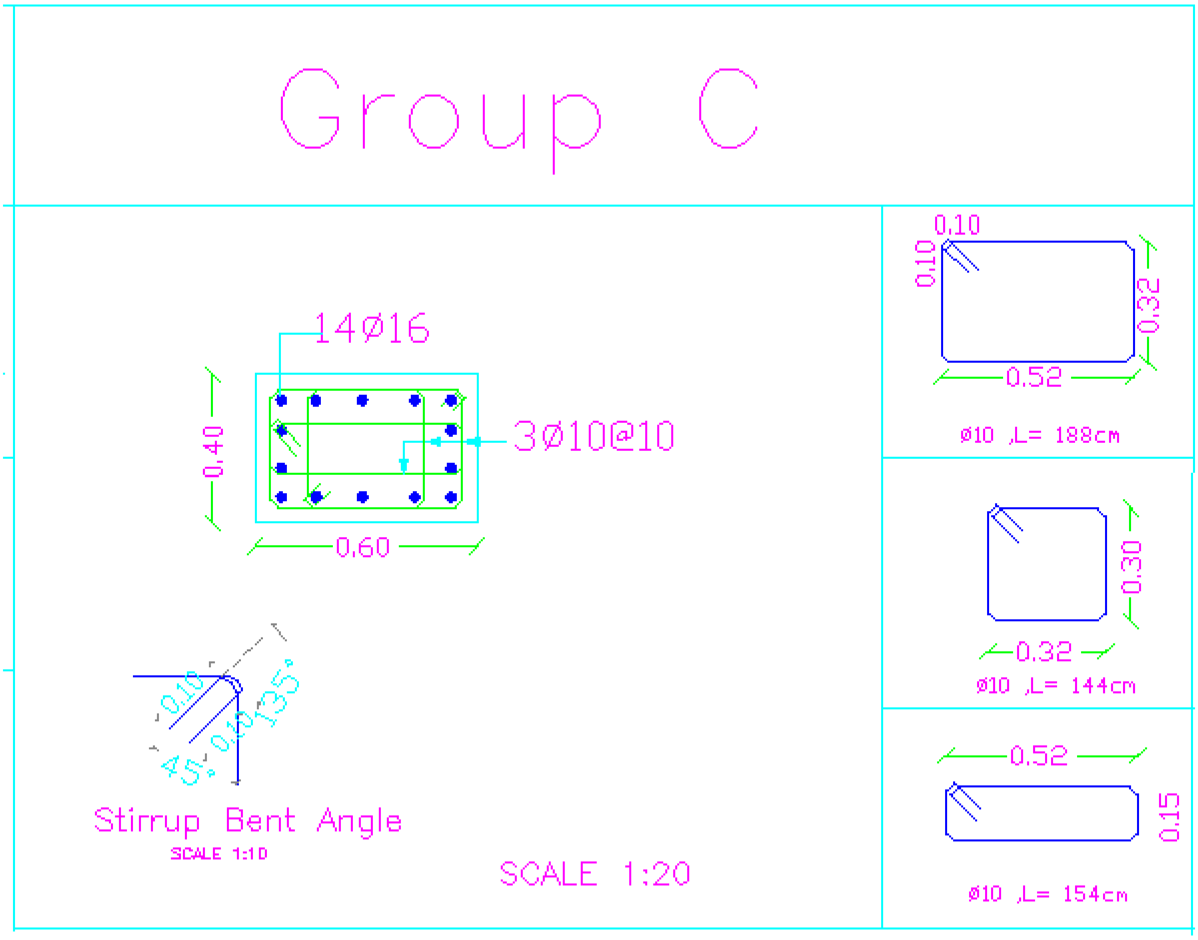


Fig 4.19: Column Reinforcement

GROUP C

C101,C100,C96,C92,C91,C87,C86,C85,C82,C77,
C73,C71,C69,C62,C60,C94,C93,C90,C88,C81,
C79,C78,C65,C64,C63,C75,C48,C82,C115,C116,
C117,C118,C119,C121,C123,C125

Note : This Group Includes (Barcking , Ground and First)

4.11 Design of Isolated Footing:

❖ **Material :-**

⇒ concrete B300 Fc' = 24 N/mm^2

⇒ Reinforcement Steel Fy = 420 N/mm^2

✓ Load Calculations :- (From Column Group D)

(4.11.1) Determination of Loads:

Dead Load = 2600 KN, Live Load = 650KN

Total services load = 2600 + 650 =3250KN

Total Factored load = 1.2*2500 + 1.6*630= 4160KN

Column Dimensions (a*b) = 600 * 600 cm

Soil density = 18 Kg/cm3

Allowable Bearing Capacity = 500 KN/m2

Assume footing to be about (75 cm) thick.

Footing weight = 25×0.75= 18.75 KN/m^2.

Soil weight above the footing = 0.5 ×18 = 9KN/m2.

q_{allow} =500 – 9– 18.75 -25*0.5 = 459.75 KN/m^2

(4.11.2) Determination of Footing Area:

$$A = \frac{3250}{459.75} = 7.07m^2$$

Try 2.8*2.8 m with area = 7.84 m^2 ≥ Areq = 7.07 ,,,,,OK.

Take B=2.8 m.

Pu=4160 KN.

q_u = 4160/7.84= 530.6 KN/m2

(4.11.3) Check for one-way shear strength:

Assume h =75 cm.
Assume, ϕ=20 mm , cover=75mm

d= 750-75-20=655mm

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

$$Vu = 530.6 * \left(\frac{2.8-0.6}{2} - 0.655\right) * 2.8 = 661.12kN$$

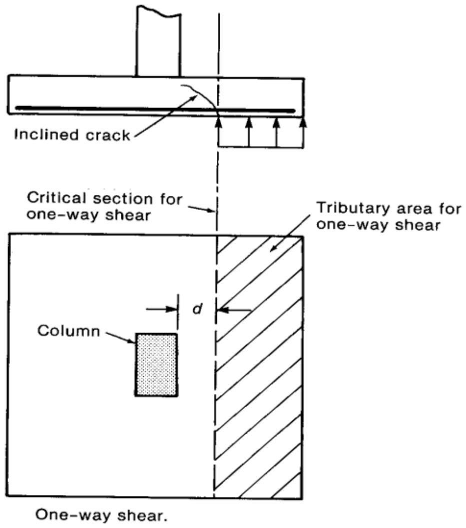


Fig (4-20):one way shear.

