

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

- **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:

Table (4-1) : Minimum Thickness of Non-prestressed Beam or One-Way Slabs Unless Deflections are Calculated. (ACI 318M-11).

thickness (h) Minimum				
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

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

- **For Rib :**

$$H_{\min} \text{ for (one end continuous) } = L/18.5 = 480/18.5 = 28.37 \text{ cm}$$

$$H_{\min} \text{ for (both end continuous) } = L/21 = 581/21 = 27.6 \text{ cm}$$

$$H_{\min} \text{ for (cantilever) } = L/8 = 203/8 = 25.4 \text{ cm}$$

Take h = 32 cm

24 cm block + 8 cm topping = 32 cm

- **For Beam :**

$$H_{\min} \text{ for (one end continuous) } = L/18.5 = 430/18.5 = 22.7 \text{ cm}$$

$$H_{\min} \text{ for (one end continuous) } = L/18.5 = 581/21 = 31.4 \text{ cm}$$

Take h = 32 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.

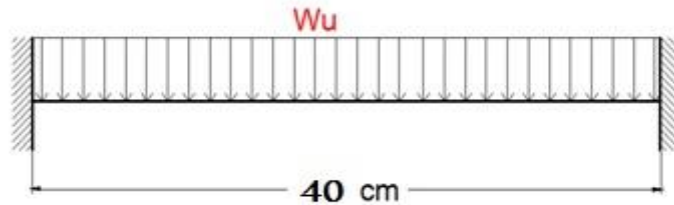


Fig 4.1: Topping Load.

✓ Load Calculations:

Dead Load:

No.	Parts of Rib	Calculation
1	Tiles	$0.03 \times 23 \times 1 = 0.69 \text{ KN/m}$
2	Mortar	$0.02 \times 22 \times 1 = 0.44 \text{ KN/m}$
3	Coarse Sand	$0.07 \times 17 \times 1 = 1.19 \text{ KN/m}$
4	Topping	$0.08 \times 25 \times 1 = 2.0 \text{ KN/m}$
5	Partitions	$1.5 \times 1 = 1.5 \text{ KN/m}$
Sum =		5.82KN/m

Table (4.2): Dead Load Calculation of Topping.

Live Load :

$$L_L = 4.8 \text{ KN/m}^2$$

$$L_L = 4.8 \text{ KN/m}^2 \times 1\text{m} = 4.8 \text{ KN/m}$$

Factored Load :

$$W_U = 1.2 \times 5.82 + 1.6 \times 4.8 = 14.66 \text{ KN/m}$$

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

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

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

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

$$M_u = \frac{w_u L^2}{12} = 0.1954 \text{ KN.m} \quad (\text{negative moment})$$

$$M_u = \frac{w_u L^2}{24} = 0.0977 \text{ KN.m} \quad (\text{positive moment})$$

$$\phi M_n \gg M_u = 0.1954 \text{ KN.m}$$

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

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:

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

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

Select $b_w = 12 \text{ cm}$

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

Select $h = 35 \text{ cm} < 3.5 \cdot 12 = 49 \text{ cm}$

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

Select $t_f = 8 \text{ cm}$

❖ Material :

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

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

❖ Section :

- ⇒ $B = 520 \text{ mm}$
- ⇒ $B_w = 120 \text{ mm}$
- ⇒ $h = 320 \text{ mm}$
- ⇒ $t = 80 \text{ mm}$
- ⇒ $d = 320 - 20 - 10 - 12/2 = 284 \text{ mm}$

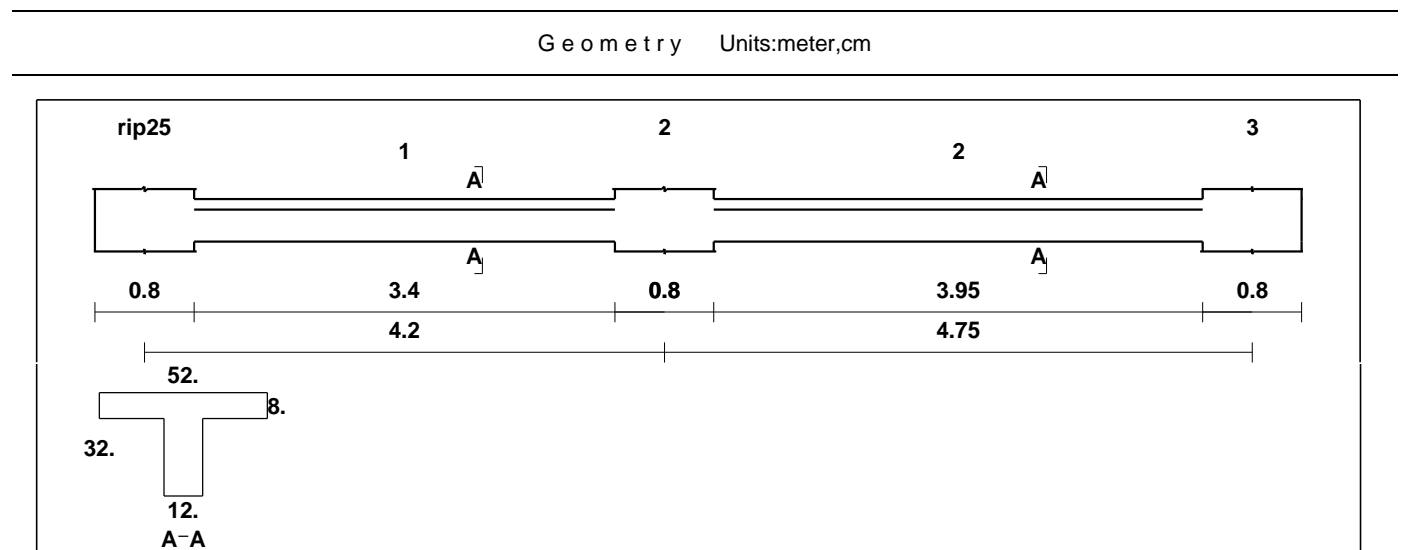
➤ Statically System and Dimensions:

Fig 4.2: One Way Rib Slab (R25).

➤ **Load Calculation:**

Dead Load:

No.	Parts of Rib	Calculation
1	Tiles	$0.03 \times 23 \times 0.52 = 0.359 \text{ KN/m/rib}$
2	Mortar	$0.03 \times 22 \times 0.52 = 0.229 \text{ KN/m/rib}$
3	Coarse Sand	$0.07 \times 17 \times 0.52 = 0.620 \text{ KN/m/rib}$
4	Topping	$0.08 \times 25 \times 0.52 = 1.04 \text{ KN/m/rib}$
5	RC. Rib	$0.24 \times 25 \times 0.12 = 0.72 \text{ KN/m/rib}$
6	Hollow Block	$0.24 \times 10 \times 0.4 = 0.96 \text{ KN/m/rib}$
7	plaster	$0.03 \times 22 \times 0.52 = 0.3432 \text{ KN/m/rib}$
8	partions	$1.5 \times 0.52 = 0.78 \text{ KN/m/rib}$
		Sum = 5.05 KN/m/rib

Table (4.3): Dead Load Calculation of Rib (R25).

Dead Load = 5.05 KN/m/rib

Live Load:

Live load = 4.8 KN/m^2

Live load /rib = $4.8 \text{ KN/m}^2 \times 0.52 \text{ m} = 2.5 \text{ KN/m}$.

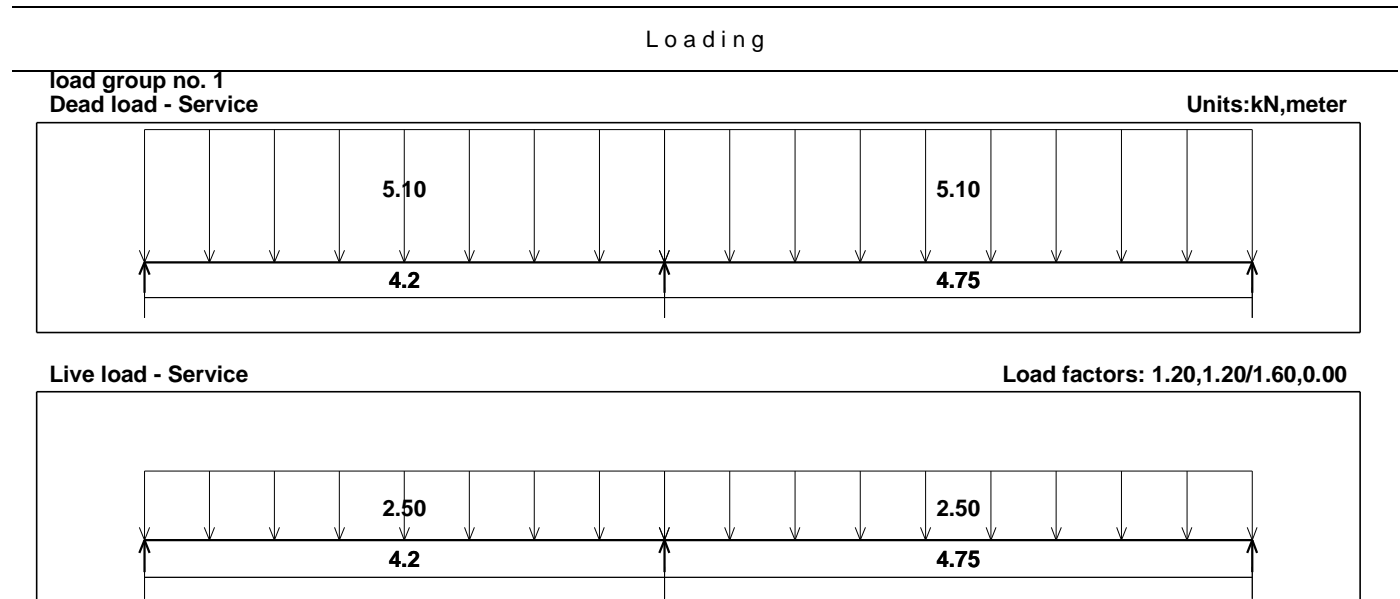


Fig 4.3: Statically System and Loads Distribution of Rib (R25).

