

4**Chapter Four****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_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,

$$\text{Strength provided} \geq \text{strength required to carry factored loads.}$$

NOTE:-

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

✓ Code:-

ACI 2008

UBC

✓ Material:-

Concrete:-B300

$f_c' = 30 \text{ N/mm}^2 (\text{MPa})$ For circular section

but for rectangular section ($f_c' = 30 * 0.8 = 24 \text{ MPa}$).

Reinforcement steel:-

The specified yield strength of the reinforcement ($f_y = 420 \text{ N/mm}^2 (\text{MPa})$).

✓ Factored loads:-

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

$$W_u = 1.2 D_L + 1.6 L_L \quad \text{ACI-code-318-08(9.2.1)}$$

4.3 Check of Minimum Thickness of Structural Member:

Minimum Thickness of Non prestressed Beam or One-Way Slabs Unless Deflections are Calculated.
(ACI 318M-11).

Table (4.1): Check of Minimum Thickness of Structural Member.

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 :-

$$h_{\min} \text{for (one end continuous)} = L/18.5 = 5.3/18.5 = 28.6 \text{ cm}$$

$$h_{\min} \text{for (both end continuous)} = L/21 = 5.3/21 = 25.2 \text{ cm}$$

$$h_{\min} \text{for (cantilever)} = L/8 = 1.26/8 = 15.8 \text{ cm}$$

Take $h = 32 \text{ cm}$

24 cm block + 8 cm topping = 32 cm

For Beam :-

$$h_{\min} \text{for (one end continuous)} = L/18.5 = 10.8/18.5 = 58.4 \text{ cm}$$

$$h_{\min} \text{for (both end continuous)} = L/21 = 8.15/21 = 38.8 \text{ cm}$$

$$h_{\min} \text{for (Simply supported)} = L/16 = 10.7/16 = 66.9 \text{ cm}$$

Take $h = 70 \text{ cm}$

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.

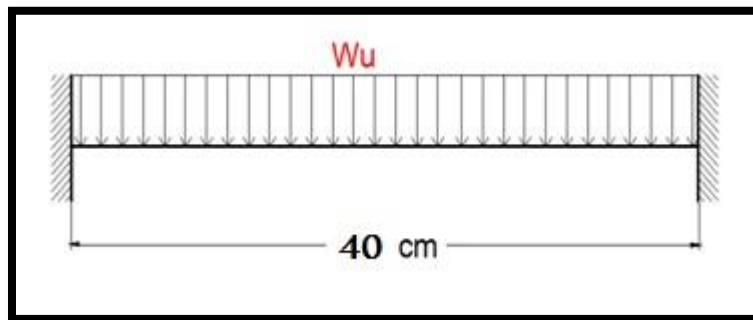


Fig 4.1: Topping Load.

✓ Load Calculations:-

Dead Load:-

Table (4.2): Dead Load Calculation of Topping.

No.	Parts of Rib	Calculation
1	Tiles	$0.03 \times 23 \times 1 = 0.69 \text{ KN/m}$
2	Mortar	$0.03 \times 22 \times 1 = 0.66 \text{ KN/m}$
3	Coarse Sand	$0.07 \times 16 \times 1 = 1.12 \text{ KN/m}$
4	Topping	$0.08 \times 25 \times 1 = 2.0 \text{ KN/m}$
5	Interior partitions	$1 \times 1 = 1 \text{ KN/m}$
Sum =		5.47KN/m

Live Load :-

$$L_L = 5 \text{ KN/m}^2$$

$$L_L = 5 \text{ KN/m}^2 \times 1 \text{ m} = 5 \text{ KN/m}$$

Factored Load :-

$$W_U = 1.2 \times 5.47 + 1.6 \times 5 = 14.56 \text{ KN/m}$$

Check the strength condition for plain concrete, $\phi M_n \geq M_u$, where $\phi = 0.55$

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

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

$$\phi M_n = 0.55 \times 0.42 \times 1 \times \sqrt{24} \times 1066666.67 \times 10^{-6} = 1.21 \text{ KN.m}$$

$$M_u = \frac{w_u L^2}{12} = 0.194 \text{ KN.m} \quad (\text{negative moment})$$

$$M_u = \frac{w_u L^2}{24} = 0.097 \text{ KN.m} \quad (\text{positive moment})$$

$$\phi M_n \gg M_u = 0.194 \text{ KN.m}$$

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

$$\rho_{\text{shrinkage}} = 0.0018 \quad \text{ACI 7.12.2.1}$$

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

Step (s) is the smallest of:

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

Take $\phi 8$ @ 200 mm in both direction , $S = 200 \text{ mm} < S_{\max} = 240 \text{ mm} \dots \text{OK}$

4.5 Design of One Way Rib Slab (R1)

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

$$bw \geq 10 \text{ cm} \dots \text{ACI(8.13.2)}$$

Select $bw = 15 \text{ cm}$

$$h \leq 3.5 \cdot bw \dots \text{ACI(8.13.2)}$$

Select $h = 32 \text{ cm} < 3.5 \cdot 15 = 52.5 \text{ cm}$

$$tf \geq L_n/12 \geq 50 \text{ mm} \dots \text{ACI(8.13.6.1)}$$

Select $tf = 8 \text{ cm}$

✓ **Material :-**

- ⇒ concrete B300 $F_c' = 24 \text{ N/mm}^2$
 ⇒ Reinforcement Steel $f_y = 420 \text{ N/mm}^2$

✓ **Section :-**

- ⇒ $B = 550 \text{ mm}$
 ⇒ $B_w = 150 \text{ mm}$
 ⇒ $h = 320 \text{ mm}$
 ⇒ $t = 80 \text{ mm}$
 ⇒ $d = 320 - 20 - 10 - 12/2 = 284 \text{ mm}$

✓ **Statically System and Dimensions:-**

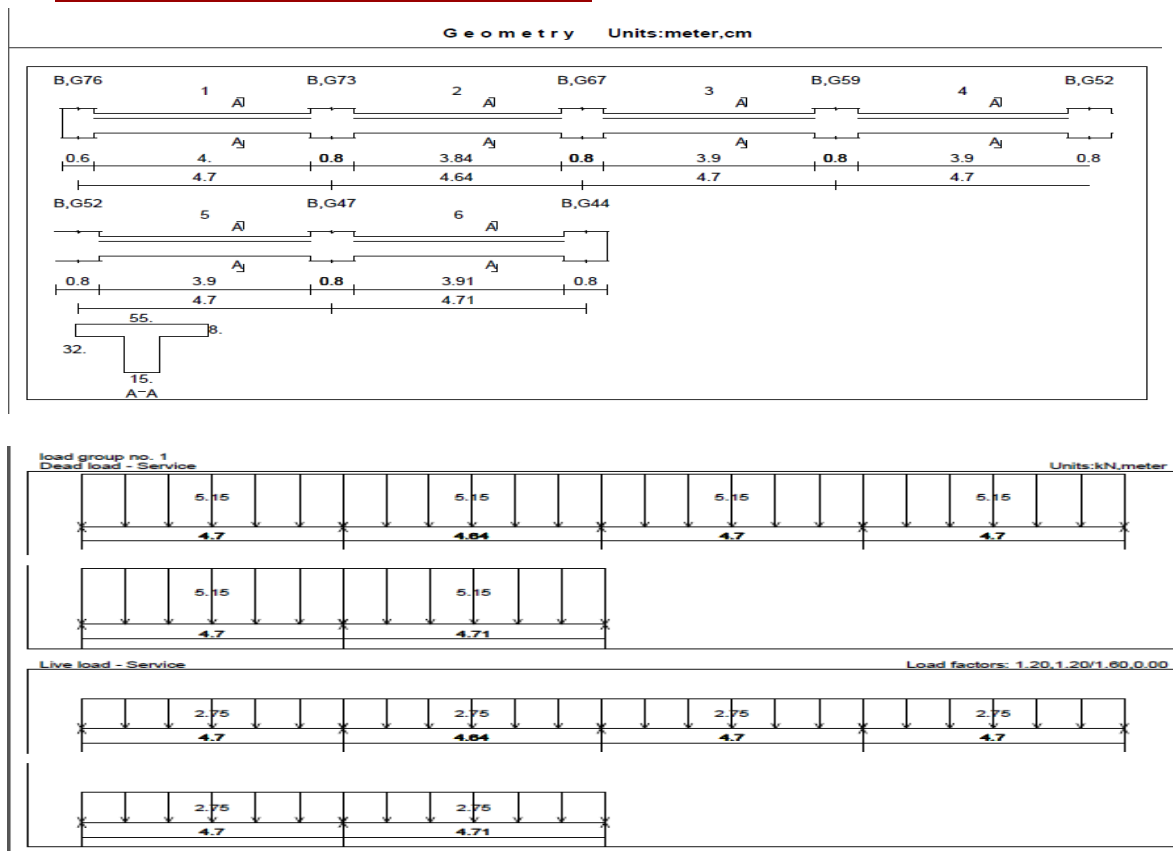


Fig 4.2: Statically System and Loads Distribution of Rib(R1).

✓ Load Calculation:-

Dead Load:-

Table (4.3): Dead Load Calculation of Rib(R1).

No.	Parts of Rib	Calculation
1	Tiles	$0.03 \times 23 \times 0.55 = 0.38 \text{ KN/m/rib}$
2	Mortar	$0.03 \times 22 \times 0.55 = 0.363 \text{ KN/m/rib}$
3	Coarse Sand	$0.07 \times 17 \times 0.55 = 0.655 \text{ KN/m/rib}$
4	Topping	$0.08 \times 25 \times 0.55 = 1.1 \text{ KN/m/rib}$
5	RC. Rib	$0.24 \times 25 \times 0.15 = 0.9 \text{ KN/m/rib}$
6	Hollow Block	$0.24 \times 10 \times 0.4 = 0.96 \text{ KN/m/rib}$
7	plaster	$0.02 \times 22 \times .55 = 0.242 \text{ KN/m/rib}$
8	partions	$1 \times 0.55 = 0.55 \text{ KN/m/rib}$
		Sum = 5.15 KN/m/rib

Dead Load /rib = 5.15 KN/m

Live Load:-

Live load = 5 KN/M^2

Live load /rib = $5 \text{ KN/m}^2 \times 0.55\text{m} = 2.75 \text{ KN/m}$.

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

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

$$b_E = L / 4 = 384 / 4 = 96\text{cm}$$

$$b_E = 15 + 16 t = 15 + 16 (8) = 143 \text{ cm}$$

$$b_E = b_e \leq \text{center to center spacing between adjacent beams} = 55 \text{ cm.}$$

Control

b_E **For T-section = 55cm .**

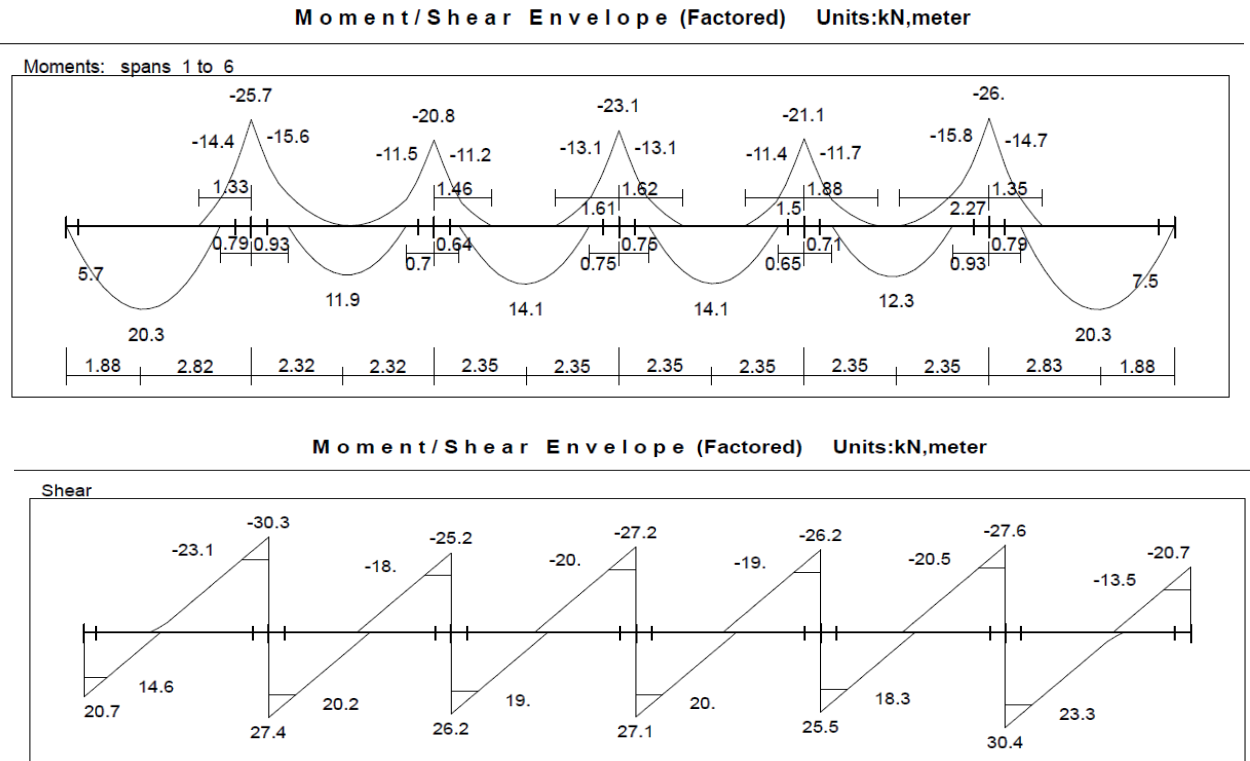


Fig 4.3: Shear and Moment Envelope Diagram of Rib (R1).