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

$$\phi.Vc = 0.75 * \frac{1}{6} * \sqrt{24} * 2800 * 655 = 1123.09KN$$

$$\phi.Vc = 1123.09KN > Vu = 661.12KN$$

$$\therefore \quad Safe$$

(4.11.4) Check for two-way shear action (punching):-

$$Vu = Pu - FR_b$$

$$FR_b = q_u * area\ \ of\ \ critical\ \ \ section$$

$$Vu = 4160 - 530.6[(0.6 + 0.655) * (0.6 + 0.655)] = 3324.3KN$$

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:

$$\beta_C = \frac{Column\ \ Length\ \ (a)}{Column\ \ Width\ \ (b)} = \frac{60}{60} = 1.0$$

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

$$b_o = 2(d + a1) + 2(d + a2) = 2(0.655 + 0.6) + 2(0.655 + 0.6) = 5.02m$$

α_s =40 for interior column

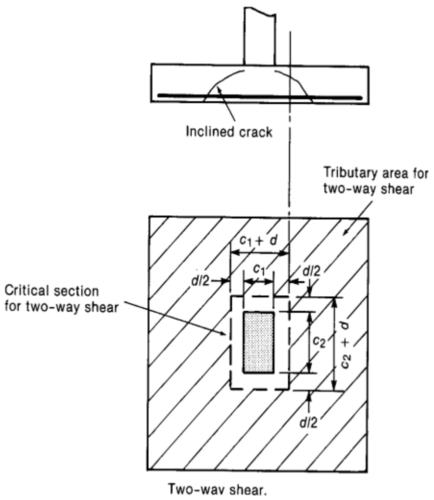


Fig (4-21) two way shear

$$\phi.V_C = \phi.\frac{1}{6}\left(1+\frac{2}{\beta_c}\right) = \frac{0.75}{6}*(1+2/1) = 0.35$$

$$\phi.V_C = \phi.\frac{1}{12}\left(\frac{\alpha_s * d}{b_o}+2\right) = \frac{0.75}{12}*\left(\frac{40*0.655}{5.02}+2\right) = 0.664$$

$$\phi.V_C = \phi.\frac{1}{3} = \frac{0.75}{3} = 0.25............control$$

$$\phi.V_c = \phi.\frac{1}{3}\sqrt{f_c'}\,b_o d = 0.25*\sqrt{24}*5020*655*10\text{^}{-3} = 4027KN$$

$$Vu = 3324.2$$

$$\phi.Vc = 4027 > Vu_C = 3324.2...........satisfied$$

(4.11.5) Design of Bending Moment:

Critical Section at the Face of Column

Mu $=530.6*2.8*1.1*\frac{1.1}{2} = 899$

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{899\times10^6}{0.9\times2800\times655^2} = 0.831\ Mpa$

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

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times0.831}{420}}\right) = .00202$

$A_{s,req} = \rho.b.d = 0.00202\times2800\times655 = 3707.5\ mm^2$

$A_{s,min} = 0.0018*2800*750 = 3780\ mm^2$ - CONTROL

$A_{s,req} = 3780\ mm^2$

Use Φ 20

n $=\frac{3780}{314.15} = 13.03$

Use 14ø20 in Both Direction, $A_{s,provided}$= 4398mm^2>$A_{s,required}$= 3780 mm^2… Ok

check for spacing :

$$s = \frac{2800 - 2*75 - 14*20}{13} = 182.3mm$$

Step "s" the smallest of

1. 450mm control
2. 3h= 3*750=2250mm

S=129.4<450 ,,,,, o k.

Check for strain:-

a $=\frac{A_{s}.f_y}{0.85b\ f_c'} = \frac{4398\times420}{0.85\times2800\times24} = 32.33\ mm$

c $= \frac{a}{\mathcal{B}_1} = \frac{32.33}{0.85} = 38.04\ mm$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{655-38.04}{38.04}\right) = 0..0486 > 0.005\ \ldots\ldots \mathbf{0}\boldsymbol{k}$$

(4.11.6) Design the column –footing joint "dowels":

Load Transfer In Footing :-

$$\Phi Pn.b = \Phi(0.85\, fc' A_1 \times \sqrt{\frac{A_2}{A_1}})$$

$A_1 = 0.60 * .060 = 0.36\ m^2$

$A_2 = 2.8*2.8 = 7.84\ m^2$

$$\ldots\ldots\ldots\ldots\ldots \sqrt{\frac{A_2}{A_1}} = 2 \quad \sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{7.84}{0.36}} = 4.67 > 2$$

$$\Phi Pn.b = 0.65\times(0.85\times24\times360\times2) = 9547.2KN$$

$$\Phi Pn = 9547.2 > Pu = 3976.........ok$$

No Need For Dowels

Load Transfer In Column :-

$$\Phi Pn.b = 0.65\times(0.85\times24\times360) = 4773.6Kn$$

$$\Phi Pn = 4773.6 > Pu = 4160kn........Not..ok$$

we Need For Dowels

As dowels =As coiumn

Use 18ø20, $A_{s,}$ = 5652 mm^2

1- Development Length In Footing :-

Tension Development Length In Footing :-

300mm $>Ld_{T\ req} = \frac{9}{10} * \frac{F_y}{\lambda\sqrt{fc}} * \frac{\psi_e\psi_s\psi_t}{\frac{ktr+cb}{db}} * db$

$$Ktr = 0\ (Nostripes)$$

$$cb = 75 + \frac{20}{2} = 85mm\ \ Or\ cb = \frac{200}{2} = 100\ mm$$

$$\frac{ktr + cb}{db} = \frac{0 + 85}{20} = 4.25 > 2.5$$

$$\frac{ktr + cb}{db} = 2.5$$

300mm>$Ld_{T\,req} = \frac{9}{10} * \frac{420}{1*\sqrt{24}} * \frac{1*1*0.8}{2.5} * 20 = 493.8\,mm$