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

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

$$b_E = L / 4 = 480 / 4 = 120 \text{ cm}$$

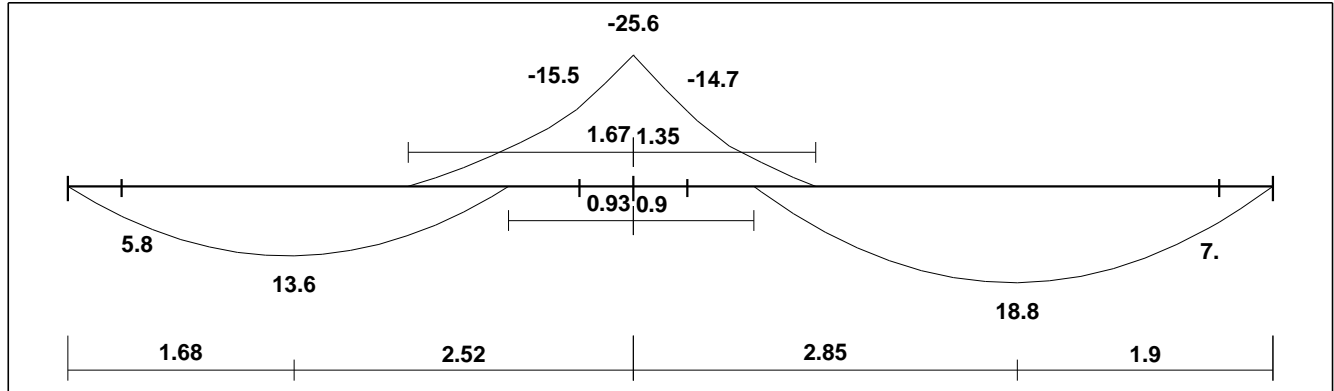
$$b_E = 12 + 16 t = 12 + 16 (8) = 140 \text{ cm}$$

$$b_E = b_e \leq \text{center to center spacing between adjacent beams} = \frac{400}{2} + \frac{400}{2} + 120 = 52 \text{ cm.}$$

b_E **For T-section = 52cm .**

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

Moments: spans 1 to 2



Shear

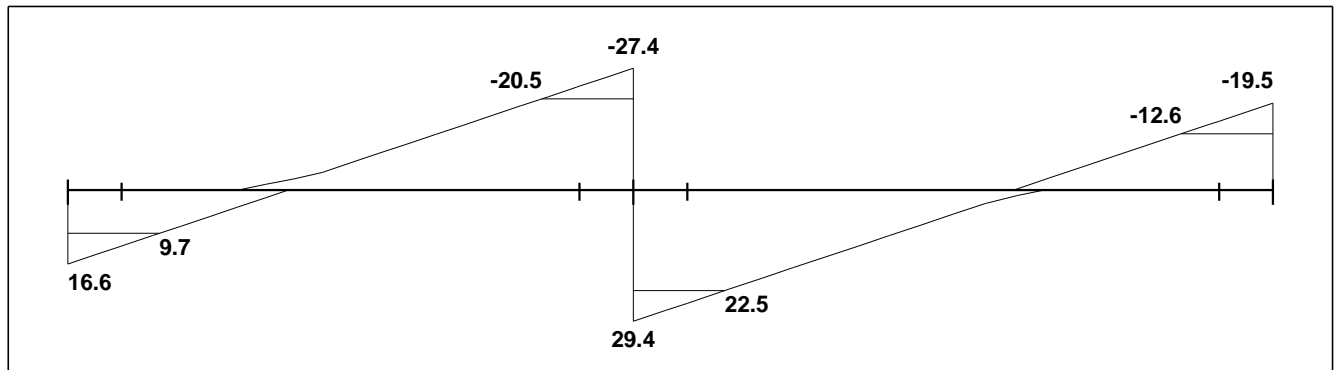


Fig 4.4: Shear and Moment Envelope Diagram of Rib (R25).

✓ Moment Design for (R 25):**Design of Positive Moment for (Rib 25): (Mu = 13.6 kN.m)**Assume bar diameter ϕ 12 for main positive reinforcement

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

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

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

$$= 0.85 \times 24 \times 520 \times 80 \times \left(284 - \frac{80}{2}\right) \times 10^{-6} = 207.06 \text{ KN.m}$$

$$M_{nf} \gg \frac{M_u}{\phi} = \frac{13.6}{0.9} = 15.11 \text{ KN.m}, \text{ the section will be designed as rectangular with } b_e = 520 \text{ mm}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{13.6 \times 10^6}{0.9 \times 520 \times 284^2} = 0.36 \text{ Mpa}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{420}}\right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.36}{420}}\right) = 0.000864$$

$$A_{s, \text{req}} = \rho \cdot b \cdot d = 0.000864 \times 520 \times 284 = 127.6 \text{ mm}^2$$

Check for A_s min:

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

$$A_s \text{ min} = \frac{\sqrt{24}}{4(420)} (120)(284) = 99.37 \text{ mm}^2$$

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

$$A_s \text{ min} = \frac{1.4}{420} (120)(284) = 113.6 \text{ mm}^2 \quad \text{control}$$

$$A_{s, \text{req}} = 127.6 \text{ mm}^2 > A_{s, \text{min}} = 113.6 \text{ mm}^2 \quad \text{OK}$$

Use 2 ϕ 10, $A_{s, \text{provided}} = 157.08 \text{ mm}^2 > A_{s, \text{required}} = 127.6 \text{ mm}^2$ Ok

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

Check for strain:

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{157.08 \times 420}{0.85 \times 520 \times 24} = 6.22 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{6.22}{0.85} = 7.313 \text{ mm}$$

$$\varepsilon_s = 0.003 \left(\frac{d - c}{c} \right) = 0.003 \left(\frac{284 - 7.313}{7.313} \right) = 0.11 > 0.005 \quad \text{OK}$$

Design of Positive Moment for (Rib25): (Mu = 18.8 KN.m)

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

$$R_n = \frac{M_u}{\phi b d^2} = \frac{18.8 \times 10^6}{0.9 \times 520 \times 284^2} = 0.49 \text{ Mpa}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 m R_n}{420}} \right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.49}{420}} \right) = 0.00118$$

$$A_{s, \text{req}} = \rho \cdot b \cdot d = 0.00118 \times 520 \times 284 = 174.26 \text{ mm}^2$$

Check for As min:

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

$$A_s \min = \frac{\sqrt{24}}{4(420)} (120)(284) = 99.37 \text{ mm}^2$$

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

$$A_s \min = \frac{1.4}{420} (120)(284) = 113.6 \text{ mm}^2 \quad \text{control}$$

$$A_{s\text{req}} = 174.26 \text{ mm}^2 > A_{s\text{min}} = 113.6 \text{ mm}^2 \quad \text{OK}$$

Use 2 ø 12 , $A_{s\text{provided}} = 226.19 \text{ mm}^2 > A_{s\text{required}} = 174.26 \text{ mm}^2$ Ok

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

Check for strain:

$$a = \frac{A_s f_y}{0.85 b f_c'} = \frac{226.19 \times 420}{0.85 \times 520 \times 24} = 8.95 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{8.95}{0.85} = 10.53 \text{ mm}$$

$$\epsilon_s = 0.003 \left(\frac{d - c}{c} \right) = 0.003 \left(\frac{284 - 10.53}{10.53} \right) = 0.0779 > 0.005 \quad \text{Ok}$$

Design of Negative Moment for (Rib25): (Mu = -15.5 KN.m)

Assume bar diameter ϕ 14 for main positive reinforcement

$$d = h - \text{cover} - d_{\text{stirrups}} - \frac{d_b}{2} = 320 - 20 - 10 - \frac{14}{2} = 283 \text{ mm}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{15.5 \times 10^6}{0.9 \times 520 \times 283^2} = 0.41 \text{ Mpa}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{420}} \right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.41}{420}} \right) = 0.00098$$

$$A_{s, \text{req}} = \rho \cdot b \cdot d = 0.00098 \times 520 \times 283 = 130.9 \text{ mm}^2$$