✓ Moment Design for (R 1):-

Design of Positive Moment for (Rib1):-($M_u=20.3\text{KN.m}$)

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

$$d = h - \text{cover} - d_{\text{stirrups}} - \frac{d_b}{2} = 320 - 20 - 10 - \frac{12}{2} = 284 \text{ mm}$$

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

$$M_{nf} = 0.85 \cdot f'_c \cdot b_e \cdot h_f \cdot \left(d - \frac{h_f}{2}\right)$$

$$= 0.85 \times 24 \times 550 \times 80 \times \left(284 - \frac{80}{2}\right) \times 10^{-6} = 219 \text{ KN.m}$$

$$M_n \gg \frac{M_u}{\phi} = \frac{20.3}{0.9} = 22.55 \text{ KN.m}, \text{ the section will be designed as rectangular section}$$

with $b_e = 550 \text{ mm}$.

$$R_n = \frac{M_u}{\phi b d^2} = \frac{20.3 \times 10^6}{0.9 \times 550 \times 284^2} = 0.508 \text{ Mpa}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{420}} \right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.508}{420}} \right) = 0.001225$$

$$A_{s, \text{req}} = \rho \cdot b \cdot d = 0.001225 \times 550 \times 284 = 191.35 \text{ mm}^2$$

Check for As min:-

$$A_s \text{ min} = \frac{\sqrt{f'_c}}{4(f_y)} (bw)(d) \text{ **ACI-318 (10.5.1)}**$$

$$A_s \text{ min} = \frac{\sqrt{24}}{4(420)} (150)(284) = 124.2 \text{ mm}^2$$

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

$$A_s \text{ min} = \frac{1.4}{420} (150)(284) = 142 \text{ mm}^2 \text{ **controls**}$$

$$A_{s, \text{req}} = 191.35 \text{ mm}^2 > A_{s, \text{min}} = 142 \text{ mm}^2 \quad \text{OK}$$

Use 2 ø 12 , $A_{s, \text{provided}} = 226 \text{ mm}^2 > A_{s, \text{required}} = 191.35 \text{ mm}^2$ Ok

$$S = \frac{150 - 40 - 20 - (2 \times 12)}{1} = 66 \text{ mm} > d_b = 12 > 25 \text{ mm} \quad \text{OK}$$

Check for strain:-

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{226 \times 420}{0.85 \times 550 \times 24} = 8.46 \text{ mm}$$

$$x = \frac{a}{\beta_1} = \frac{8.46}{0.85} = 9.95 \text{ mm}$$

$$\varepsilon_s = 0.003 \left(\frac{d - x}{x} \right) = 0.003 \left(\frac{284 - 9.95}{9.95} \right) = 0.0826 > 0.005 \quad \mathbf{Ok}$$

Design of Negative Moment for(Rib1):- (Mu=-15.8KN.m)

Assume bar diameter ϕ 12 for main positive reinforcement

$$d = h - \text{cover} - d_{\text{stirrups}} - \frac{d_b}{2} = 320 - 20 - 10 - \frac{12}{2} = 284 \text{ mm}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{15.8 \times 10^6}{0.9 \times 150 \times 284^2} = 1.45 \text{ Mpa}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 m R_n}{420}} \right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 1.45}{420}} \right) = 0.00358$$

$$A_{s, \text{req}} = \rho \cdot b \cdot d = 0.00358 \times 150 \times 284 = 152.5 \text{ mm}^2$$

Check for As min:-

$$A_s \text{ min} = \frac{\sqrt{f'_c}}{4(f_y)} (b_w)(d) \text{ ACI-318 (10.5.1)}$$

$$A_s \text{ min} = \frac{\sqrt{24}}{4(420)} (150)(284) = 124.22 \text{ mm}^2$$

$$A_s \text{ min} = \frac{1.4}{(f_y)} (b_w)(d)$$

$$A_s \text{ min} = \frac{1.4}{420} (150)(284) = 142 \text{ mm}^2 \text{ controls}$$

$$A_{s, \text{req}} = 152.5 \text{ mm}^2 > A_{s, \text{min}} = 142 \text{ mm}^2 \mathbf{OK}$$

Use 2 ϕ 10 , $A_{s, \text{provided}} = 157 \text{ mm}^2 > A_{s, \text{required}} = 152.5 \text{ mm}^2 \dots \text{Ok}$

$$S = \frac{150-40-20-(2 \times 10)}{1} = 70 \text{ mm} > d_b = 10 > 25 \text{ mm} \quad \text{OK}$$

Check for strain:-

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{157 \times 420}{0.85 \times 150 \times 24} = 21.55 \text{ mm}$$

$$x = \frac{a}{B_1} = \frac{21.55}{0.85} = 25.35 \text{ mm}$$

$$\epsilon_s = 0.003 \left(\frac{d - x}{x} \right) = 0.003 \left(\frac{284 - 25.35}{25.35} \right) = 0.030 > 0.005 \quad \text{OK}$$

✓ Shear Design for (R 1):-

V_u at distance d from support = 23.3 KN

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).

$$V_c = \frac{1.1}{6} \sqrt{f'_c} b_w d = \frac{1.1}{6} \sqrt{24} \times 150 \times 284 \times 10^{-3} = 38.26 \text{ KN}$$

$$\phi V_c = 0.75 \times 38.26 = 28.70 \text{ KN}$$

$$0.5 \phi V_c = 0.5 \times 28.70 = 14.35 \text{ KN}$$

$$0.5 \phi V_c < V_u < \phi V_c$$

Case (2) for shear design, minimum shear reinforcement is required ($A_{v,min}$), exception for Ribbed slab, No shear Reinforcement.

Use stirrups U-shape as montage (2 leg stirrups) $\phi 8 @ 250 \text{ mm}$, $A_v = 2 \times 50.24 = 100.48 \text{ mm}^2$.

$$A_v = \frac{2 \times 50.3}{0.25} = 401.92 \text{ mm}^2/\text{m}_{\text{strip}}$$

4.6 Design of One Way Solid Slab (S1).

✓ Material :-

⇒ concrete B300 $f_c' = 24 \text{ N/mm}^2$
,
⇒ Reinforcement Steel $f_y = 420 \text{ N/mm}^2$

✓ Slabs Thickness calculation:-

The overall depth must satisfy ACI Table (9.5.a):

→ from *ACI-318-08 table (9.5a)*

Min h (deflection requirement) \geq :

- For simply supported one-way solid:

$$\frac{L}{20} = \frac{5.45}{20} = 0.2725 \text{ m}$$

For One way solid slab ,will use thickness of slab **25 cm** .

✓ **Statically System and Dimensions:-**

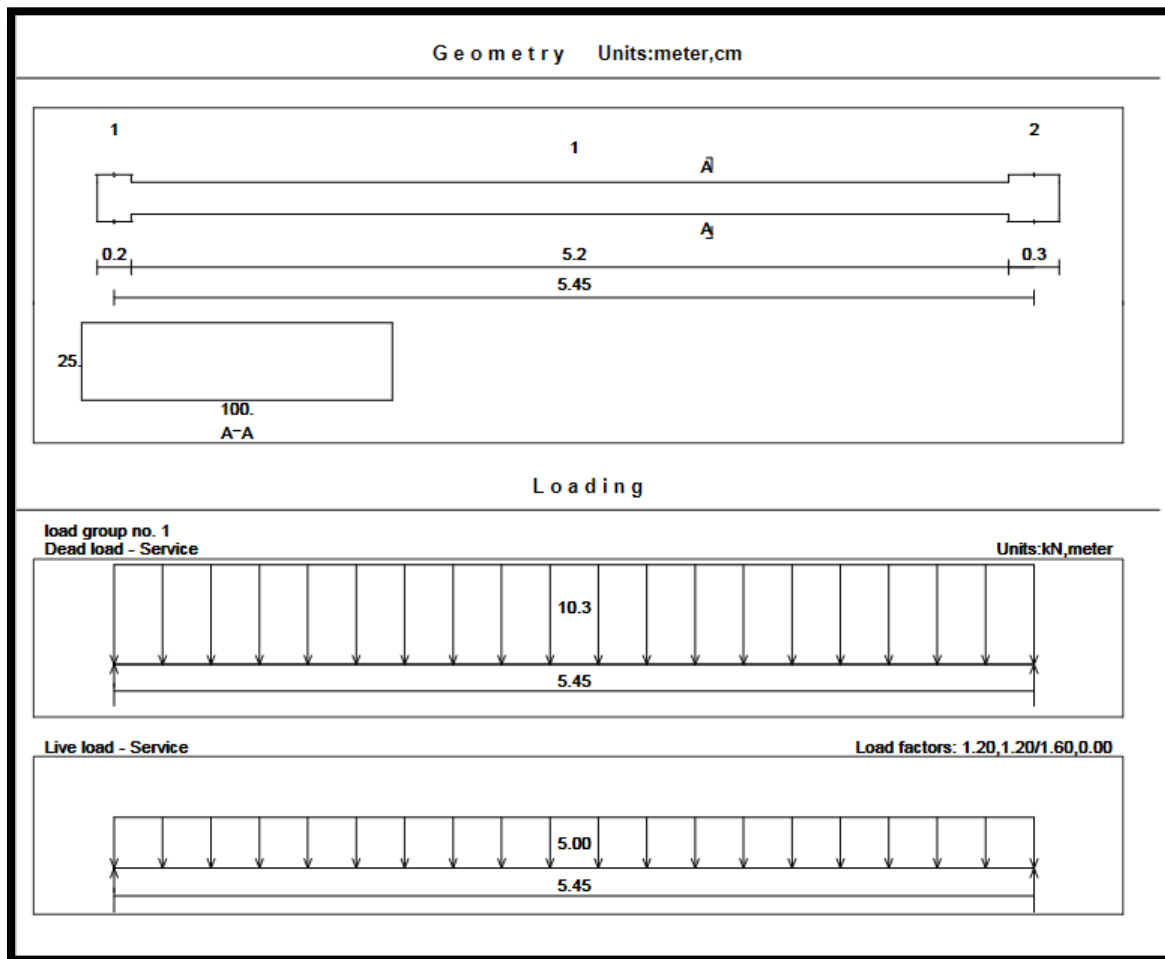


Fig 4.4: Statically System and Loads Distribution of Solid Slab(S1).

✓ Load Calculations:-

Dead Load:-

Table (4.4): Dead Load Calculation of Solid slab (S1) .

No.	Parts of Beam	Calculation
1	Tiles	$0.03 \times 23 \times 1 = 0.69 \text{ KN/m}$
2	Mortar	$0.03 \times 22 \times 1 = 0.66 \text{ KN/m}$
3	Coarse Sand	$0.07 \times 16 \times 1 = 1.12 \text{ KN/m}$
5	RC. Slab	$1 \times 0.25 \times 25 = 6.25 \text{ KN/m}$
7	plaster	$0.03 \times 22 \times 1 = 0.66 \text{ KN/m}$
8	partions	$1 \times 1 = 1 \text{ KN/m}$
		Sum = 10.38 KN/m

Live Load:-

LL=5KN/m .

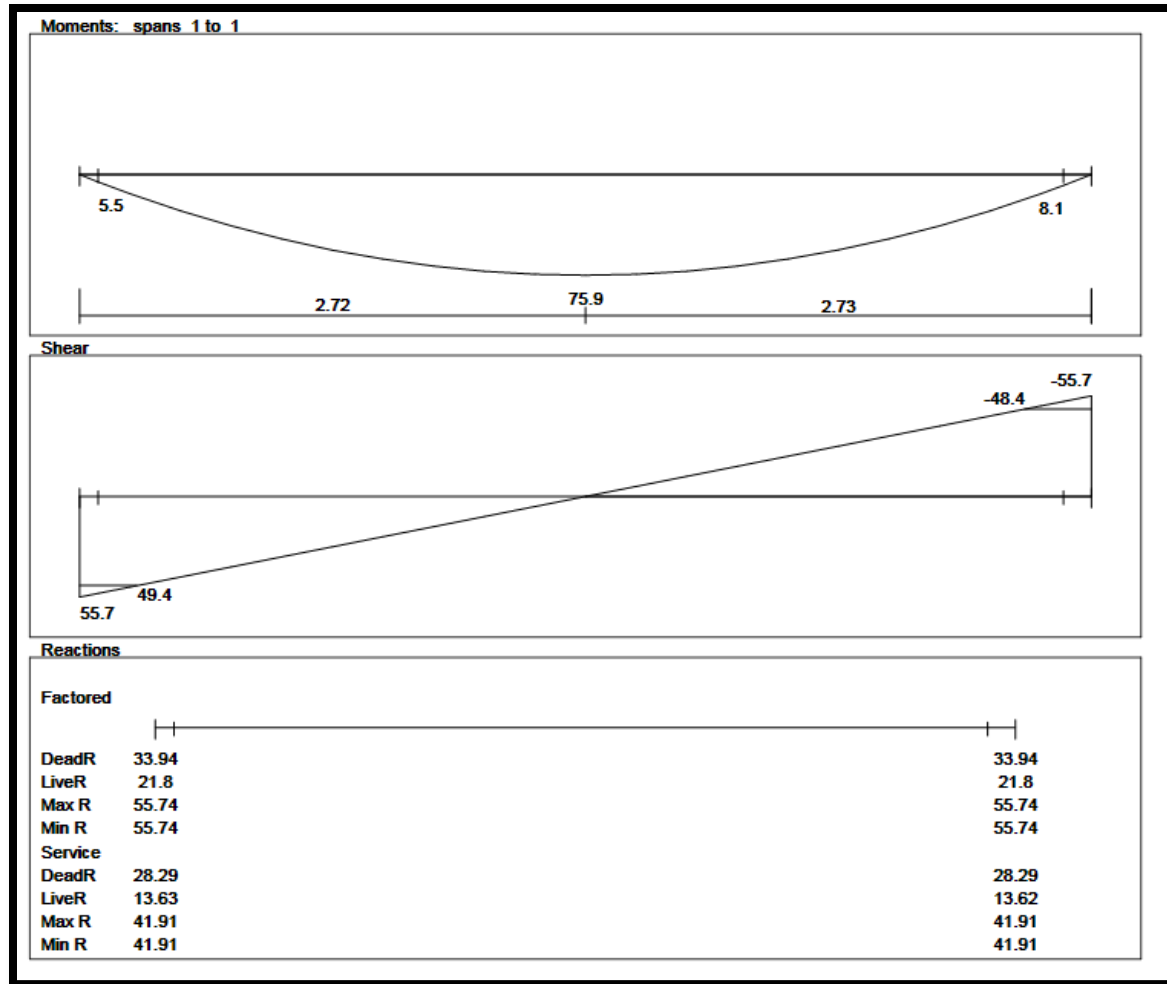


Fig 4.5: Shear and Moment Envelope Diagram of Solid Slab(S1).

✓ Design of slab:-

Assume bar diameter $\phi 10$ for main reinforcement.

$$d = 250 - 20 - \frac{10}{2} = 225 \text{ mm}$$

• For shear:

check whether thickness is adequate for shear:

$$V_{u,max} = 49.4 \text{ KN/m strip}$$

$$\phi V_c = \frac{1}{6} * 0.75 * \sqrt{f_c'} * b_w * d$$

$$= \frac{1}{6} * 0.75 * \sqrt{24} * 1000 * 225 = 137.78 \text{ KN / 1m strip}$$

$$\frac{1}{2} \phi V_c = \frac{1}{2} * 137.78 = 68.89 \text{ KN / 1m strip}$$

$$V_{u,max} \leq \frac{1}{2} \phi V_c - \text{No shear reinforcement is required}$$

- For positive Moment:**

$$M_u = 75.9 \text{ KN.m /m}$$

$$m = \frac{f_y}{0.85 * f_c'} = \frac{420}{0.85 * 24} = 20.59$$

$$R_n = \frac{M_u / \phi}{b * d^2}$$

$$R_n = \frac{75.9 * 10^{-3} / 0.9}{1 * (0.225)^2} = 1.67 \text{ N/mm}^2 \text{ (Mpa)}$$

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

$$\rho = \frac{1}{20.59} \left(1 - \sqrt{1 - \frac{2(20.59)(1.67)}{420}} \right) = 0.00415 > \rho_{min} = 0.0018 \quad \text{ok}$$

$$A_s = \rho * b * d = 0.00415 * 1000 * 225 = 933.75 \text{ mm}^2$$

Check Minimum Reinforcement $A_s \text{ min} \dots (\text{ACI- 318M-08} - (10.5.1))$

$$A_s \text{ min} = \rho_{min} * b * h = 0.0018 * 1000 * 250 = 450 \text{ mm}^2 \quad (\text{control})$$

$$A_s > A_{s \min}$$

$$A_{s \min} = 450 \text{ mm}^2 < A_{s \text{ req}} = 933.75 \text{ mm}^2 \text{ .OK .}$$

$$\Rightarrow \text{Use } \Phi 12 / 10 \text{ cm , } A_{s \text{ prov}} = 1130.4 \text{ mm}^2/\text{m}$$

step (s) is the smallest of :-

$$\leq 380 \left(\frac{280}{f_s} \right) - 2.5 * C_c$$

$$\leq 380 * \left(\frac{280}{\frac{2}{3} f_y} \right) - 2.5 * 20 = 380 * \left(\frac{280}{\frac{2}{3} * 420} \right) - 2.5 * 20 = 330 \text{ mm}$$

$$\leq 300 \left(\frac{280}{f_s} \right) = 300 * \left(\frac{280}{\frac{2}{3} f_y} \right) = 300 * \left(\frac{280}{\frac{2}{3} * 420} \right) = 300 \text{ mm (control)}$$

$$\leq 3 * h = 3 * 250 = 750 \text{ mm}$$

$$\leq 450 \text{ mm.}$$

$$S = 100 \text{ mm} \leq S_{\max} = 300 \text{ mm}$$

∴ Use $\Phi 12$ @ 10 cm in main directions.