$Ld_{T\ available} = \frac{2800-600}{2}$ -75= 1025 mm

$Ld_{T\ available}$ = 825 mm >$ld_{req} = 395.054\,mm$…….. **OK**

Compression Development Length In Footing :-

200mm> 0.043*Fy*dB >$Ld_{Creq} = \frac{0.24*Fy*dB}{\sqrt{24}}$

200mm> 0.043*420*20= 361.2>$Ld_{Creq} = \frac{0.24*420*20}{\sqrt{24}}$ = 411.51

Ld_{Creq}= 411mm

Ld_{Creq}= 411mm …….. **Ok**>$Ldc_{available}$ = 750– 75 – 20 – 20 = 635mm

Lap Splice of Dowels In Column :-

Lsc = 0.071 × fy × db = 0.071 × 420 × 20 = 596.4 mm > 300 m

Select Lsc = 600 mm

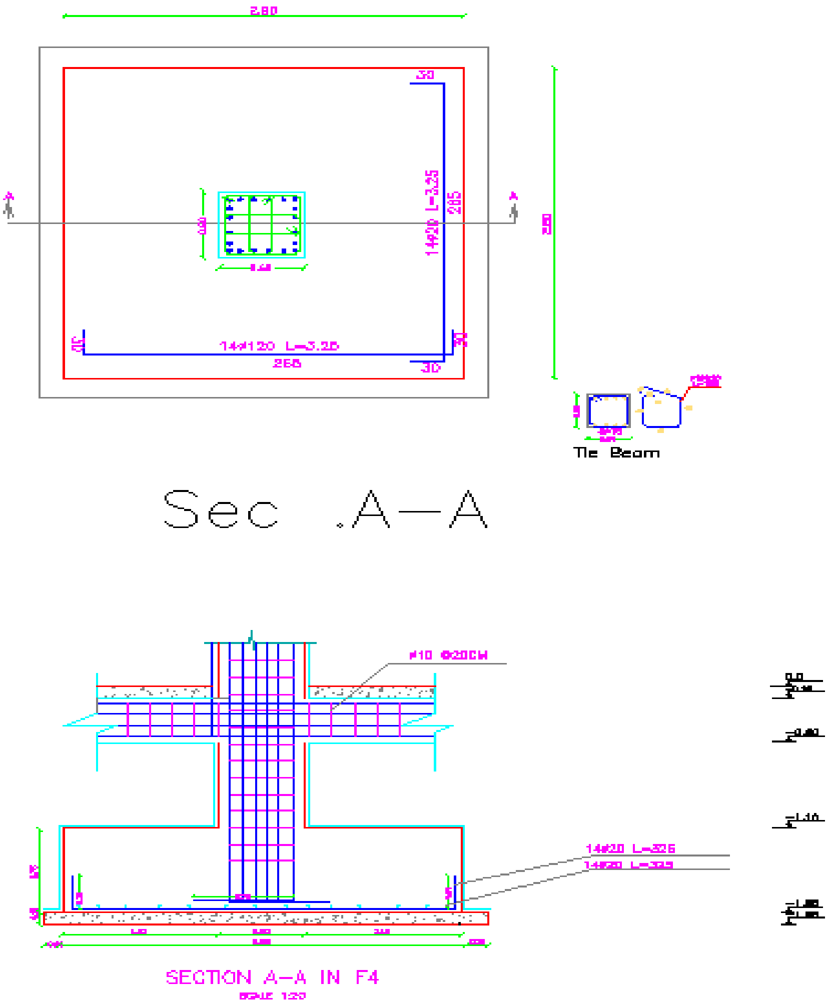
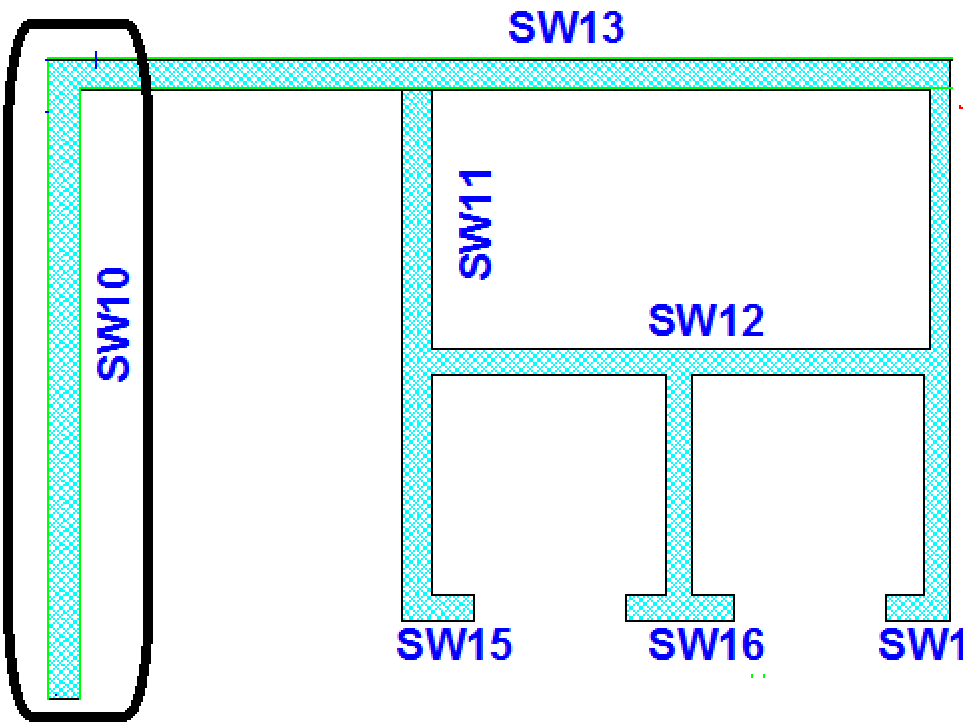


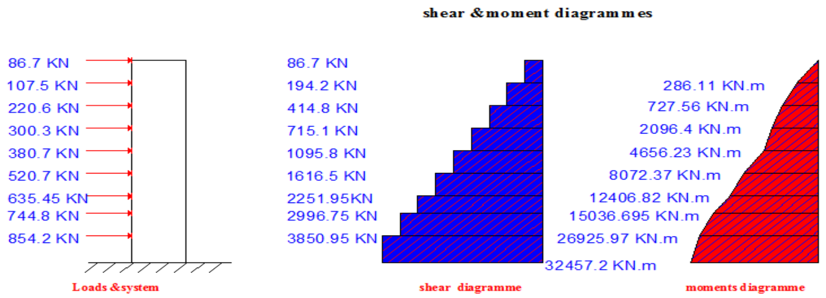
Fig (4-22) :Top Plan and section of footing.

4.12 Design of Shear Wall (SW1):

We designed the shear wall by using **Etabs** program.



Fig(4-23) : Plan of the shear wall.