Check for As min:

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

$$A_s \text{ min} = \frac{\sqrt{24}}{4(420)} (120)(283) = 99.02 \text{ mm}^2$$

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

$$A_s \text{ min} = \frac{1.4}{420} (120)(283) = 113.2 \text{ mm}^2 \quad \text{control}$$

$$A_{s, \text{req}} = 130.9 \text{ mm}^2 > A_{s, \text{min}} = 113.2 \text{ mm}^2 \quad \text{OK}$$

Use 2 ϕ 12 . $A_{s,provided} = 226.19 \text{ mm}^2 > A_{s,required} = 130.9 \text{ mm}^2 \dots \text{Ok}$

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

Check for strain:

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{226.19 \times 420}{0.85 \times 520 \times 24} = 8.95 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{8.95}{0.85} = 10.5 \text{ mm}$$

$$\varepsilon_s = 0.003 \left(\frac{d - c}{c} \right) = 0.003 \left(\frac{284 - 10.5}{10.5} \right) = 0.078 > 0.005 \quad \text{Ok}$$

✓ Shear Design for (R25):

V_u at distance d from support = 22.5 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).

$$V_c = \frac{1.1}{6} \sqrt{f'_c} b_w d = \frac{1.1}{6} \sqrt{24} \times 120 \times 284 \times 10^{-3} = 30.6 \text{ KN}$$

$$\phi V_c = 0.75 \times 30.6 = 22.9 \text{ KN}$$

$$0.5 \phi V_c = 0.5 \times 22.9 = 11.45 \text{ KN}$$

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

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

exception for Ribbed slab , No shear Reinforcement .

Use stirrups (2 leg stirrups) ϕ 8@150 mm , $A_v = 2 \times 50.26 = 100.53 \text{ mm}^2$

$$A_{v_{\min}} = \frac{1}{16} \sqrt{f'_c} \frac{b_w s}{f_{yt}} \geq \frac{1}{3} \frac{b_w s}{f_{yt}}$$

$$A_{v_{\min}} = 100.53 = \frac{1}{16} \sqrt{24} \frac{120s}{420} \rightarrow s = 1.145m$$

$$100.53 = \frac{1}{3} * \frac{120s}{420} \rightarrow s = 1.055m$$

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

$$S_{\max} \rightarrow \leq 600 \text{ mm}$$

Take (2 leg stirrups) ϕ 8 @ 150 mm

$$A_v = \frac{2 * 50.26}{0.15} = 670.2 \text{ mm}^2/\text{m strip}$$

4.6 Design of Beam :

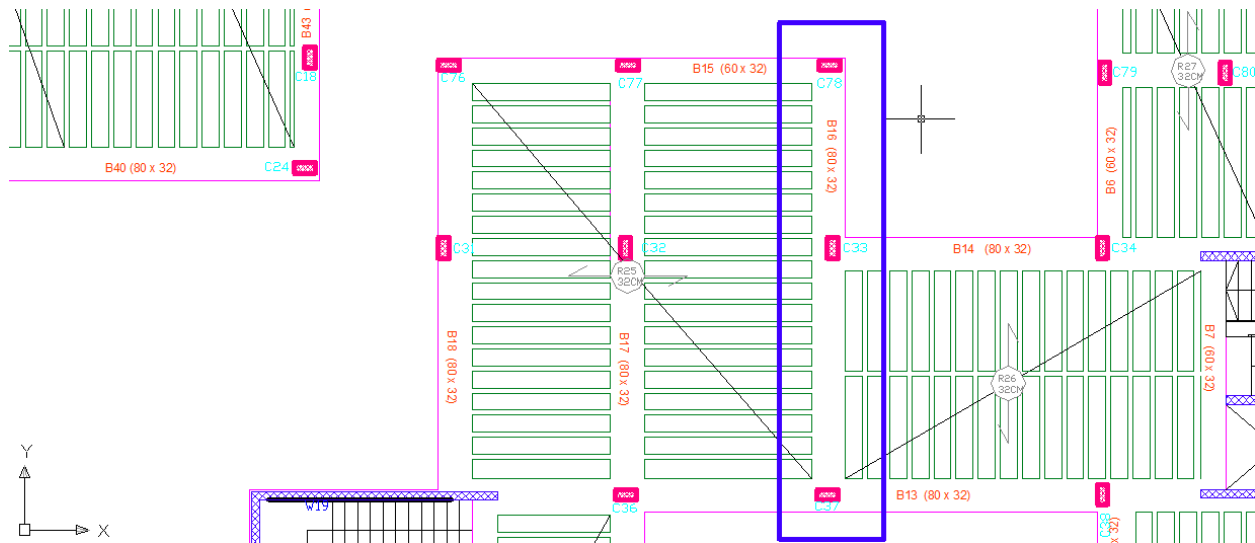


Fig 4.5: Design of Beam (16).

❖ **Material :**

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

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

❖ **Section :**

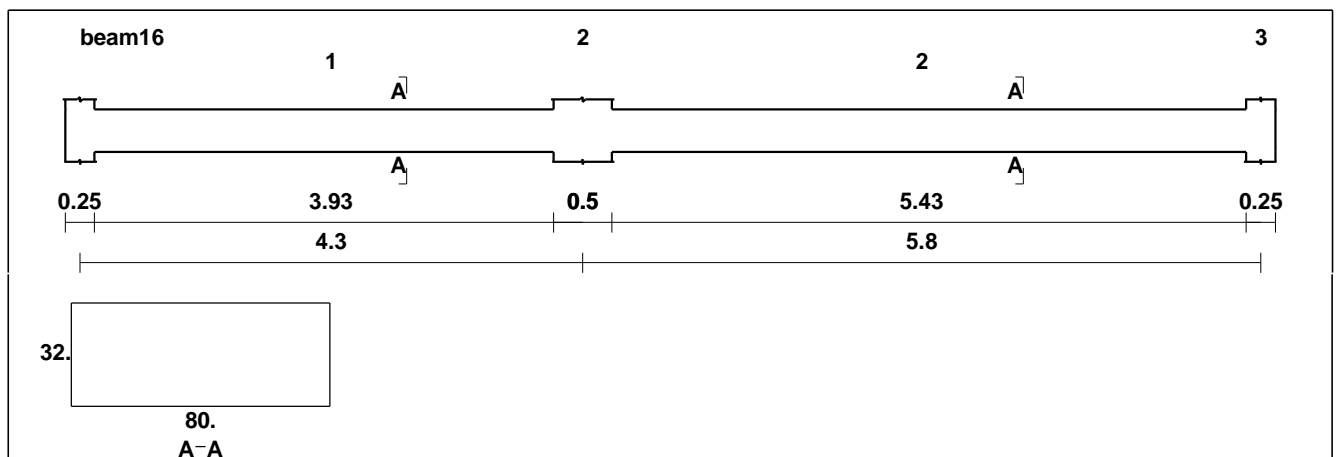
⇒ $B = 80 \text{ cm}$

⇒ $h = 32 \text{ cm}$

⇒ $d = 320 - 40 - 10 - 20/2 = 260 \text{ mm}$

➤ **Statically System and Dimensions:**

Geometry Units: meter, cm



✓ Load Calculations:

Dead Load Calculations for Beam (B 16):

The distributed Dead and Live loads acting upon B16 can be defined from the support reactions of the R25 .

From Rib 25

The maximum support reaction from Dead Loads for R25 upon B16 is 9.39 KN ,The distributed Dead Load from the R25 on B16 .

$$DL = (9.39 / 0.52) = 18.05 \text{ KN / m}$$

$$\text{Self weight of beam} = 6.14 \text{ KN / m}$$

$$DL = 18.05 + 6.14 = 24.19 \text{ KN / m}$$

Live Load calculations for Beam (B16):

From Rib 25

The maximum support reaction from Live Loads for R25 upon B16 is 5.15 KN The distributed Live Load from the Rib25 on B16.