Temperature and Shrinkage :

$$\rightarrow \rho = 0.0018$$

$$A_{s \min} = \rho_{\min} * b * h = 0.0018 * 1000 * 250 = 450 \text{ mm}^2 \text{ (control)}$$

Use $\Phi 12$ @ 250 mm

4.7 Design of Beam (B,G59)

✓ Material :-

⇒ concrete B300 $F_c' = 24 \text{ N/mm}^2$

⇒ Reinforcement Steel $f_y = 420 \text{ N/mm}^2$

✓ Section :-

⇒ $B = 80\text{cm}$

⇒ $h = 70 \text{ cm}$

⇒ $d = 700 - 40 - 10 - 20/2 = 640 \text{ mm}$

✓ Statically System and Dimensions:-

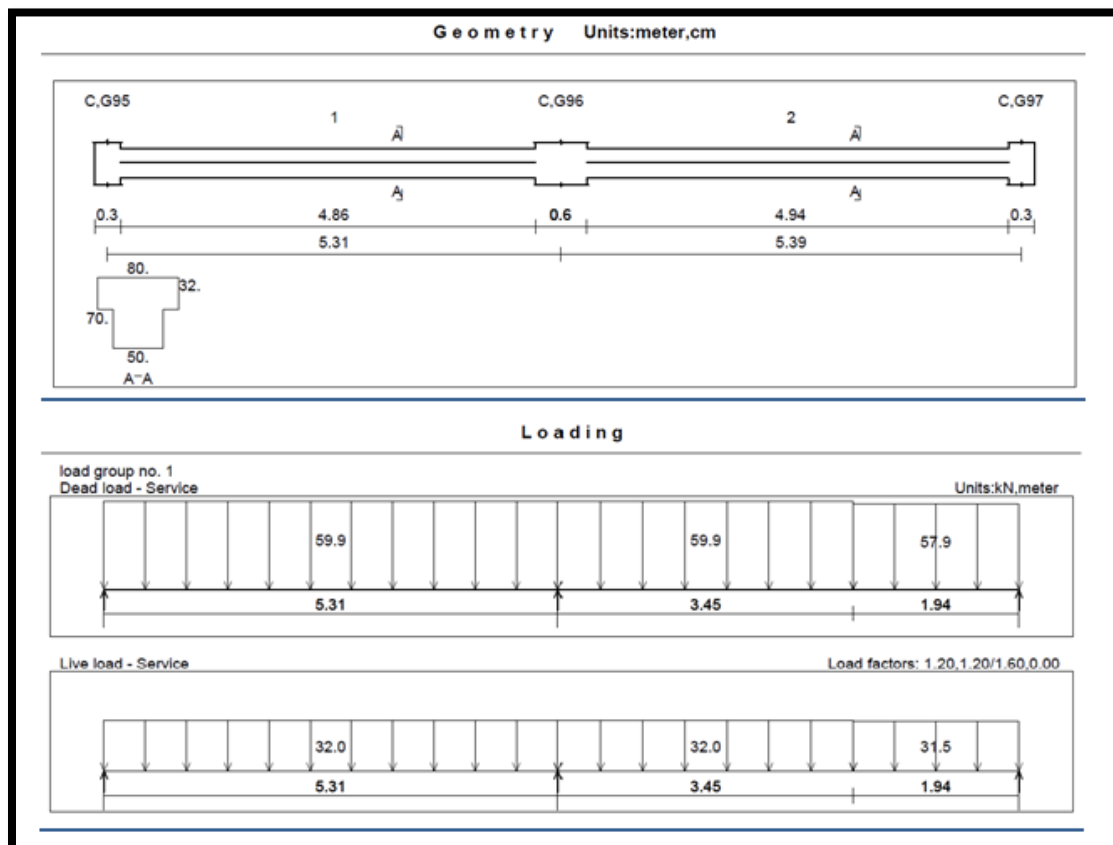


Fig 4.6: Statically System and Loads Distribution of Beam (B,G59).

✓ Load Calculations:-

Dead Load Calculations for Beam(B,G59):-

The distributed Dead and Live loads acting upon (B,G59) can be defined from the support reactions of the R1 and R2.

Dead Load:-

Table (4.5): Dead Load Calculation of Beam(B,G59).

No.	Parts of Beam	Calculation
1	Tiles	$0.03 \times 23 \times 0.8 = 0.55 \text{ KN/m}$
2	Mortar	$0.03 \times 22 \times 0.8 = 0.53 \text{ KN/m}$
3	Coarse Sand	$0.07 \times 16 \times 0.8 = 0.89 \text{ KN/m}$
5	RC. Beam	$((0.32 \times 0.8) + (0.38 \times 0.5)) \times 25 = 11.15 \text{ KN/m}$
7	plaster	$0.03 \times 22 \times 1.56 = 1.03 \text{ KN/m}$
8	partions	$1 \times 0.8 = 0.8 \text{ KN/m}$
		Sum = 14.95 KN/m

From Rib1

The maximum support reaction from Dead Loads for R1 upon B,G59 is 24.73 KN,

The distributed Dead Load from the R1 on B,G59.

$$DL = (24.73 / 0.55) = 44.96 \text{ KN / m}$$

$$\text{Self weight of beam} = 14.95 \text{ KN / m}$$

$$DL = 44.96 + 14.95 = 59.91 \text{ KN / m}$$

From Rib2

The maximum support reaction from Dead Loads for R2 upon B,G59 is 23.65KN,

The distributed Dead Load from the R2 on B,G59

$$DL = (23.65 / 0.55) = 43 \text{ KN / m}$$

$$\text{Self weight of beam} = 14.95 \text{ KN / m}$$

$$DL = 43 + 14.95 = 57.95 \text{ KN/m}$$

Live Load calculations for Beam (B.G59):-

From Rib1

The maximum support reaction from Live Loads for R1 upon B,G59 is 15.41KN The distributed Live Load from the Rib 1 on B,G59.

$$LL = 15.4 / 0.55 = 28.01 \text{ KN/m.}$$

$$\text{Nominal Total live load} = 5 * 0.8 = 4 \text{ KN/m}$$

$$\text{Total LL} = 28.01 + 4 = 32.01 \text{ KN/m}$$

from Rib2

The maximum support reaction from Live Loads for R2 upon B,G59 is 15.12Kn The distributed Live Load from the Rib2 on B,G59.

$$LL = 15.12 / 0.55 = 27.5 \text{ KN/m.}$$

$$\text{Nominal Total live load} = 5 * 0.8 = 4 \text{ KN/m}$$

$$\text{Total LL} = 27.5 + 4 = 31.5 \text{ KN/m}$$

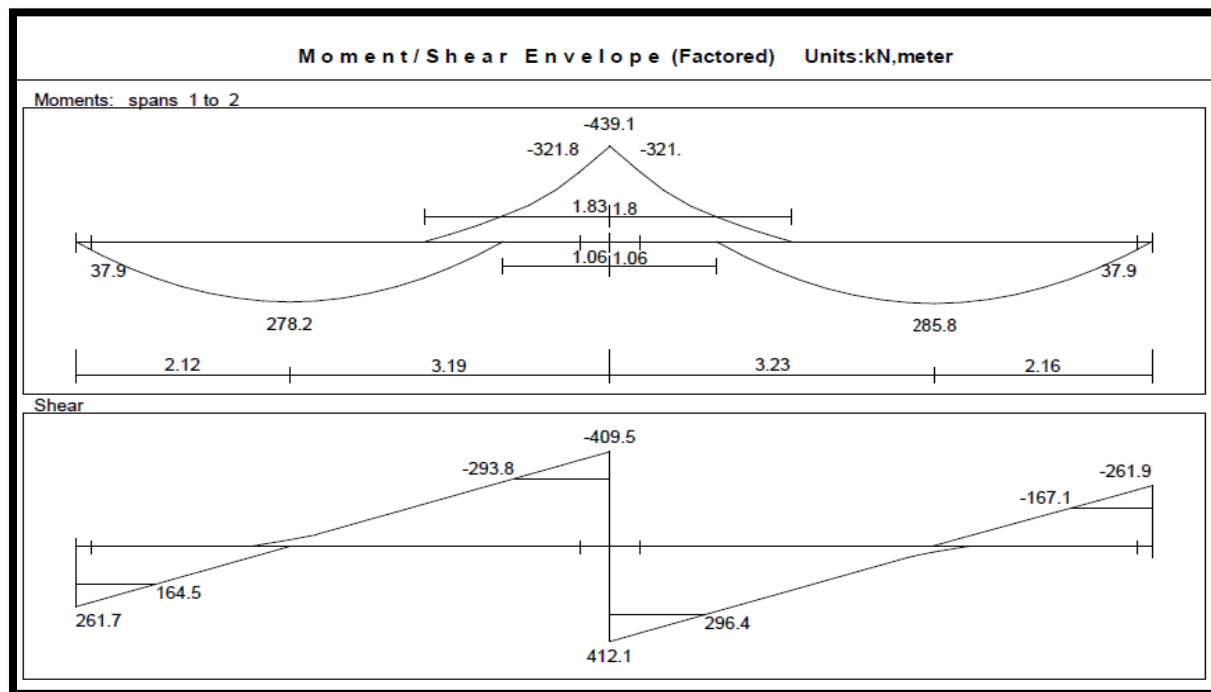


Fig 4.7: Shear and Moment Envelope Diagram of Beam (B,G59).

✓ Moment Design for (B,G59):-

Flexural Design of Positive Moment for(B,G59):-($M_u=285.8\text{KN.m}$)

Determine of $M_{n,\max}$

$$d = 700 - 40 - 10 - 20/2 = 640 \text{ mm}$$

$$x = \frac{3}{7}d = \frac{3}{7} * 640 = 274.28 \text{ mm}$$

$$a = \beta_1 x = 0.85 * 274.28 = 233.14 \text{ mm}$$

$$M_{n,\max} = 0.85 * f'_c * a * b * (d - \frac{a}{2}) = 0.85 * 24 * 233.14 * 800 * (640 - 233.14/2) * 10^{-6} = 1991.57 \text{ KN.m}$$

$$\phi M_{n,\max} = 0.82 * 1991.57 = 1633.08 \text{ KN.m} > 285.8 \text{ KN.m}$$

Design as singly reinforcement

$$R_n = \frac{M_u}{\phi b d^2} = \frac{285.8 \times 10^6}{0.9 \times 800 \times 640^2} = 0.969 \text{ Mpa}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 R_n}{f_y}} \right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 \times 0.969}{420}} \right) = 0.00236$$

$$A_s = \rho * b * d = 0.00236 \times 800 \times 640 = 1208.32 \text{ mm}^2$$

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

$$A_{s,\min} = \frac{\sqrt{f'_c}}{4(f_y)} (b_w)(d) = \frac{\sqrt{24}}{4 * 420} * 500 * 640 = 933.14 \text{ mm}^2$$

$$A_{s,\min} = \frac{1.4}{(f_y)} (b_w)(d) = \frac{1.4}{420} * 500 * 640 = 1066.67 \text{ mm}^2 \text{ Controls}$$

$$A_s = 1208.32 \text{ mm}^2$$

Use 4ø 20 Bottom, $A_{s,\text{provided}} = 1256 \text{ mm}^2 > A_{s,\text{required}} = 1208.32 \text{ mm}^2 \dots \text{Ok}$

Check spacing :-

$$S = \frac{800 - 40 \times 2 - 20 - (4 \times 20)}{4} = 155 \text{ mm} > d_b = 20 > 25 \text{ mm} \quad \text{OK}$$

Check for strain:-

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{1256 \times 420}{0.85 \times 800 \times 24} = 32.32 \text{ mm}$$

$$x = \frac{a}{\beta_1} = \frac{32.32}{0.85} = 38.02 \text{ mm}$$

$$\varepsilon_s = 0.003 \left(\frac{d - x}{x} \right) = 0.003 \left(\frac{640 - 38.02}{38.02} \right) = 0.0475 > 0.005 \quad \text{OK}$$

Flexural Design of Positive Moment for(B,G59):-($M_u=278.2\text{KN.m}$)

$$R_n = \frac{M_u}{\phi b d^2} = \frac{278.2 \times 10^6}{0.9 \times 800 \times 640^2} = 0.943 \text{ Mpa.}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{420}} \right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.943}{420}} \right) = 0.00229$$

$$A_s = \rho \cdot b \cdot d = 0.00229 \times 800 \times 640 = 1172.48 \text{ mm}^2.$$

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

$$A_{s\min} = \frac{\sqrt{f'_c}}{4(f_y)} (b_w)(d) = \frac{\sqrt{24}}{4 \times 420} * 500 * 640 = 933.14 \text{ mm}^2$$

$$A_{s\min} = \frac{1.4}{(f_y)} (b_w)(d) = \frac{1.4}{420} * 500 * 640 = 1066.7 \text{ mm}^2 \text{ Controls}$$

$$A_s = 1172.48 \text{ mm}^2$$

Use 4φ20Bottom, $A_{s,provided}=1256 \text{ mm}^2 > A_{s,required}=1172.48 \text{ mm}^2 \dots \text{Ok}$

Check spacing :-

$$S = \frac{700 - 40 \times 2 - 20 - (4 \times 20)}{4} = 155 \text{ mm} > d_b = 20 > 25 \quad \text{OK}$$

Check for strain:-

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{1256 \times 420}{0.85 \times 800 \times 24} = 32.32 \text{ mm}$$

$$x = \frac{a}{\beta_1} = \frac{32.32}{0.85} = 38 \text{ mm}$$

$$\varepsilon_s = 0.003 \left(\frac{d - x}{x} \right) = 0.003 \left(\frac{640 - 38}{38} \right) = 0.0475 > 0.005 \quad \text{Ok}$$

Flexural Design of Negative Moment for(B,G59):-($M_u=321.8 \text{ KN.m}$)

$$R_n = \frac{M_u}{\phi b d^2} = \frac{321.8 \times 10^6}{0.9 \times 800 \times 640^2} = 1.09 \text{ Mpa}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 m R_n}{420}} \right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 1.09}{420}} \right) = 0.00266$$

$$A_s = \rho \cdot b \cdot d = 0.00266 \times 800 \times 640 = 1361.92 \text{ mm}^2$$