Fig(4-24) : shear and moment diagrams

❖ **Material and Sections:- (From Shear Wall 2)**

⇒ concrete B300 Fc' = 24 N/mm^2

⇒ Reinforcement Steel Fy = 420 N/mm^2

⇒ Shear Wall Thickness h = 30 cm

⇒ Shear Wall Width Lw = 5.85m

⇒ Shear Wall Height Hw = 28 m

✓ **Design of Horizontal Reinforcement:-**

$$\sum Fx = Vu = 3850.8 \ KN$$

The critical Section is the smaller of:

$$\frac{lw}{2} = \frac{5.58}{2} = 2.925m$$

$$\frac{hw}{2} = \frac{28.1}{2} = 14m$$

$$storyheigh(Hw) = 21.8m........Control$$

$$d = 0.8 \times Lw = 0.8 \times 5.85 = 4.68m$$

$$\emptyset V_{nmax} = \emptyset \frac{5}{6}\sqrt{f_c{}'}hd$$
$$= 0.75 * 0.83 * \sqrt{24} * 300 * 4680 \ = 4281.65\ KN > V_u \ = 3850.8KN$$

V_c is the smallest of :

$$1 - \ V_c = \frac{1}{6}\sqrt{f_c{}'}hd = \frac{1}{6}\sqrt{24} * 300 * 4680 = 1146.4 \ \text{........ Control}$$

$$2 - \ V_c = 0.27\sqrt{f_c{}'}hd + \frac{N_u d}{4l_w} = 0.27\sqrt{24} * 300 * 4680 + 30.4 = 1650.8KN$$

$$3 - \ V_c = \left[0.05\sqrt{f_c} + \frac{l_w\left(0.1\sqrt{f_c{}'} + 0.2\frac{N_u}{l_w h}\right)}{\frac{M_u}{V_u} - \frac{l_w}{2}}\right]hd = 3239.41KN$$

$$\frac{6123.1 - 3637.3}{3.24} = \frac{M_u - 3637.3}{0.24} \Longrightarrow M_u = 3821.43KN.m$$

Vc =1146.4KN

$\emptyset * vc + \emptyset vs = vu$

$\emptyset * vs$=vu-$\emptyset * vc$

Vs=vu/$\emptyset - vc$

Vs=3850.8/0.75-1146.4= 3988kn **No need reinforcement**

Minimum shear reinforcementis required:

Min(Avh/Sh)=0.0025*h

=0.0025*300=0.75

Select $\emptyset 12$,two layers

Avh=2* π *12^2/4=226.2 mm^2

226.2/Sh=0.75

Sh=226.2/0.75=301.5

Select $\emptyset$12 in Two Layer

Select Sh=200mm≤Smax=Lw/5=585/5=117 cm.

=3*h =3*30=90 cm.

✓ Design of Vertical Reinforcement:-

$$\frac{A_{vv}}{S_v} = \left[0.0025 + 0.5\left(2.5 - \frac{h_w}{Lw}\right)\left(\frac{A_{vh}}{S_h * h} - 0.0025\right)\right]*250$$

$$\frac{A_{vv}}{S_v} = \left[0.0025 + 0.5\left(2.5 - \frac{28.1}{5.85}\right)\left(\frac{226.2}{200*300} - 0.0025\right)\right]*250$$

$$\frac{A_{vv}}{S_v} = 0.26$$

Select $\emptyset$12 in Two Layer

$$A_{vh} = \frac{2*\pi*12^2}{4} = 226.2 \text{ mm2}$$

$$\frac{226.2}{S_v} = 0.26$$

S_v= 870 mm

- Maximum spacing is the least of :

$= \frac{5850}{3} = 1950 \text{ mm} \frac{Lw}{3}$

3*h = 3*300 = 900mm

450 mm ……. Control

Use ϕ12/200 mm for two layers

✓ Design of Bending Moment:-

$$A_{st} = \left(\frac{5850}{200}\right) * 2 * 113.1 = 6616.35\ mm^2$$

$$w = \left(\frac{A_{st}}{L_w h}\right)\frac{f_y}{f_c{}'} = \left(\frac{6616.35}{5850 * 300}\right)\frac{420}{24} = 0.066$$

$$\alpha = \frac{P_u}{l_w h f_c{}'} = 0$$

$$\frac{C}{l_w} = \frac{w+\alpha}{2w + 0.85\beta_1} = \frac{0.066 + 0}{2 * 0.066 + 0.85 * 0.85} = 0.077$$

$$\emptyset M_n = \emptyset\left[0.5 A_{st} f_y l_w (1 + \frac{P_u}{A_{st} f_y})(1 - \frac{c}{2l_w})\right]$$

$$= 0.9[0.5 * 6616.35 * 420 * 5850(1+0)(1-0.077/2)] = 7033.72\ KN \geqq 32457.2 KN.m$$