$$LL = 5.15 / 0.52 = 9.9 \text{ KN/m.}$$

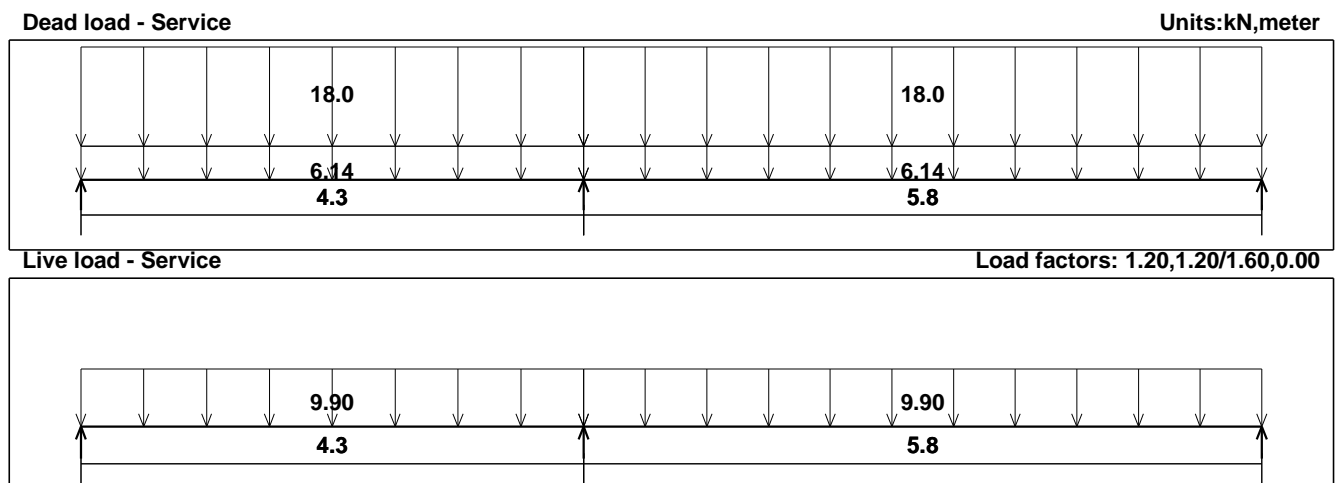


Fig 4.6: Static System and Loads Distribution of Beam (B 16)

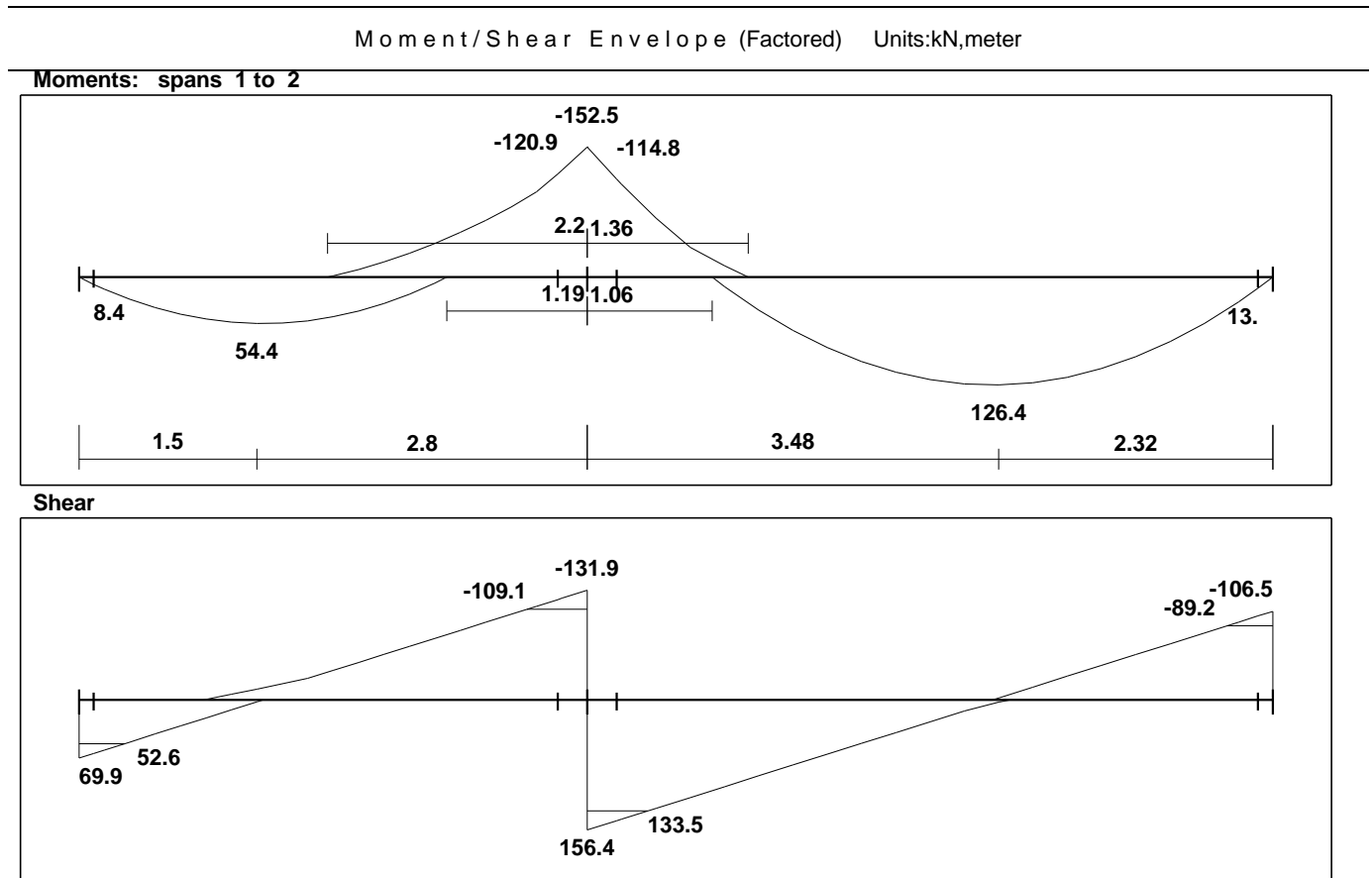
✓ Moment Design for (B16):

Fig 4.7: Shear and Moment Envelope Diagram of Beam (B16).

Flexural Design of Positive Moment for (B16): (Mu = 54.4 KN.m)

Determine of $M_{n,max}$

$$d = 320 - 40 - 10 - 20/2 = 260 \text{ mm}$$

$$x = \frac{3}{7}d = \frac{3}{7} \cdot 260 = 111.4 \text{ mm}$$

$$a = \beta_1 x = 111.4 \cdot 0.85 = 94.69 \text{ mm}$$

$$M_{n,max} = 0.85 \cdot f'_c \cdot a \cdot b \left(d - \frac{a}{2} \right) = 0.85 \cdot 24 \cdot 94.69 \cdot 800 \cdot \left(260 - \frac{94.69}{2} \right) \cdot 10^{-6} = 328.62 \text{ KN.m}$$

$$\phi M_{n,max} = 0.9 \cdot 328.62 = 295.76 \text{ KN.m} > 54.4 \text{ KN.m}$$

Design as singly reinforcement

$$R_n = \frac{M_u}{\phi b d^2} = \frac{54.4 \times 10^6}{0.9 \times 800 \times 260^2} = 1.11 \text{ Mpa}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{420}} \right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 1.11}{420}} \right) = 0.00272$$

$$A_s = \rho \cdot b \cdot d = 0.00272 \times 800 \times 260 = 565.76 \text{ mm}^2$$

Check for $A_{s,min}$:

$$A_{s,min} = \frac{\sqrt{f'_c}}{4(f_y)} (b_w)(d) = \frac{\sqrt{24}}{4 \cdot 420} \cdot 800 \cdot 260 = 606.5 \text{ mm}^2$$

$$A_{s,min} = \frac{1.4}{(f_y)} (b_w)(d) = \frac{1.4}{420} \cdot 800 \cdot 260 = 693.33 \text{ mm}^2 \quad \text{Control}$$

$$A_{s,req} = 693.33 \text{ mm}^2$$

Use 4 ϕ 16 Bottom , $A_{s,provided} = 804.24 \text{ mm}^2 > A_{s,required} = 693.33 \text{ mm}^2 \dots$ Ok

Check spacing :

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

Check for strain:

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{804.24 \times 420}{0.85 \times 800 \times 24} = 20.7 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{20.7}{0.85} = 24.35 \text{ mm}$$

$$\varepsilon_s = 0.003 \left(\frac{d - c}{c} \right) = 0.003 \left(\frac{262 - 24.35}{24.35} \right) = 0.029 > 0.005 \quad \text{OK}$$

Flexural Design of Positive Moment for (B16): (Mu=126.4 KN.m)

$$R_n = \frac{M_u}{\phi b d^2} = \frac{126.4 \times 10^6}{0.9 \times 800 \times 260^2} = 2.59 \text{ Mpa}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 R_n}{f_y}} \right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 \times 2.59 \times 10^6}{420 \times 260^2}} \right) = 0.00661$$

$$A_s = \rho \cdot b \cdot d = 0.00661 \times 800 \times 260 = 1374.88 \text{ mm}^2$$

Check for $A_{s,min}$:

$$A_{s,min} = \frac{\sqrt{f_c'}}{4(f_y)}(bw)(d) = \frac{\sqrt{24}}{4 \times 420} \times 800 \times 260 = 606.5 \text{ mm}^2$$

$$A_{s,min} = \frac{1.4}{(f_y)}(bw)(d) = \frac{1.4}{420} \times 800 \times 260 = 693.33 \text{ mm}^2 \quad \textbf{Control}$$

$$A_{s, req} = 1374.88 \text{ mm}^2$$

Use 6 ϕ 18 Bottom , $A_{s,provided} = 1526.8 \text{ mm}^2 > A_{s,required} = 1374.88 \text{ mm}^2 \dots \text{Ok}$