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

$$A_{s,min} = \frac{\sqrt{f'_c}}{4(f_y)} (b_w)(d) = \frac{\sqrt{24}}{4 \times 420} * 500 * 640 = 933.14 \text{ mm}^2$$

$$A_{s,min} = \frac{1.4}{(f_y)} (b_w)(d) = \frac{1.4}{420} * 500 * 640 = 1066.7 \text{ mm}^2 \text{Controls}$$

$$A_s = 1361.92 \text{ mm}^2 \text{Controls}$$

Use 5ø 20 , $A_{s,provided} = 1570 \text{ mm}^2 > A_{s,required} = 1361.92 \dots$ Ok

Check spacing :-

$$S = \frac{800 - 40 \times 2 - 20 - (5 \times 20)}{4} = 150 \text{ mm} > d_b = 20 > 25 \quad \text{OK}$$

Check for strain:-

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{1570 \times 420}{0.85 \times 800 \times 24} = 40.4 \text{ mm}$$

$$x = \frac{a}{\beta_1} = \frac{40.4}{0.85} = 47.53 \text{ mm}$$

$$\varepsilon_s = 0.003 \left(\frac{d - x}{x} \right) = 0.003 \left(\frac{640 - 47.53}{47.53} \right) = 0.0374 > 0.005 \quad \text{Ok}$$

✓ Shear Design for (B ,G59):-

1. Case 3 :-

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

Use stirrups (2 leg stirrups) ø10/ 250 mm , $A_v = 2 \times 79 = 157 \text{ mm}^2$

$$\mathbf{V_u = 296.4 \text{ KN}}$$

$$V_c = \frac{1}{6} \sqrt{f'_c} b_w d = \frac{1}{6} \sqrt{24} * 500 * 640 = 261.28 \text{ KN}$$

$$\Phi V_c = 0.75 * 261.28 = 195.96 \text{ KN}$$

$$\Phi V_{smin} \geq 0.75 \left(\frac{1}{3} \right) * b_w * d = 0.75 * \left(\frac{1}{3} \right) * 500 * 640 * 10^{-3} = 80 \text{ KN Controls}$$

$$\Phi V_{smin} \geq 0.75 \left(\frac{\sqrt{f_c'}}{16} \right) * b_w * d = 0.75 * \left(\frac{\sqrt{24}}{16} \right) * 500 * 640 * 10^{-3} = 73.48 \text{ KN}$$

$$\Phi V_c < V_u \leq \Phi V_c + \Phi V_{smin}$$

$$195.96 < 296.4 \leq 275.96 \dots \text{not satisfied}$$

Cases 1&2&3 is not suitable

Case 4 :-

$$v_{s'} = \frac{1}{3} \sqrt{f_c'} b_w d = \frac{1}{3} \sqrt{24} * 500 * 640 = 522.55 \text{ KN}$$

$$\Phi(v_c + v_{s,min}) < v_u \leq \Phi(v_c + v_{s'})$$

$$0.75(261.28 + 106.67) < 296.4 < 0.75(261.28 + 522.55)$$

$$275.9 < 296.4 < 587.87$$

shear reinforcement are required

Use 2 leg Φ 10

$$A_s = 157 \text{ mm}^2$$

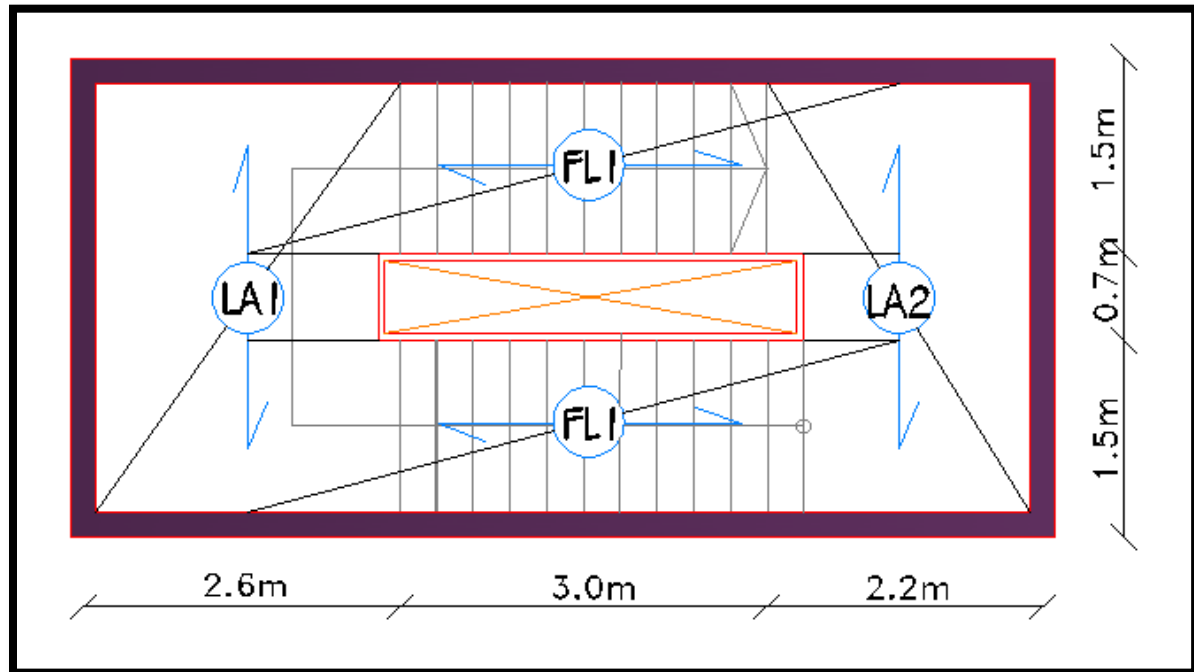
$$V_s = V_u - V_c = \frac{296.4}{0.75} - 261.28 = 133.92 \text{ KN}$$

$$S = \frac{A_v f_{yt} d}{v_s} = \frac{157 * 420 * 640}{133.92 * 1000} = 315.13 \text{ mm}$$

$$s_{max} \leq \frac{d}{2} = \frac{640}{2} = 320 \text{ mm} \quad \text{control}$$

$$\text{or} \quad s_{max} \leq 600 \text{ mm}$$

Use 2 leg Φ 10 @250mm

4-8 Design of Stair (Stair#4)**Fig 4.8: Stair Plan.****✓ Material :-**

⇒ concrete B300 $F_c' = 24 \text{ N/mm}^2$

⇒ Reinforcement Steel $F_y = 420 \text{ N/mm}^2$

1- Design of Flight :-**✓ Determination of Thickness:-**

$$h_{\min} = L/20$$

$$h_{\min} = 3.0/20 = 15 \text{ cm}$$

Take $h = 20 \text{ cm}$

The Stair Slope by $\theta = \tan^{-1}(15 / 30) = 26.5^\circ$

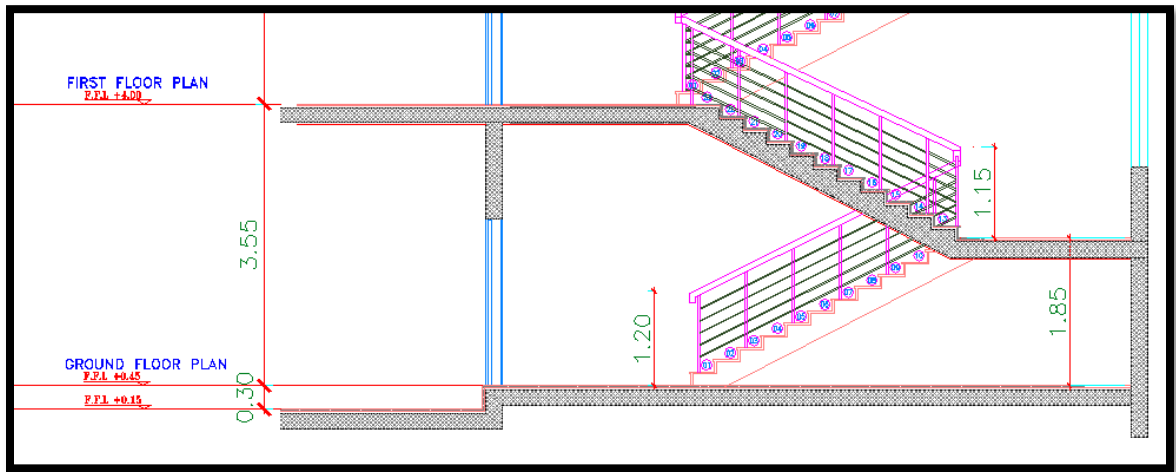


Fig 4.9 : Stair Section.

Dead Load For Flight For 1m Strip:-**Table (4.6): Dead Load Calculation of Flight.**

No.	Parts of Flight	Calculation
1	Tiles	$23 \times 0.03 \times 1 \times ((0.3 + 0.2) / 0.3) = 1.15 \text{ KN/m}$
2	Mortar	$22 \times 0.03 \times 1 \times ((0.3 + 0.2) / 0.3) = 1.1 \text{ KN/m}$
3	Stair	$25 \times 0.5 \times 0.2 \times 1 = 2.5 \text{ KN/m}$
4	R.C	$25 \times 0.2 \times 1 / \cos 26.5^\circ = 5.59 \text{ KN/m}$
5	Plaster	$22 \times 0.02 \times 1 / \cos 26.5^\circ = 0.49 \text{ KN/m}$
		Sum = 10.83 KN/m

Live Load For Landing For 1m Strip = $5 \times 1 = 5 \text{ KN/m}$

✓ **System of Flight:-**

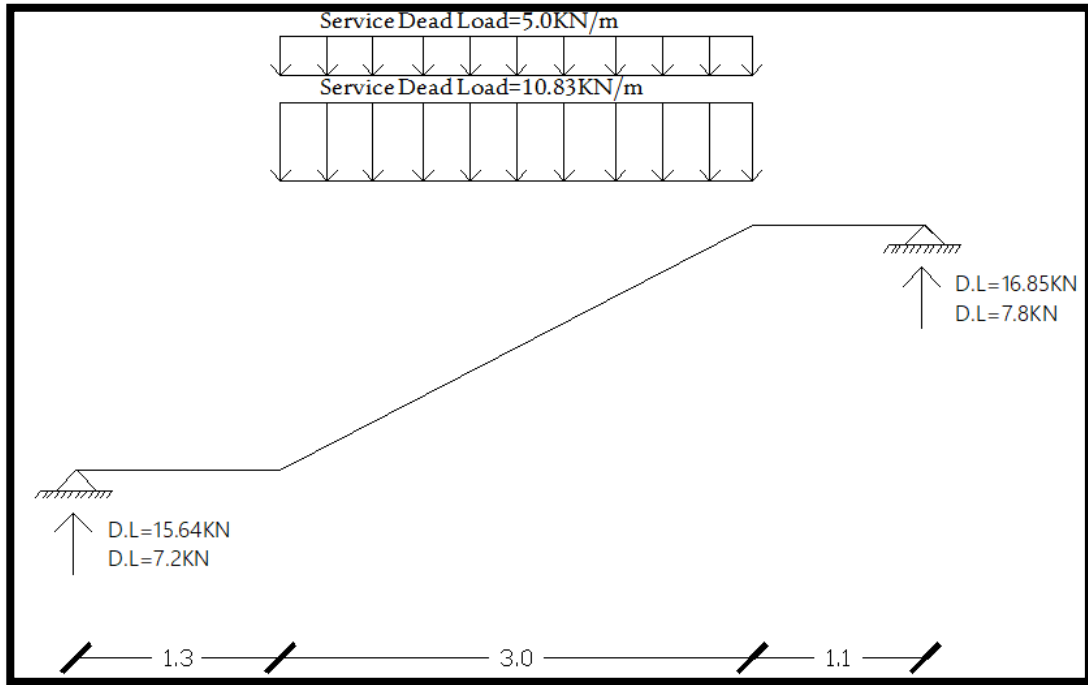


Fig 4.10: Statically System and Loads Distribution of Flight.

Factored Load For Flight :-

$$W_U = 1.2 \times 10.83 + 1.6 \times 5 = 21 \text{ kN/m}$$

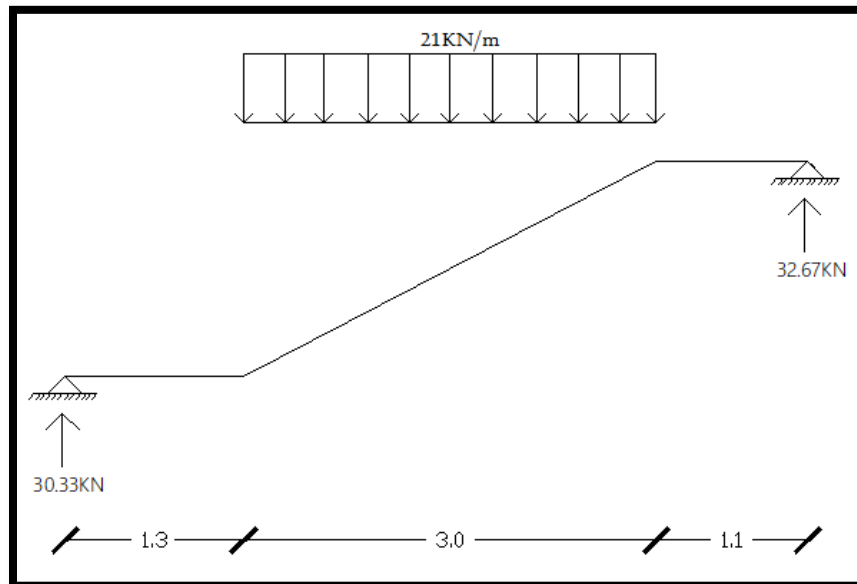


Fig 4.11: Statically System and Loads Distribution of Flight.

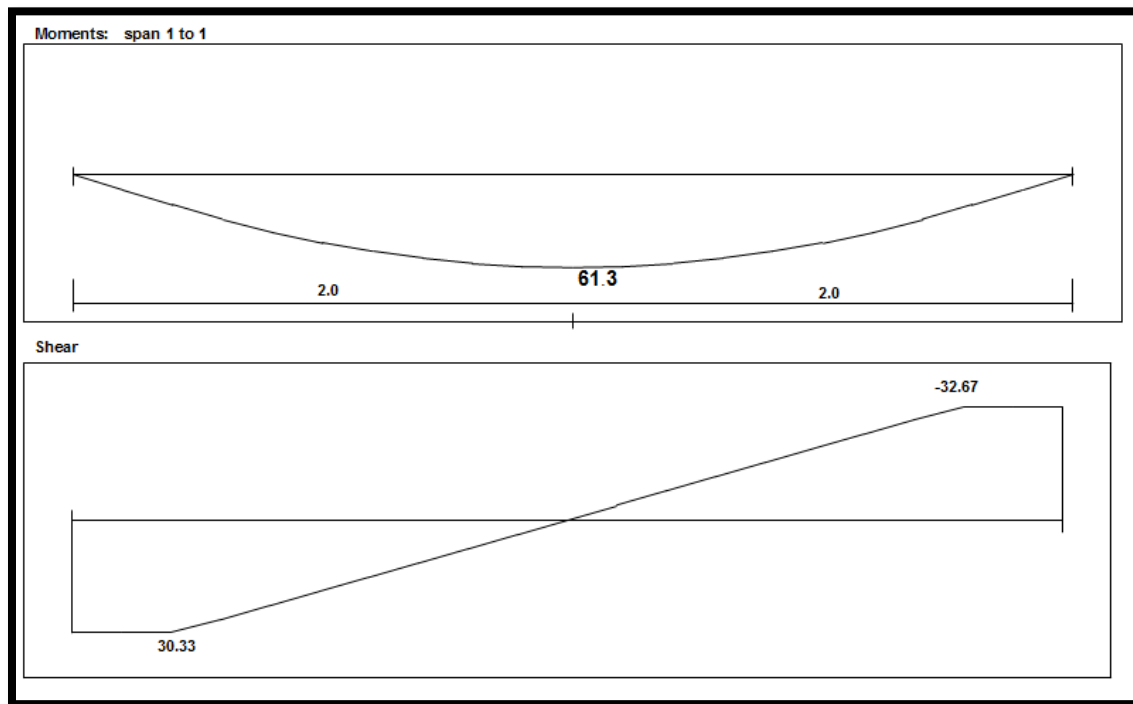


Fig 4.12: Shear and Moment Envelope Diagram of Flight.