Mub=Mu-ØMn=32457.2 − 7033.72= 25423.48 KN.m

X≥ $\frac{lw}{600*\frac{\Delta h}{hw}} = \frac{5850}{600*.007}$=1392.8 mm

Lb≥ $\frac{X}{2} = 696.4\ mm$

Mub (moment carried by boundary steel)=25423.48 KN.m

Asb=Mn/{Fy*(Lw-Lb)}= $\frac{(25423.48*10^6)/0.9}{420*(5850-700)}$ = 1360 mm^2

select 8Ø 16 with As=1608 mm^2 for each boudary element

4.13 Design of stair:

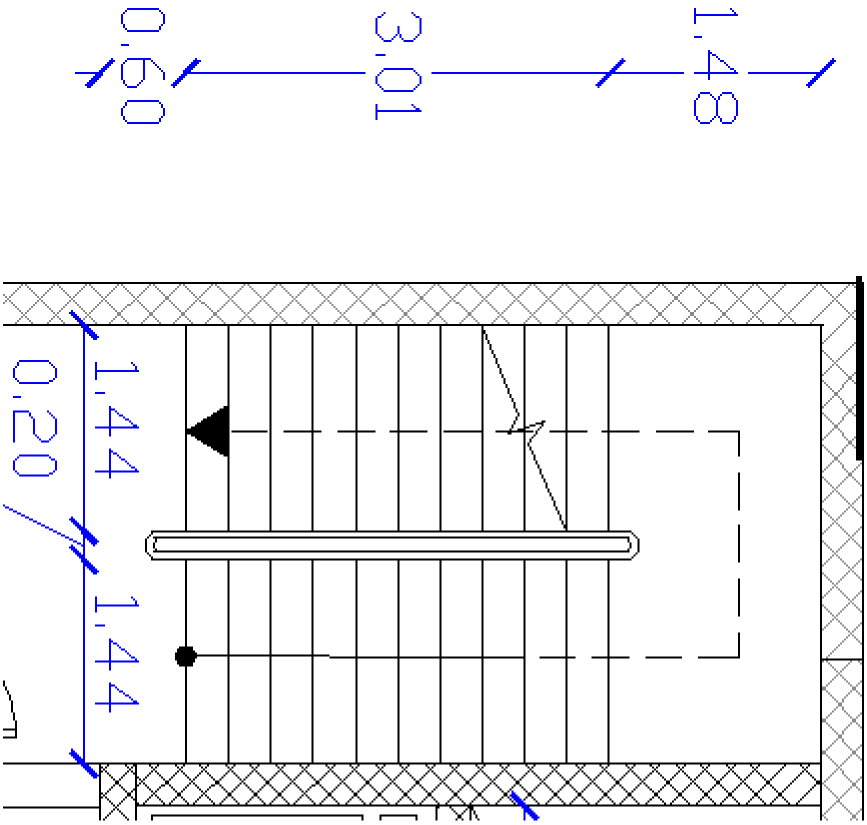


Figure (4-25): Top view of stair

❖ **Material :-**

⇒ concrete B300 $Fc' = 24\ N/mm^2$

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

1- Design of Flight :-

✓ **Determination of Thickness:-**

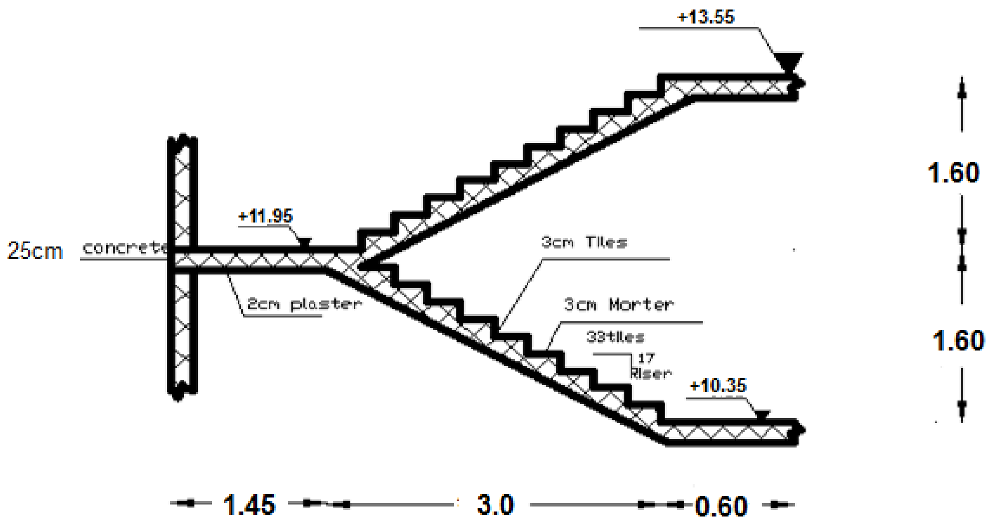
$h_{min} = L/20$

$h_{min} = 5.05/20 = 25$ cm

Take h = 25 cm

The Stair Slope by $\theta = \tan^{-1}(17/30) = 29.56^{o}$

✓ **Load Calculation:-**



Fi g(4.26): Stair Section.

Dead Load For Flight For 1m Strip:-

Table (4.5): Dead Load Calculation on Flight.

No.	Parts of Flight	Calculation	
1	Tiles	23*0.03*1*(0.33+0.17/0.3) = 1.15 KN/m	
2	Mortar	22*0.03*1*(0.3+0.17/0.3) = 1.04 KN/m	
3	Stair	25*0.5*0.17*1 = 2.13 KN/m	
4	R.C	25*0.25*1/cos29.56° 7.18KN/m	
5	Plaster	22*0.02*1 / cos 29.56° = 0.51 KN/m	
		Sum	12 KN/m

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

Factored Load for Flight:-

W_U = 1.2 ×12+ 1.6×5 =22.4 KN/m

✓ System of Flight:-

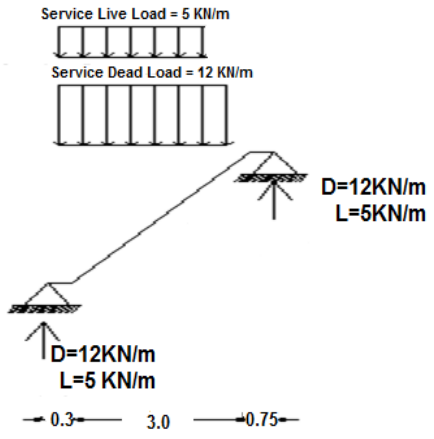


Fig (4.27): Static System and Loads Distribution of Flight.

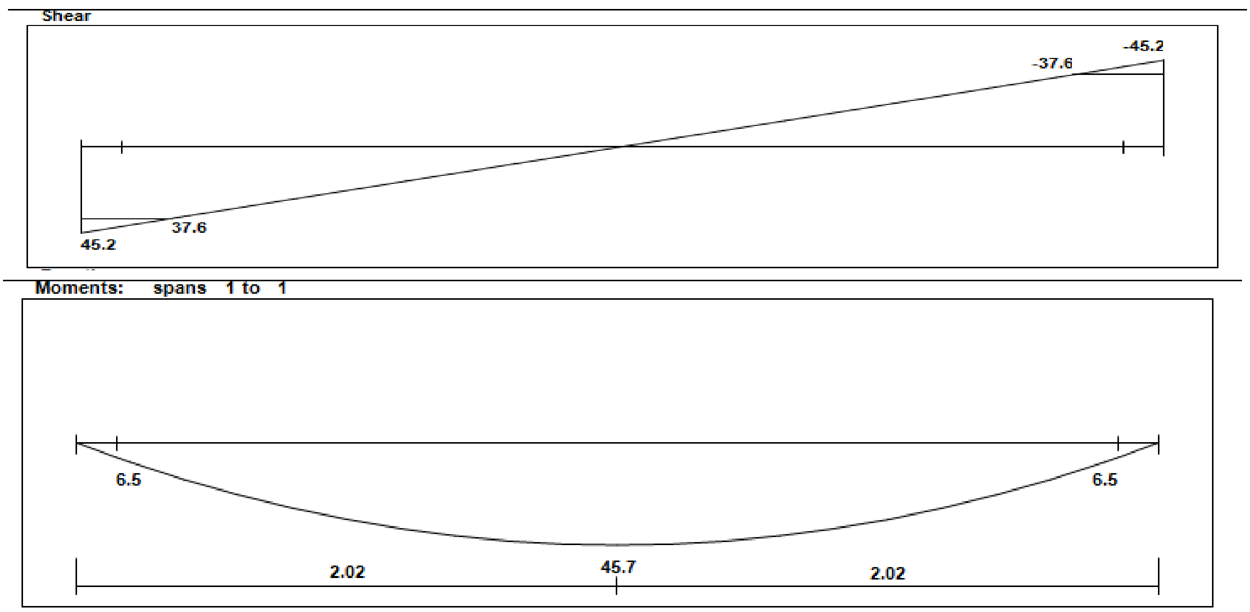


Fig (4.28): Shear and Moment Diagram of Flight

1- Design of Shear for Flight :- (Vu=37.6 KN.m)

Assume bar diameter ø 12 for main reinforcement

d =h- cover $-\frac{d_b}{2} = 250 - 20 - \frac{12}{2} = 224\ mm$

$V_c = \frac{1}{6}\sqrt{fc'}b_w\ d = \frac{1}{6}\sqrt{24} * 1000 * 224 = 182.9KN$