Check spacing :

$$S = \frac{800 - 40 \times 2 - 20 - (6 \times 18)}{5} = 118.4 \text{ mm} > d_b = 18 > 25 \text{ mm} \quad \textbf{OK}$$

Check for strain:

$$a = \frac{A_s f_y}{0.85 b f_c'} = \frac{1526.8 \times 420}{0.85 \times 800 \times 24} = 39.29 \text{ mm}$$

$$c = \frac{a}{B_1} = \frac{39.29}{0.85} = 46.22 \text{ mm}$$

$$\varepsilon_s = 0.003 \left(\frac{d - c}{c} \right) = 0.003 \left(\frac{261 - 46.22}{46.22} \right) = 0.013 > 0.005 \quad \textbf{Ok}$$

Flexural Design of Negative Moment for (B16): ($M_u = -120.9 \text{ KN.m}$)

$$R_n = \frac{M_u}{\phi b d^2} = \frac{120.9 \times 10^6}{0.9 \times 800 \times 260^2} = 2.48 \text{ Mpa}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{420}} \right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 2.48}{420}} \right) = 0.00631$$

$$A_s = \rho \cdot b \cdot d = 0.00631 \times 800 \times 260 = 1313.64 \text{ mm}^2$$

Check for $A_{s,min}$:

$$A_{s,min} = \frac{\sqrt{f_c'}}{4(f_y)} (b_w)(d) = \frac{\sqrt{24}}{4 \times 420} * 800 * 260 = 606.5 \text{ mm}^2$$

$$A_{s,min} = \frac{1.4}{(f_y)} (b_w)(d) = \frac{1.4}{420} * 800 * 260 = 693.33 \text{ mm}^2 \quad \textbf{Control}$$

$$A_{s, req} = \mathbf{1313.64 \text{ mm}^2}$$

Use 6 ϕ 18 Bottom , $A_{s,provided} = 1526.8 \text{ mm}^2 > A_{s,required} = 1313.64 \text{ mm}^2 \dots \text{Ok}$

Check spacing :

$$S = \frac{800 - 40 \times 2 - 20 - (6 \times 18)}{5} = 118.4 \text{ mm} > d_b = 18 > 25 \text{ mm} \quad \textbf{OK}$$

Check for strain:

$$a = \frac{A_s f_y}{0.85 b f_c'} = \frac{1526.8 \times 420}{0.85 \times 800 \times 24} = 39.29 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{39.29}{0.85} = 46.22 \text{ mm}$$

$$\varepsilon_s = 0.003 \left(\frac{d - c}{c} \right) = 0.003 \left(\frac{261 - 46.22}{46.22} \right) = 0.013 > 0.005 \quad \textbf{Ok}$$

✓ **Shear Design for (B 16):**

$$V_{u,\max} = 133.5 \text{ KN.}$$

$$d = h - \text{cover} - d_{\text{stirrup}} - \frac{d_b}{2} = 320 - 40 - 10 - \frac{18}{2} = 261 \text{ mm.}$$

$$V_c = \frac{1}{6} \sqrt{f'_c} b \cdot d = \frac{1}{6} \sqrt{24} \times 800 \times 261 \times 10^{-3} = 187.53 \text{ KN}$$

$$\Phi V_c = 0.75 * 187.53 = 140.65 \text{ KN}$$

$$0.5 * \Phi V_c = 0.5 * 0.75 * 187.53 = 70.325 \text{ KN}$$

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

$$S_{\max} \leq \frac{d}{2} = \frac{261}{2} = 130.5 \text{ mm} \quad \text{OR} \quad S_{\max} \leq 600 \text{ mm}$$

$$S_{\max} = 130.5 \text{ mm} \quad \text{control.}$$

$$\text{By using } \phi 10 \text{ double legs stirrups, } A_v = 316 \text{ mm}^2$$

$$s = \frac{A_v f_{yt}}{V_s} d = \frac{316 \times 420 \times 261}{69.6 \times 800} = 622.2 \text{ mm}$$

Use 4 leg $\phi 10$ @ 100mm

For all spans 4 leg $\phi 10$ @ 100mm will be used for stirrups.

4.7 Design of One Way Solid Slab

❖ Material:-

⇒ concrete B300 $F_c' = 24 \text{ MPa}$

⇒ Reinforcement Steel $F_y = 420 \text{ MPa}$

✓ Slab Thickness Calculation:-

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

Min H (deflection requirement):-

-For simply supported :-

$$\frac{L}{20} = \frac{3.25}{20} = 0.1625$$

For One way solid slab, will use thickness of slab 20 cm.

$$d = h - \text{cover} - \frac{d_b}{2} = 200 - 20 - \frac{12}{2} = 174 \text{ mm}$$

✓ Load Calculation:-

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

-Load Calculation For the Horizontal Slab:- (For one Meter Strip)

#	material	calculation
1	Tiles	$0.03 * 22 = 0.66$
2	mortar	$0.02 * 22 = 0.44$
3	Coarse sand	$0.07 * 16 = 1.12$
4	RC concrete	$0.20 * 25 = 5.00$
5	plaster	$0.02 * 22 = 0.44$
	Sum	9.66

Table 4.4: Dead Load Calculation of Solid Slab.

Live load = 5 KN/m

✓ Design of Positive Moment :

Design of Positive Moment :- (Mu = 25.9 KN.m)

Assume bar diameter $\Phi 12$ for main reinforcement

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

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

$$R_n = \frac{25.9 * 10^6 / 0.9}{1000 * (174)^2} = 1.33 \text{ (MPa)}$$

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

$$\rho = \frac{1}{20.59} \left(1 - \sqrt{1 - \frac{2(20.59)(1.33)}{420}} \right) = 0.00328$$

$$A_s = \rho * b * d = 0.00328 * 1000 * 174 = 570.7 \text{ mm}^2$$

Check for A_s min:-

$$A_s \text{ min} = \rho_{\min} * b * h = 0.0018 * 1000 * 200 = 360 \text{ mm}^2$$

$$A_{s\text{req}} = 570.7 \text{ mm}^2 > A_{s\text{min}} = 360 \text{ mm}^2 \quad \text{OK}$$

Use ϕ 12/20cm , $A_{s,\text{provided}} = 4.52 \text{ cm}^2 > A_{s,\text{required}} = 3.6 \text{ cm}^2$ Ok

❖ **Shear Design:-**

Check Whether Thickness Is Adequate For Shear:-

$$V_{u,\text{max}} = 22.8 \text{ KN/ 1m strip}$$

$$d = h - 20 - db = 200 - 20 - (12 / 2) = 174 \text{ mm}$$

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

$$= \frac{1}{6} * 0.75 * \sqrt{24} * 1000 * 174 = 106.55 \text{ KN / 1 m strip}$$

$$\Phi V_c = 106.55 \text{ KN} > V_{u, \max} = 22.8 \text{ KN/ 1m strip}$$

The thickness of the slab is adequate enough.

4.8 Design of Stair:

❖ **Material :-**

⇒ concrete B300 $F_c' = 24 \text{ MPa}$

⇒ Reinforcement Steel $F_y = 420 \text{ MPa}$

✓ **Design of Flight :-**

✓ **Determination of Thickness:-**

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

$$h_{\min} = 3.5/20 = 17.5 \text{ cm}$$

Take $h = 20 \text{ cm}$, Rise = 16 cm , Run = 30 cm

The Stair Slope by $\theta = \tan^{-1}(16 / 30) = 28.07^\circ$

✓ Load Calculation:-

Dead Load For Flight For 1m Strip:-

No.	Parts of Flight	Calculation
1	Tiles	$22 \times 0.03 \times 1 \times (0.30 + 0.16/0.3) = 1.056 \text{ KN/m}$
2	Mortar	$22 \times 0.02 \times 1 \times (0.30 + 0.16/0.3) = 0.704 \text{ KN/m}$
3	Stair	$25 \times 1 \times (0.30 \times 0.16/2) / 0.3 = 2.00 \text{ KN/m}$
4	Slab	$25 \times 0.20 \times 1 / \cos 28.07 = 5.70 \text{ KN/m}$
5	Plaster	$22 \times 0.02 \times 1 / \cos 28.07 = 4.98 \text{ KN/m}$
Sum		14.44 KN/m

Table 1-5: Dead Load Calculation of Flight.

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

Factored Load For Flight :-

$$W_U = 1.2 \times 14.44 + 1.6 \times 5 = 25.32 \text{ KN/m}$$

$$R = (W \times L) / 2 = 25.32 \times 3.5 / 2 = 44.32 \text{ KN}$$

1- Design of Shear for Flight :- ($V_u=27.45$ KN)

Assume bar diameter ϕ 14 for main reinforcement

$$d = h - \text{cover} - \frac{d_b}{2} = 200 - 20 - \frac{12}{2} = 174 \text{ mm}$$

$$V_u = 44.32 \text{ KN}$$

$$V_c = \frac{1}{6} \sqrt{f_c'} b_w d = \frac{1}{6} \sqrt{24} * 1000 * 174 = 142.1 \text{ KN/m}$$

$$\Phi V_c = 0.75 * 142.1 = 106.55 \text{ KN /m}$$

$$V_u = 44.32 < \Phi V_c = 106.55 \text{ KN /m}$$

The thickness is enough .