✓ **Design of Shear for Flight :- ($V_u=32.67$ KN)**

Assume bar diameter ϕ 14 for main reinforcement

$$d = h - \text{cover} - \frac{d_b}{2} = 200 - 20 - \frac{14}{2} = 173 \text{ mm}$$

$$V_c = \frac{1}{6} \sqrt{f'c} b_w d = \frac{1}{6} \sqrt{24} * 1000 * 173 = 141.25 \text{ Kn}$$

$\Phi V_c = 0.75 * 141.25 = 105.94 \text{ KN} > V_u = 32.67 \text{ KN} \dots\dots \text{No shear reinforcement are required}$

✓ **Design of Bending Moment for Flight :- ($M_u=61.3$ KN.m)**

$$R_n = \frac{M_u}{\phi b d^2} = \frac{61.3 \times 10^6}{0.9 \times 1000 \times 173^2} = 2.27 \text{ Mpa}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{420}} \right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 2.27}{420}} \right) = 0.00574$$

$$A_{s, \text{req}} = \rho \cdot b \cdot d = 0.00574 \times 1000 \times 173 = 993.02 \text{ mm}^2/\text{m}$$

$$A_{s, \text{min}} = 0.0018 \times 1000 \times 200 = 360 \text{ mm}^2/\text{m}$$

$$A_{s, \text{req}} = 993.02 \text{ mm}^2 > A_{s, \text{min}} = 360 \text{ mm}^2/\text{m}$$

Check for Spacing :-

$$S = 3h = 3 \times 200 = 600 \text{ mm}$$

$$S = 380 \times \left(\frac{\frac{280}{\frac{2}{3} \times 420}}{3} \right) - 2.5 \times 20 = 330$$

$$S = 450 \text{ mm}$$

$$S = 330 \text{ mm} \dots\dots\dots \text{is control}$$

Use $\phi 12$ @ 100 mm , $A_{s, \text{provided}} = 1130 \text{ mm}^2 > A_{s, \text{required}} = 993.02 \text{ mm}^2 \dots \text{Ok}$

Check for strain:-

$$a = \frac{A_s \cdot f_y}{0.85 b f'_c} = \frac{1130 \times 420}{0.85 \times 1000 \times 24} = 23.26 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{23.26}{0.85} = 27.36 \text{ mm}$$

$$\varepsilon_s = 0.003 \left(\frac{d - c}{c} \right) = 0.003 \left(\frac{173 - 27.36}{27.36} \right) = 0.016 > 0.005 \dots\dots \mathbf{Ok}$$

✓ Lateral or Secondary Reinforcement For Flight :-

$$A_{s, \text{req}} = A_{s, \text{min}} = 0.0018 \times 1000 \times 220 = 360 \text{ mm}^2$$

Use $\phi 10 @ 200 \text{ mm}$, $A_{s, \text{provided}} = 395 \text{ mm}^2 > A_{s, \text{required}} = 360 \text{ mm}^2 \dots \text{Ok}$

2- Design of Middle Landing :-

✓ Determination of Thickness:-

$$h_{\min} = L/20$$

$$h_{\min} = 3.70 / 20 = 18.5 \text{ cm}$$

Take $h = 25 \text{ cm}$

✓ Load Calculation:-

Dead Load For (LA1) Landing For 1m Strip:-

Table (4.7) : Dead Load Calculation of Middle Landing.

No.	Parts of Landing	Calculation
1	Tiles	$23 * 0.03 * 1 = 0.69 \text{ KN/m}$
2	Mortar	$22 * 0.03 * 1 = 0.66 \text{ KN/m}$
4	R.C	$25 * 0.25 * 1 = 6.25 \text{ KN/m}$
5	Plaster	$22 * 0.02 * 1 = 0.44 \text{ KN/m}$
		Sum = 8.04 KN/m

Live Load For Landing = $5 * 1 = 5 \text{ KN/m}$

Factored Load For Landing :-

$$W_U = 1.2 \times 8.04 + 1.6 \times 5 = 17.65 \text{ kN/m}$$

Factored Load From Flight :-

$$W_{LA1} = \frac{W_{FL1}}{L} = \frac{30.33}{1.5} = 20.22 \text{ kN/m}$$

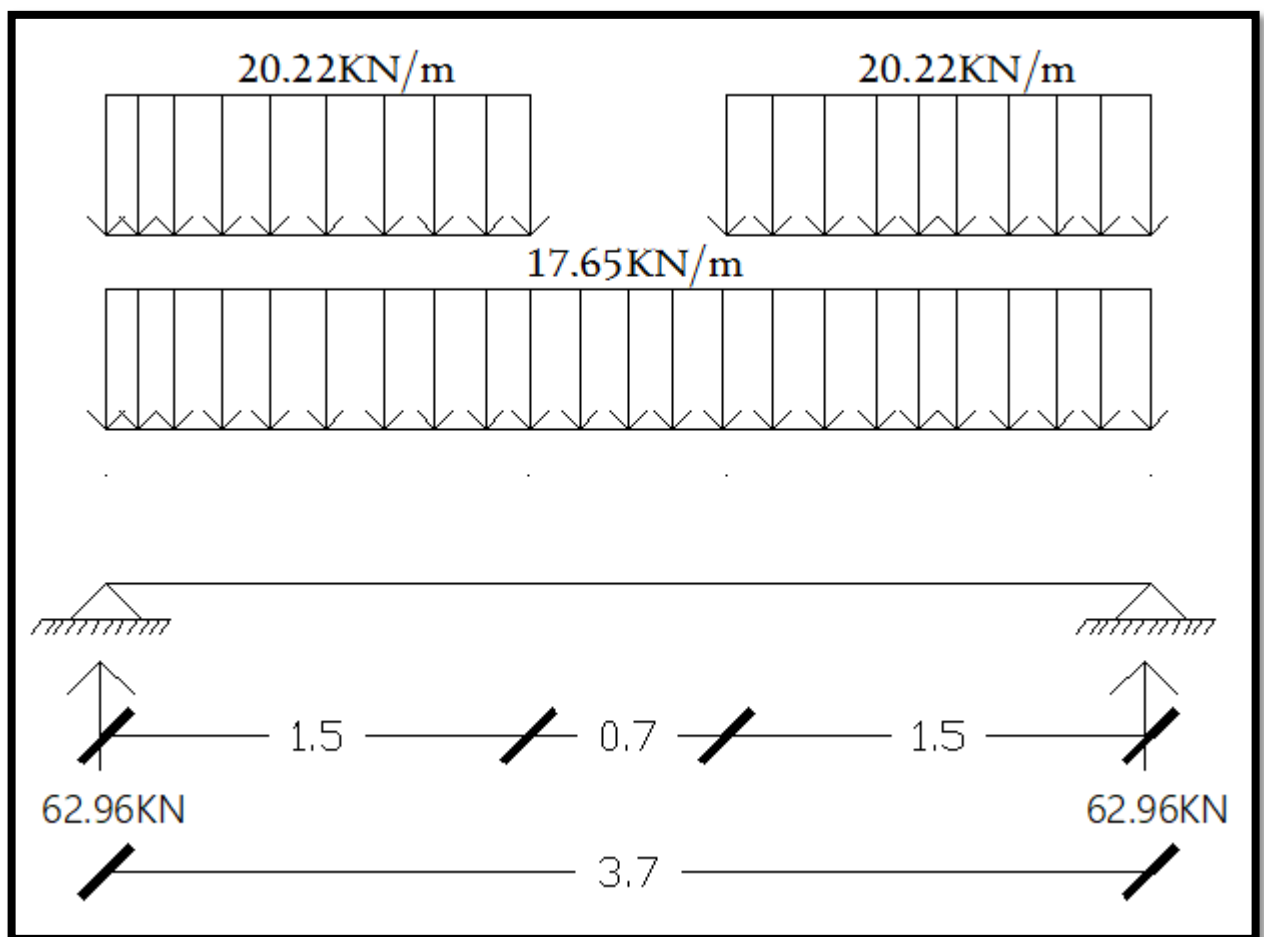
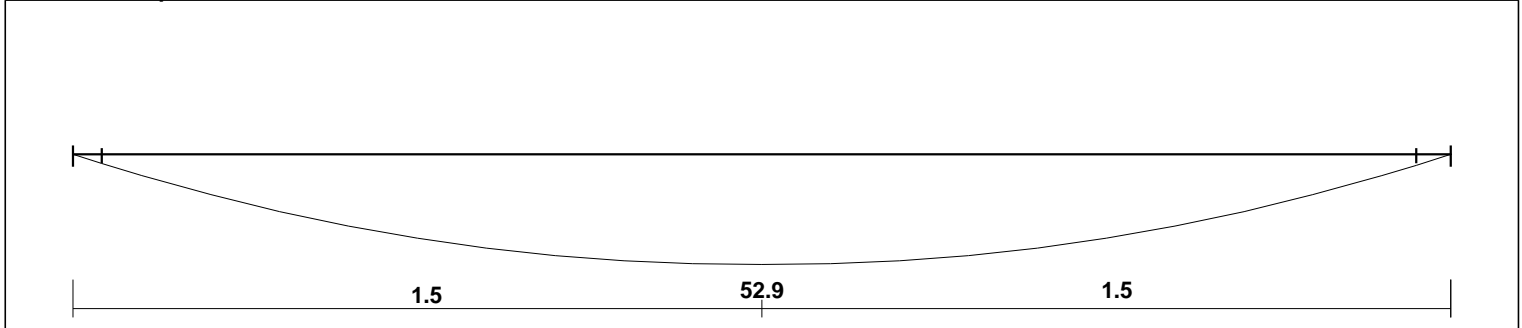
✓ System of Landing:-

Fig 4.13: Static System and Loads Distribution Of Middle Landing.

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

Moments: span 1 to 1



Shear

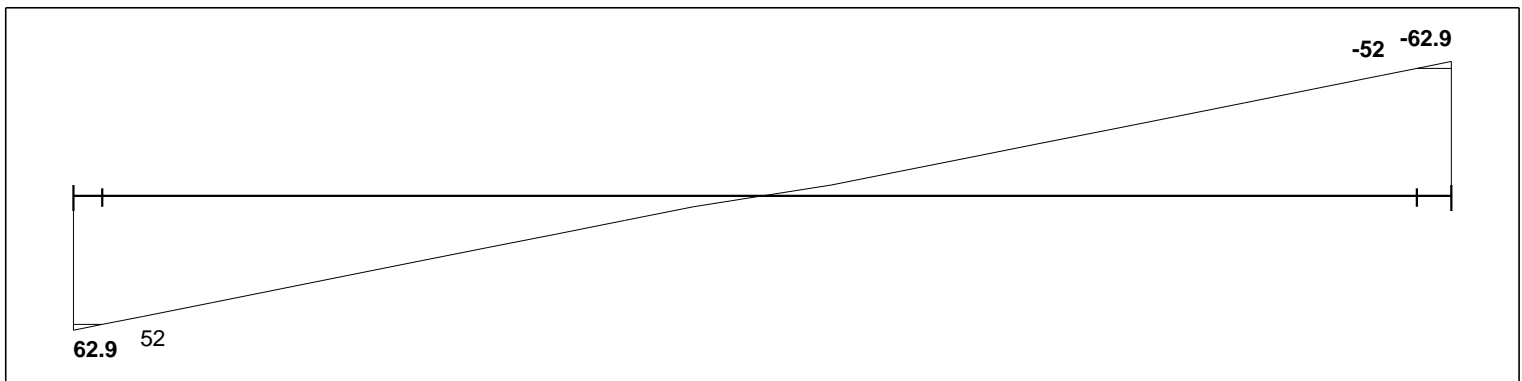


Fig 4.14: Shear and Moment Envelope Diagram of Middle Landing.

✓ Design of Shear:- ($V_u=52\text{KN}$)Assume bar diameter $\phi 14$ for main reinforcement

$$d = h - \text{cover} - \frac{d_b}{2} = 250 - 20 - \frac{14}{2} = 223 \text{ mm}$$

$$V_c = \frac{1}{6} \sqrt{f_c'} b_w d = \frac{1}{6} \sqrt{24} * 1000 * 223 = 182.1 \text{ KN}$$

$$\Phi * V_c = 0.75 * 182.1 = 136.6 \text{ KN} > V_u = 52 \text{ KN} \dots \text{No shear reinforcement are required}$$

✓ Design of Bending Moment :- (Mu=52.9KN.m)

Assume bar diameter ϕ 14 for main reinforcement

$$d = h - \text{cover} - \frac{d_b}{2} = 250 - 20 - \frac{14}{2} = 223 \text{ mm}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{52.9 \times 10^6}{0.9 \times 1000 \times 223^2} = 1.18 \text{ Mpa}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{420}} \right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 1.18}{420}} \right) = 0.00289$$

$$A_{s, \text{req}} = \rho \cdot b \cdot d = 0.00289 \times 1000 \times 223 = 644.47 \text{ mm}^2$$

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

$$A_{s, \text{req}} = 644.47 \text{ mm}^2 \dots \dots \text{is control}$$

Check for Spacing:-

$$S = 3h = 3 \times 250 = 750 \text{ mm}$$

$$S = 380 \times \left(\frac{\frac{280}{2}}{\frac{2}{3} \times 420} \right) - 2.5 \times 20 = 330$$

$$S = 450 \text{ mm}$$

$$S = 330 \text{ mm} \dots \dots \text{is control}$$

Use $\phi 12 @ 150 \text{ mm}$, $A_{s, \text{provided}} = 753 \text{ mm}^2 > A_{s, \text{required}} = 644.47 \text{ mm}^2 \dots \text{Ok}$

Check for strain:-

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{770 \times 420}{0.85 \times 1000 \times 24} = 15.85 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{15.85}{0.85} = 18.64 \text{ mm}$$

$$\epsilon_s = 0.003 \left(\frac{d - c}{c} \right) = 0.003 \left(\frac{223 - 18.64}{18.64} \right) = 0.033 > 0.005 \dots \dots \text{Ok}$$

Lateral or Secondary Reinforcement For Landing:-

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

Use $\phi 8$ @ 100 mm , $A_{s,provided} = 502.4 \text{ mm}^2 > A_{s,required} = 450 \text{ mm}^2 \dots$ Ok

3- Design of Main Landing:-

✓ Determination of Thickness:-

$$h_{min} = L/20$$

$$h_{min} = 3.20 / 20 = 16 \text{ cm}$$

Take $h = 32 \text{ cm}$

✓ Load Calculation:-

Dead Load For middle Landing For 1m Strip:-

Table (4.8): Dead Load Calculation of Main Landing.