$\Phi\ V_c = 0.75*182.9 = 137.2KN > Vu = 37.6$ KN…… **No shear reinforcement are required**

2- Design of Bending Moment for Flight :- (Mu=45.7KN.m)

$R_n = \frac{M_u}{\emptyset bd^2} = \frac{45.7\times10^6}{0.9\times1000\times224^2} = 1.01\ Mpa$

$m = \frac{fy}{0.85f_c'} = \frac{420}{0.85\times24} = 26$

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times2.6\times1.01}{420}}\right) = 0.0247$

$A_{s,req} = \rho.b.d = 0.0247\times1000\times224 = 553.83mm^2$

$A_{s,min} = 0.0018*1000*250 = 450\ mm^2$

$As_{req} = 553.83mm^2 > A_{s,min} = 450\ mm^2$……… **is ok**

Check for Spacing :-

S = 3h = 3*250 =750 mm

S = 450 mm

S = 450 mm ……… **is control**

Use ø12 @ 200mm , $A_{s,provided} = 565mm^2 > A_{s,required} = 553.83mm^2$… Ok

Check for strain:-

$a = \frac{A_{s.}f_y}{0.85b\ f_c'} = \frac{565\times420}{0.85\times1000\times24} = 11.63\ mm$

$c = \frac{a}{\mathcal{B}_1} = \frac{11.63}{0.85} = 13.68mm$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{224-13.68}{13.68}\right) = 0.0046 > 0.005\quad \dots\dots \boldsymbol{Ok}$$

3- Lateral or Secondary Reinforcement For Flight :-

$A_{s,req} = A_{s,min} = 0.0018*1000*250 = 450mm^2$

Use ø12 @ 200 mm , $A_{s,provided}$= 565mm^2>$A_{s,required}$= 450 mm^2… Ok

1- Design of Landing :- (For First One Meter)

Determination of Thickness:-

Table (4.6): Dead load calculation of landing

h_{min} = L/20

h_{min} = 3.1/20 = 15.5cm, take h=25 cm

No.	Parts of Landing	Calculation	
1	Tiles	22*0.03*1= 0.66 KN/m	
2	Mortar	22*0.03*1= 0.66 KN/m	
4	R.C	25*0.25*1= 6.25KN/m	
5	Plaster	22*0.02*1= 0.44 KN/m	
		Sum	8KN/m