2- Design of Bending Moment for Flight :- ($M_u = 66.8$ KN.m)

$$R_n = \frac{M_u}{\phi b d^2} = \frac{66.8 \times 10^6}{0.9 \times 1000 \times 174^2} = 2.45 \text{ Mpa}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 m R_n}{420}} \right) = \frac{1}{20.59} \left(1 - \sqrt{1 - \frac{2 \times 20.59 \times 2.45}{420}} \right) = 0.00623$$

$$A_{s, \text{req}} = \rho \cdot b \cdot d = 0.00623 \times 1000 \times 174 = 1084.64 \text{ mm}^2$$

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

$$A_{s, \text{req}} > A_{s, \text{min}}$$

$$A_{s, \text{req}} = 1084.64 \text{ mm}^2$$

Check for spacing :-

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

2) $S = 380 \times (280 / (2/3 \times 420)) - 2.5 \times 20 = 330 \leq S = 300 \times (280 / (2/3 \times 420)) = 330 \text{ mm}$

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

Use $\phi 12 @ 100 \text{ mm}$, $A_{s,provided} = 1131 \text{ mm}^2 > A_{s,required} = 1084.64 \text{ mm}^2 \dots \text{Ok}$

3- Lateral or Secondary Reinforcement For Flight :-

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

Use $\phi 10 @ 200 \text{ mm}$, $A_{s,provided} = 392 \text{ mm}^2 > A_{s,required} = 360 \text{ mm}^2 \dots \text{Ok}$

✓ Design of Landing :

➤ Load Calculation:-

Dead Load For Landing For 1m Strip:-

No.	Parts of Flight	Calculation
1	Tiles	$22 \times 0.03 \times 1 = 0.66 \text{ KN/m}$
2	Mortar	$22 \times 0.02 \times 1 = 0.44 \text{ KN/m}$
3	Slab	$25 \times 0.20 \times 1 = 5.00 \text{ KN/m}$
4	Plaster	$22 \times 0.03 \times 1 = 0.66 \text{ KN/m}$
Sum		6.76 KN/m

Table 2-6: Dead Load Calculation of landing.

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

Factored Load For Landing :-

$$W_U = 1.2 \times 6.76 + 1.6 \times 5 = 16.11 \text{ KN/m}$$

➤ **System of Landing :-**

1- Design of Shear:- ($V_u = 73.3$ KN)

Assume bar diameter ϕ 12 for main reinforcement

$$d = h - \text{cover} - \frac{d_b}{2} = 200 - 20 - \frac{12}{2} = 174 \text{ mm}$$

$$V_c = \frac{1}{6} \sqrt{f'c} b_w d = \frac{1}{6} \sqrt{24} * 1000 * 174 = 142.1 \text{ KN}$$

$$\Phi * V_c = 0.75 * 142.1 = 106.55 \text{ KN} > V_u = 73.3 \text{ KN}$$

Thickness of slab is enough

2- Design of Bending Moment :- ($M_u = 77.06$ KN.m)

Assume bar diameter ϕ 12 for main reinforcement

$$d = h - \text{cover} - \frac{d_b}{2} = 200 - 20 - \frac{12}{2} = 174 \text{ mm}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{77.06 \times 10^6}{0.9 \times 1000 \times 174^2} = 2.82 \text{ Mpa}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{420}} \right) = \frac{1}{20.59} \left(1 - \sqrt{1 - \frac{2 \times 20.59 \times 2.82}{420}} \right) = 0.00725$$

$$A_{s, \text{req}} = \rho \cdot b \cdot d = 0.00725 \times 1000 \times 174 = 1262.66 \text{ mm}^2$$

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

$$A_{s, \text{req}} = 1262.66 \text{ mm}^2 \dots\dots\dots \text{is control}$$

Check for Spacing:-

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

5) $S = 380 \times (280 / (2/3 \times 420)) - 2.5 \times 20 = 330 \leq S = 300 \times (280 / (2/3 \times 420)) = 330\text{mm}$

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

$S = 100\text{ mm}$ is control

Use $\phi 12$ @ 100 mm

4.9 Design of Column

❖ Material :-

⇒ concrete B350 $F_c' = 24 \text{ MPa}$

⇒ Reinforcement Steel $F_y = 420 \text{ MPa}$

➤ Load Calculation:-

Service Load:-

Dead Load = 638.3 KN

Live Load = 252.6 KN

Factored Load:-

$$P_U = 1.2 \times 638.3 + 1.6 \times 252.6 = 1170.12 \text{ KN}$$

➤ Dimensions of Column:-

Assume $\rho_g = 0.01$

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

$$1170.12 \times 1000 = 0.65 \times 0.8 \times A_g \{0.85 * 24 (1 - 0.01) + 0.01 * 420\}$$

$$A_g = 57938 \text{ mm}^2$$

Assume Rectangular Section

Try $h = 500 \text{ mm}$

$$b = 250$$

Selecting Longitudinal Bars:

$$1170.12 \times 1000 = 0.65 \times 0.8 \times A_g \{0.85 \times 24 (240000 - A_{st}) + A_{st} \times 420\}$$

$$A_{st} = 5980 \text{ mm}^2$$

Use 10 ϕ 20 , $A_{st}(\text{prov}) = 6308 \text{ mm}^2 > A_{st} = 5980 \text{ mm}^2$

$$\rho_g = A_{st}/A_g = 0.0125$$

✓ Design of the tie reinforcement :

$$S \leq 16 \text{ db (longitudinal bar diameter)}$$

$$S \leq 48 \text{ dt (tie bar diameter).}$$

$$S \leq \text{Least dimension.}$$

$$\text{spacing} \leq 16 \times d_b = 16 \times 2.0 = 44.8 \text{ cm}$$

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

$$\text{spacing} \leq \text{least.dim} = 25 \text{ cm control}$$

Use ϕ 10@20 cm

4.10 Design of shear wall:

$$h_w = 11.25 \text{ m} , L_w = 5.2 \text{ m}$$

$$d \leq 0.8 * L_w = 0.8 * 5.2 = 4.16 \text{ m} \dots \text{control}$$

$$d \leq 0.8 * h_w = 0.8 * 11.25 = 9 \text{ m}$$

$$L_w / 2 = 2.6 \text{ m} \dots\dots \text{control}$$

$$h_w / 2 = 5.625 \text{ m}$$

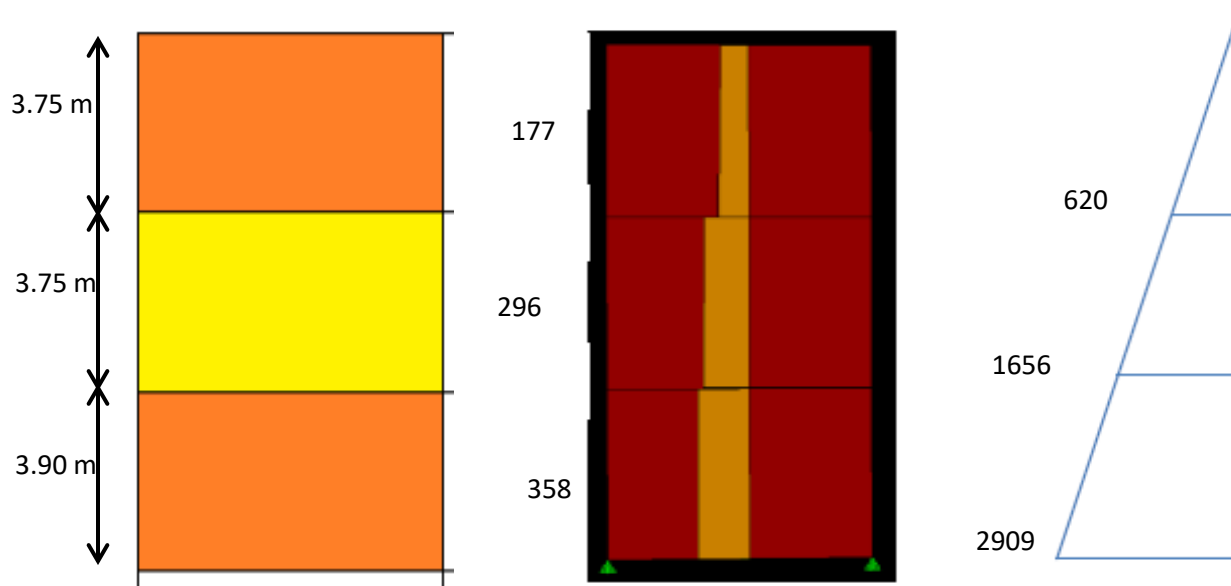


Fig. 4.8 Shear force and moment on the wall

✓ Design horizontal reinforcement :

$$V_{c1} = \frac{\sqrt{f_c'}}{6} \times b \times d$$

$$V_{c1} = \frac{\sqrt{24}}{6} \times 200 \times 4160 = 901.4 \text{ KN (control)}$$

$$V_{c2} = \frac{\sqrt{f_c'} \times b \times d}{4} + \frac{N_u \times d}{4 \times L_w}$$

$$N_u = 0.0 \text{ KN}$$

$$V_{c2} = \frac{\sqrt{24} \times 200 \times 4160}{4} + 0.0 = 1273.72 \text{ KN}$$

$$Mu(1) = 1656 + 358 * (3.75 - 2.60) = 1745.5 \text{ kN.m}$$

$$V_{c3} = \left[\frac{\sqrt{f_c'}}{2} + \frac{l_w \left(\sqrt{f_c'} + \frac{2 \times N_u}{l_w \times h} \right)}{\left\langle \frac{Mu(1)}{V_u} - \frac{l_w}{2} \right\rangle} \right] \times \frac{h \times d}{10}$$

$$V_{c3} = \left[\frac{\sqrt{24}}{2} + \frac{5.2(\sqrt{24} + 0.0)}{\left\langle 358 - \frac{5.2}{2} \right\rangle} \right] \times \frac{200 \times 4160}{10} = 2542.67 \text{ KN}$$

So thickness of wall is safe.