No.	Parts of Landing	Calculation
1	Tiles	$23 * 0.03 * 1 = 0.69 \text{ KN/m}$
2	Mortar	$22 * 0.03 * 1 = 0.66 \text{ KN/m}$
4	R.C	$25 * 0.32 * 1 = 8.0 \text{ KN/m}$
5	Plaster	$22 * 0.02 * 1 = 0.44 \text{ KN/m}$
		Sum = 9.79 KN/m

Live Load For Landing = $5 \times 1 = 5 \text{ KN/m}$

Factored Load For Landing :-

$$W_U = 1.2 \times 9.79 + 1.6 \times 5 = 19.75 \text{ KN/m}$$

Factored Load From Flight :-

$$W_{LA2} = \frac{W_{FL1}}{L} = \frac{32.67}{1.5} = 21.78 \text{ KN/m}$$

✓ **System of Landing:-**

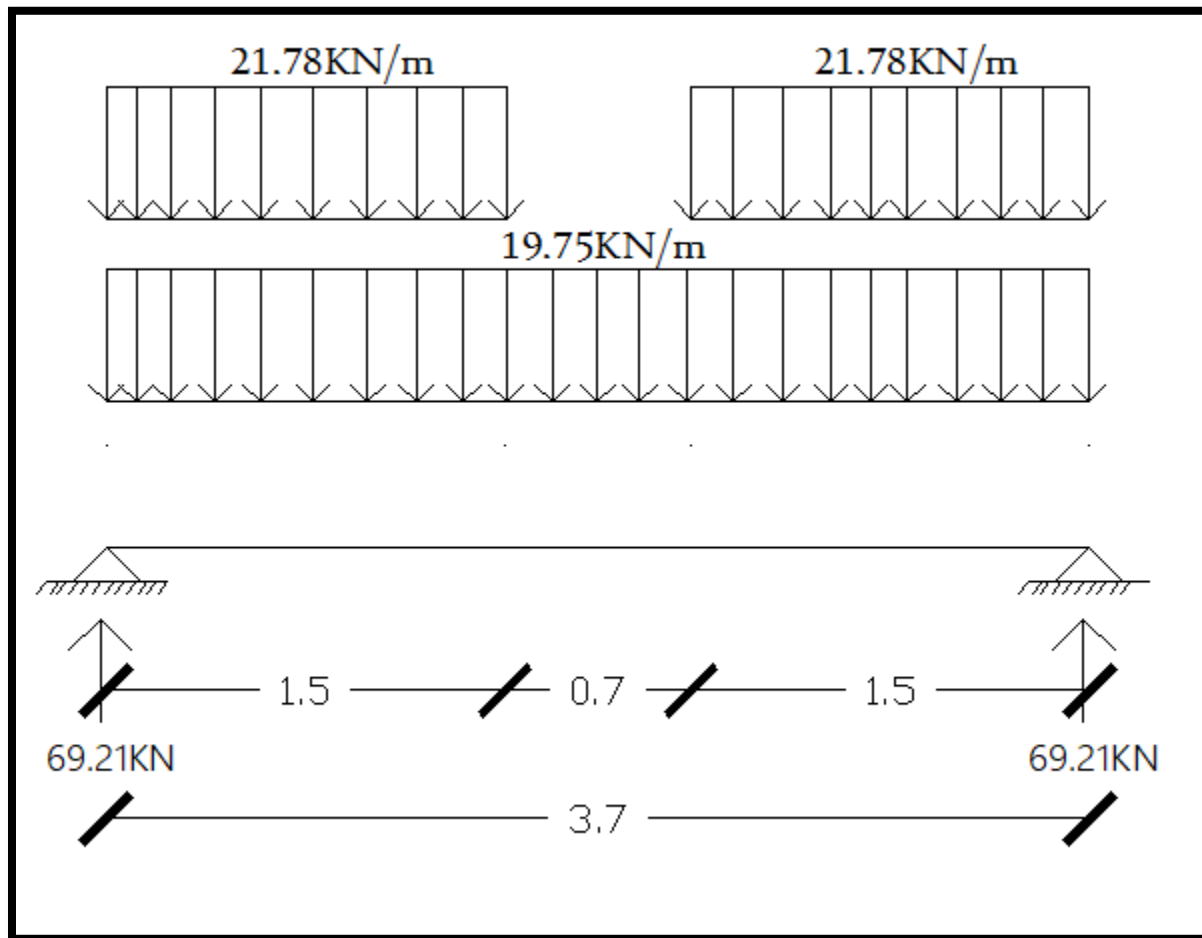
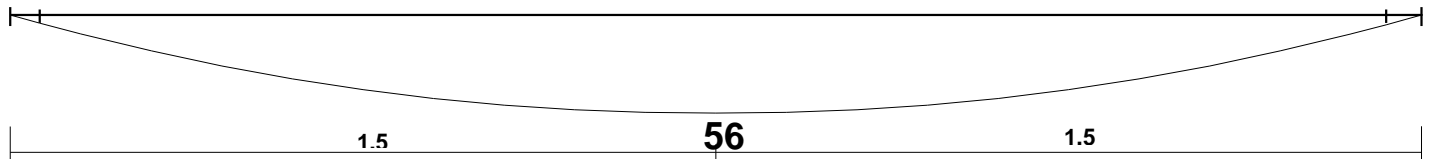


Fig 4.15 : Statically System and Loads Distribution of Main Landing.

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

Moments: span 1 to 1



Shear

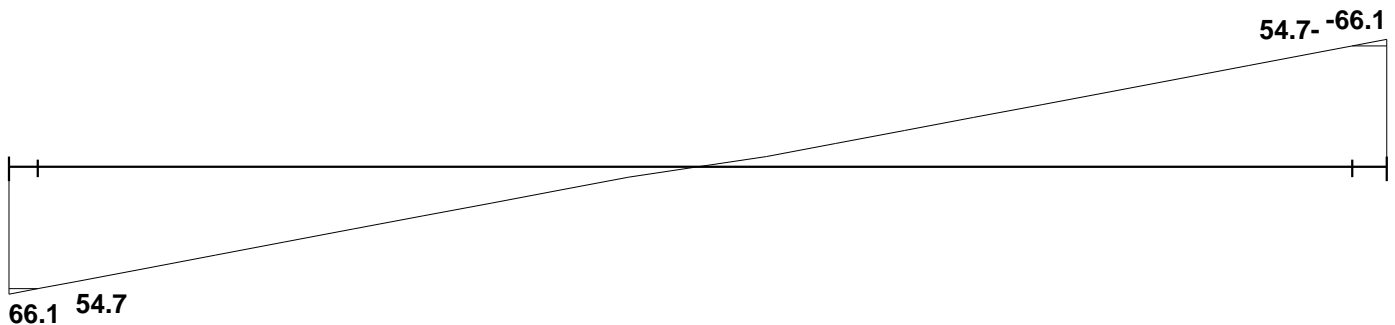


Fig 4.16 : Shear and Moment Envelope Diagram of Main Landing.

✓ Design of Shear:- ($V_u = 54.7$ KN)

Assume bar diameter $\phi 14$ for main reinforcement

$$d = h - \text{cover} - \frac{d_b}{2} = 320 - 20 - \frac{14}{2} = 293 \text{ mm}$$

$$V_c = \frac{1}{6} \sqrt{f_c'} b_w d = \frac{1}{6} \sqrt{24} * 1000 * 293 = 293.2 \text{ KN}$$

$$\Phi * V_c = 0.75 * 293.2 = 219.9 \text{ KN} > V_u = 54.7 \text{ KN} \dots \text{No shear reinforcement are required}$$

✓ Design of Bending Moment :- (Mu=56KN.m)

Assume bar diameter ϕ 14 for main reinforcement

$$d = h - \text{cover} - \frac{d_b}{2} = 320 - 20 - \frac{14}{2} = 293 \text{ mm}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{56 \times 10^6}{0.9 \times 1000 \times 293^2} = 0.72 \text{ Mpa}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2mR_n}{420}} \right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.72}{420}} \right) = 0.00174$$

$$A_{s, \text{req}} = \rho \cdot b \cdot d = 0.00174 \times 1000 \times 293 = 509.82 \text{ mm}^2$$

$$A_{s, \text{min}} = 0.0018 \times 1000 \times 320 = 576 \text{ mm}^2$$

$$A_{s, \text{req}} = 509.82 \text{ mm}^2 < A_{s, \text{min}} = 576 \text{ mm}^2 \dots \dots \text{is control}$$

$$A_{s, \text{min}} = 576 \text{ mm}^2 \dots \dots \text{is control}$$

Check for Spacing:-

$$S = 3h = 3 \times 320 = 960 \text{ mm}$$

$$S = 380 \times \left(\frac{280}{\frac{2}{3} \times 420} \right) - 2.5 \times 20 = 330$$

$$S = 450 \text{ mm}$$

$$S = 330 \text{ mm} \dots \dots \text{is control}$$

Use $\phi 12 @ 15 \text{ mm}$, $A_{s, \text{provided}} = 753 \text{ mm}^2 > A_{s, \text{required}} = 576 \text{ mm}^2 \dots \text{Ok}$

Check for strain:-

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{753 \times 420}{0.85 \times 1000 \times 24} = 15.5 \text{ mm}$$

$$c = \frac{a}{B_1} = \frac{21.14}{0.85} = 18.23 \text{ mm}$$

$$\varepsilon_s = 0.003 \left(\frac{d - c}{c} \right) = 0.003 \left(\frac{293 - 18.23}{18.23} \right) = 0.045 > 0.005 \dots \dots \text{Ok}$$

✓ Lateral or Secondary Reinforcement For Landing:-

$$A_{s,req} = A_{s,min} = 0.0018 * 1000 * 320 = 576 \text{ mm}^2$$

Use $\phi 12$ @ 150 mm , $A_{s,provided} = 753 \text{ mm}^2 > A_{s,required} = 576 \text{ mm}^2 \dots \text{Ok}$

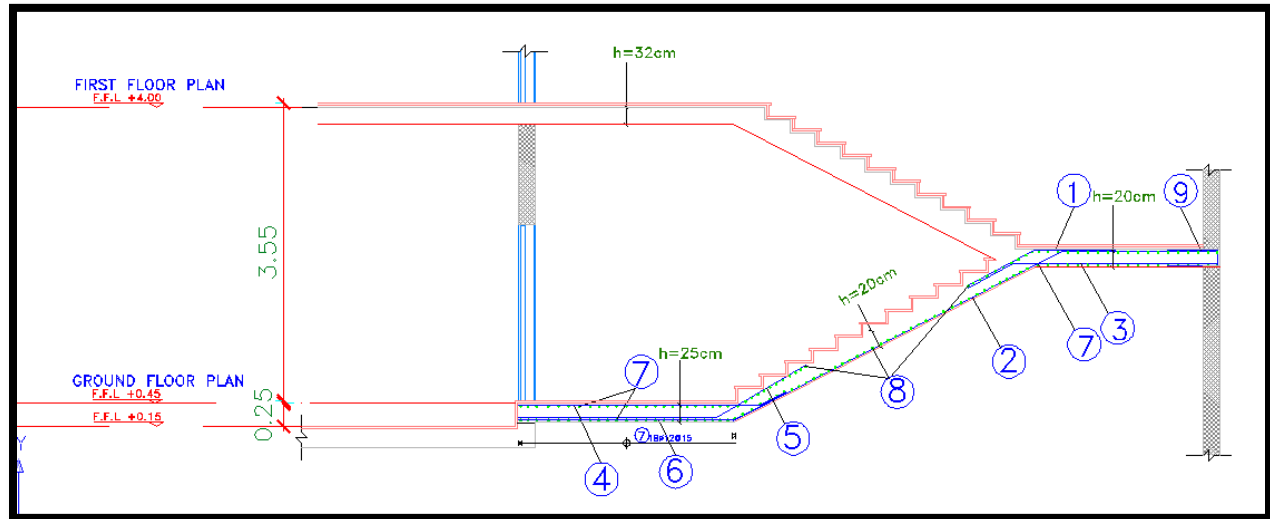


Fig 4.17: Stair Reinforcement Details.

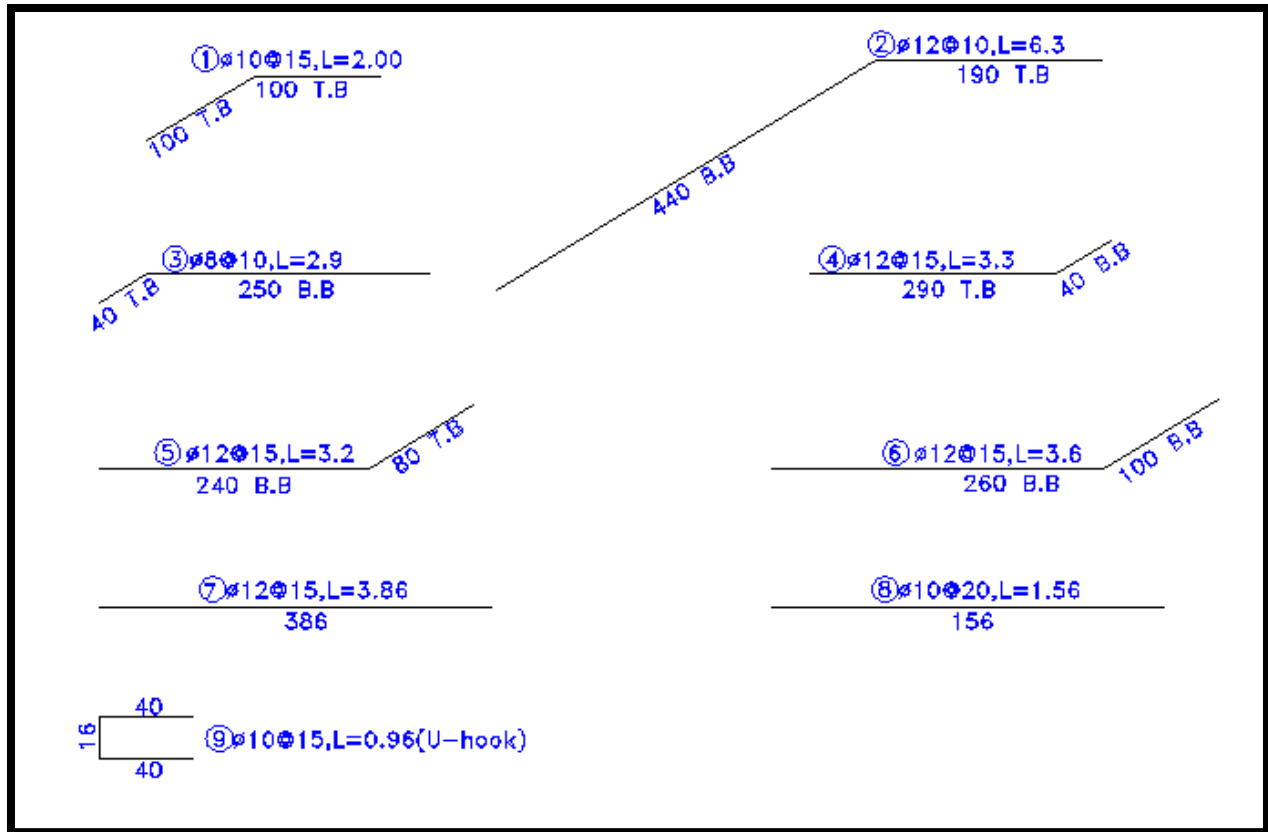


Fig 4.18: Stair Reinforcement Details.