- ✓ **Load Calculation:-**

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

Reaction From Flight:-

DL =24.24 KN/m
LL = 10KN/m

Total Dead Load = 8 + 24.24 = 32.24 KN/m

Total Live Load = 5 + 10 = 15 KN/m

Factored Load For Landing :-

W_U = 1.2 ×32.24+ 1.6×15=62.7KN/m

✓ System of Landing:-

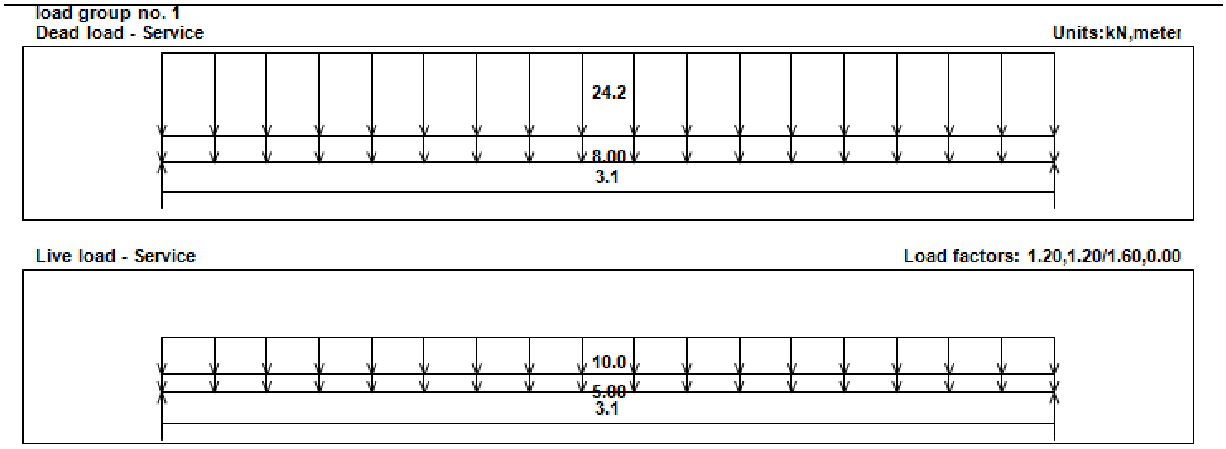


Fig (4.29): Static System and Loads Distribution At First 1m 0f Landing

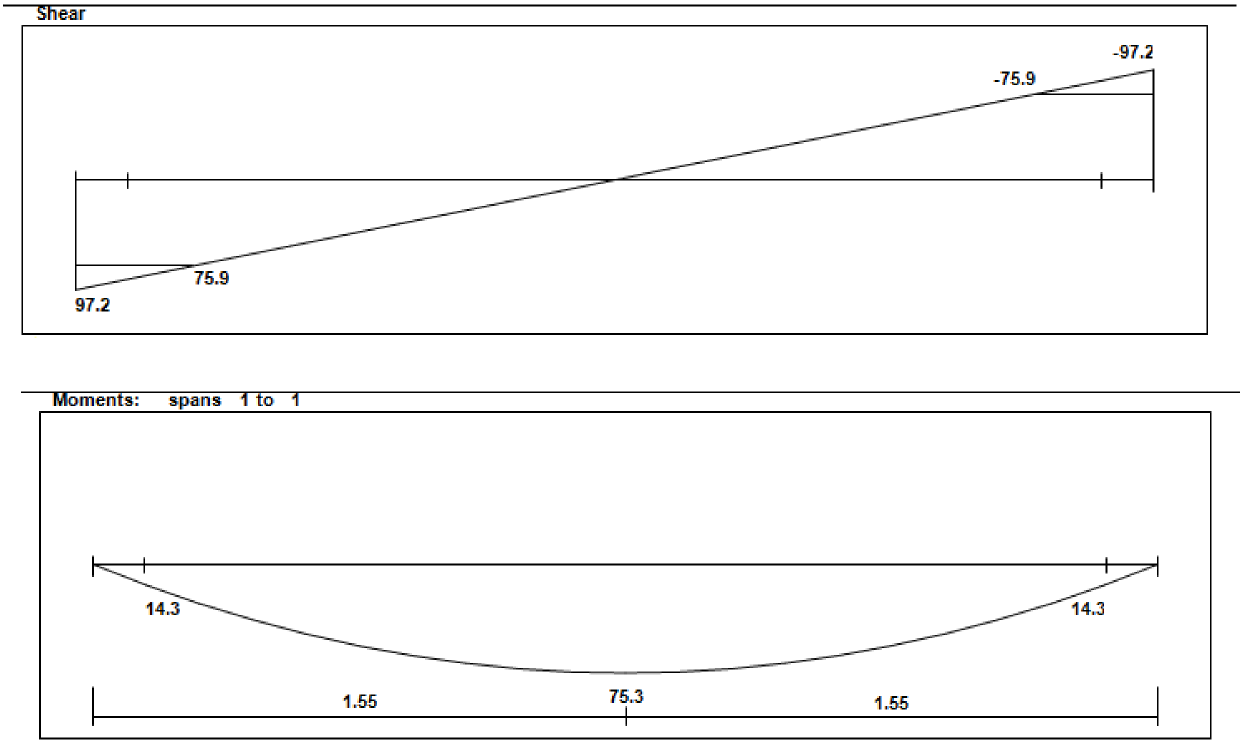


Fig (4.30): Shear and Moment Diagram At First 1m of Landing.

1- Design of Shear:- (Vu=75.9 KN)

Assume bar diameter ø 12 for main reinforcement d =h- cover $-\frac{d_b}{2} = 300 - 20 - \frac{12}{2} = 224\ mm$

$V_c = \frac{1}{6}\sqrt{fc'}b_w\ d == \ \frac{1}{6}\sqrt{24} * 1000 * 224 = 182.9$KN

Φ* V_c = 0.75* 182.9= 137.2 KN> Vu = 75.9 KN…... **No shear reinforcement are required**

2- Design of Bending Moment :- (Mu=75.3 KN.m)

Assume bar diameter ø 12 for main reinforcement

d =h- cover $-\frac{d_b}{2} = 300 - 20 - \frac{12}{2} = 224\ mm$

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{75.3\times10^6}{0.9\times1000\times224^2} = 1.667\ \ Mpa$

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

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2\times20.6\times1.667}{420}}\right) = 0.0415$

$A_{s,req}$ = ρ.b.d = 0.0415×1000×224 = 929 mm^2

$A_{s,min}$ =0.0018*1000*250= 450 mm^2

$A_{s,req}$ 929 mm^2

Check for Spacing :-

S = 3h = 3*250=750 mm

S = 450 mm……… **is control**

Use ø14@ 15 cm , $A_{s,provided}$= 1025.64 mm^2>$A_{s,required}$= 929mm^2… Ok

Check for strain:-

$a = \frac{A_{s.}f_y}{0.85b\ f_c'} = \frac{1025.64\times420}{0.85\times1000\times24} = 21.11\ \ mm$

$c = \frac{a}{\beta_1} = \frac{21.11}{0.85} = 24.85mm$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{224-24.85}{24.85}\right) = 0.0240 > 0.005\ \ \dots\dots\boldsymbol{0k}$$