✓ Design for horizontal reinforcement :

$$A_{vh} \text{ min.} = 0.0025 * s * h$$

$$A_{vh} = 2 \Phi 10 = 158 \text{ mm}^2$$

$$\left(\frac{2 * 79}{s} \right) = 0.5$$

$$S = 316 \text{ mm}$$

$$S_{\max} \leq L_w/5 = 5200/5 = 1040 \text{ mm}$$

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

$$\leq 3 * h = 3 * 200 = 600 \text{ mm} \quad \text{Take } s = 300 \text{ mm} < s_{\max}$$

Select $\Phi 10 @ 15 \text{ cm}$

✓ Design for Vertical reinforcement:-

$$A_{vv} = \left\{ 0.0025 + 0.5 \left(2.5 - \frac{h_w}{l_w} \right) * \left(\frac{A_{vh}}{S_2 * h} - 0.0025 \right) \right\} * s * h$$

$$A_{vh} = 2 \Phi 10 = 158 \text{ mm}^2$$

$$A_{vv} = \left\{ 0.0025 + 0.5 \left(2.5 - \frac{11.25}{5.2} \right) * \left(\frac{2 * 79}{300 * 200} - 0.0025 \right) \right\} * s * 200$$

$$A_{vv} = 0.0025 * s * h$$

$$\left(\frac{A_{vv}}{s} \right) = 0.53$$

$$A_{vv} = 2 \Phi 10 = 158 \text{ mm}^2 \quad \text{-----} > \quad s = 298$$

$$S_{\max} \leq L_w/3 = 5200/3 = 1733 \text{ mm} \quad \leq 450 \text{ mm}$$

$$\leq 3 * h = 3 * 200 = 600 \text{ mm} \quad \text{Take } s = 250 \text{ mm} < s_{\max}$$

Select $\Phi 12 - 15 \text{ cm}$

4.11 Design of Basement wall:

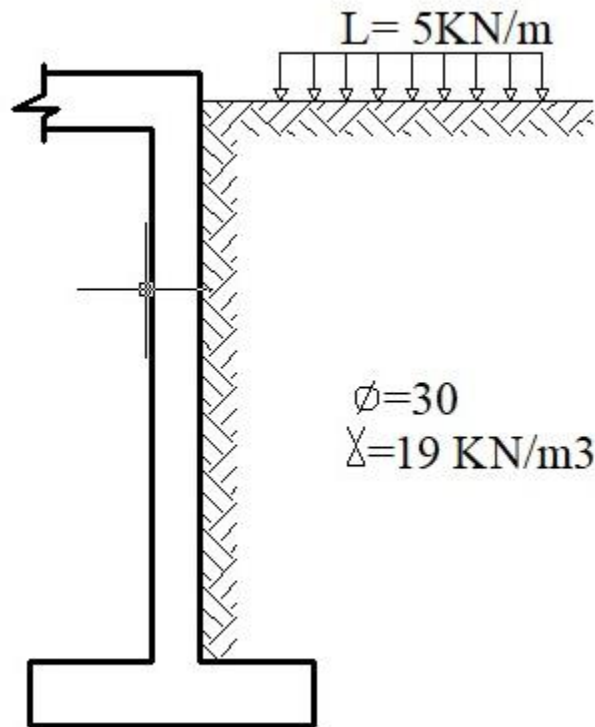


Figure (4-9): Geometry of basement.

Material:-

Concrete B350 $F_c' = 24 \text{ MPa}$
Reinforcement Steel $f_y = 420 \text{ MPa}$

$\phi = 30^\circ$ $\gamma = 19.0 \text{ kN/m}^3$

- Soil at rest

$$\begin{aligned} K_o &= 1 - \sin \phi \\ &= 1 - \sin 30 \\ &= 0.50 \end{aligned}$$

4.10.1) Load on basement wall:

For 1m length of wall:

- Weight of backfill:

$$\begin{aligned} e &= K_o * \gamma * h \\ &= 0.50 \times 19.0 \times 5.85 = 55.5 \text{ KN/m} \end{aligned}$$

$$q_1 (\text{Factored}) = 1.6 \times e$$

$$q_1 (\text{Factored}) = 1.6 \times 55.5 = 88.8 \text{ KN/m}$$

- Load from live load:

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

$$\begin{aligned} q_2 &= K_o \times LL \\ &= 0.50 \times 5 = 2.50 \text{ KN/m} \end{aligned}$$

$$q_2 (\text{Factored}) = 1.6 \times 2.50 = 4.0 \text{ KN/m}$$

4.10.2) Design of the shear force:

- Assume $\phi 12$ for main reinforcement
- Assume $h = 300 \text{ mm}$,

$$d = 300 - 20 - 12 = 268 \text{ mm}$$

By using **ATIR** program, we get the envelope moment and shear force diagram

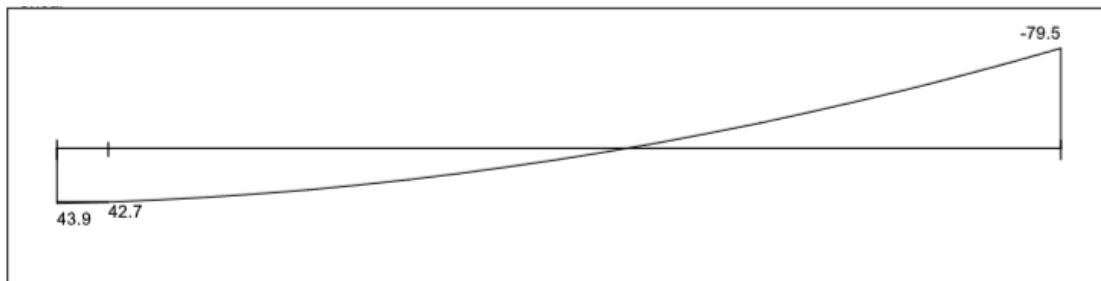


Figure (4.10) shear of basement

Max $V_u = 79.5$ KN.

$$\phi V_c = \frac{\phi \sqrt{f'_c} * b_w * d}{6}$$

$$\phi V_c = \frac{0.75 \times \sqrt{28} \times 1000 \times 268}{6} = 177.27 \text{ KN}$$

$V_u = 79.5 \text{ KN} < \phi V_c = 177.27 \text{ KN}$.

No shear Reinforcement is required.

4.10.3) Design of bending moment:

By using **ATIR** program, we get the envelope moment and moment force diagram

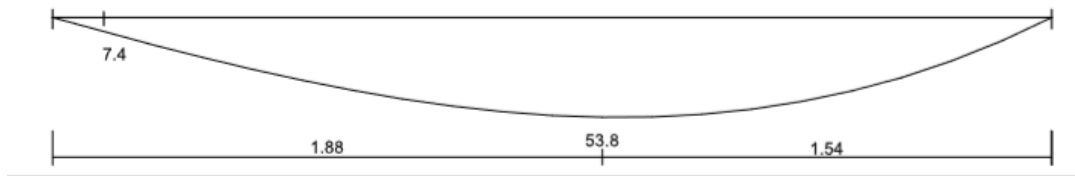


Figure (4.11) moment of basement

$M_u \text{ max} = 53.8 \text{ KN.m}$

$$M_n = \frac{M_u}{0.9} = \frac{53.8}{0.9} = 59.8 \text{ KN.m}$$

$$R_n = \frac{M_n \times 10^6}{b \times d^2} = \frac{59.8 \times 10^6}{1000 \times 268^2} = 0.832 \text{ Mpa}$$

$$m = \frac{F_y}{0.85 \times f'_c} = \frac{420}{0.85 \times 24} = 20.58$$

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

$$= \frac{1}{20.58} \times \left(1 - \sqrt{1 - \frac{2 \times 0.832 \times 20.58}{420}} \right)$$

$$= 2.02 \times 10^{-3}$$

$$A_{sreq} = \rho \times b \times d = 2.02 \times 10^{-3} \times 1000 \times 268 = 5.4049 \text{ cm}^2/\text{m}$$

$$A_{smin} = 0.0012 \times b \times h = 0.0012 \times 1000 \times 300 = 3.60 \text{ cm}^2/\text{m}$$

$$A_{smin} = 3.60 \text{ cm}^2/\text{m} \leq A_{sreq} = 5.4049 \text{ cm}^2/\text{m}$$

Use $\Phi 12@ 20\text{cm}$

$$A_s \text{ provided} = 5.65 \text{ cm}^2/\text{m} > A_{sreq} = 5.4049 \text{ cm}^2/\text{m}.$$

Step(s) is the smallest of :

- $3h = 3 \times 200 = 600\text{mm}$.
- 450mm
- $S = 380 \left(\frac{280}{f_s} \right) - 2.5c_c = 380 \left(\frac{280}{280} \right) - 2.5 \times 20 = 330\text{mm}$.

$$S = 200\text{mm} < S_{max}$$

Select $\Phi 12@ 15\text{cm/m}$ in both direction.

$$\text{With } a_s = 5.65 \text{ cm}^2/\text{m}$$

4.10.4) Design of the horizontal reinforcement:

$$A_{s(min)} = 0.0012 \times b \times h = 0.002 \times 1000 \times 300 = 360 \text{ cm}^2/\text{m}$$

Select $\Phi 12@ 20\text{cm/m}$, in two layer.