4-9 Design of Column (C,151)

✓ Material :-

⇒ concrete B350 $F_c' = 28 \text{ N/mm}^2$

⇒ Reinforcement Steel $F_y = 420 \text{ N/mm}^2$

✓ Load Calculation:-

Service Load:-

Dead Load = 900 kN

Live Load = 450 kN

Factored Load:-

$$P_U = 1.2 \times 900 + 1.6 \times 450 = 1800 \text{ kN}$$

✓ Dimensions of Column:-

Assume $\rho_g = 0.01$

$$\phi * P_n = 0.65 \times 0.8 \times A_g \{ 0.85 f_c' (1 - \rho_g) + \rho_g * F_y \}$$

$$2080 = 0.65 \times 0.8 \times A_g \{ 0.85 * 28 (1 - 0.01) + 0.01 * 420 \}$$

$$A_g = 124686.2 \text{ mm}^2$$

Assume Rectangular Section

$$h = 300 \text{ mm}$$

$$b = 124686.2 / 300 = 415.6 \text{ mm}$$

Select $b = 600 \text{ mm}$

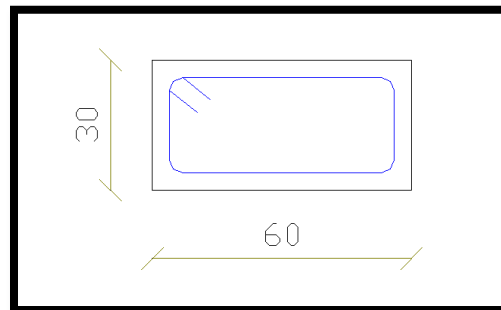


Fig 4.19 : Column section

✓ **Check Slenderness Parameter:-**

$$\frac{klu}{r} < 34 - 12 \frac{M1}{M2} \leq 40$$

Lu: Actual unsupported (Unbraced) length.

K: effective length factor. According to ACI 318-2002 (10.10.6.3) The effective length factor k, shall be permitted to be taken as 1.0.

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

$$Lu = 3.55 - 0.7 = 2.85 \text{ m}$$

$$M1/M2 = 1$$

K=1 for braced frame.

- **about Y-axis (b= 0.60 m)**

$$\frac{klu}{r} < 34 - 12 \frac{M1}{M2} \leq 40$$

- $\frac{1 \times 2.85}{0.3 \times 0.60} = 15.83 < 22$

Column Is Short About Y-axis

- **about X-axis (h= 0.30m)**

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

$$\frac{1 \times 2.85}{0.3 \times 0.30} = 31.67 > 22$$

Column Is Long About X-axis

✓ **Minimum Eccentricity:-**

$$e_y = \frac{M_{ux}}{P_u} = 0$$

$$\min e_y = 15 + 0.03 \times h = 15 + 0.03 \times 300 = 24 \text{ mm} = 0.024 \text{ m}$$

$$e_y = 0.024 \text{ m}$$

✓ **Magnification Factor:-**

$$\delta_{ns} = \frac{C_m}{1 - \frac{P_u}{0.75 P_c}} \geq 1.0 \text{ and } \leq 1.4$$

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

$$C_m = 0.6 + 0.4 * 1 = 1 \geq 0.4$$

$$P_{cr} = \frac{\pi^2 EI}{(KL_u)^2}$$

$$EI = 0.4 \frac{E_c I_g}{1 + \beta_d}$$

$$E_c = 4700 \sqrt{f_c'} = 4700 \times \sqrt{28} = 24870.6 \text{ Mpa}$$

$$\beta_d = \frac{1.2 DL}{P_u} = \frac{1.2 * (900)}{1800} = 0.6 < 1$$

$$I_g = \frac{b \times h^3}{12} = \frac{0.60 \times 0.30^3}{12} = 0.00135 \text{ m}^4$$

$$EI = \frac{0.4 \times 24870 \times 0.00135}{1 + 0.6} = 8.39 \text{ MN.m}^2$$

$$P_{cr} = \frac{\pi^2 * 8.39}{(1 * 2.85)^2} = 10.19 \text{ MN}$$

$$\delta_{ns} = \frac{1}{1 - \frac{1800}{0.75 \times 10190}} = 1.308 \geq 1.0 \text{ and } \leq 1.4$$

✓ Interaction Diagram:-

$$e_y = e_{\min} \times \delta_{ns} = 0.024 \times 1.308 = 0.0314m$$

$$\frac{e_y}{h} = \frac{0.0314}{0.6} = 0.052$$

$$\frac{\gamma}{h} = \frac{300 - 2 \times 40 - 2 \times 10 - 16}{350} = 0.613$$

From the interaction diagram chart

from chart A9- a for $\frac{\gamma}{h} = 0.6 \rightarrow \rho_g = 0.01$

from chart A9- b for $\frac{\gamma}{h} = 0.75 \rightarrow \rho_g = 0.01$

then for $\frac{\gamma}{h} = 0.613 \rightarrow \rho_g = 0.01$

Select reinforcement

$$A_{st} = \rho_g \times A_g = 0.01 \times 300 \times 600 = 1800mm^2$$

Select 10 $\phi 16$ with $A_s = 2010mm^2 > A_{st} = 1800mm^2$.

✓ **Design of the Stirrups:-**

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

$$spacing \leq 16 \times d_b = 16 \times 2.0 = 25.6 \text{ cm}$$

$$spacing \leq 48 \times d_s = 48 \times 1.0 = 48 \text{ cm}$$

$$spacing \leq 40 \text{ cm}$$

Use $\phi 10 @ 20 \text{ cm}$

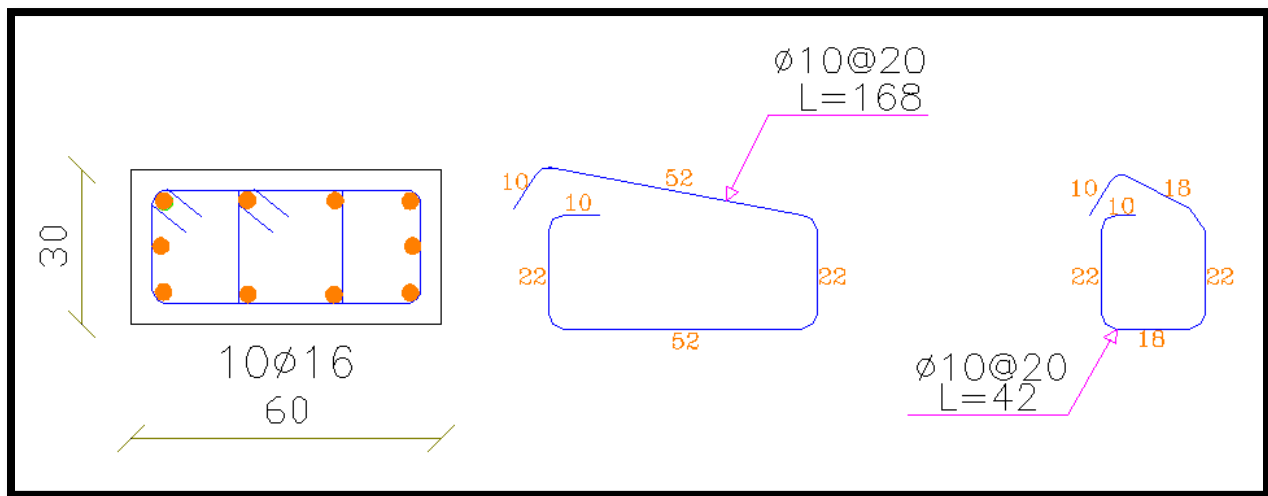


Fig 4.20: Column Reinforcement Details.

4.10 Design of Shear Wall (SW,16)

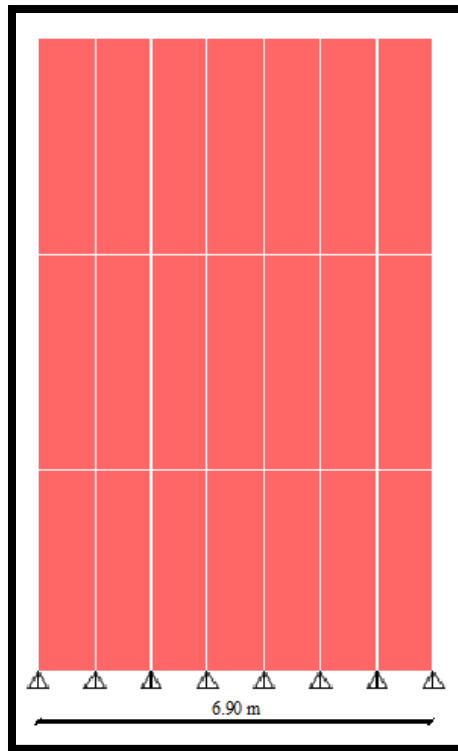


Fig 4.21:Shear Wall.

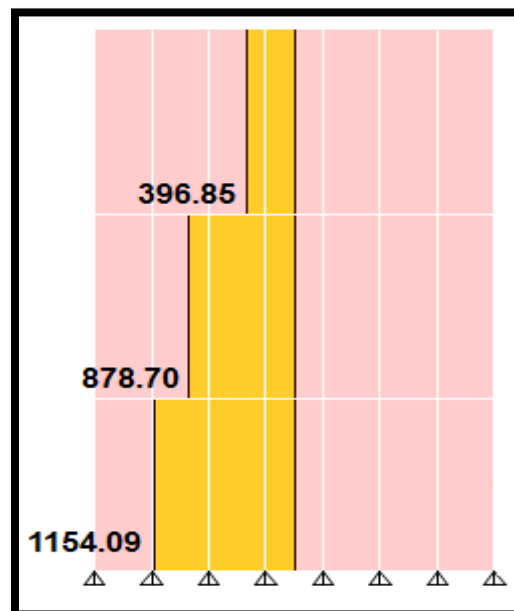


Fig 4.22: Shear Diagram of Shear Wall.

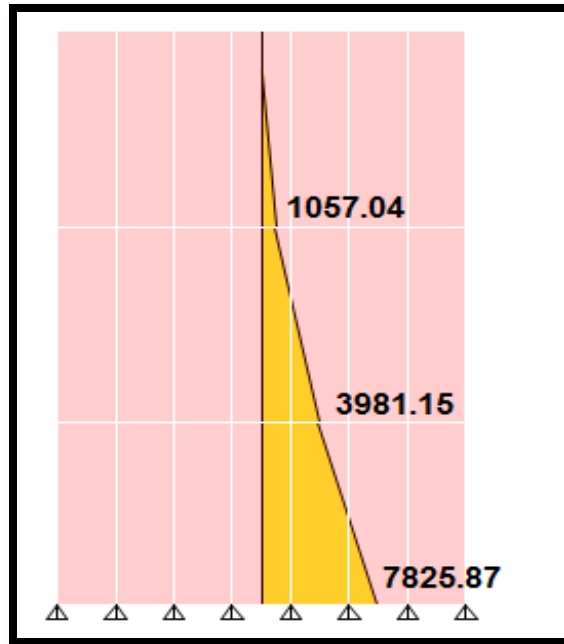


Fig 4.23: Moment Diagram of Shear Wall.

✓ **Material and Sections:- (From Shear Wall 16)**

- ⇒ concrete B300 $F_c' = 24 \text{ N/mm}^2$
- ⇒ Reinforcement Steel $F_y = 420 \text{ N/mm}^2$
- ⇒ Shear Wall Thickness $h = 20 \text{ cm}$
- ⇒ Shear Wall Width $L_w = 6.9 \text{ m}$
- ⇒ Shear Wall Height $H_w = 3.55 \text{ m}$

✓ Design of Horizontal Reinforcement:-

$$\sum Fx = Vu = 1154.09 \text{ KN}$$

The critical Section is the smaller of:

$$\frac{l_w}{2} = \frac{6.9}{2} = 3.45m \dots \text{Control}$$

$$\frac{h_w}{2} = \frac{11.15}{2} = 5.58m$$

$$\text{storyheight}(H_w) = 3.55m.$$

$$d = 0.8 \times L_w = 0.8 \times 6.9 = 5.52m$$

$$\begin{aligned} \phi V_{nmax} &= \phi \frac{5}{6} \sqrt{f_c'} h d \\ &= 0.75 * 0.833 * \sqrt{24} * 200 * 5520 = 3378.9 \text{ KN} > V_u = 1154.094 \text{ KN} \end{aligned}$$

V_c is the smallest of :

$$1 - V_c = \frac{1}{6} \sqrt{f_c'} h d = \frac{1}{6} \sqrt{24} * 200 * 5520 = 901.4 \text{ KN} \dots \dots \text{Control}$$

$$2 - V_c = 0.27 \sqrt{f_c'} h d + \frac{N_u d}{4 l_w} = 0.27 \sqrt{24} * 200 * 5520 + 0 = 1460.3 \text{ KN}$$

$$\begin{aligned} 3 - V_c &= \left[0.05 \sqrt{f_c'} + \frac{l_w \left(0.1 \sqrt{f_c'} + 0.2 \frac{N_u}{l_w h} \right)}{\frac{M_u}{V_u} - \frac{l_w}{2}} \right] h d \\ &= \left[0.05 \sqrt{24} + \frac{6.9 (0.1 \sqrt{24} + 0)}{0.1} \right] 200 * 5520 = 37585 \text{ KN} \end{aligned}$$

$$\frac{7825.87 - 3981.15}{3.55} = \frac{M_u - 3981.15}{3.55 - 3.45} \Rightarrow M_u = 4089.45 \text{ KN.m}$$

$$\frac{M_u}{V_u} - \frac{l_w}{2} = \frac{4089.45}{1154.09} - \frac{6.9}{2} = 0.1$$

$$V_c = 901.4 \text{ kN}$$

$$V_u = 1154.09 \text{ kN} > \frac{1}{2} * 0.75 * 901.4 = 338 \text{ kN} \quad \text{Needs reinforcement}$$

$$\phi * v_c + \phi v_s = v_u$$

$$\phi * v_s = v_u - \phi * v_c$$

$$V_s = v_u / \phi - v_c$$

$$V_s = 1154.09 / 0.75 - 901.4 = 637.4 \text{ kN}$$

$$\frac{A_{vh}}{s_h} = \frac{v_s}{f_{yd}} = \frac{637.4}{420 * 5520} = 0.000275 \text{ mm}^2 / \text{m}$$