1- Lateral or Secondary Reinforcement For Landing :-

$A_{s,req} = A_{s,min} = 0.0018*1000*250 = 450mm^2$

Use ø12@ 200 mm , $A_{s,provided} = 565mm^2 > A_{s,required} = 450mm^2$… Ok

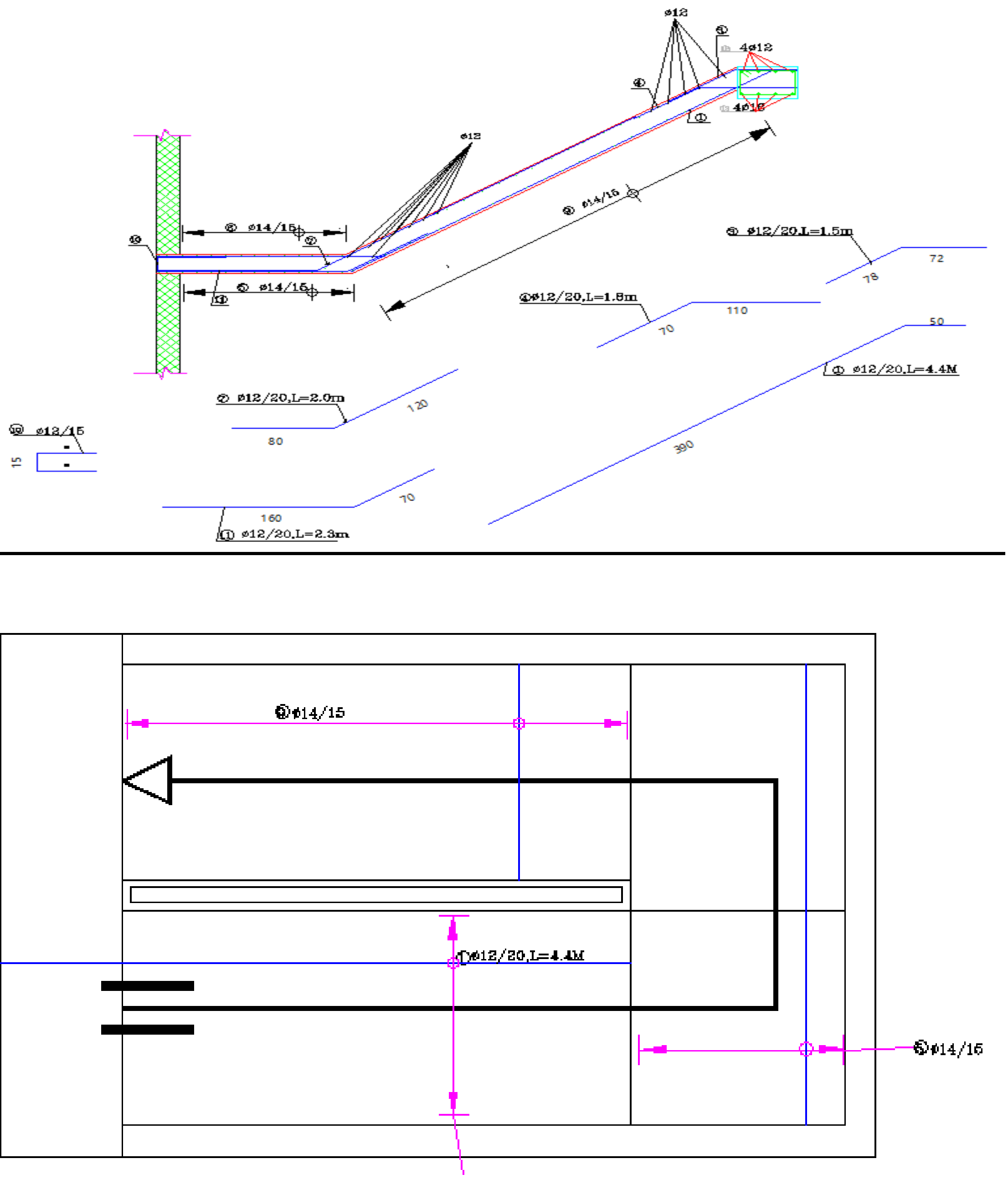


Fig (4.31): Reinforcement of stairs

4.11 Design of Basement Wall:

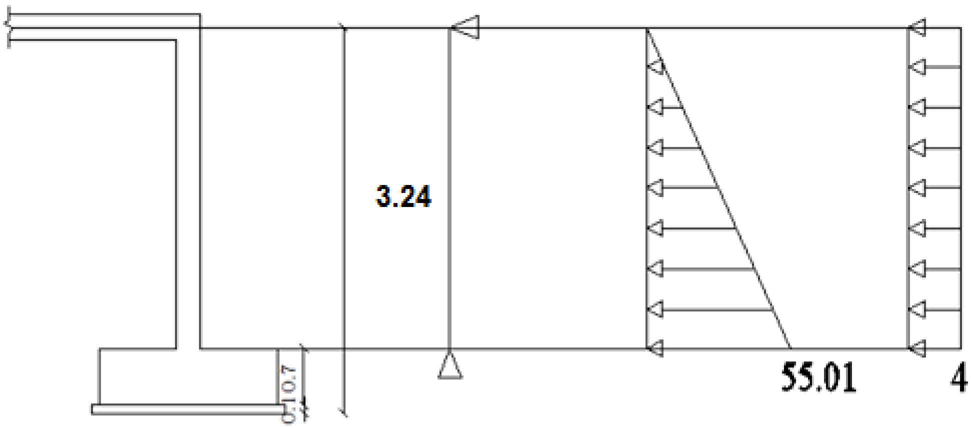


Figure (4-32): Geometry of basement.

❖ **Material :-**

⇒ concrete B300 Fc' = 24 N/mm^2

⇒ Reinforcement Steel Fy = 420 N/mm^2

$\emptyset = 30° \gamma = 18.00 \mathrm{KN/m^3}$

$18.00 \mathrm{KN/m^3}$

$$\begin{aligned} Ko &= 1 - \sin \emptyset \\ &= 1 - \sin 30 \\ &= 0.50 \end{aligned}$$

4.14.1 Load on basement wall:

For 1m length of wall:

* **Weight of backfill:**

$q1 = Ko * \gamma * h$

= 0.50*18.0*3.24 =29.16 KN/m

$q_{1\ (Factored)}$ = 1.2 *29.16 = 35 KN/m

*** Load from live load:**

LL=5 KN/m2

$q2 = Ko * LL$

= 0.50 * 5 =2.50 KN/m

$q_{2\ (Factored)}$ = 1.6 *2.50 =4.0 KN/m

4.14.2 Design of the shear force:

Assume h = 300 mm,

$d = 300 - 20 - 14 = 266\ mm$

Vmax =77.77 KN

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

$$\phi Vc = \frac{\phi\sqrt{24} * 1000 * 266}{6} = 162.9KN$$

Vu $< \phi Vc$

No shear Reinforcement is required.

4.14.3 Design of bending moment:

Mu max =61.1 KN.m

$$Mn = \frac{Mu}{0.9} = \frac{61.1}{0.9} = 67.88\ KN.m$$

$$Kn = \frac{Mn * 10^6}{b * d^2} = \frac{67.88 * 10^6}{1000 * 266^2} = 0.96\ Mpa$$

$$m = \frac{Fy}{0.85 * fc'} = \frac{420}{0.85 * 24} = 20.58$$

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

$$= \frac{1}{20.58} * \left(1 - \sqrt{1 - \frac{2 * 0.96 * 20.58}{420}}\right)$$

$$= 2.343 * 10^{-3}$$

$$Asreq = \rho * b * d = 2.34 * 10^{-3} * 1000 * 266 = \mathbf{6.23\ cm^2/m}$$

$$Asmin\ = 0.0012 * b * h = 0.0012 * 1000 * 300 = 3.60\ cm^2/m$$

$$Amin\ \leq Areq$$

Select $\emptyset 12@15cm/m$

Vertical reinforcement at compression face:

As req =As min $= 3.60\ cm^2/m$

$\boldsymbol{\emptyset 10@15cm/m}$

4.14.4 Design of the horizontal reinforcement:

$$Asmin\ = 0.0012 * b * h = 0.002 * 1000 * 300 = 360cm^2/m$$

Select$\boldsymbol{\emptyset 10@20cm/m}$, in two layer.

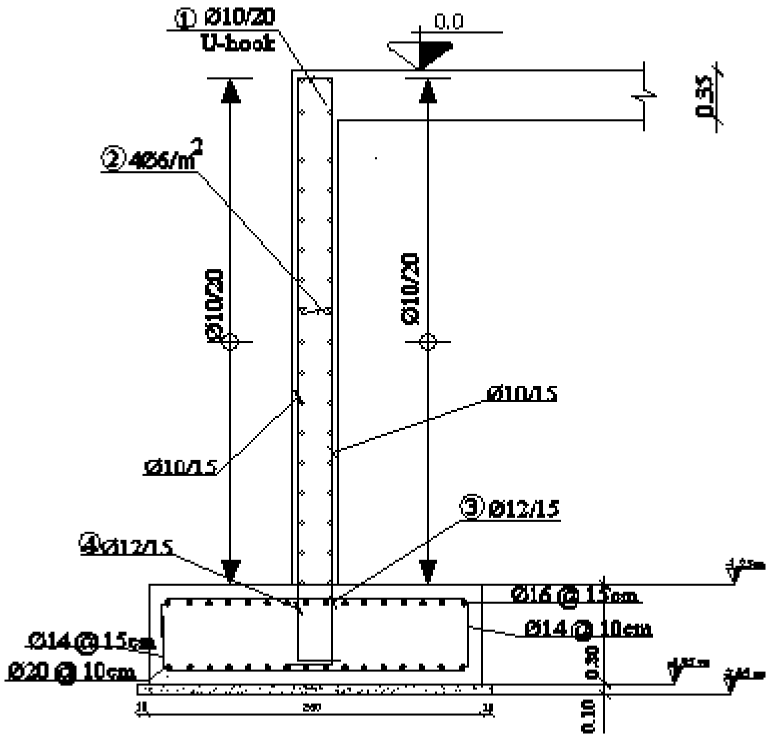


Figure (4-33): Reinforcement for Basement Wall.