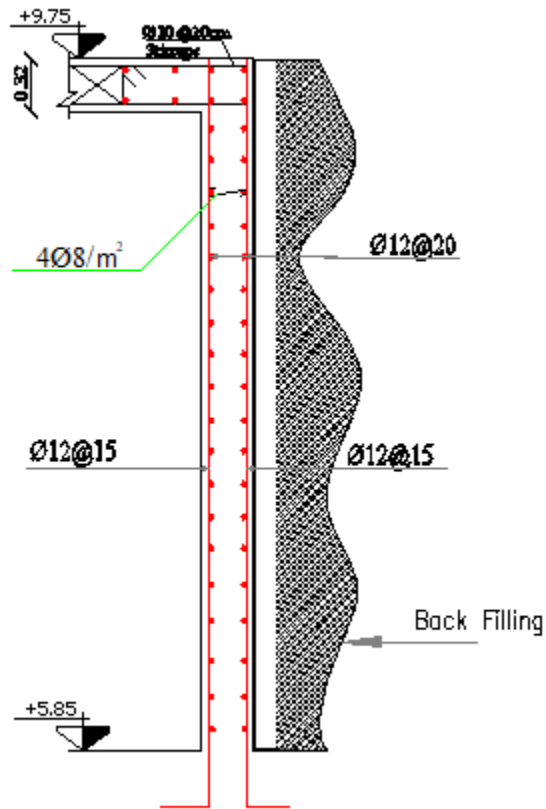


Figure (4.12): Reinforcement for basement wall.

4.12 Design of Footing:**❖ Material :-**

⇒ concrete B350 $F_c' = 24 \text{ MPa}$

⇒ Reinforcement Steel $F_y = 420 \text{ MPa}$

✓ Load Calculations :-

Dead Load = 638.3 KN , Live Load = 252.6 KN

Total Factored load = $1.2 \times 638.3 + 1.6 \times 252.6 = 1170.12 \text{ KN}$

Column Dimensions (a*b) = 25*50 cm

Soil density = 19 Kg/cm³

Allowable Bearing Capacity = 400 KN/m²

Assume h = 50cm

$q_{net-allow} = 400 - 25 \times 0.5 - 19 \times 0.5 - 25 \times 0.7 = 360.5 \text{ KN/m}^2$

✓ Area of Footing :-

$$A = \frac{Pt}{q_{net-allow}} = \frac{890.9}{360.5} = 2.47 \text{ m}^2$$

Assume the area = 1.85 * 1.60 = 2.88 m²

✓ Bearing Pressure :-

$q_u = 1170.12 / 1.85 \times 1.60 = 406.3 \text{ KN/m}^2$

✓ **Design of Footing :-**

✓ **Design of One Way Shear Strength :-**

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

Assume h = 50cm , bar diameter ϕ 14 for main reinforcement and 7.5 cm Cover

$$V_u = \left(\frac{l}{2} - \frac{a}{2} - d \right) * q_u * b = \left(\frac{1.85}{2} - \frac{0.50}{2} - d \right) * 406.3 * 1.6$$

$$\phi V_c = \frac{0.75}{6} \sqrt{24} * 1.6 * d * 10^3$$

$$\text{Let, } \phi V_c = V_u$$

$$d = 0.448m$$

$$h = 448 + 75 + 14 = 537mm$$

$$\text{Try } h = 550 \text{ mm} \dots d = 550 - 75 - 14 = 461 \text{ mm}$$

✓ **Design of Two Way Shear Strength :-**

$$V_u = P_u - FR_b$$

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

$$V_u = 1170.12 - 406.3[(0.5 + 0.461) * (0.25 + 0.461)] = 892.5KN$$

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:-

$$\beta_c = \frac{\text{Column Length (a)}}{\text{Column Width (b)}} = \frac{50}{25} = 2$$

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

$$b_o = 2 * (46.1 + 50) + 2 * (46.1 + 25) = 334.4 \text{ cm}$$

$\alpha_s = 40$ for interior column

$$\phi.V_c = \phi \cdot \frac{1}{6} \left(1 + \frac{2}{\beta_c} \right) \sqrt{f'_c} b_o d = \frac{0.75}{6} * \left(1 + \frac{2}{2} \right) * \sqrt{24} * 3344 * 461 = 2215.4 \text{ KN}$$

$$\phi.V_c = \phi \cdot \frac{1}{12} \left(\frac{\alpha_s}{b_o/d} + 2 \right) \sqrt{f'_c} b_o d = \frac{0.75}{12} * \left(\frac{40 * 461}{3344} + 2 \right) * \sqrt{24} * 3344 * 461 = 1107 \text{ KN}$$

$$\phi.V_c = \phi \cdot \frac{1}{3} \sqrt{f'_c} b_o d = \frac{0.75}{3} * \sqrt{24} * 3344 * 461 = 2215 \text{ KN}$$

$$\Phi V_c = 2215 \text{ KN} > V_u = 1170 \text{ KN}$$

✓ Design of Bending Moment :-

Critical Section at the Face of Column

$$FR = q_u * \left(\frac{B-a}{2}\right) * L = 406.3 * \left(\frac{1.85-0.25}{2}\right) * 1.6 = 520 \text{ KN}$$

$$M_u = 520 * 0.465 = 245.7 \text{ KN.m}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{245.7 \times 10^6}{0.9 \times 1850 \times 461^2} = 0.54 \text{ Mpa}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{420}} \right) = \frac{1}{17.6} \left(1 - \sqrt{1 - \frac{2 \times 20.59 \times 0.54}{420}} \right) = 0.00152$$

$$A_{s, \text{req}} = \rho \cdot b \cdot d = 0.00152 \times 1850 \times 461 = 1480 \text{ mm}^2$$

$$A_{s, \text{min}} = 0.0018 \times 1850 \times 550 = 2052 \text{ mm}^2$$

$$A_{s, \text{req}} < A_{s, \text{min}} = 2052 \text{ mm}^2 \text{ is control}$$

Check for Spacing :-

$$S = 3h = 3 \times 55 = 165 \text{ cm}$$

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

$$S = 45 \text{ cm is control}$$

Use 16ø14, $A_{s, \text{provided}} = 2618 \text{ mm}^2 > A_{s, \text{required}} = 2052 \text{ mm}^2$... Ok

And In Another Direction Use 8 ø14

✓ Development Length In Footing :-

Tension Development Length In Footing :-

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

$$cb = 75 + \frac{14}{2} = 82\text{mm} \text{ Or } cb = \frac{150}{2} = 75\text{ mm}$$

$$\frac{ktr + cb}{db} = \frac{0 + 75}{14} = 5.3 > 2.5$$

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

$$L_{d_{req}} = \frac{9}{10} * \frac{420}{1 * \sqrt{24}} * \frac{1 * 1 * 0.8}{2.5} * 16 = 395.75\text{ mm} > 300\text{mm}$$

$$L_{d_{T \text{ available}}} = \frac{1850 - 500}{2} - 75 = 625\text{ mm}$$

$$L_{d_{T \text{ available}}} = 625\text{ mm} > L_{d_{req}} = 395.054\text{ mm} \dots\dots \text{OK}$$

Compression Development Length In Footing :-

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

$$L_{d_{Creq}} = \frac{0.24 * 420 * 14}{\sqrt{24}} = 288.05 > 0.043 * 420 * 14 = 252.84 > 200\text{mm}$$

$$L_{d_{C \text{ available}}} = 550 - 75 - 14 - 14 = 497\text{mm} > L_{d_{Creq}} = 288.0\text{ mm} \dots\dots \text{Ok}$$

Lap Splice of Dowels In Column :-

$$L_{sc} = 0.071 * f_y * db = 0.071 * 420 * 14 = 477.48\text{ mm} > 300\text{ mm}$$

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