- Maximum spacing is the least of:

$$\frac{L_w}{5} = \frac{6900}{5} = 1380 \text{ mm}$$

$$3 * h = 3 * 200 = 600 \text{ mm}$$

450 mm Control

Take $\rho = 0.0025$

Try $\phi 10$ ($A_s = 78.5 \text{ mm}^2$) two layers

$$\rho = \frac{A_{vh}}{h s_h} = \frac{2 * 78.5}{200 s_h} = 0.0025$$

$$s_h = 314 \text{ mm}$$

→ use $\phi 10 @ 250 \text{ mm}$ in tow layer

✓ Design of Vertical Reinforcement:-

$$\frac{A_{vv}}{s_v} = \left[0.0025 + 0.5 \left(2.5 - \frac{h_w}{L_w} \right) \left(\frac{A_{vh}}{s_h * h} - 0.0025 \right) \right] * 300$$

$$\frac{A_{vv}}{s_v} = \left[0.0025 + 0.5 \left(2.5 - \frac{11.15}{6.9} \right) \left(\frac{157}{200 * 200} - 0.0025 \right) \right] * 300$$

$$\frac{A_{vv}}{s_v} = 0.938$$

Try $\phi 12$ ($A_s = 113.1 \text{ mm}^2$) two layers

$$\frac{2 * 113.1}{S_v} = 0.938$$

$$S_v = 241.2 \text{ mm}$$

- Maximum spacing is the least of :

$$\frac{L_w}{3} = \frac{6900}{3} = 2300 \text{ mm}$$

$$3 * h = 3 * 200 = 600 \text{ mm}$$

$$450 \text{ mm} \dots\dots \text{Control}$$

→ use $\emptyset 12 @ 250 \text{ mm}$ in tow layer

✓ Design of Bending Moment:-

$$A_{st} = \left(\frac{6900}{250} \right) * 2 * 113.1 = 6243.12 \text{ mm}^2$$

$$w = \left(\frac{A_{st}}{L_w h} \right) \frac{f_y}{f_c'} = \left(\frac{6243.12}{6900 * 200} \right) \frac{420}{24} = 0.0791$$

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

$$\frac{C}{l_w} = \frac{w + \alpha}{2w + 0.85\beta_1} = \frac{0.0791 + 0}{2 * 0.0791 + 0.85 * 0.85} = 0.0898$$

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

$$= 0.9 [0.5 * 6243.12 * 420 * 6900 (1 + 0) (1 - 0.0898)] = 7410.5 \text{ KN} \geq 4089.45 \text{ KN.m}$$

$$M_{ub} = M_u - \phi M_n = 4089.45 - 7410.5 = -3321.05 \text{ KN.m}$$

$$X \geq \frac{l_w}{600 * \frac{\Delta h}{h_w}} = \frac{6900}{600 * 1} = 11.5 \text{ mm}$$

$$L_b \geq \frac{X}{2} = 5.75 \text{ mm}$$

Since Smallest value of L_b & M_{ub} not requires Boundary.

4.11 Design of Footing (F11)

✓ Material :-

⇒ concrete B350 $F_c' = 28 \text{ N/mm}^2$

⇒ Reinforcement Steel $F_y = 420 \text{ N/mm}^2$

✓ Load Calculations :- (From Column C125)

Dead Load = 987Kn , Live Load = 557 Kn

Total services load = 987 + 557 = 1544 Kn

Total Factored load = $1.2 \times 987 + 1.6 \times 557 = 2075.6 \text{ Kn}$

Column Dimensions (a*b) = 50*50 cm

Soil density = 18 Kg/cm³

Allowable Bearing Capacity = 400 Kn/m²

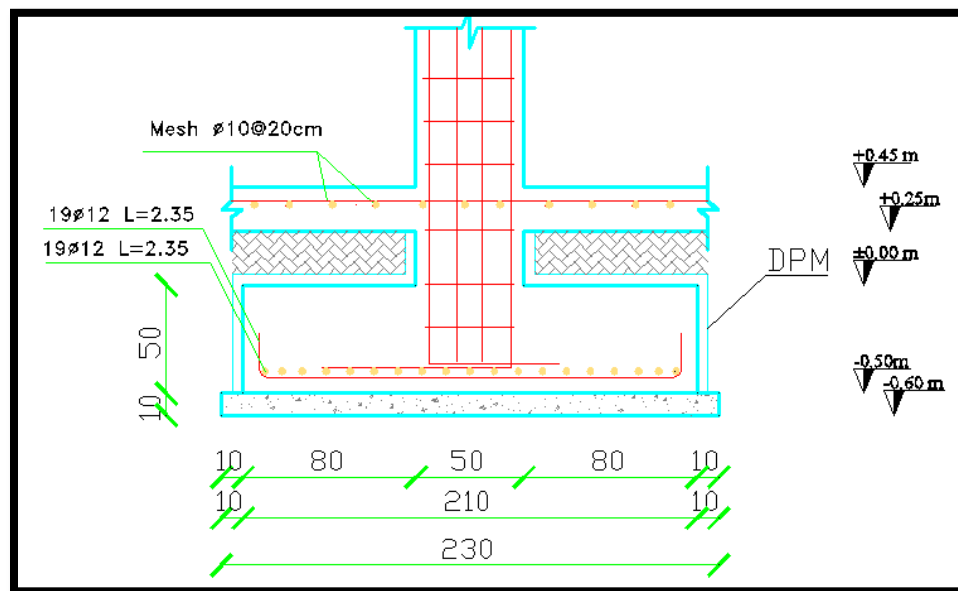


Fig 4.24 :Foot Section.

Assume $h = 50\text{cm}$

$$q_{\text{net-allow}} = 400 - 18 \times 0.25 - 25 \times 0.60 = 384.9 \text{ kN/m}^2$$

✓ Area of Footing :-

$$A = \frac{Pt}{q_{\text{net-allow}}} = \frac{1544}{384.9} = 4.01 \text{ m}^2$$

Assume Square Footing

B required = 2.01 m

Select B = 2.1 m

✓ Bearing Pressure :-

$$q_u = 2075.6 / 2.1 \times 2.1 = 470.6 \text{ KN/m}^2$$

✓ Design of Footing :-

1- Design of One Way Shear Strength :-

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

Assume $h = 50\text{cm}$, bar diameter $\phi 12$ for main reinforcement and 7.5 cm Cover

$$d = 500 - 75 - 12 = 413 \text{ mm}$$

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

$$V_u = 470.6 \times \left(\frac{2.1 - 0.50}{2} - 0.413 \right) \times 2.1 = 382.5 \text{ KN}$$

$$\phi.V_c = \phi \cdot \frac{1}{6} * \sqrt{f_c'} * b_w * d$$

$$\phi.V_c = 0.75 * \frac{1}{6} * \sqrt{28} * 2100 * 413 = 573.6Kn$$

$$\phi.V_c = 573.6Kn > V_u = 382.5Kn$$

\therefore Safe

2- Design of Two Way Shear Strength :-

$$V_u = P_u - FR_b$$

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

$$V_u = 2075.6 - 470.6[(0.5 + 0.413) * (0.5 + 0.413)] = 1216.3Kn$$

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

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

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

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

Where:-

$$\beta_c = \frac{\text{Column Length (a)}}{\text{Column Width (b)}} = \frac{50}{50} = 1$$

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

$$b_o = 2 * (41.3 + 50) + 2 * (41.3 + 50) = 365.2cm$$

$$\alpha_s = 40 \text{ for interior column}$$

$$\phi.V_c = \phi.\frac{1}{6}\left(1 + \frac{2}{\beta_c}\right)\sqrt{f'_c} b_o d = \frac{0.75}{6} * \left(1 + \frac{2}{1}\right) * \sqrt{28} * 3652 * 413 = 2992.8 \text{ Kn}$$

$$\phi.V_c = \phi.\frac{1}{12}\left(\frac{\alpha_s}{b_o/d} + 2\right)\sqrt{f'_c} b_o d = \frac{0.75}{12} * \left(\frac{40 * 413}{3652} + 2\right) * \sqrt{28} * 3652 * 413 = 3254 \text{ Kn}$$

$$\phi.V_c = \phi.\frac{1}{3}\sqrt{f'_c} b_o d = \frac{0.75}{3} * \sqrt{28} * 3652 * 413 = 1995.3 \text{ Kn}$$

$$\Phi V_c = 1995.3 \text{ Kn} > V_u = 1216.3 \text{ Kn}$$

3- Design of Bending Moment :-

Critical Section at the Face of Column

$$F_R = q_u * \left(\frac{B-a}{2}\right) * L = 470.6 * \left(\frac{2.1-0.50}{2}\right) * 2.1 = 790.6 \text{ Kn}$$

$$M_u = 470.6 * 2.1 * 0.8 * 0.8 / 2 = 316.3 \text{ Kn.m}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{316.3 \times 10^6}{0.9 \times 2100 \times 413^2} = .98 \text{ Mpa}$$

$$m = \frac{f_y}{0.85 f'_c} = \frac{420}{0.85 \times 28} = 17.65$$

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2m.R_n}{420}}\right) = \frac{1}{17.6} \left(1 - \sqrt{1 - \frac{2 \times 17.65 \times 0.98}{420}}\right) = 0.0024$$

$$A_{s, \text{req}} = \rho.b.d = 0.0024 \times 2100 \times 413 = 2080.5 \text{ mm}^2$$

$$A_{s, \text{min}} = 0.0018 * 2100 * 500 = 1890 \text{ mm}^2$$

$$A_{s, \text{req}} > A_{s, \text{min}} \quad 1890 \text{ mm}^2$$

As,req = 2080.5..... is control

Check for Spacing :-

$$S = 3h = 3 * 50 = 150 \text{ cm}$$

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

S = 45 cm is control

Use 19ø12in Both Direction, $A_{s,provided} = 2147.8 \text{ mm}^2 > A_{s,required} = 2080.5 \text{ mm}^2 \dots \text{Ok}$

Check for strain:-

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{2147.8 \times 420}{0.85 \times 2100 \times 28} = 18 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{18}{0.85} = 21.1 \text{ mm}$$

$$\varepsilon_s = 0.003 \left(\frac{d - c}{c} \right) = 0.003 \left(\frac{413 - 21.1}{21.1} \right) = 0.055 > 0.005 \dots \dots \text{Ok}$$

4- Design of Dowels :-

Load Transfer In Footing :-

$$\Phi P_n.b = \Phi (0.85 f'_c A_1 \times \sqrt{\frac{A_2}{A_1}})$$

$$A_1 = 50 \times 50 = 0.25 \text{ m}^2$$

$$A_2 = 210 \times 210 = 4.41 \text{ m}^2$$

$$\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{4.41}{0.25}} = 4.2 > 2 \dots \dots \dots \sqrt{\frac{A_2}{A_1}} = 2$$

$$\Phi P_n.b = 0.65 \times (0.85 \times 28 \times 250 \times 2) = 7735 \text{ Kn}$$

$$\Phi P_n = 7735 > P_u = 2075.6 \dots \dots \dots \text{ok}$$

No Need For Dowels

Load Transfer In Column :-

$$\Phi P_n.b = 0.65 \times (0.85 \times 28 \times 250) = 3867.5 \text{ Kn}$$

$$\Phi P_n = 3867.5 > P_u = 2075.6 \text{ kn} \dots \dots \dots \text{ok}$$

No Need For Dowels

$$A_{s,min} = 0.005 \times A_c = 0.005 \times 500 \times 500 = 1250 \text{ mm}^2$$

Use 14ø16, $A_{s,provided} = 2813.5 \text{ mm}^2 > A_{s,required} = 1250 \text{ mm}^2 \dots \text{Ok}$

5- Development Length In Footing :-

Tension Development Length In Footing :-

$$L_{d_{req}} = \frac{9}{10} * \frac{F_y}{\lambda \sqrt{f_c}} * \frac{\psi_e \psi_s \psi_t}{\frac{ktr+cb}{db}} * db > 300\text{mm}$$

$$Ktr = 0 \text{ (No stripes)}$$

$$cb = 50 + \frac{16}{2} = 58\text{mm} \text{ Or } cb = \frac{110}{2} = 55\text{ mm}$$

$$\frac{ktr + cb}{db} = \frac{0 + 55}{16} = 3.4 > 2.5$$

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

$$L_{d_{req}} = \frac{9}{10} * \frac{420}{1 * \sqrt{28}} * \frac{1 * 1 * 0.8}{2.5} * 16 = 365.75\text{ mm} > 300\text{mm}$$

$$L_{d_{available}} = \frac{2100 - 500}{2} - 75 = 725\text{ mm}$$

$$L_{d_{available}} = 725\text{ mm} > L_{d_{req}} = 365.75\text{ mm} \dots\dots \text{OK}$$

Compression Development Length In Footing :-

$$L_{d_{req}} = \frac{0.24 * F_y * dB}{\sqrt{24}} > 0.043 * F_y * dB > 200\text{mm}$$

$$L_{d_{req}} = \frac{0.24 * 420 * 16}{\sqrt{28}} = 304.8 > 0.043 * 420 * 16 = 288.96 > 200\text{mm}$$

$$L_{d_{req}} = 304.8\text{ mm}$$

$$L_{d_{available}} = 500 - 75 - 16 - 16 = 393\text{mm} > L_{d_{req}} = 304.8\text{ mm} \dots\dots \text{Ok}$$

Lap Splice of Dowels In Column :-

$$L_{sc} = 0.071 \times f_y \times d_b = 0.071 \times 420 \times 16 = 477.12 \text{ mm} > 300 \text{ mm}$$

elect $L_{sc} = 500 \text{ mm}$

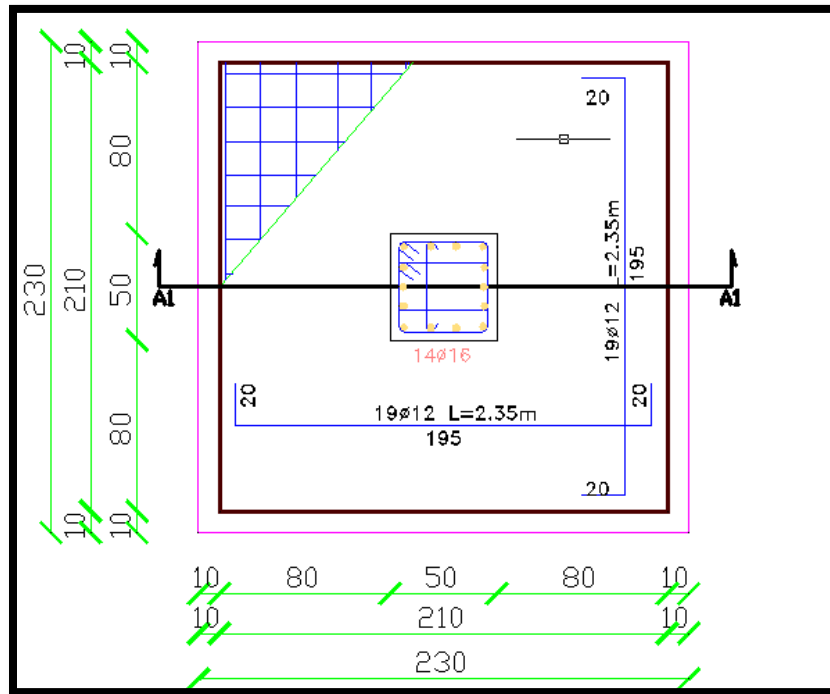


Fig 4.25 :Foot Reinforcement Details.