

4

Chapter Four

Structural Analysis and Design

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4-1 Introduction

Concrete is a construction material composed of cement (commonly Portland cement) as well as other cementations materials such as fly ash and slag cement, aggregate (generally a coarse aggregate such as gravel, limestone, or granite, plus a fine aggregate such as sand), water, and chemical admixtures. The word concrete comes from the Latin word "concretus", which means "hardened" or "hard".

Concrete solidifies and hardens after mixing with water and placement due to a chemical process known as hydration. The water reacts with the cement, which bonds the other components together, eventually creating a stone-like material. Concrete is used to make pavements, architectural structures, foundations, motorways/roads, bridges/overpasses, parking structures, brick/block walls and footings for gates.

In This Project, there are three types of slabs: solid slabs, one-way ribbed and two-way ribbed slabs. They would be analyzed and designed by using finite element method of design, with aid of a computer Program called " ATIR- Software" to find the internal forces, deflections and moments for ribbed slabs, and then hand calculation would be made to find the required steel for some members.

The design strength provided by a member, its connections to other members, and its cross-sections in terms of flexure, and load, shear, and torsion is taken as the nominal strength calculated in accordance with the requirements and assumptions of ACI-code.

In this project we use concrete type B300 with compressive strength $f_c' = 30 \text{ N/mm}^2 (\text{MPa})$ for circular section and $f_c' = 30 * 0.8 = 24 \text{ MPa}$ for rectangular section, and reinforcement steel with specified yield strength $\{f_y = 420 \text{ N/mm}^2 (\text{MPa})\}$.

4.2 Factored Loads :-

According to ACI-Code-318-11, the minimum thickness of nonprestressed beams or one way slabs unless deflections are computed as follow:

$$W_u = 1.2 D_L + 1.6 L_L \quad \text{ACI-code-318-11(9.2.1)}$$

4.3 Determination of Thickness

4.3.1 Determination of Thickness for One Way Rib Slab:

According to ACI-Code-318-05, the minimum thickness of nonprestressed beams or one way slabs unless deflections are computed as follow:

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

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

-The maximum span length for one- end continuous (for ribs):

$$h_{\min} = \frac{L}{18.5} = \frac{680}{18.5} = 36.75 \text{ cm}$$

-The maximum span length for both -end continuous (for ribs):

$$h_{\min} = \frac{L}{21} = \frac{500}{21} = 23.8 \text{ cm}$$

-The maximum span length for cantilever (for ribs):

$$h_{\min} = \frac{L}{8} = \frac{210}{8} = 26.25 \text{ cm}$$

Take slab thickness $h=35$ cm. (deflection will be checked)

$h=35$ cm (27cm Hollow block + 8cm Topping).

4.3.2 Determination of Thickness for Two Way Rib Slab:

****In Two way rib we use kalkal block instead of concrete block with dimension (50*20) cm.**

$$I_b = \frac{bh^3}{12} = \frac{80(40)^3}{12} = 426667$$

$$I_{rib} = 136533$$

$$I_{s1} = \frac{136533}{64} \times (1040/2 + 80) = 1279997$$

$$\alpha_1 = \frac{I_b}{I_s} = \frac{426667}{1279997} = 0.33$$

$$I_{s2} = \frac{136533}{64} \times (1170/2 + 80) = 1418663$$

$$\alpha_2 = \frac{I_b}{I_s} = \frac{426667}{1418663} = 0.3$$

$$\alpha_{fm} = 0.315$$

$$0.2 < \alpha_{fm} < 2$$

$$h_{\min} = \frac{Ln(0.8 + f_y/1400)}{36 + 5\beta(\alpha_{fm} - 0.2)}$$

$$\beta = \frac{11.7}{10.4} = 1.125$$

$$h_{\min} = \frac{Ln(0.8 + f_y/1400)}{36 + 5\beta(\alpha_{fm} - 0.2)} = \frac{11700(0.8 + \frac{412}{1400})}{36 + 5 \times 1.125 \times (0.315 - 0.2)} = 34.94 \text{ cm}$$

Select $h_{\min} = 40 \text{ cm}$

4-4 –Load Calculation

4.4.1: One - way ribbed slab.

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

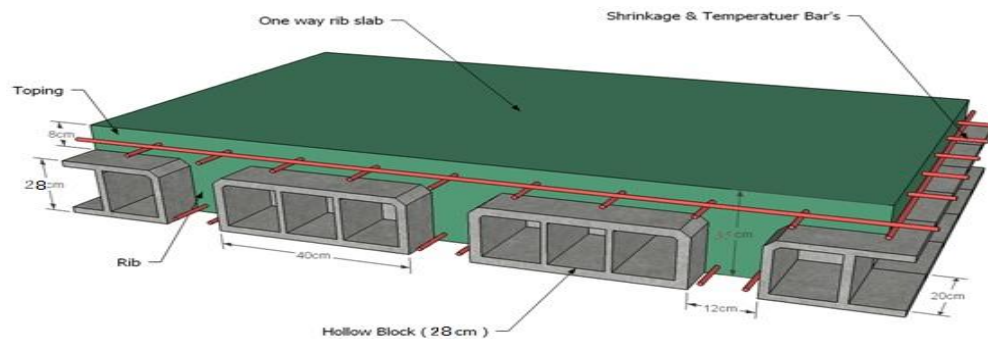


Fig. (4-1) One way rib slab

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

No.	Parts of Rib	Density	Calculation
1	Rib	25	$0.12 \times 0.27 \times 25 = 0.81 \text{ KN/m}$
2	Top Slab	25	$0.08 \times 0.52 \times 25 = 1.04 \text{ KN/m}$
3	Plaster	22	$0.03 \times 0.52 \times 22 = 0.3432 \text{ KN/m}$
4	Block	10	$0.27 \times 0.4 \times 10 = 1.08 \text{ KN/m}$
5	Sand Fill	17	$0.07 \times 0.52 \times 17 = 0.6188 \text{ KN/m}$
6	Tile	23	$0.03 \times 0.52 \times 23 = 0.3588 \text{ KN/m}$
7	Mortar	22	$0.03 \times 0.52 \times 22 = 0.3432 \text{ KN/m}$
8	partition		$2.3 \times 0.52 = 1.196 \text{ KN/m}$
			5.79 KN/m

Table (4 – 2) Calculation of the total dead load for one way rib slab.

Nominal Total Dead Load:

$$D.L._{\text{total}} = 0.81 + 1.04 + 0.3432 + 1.08 + 0.6188 + 0.3588 + 0.3432 + 1.196 = 5.79 \text{ KN/m of rib}$$

$$\text{Live load} = 5 \times 0.52 = 2.6 \text{ KN/m of rib}$$

4.4.2: Two-way ribbed slab :

Calculation of the total dead load for two way rib slab "Kalkal (50*20)" is shown in the following table:

Dead load:

No.	Parts of Rib	Density	Calculation
1	Tiles	22	$0.03 \times 0.54 \times 0.64 \times 22 = 0.2281$
2	Mortar	22	$0.02 \times 0.54 \times 0.64 \times 22 = 0.1521$
3	Coarse Sand fill	16	$0.07 \times 0.54 \times 0.64 \times 16 = 0.3871$
4	Topping	25	$0.08 \times 0.54 \times 0.64 \times 25 = 0.6912$
5	Concrete Rib	25	$0.32 \times (0.4 + 0.64) \times 0.14 \times 25 = 1.1648$
6	Block	9	$0.06 \times 0.5 \times 0.4 \times 9 = 0.108$

7	Kalkal	0.15	$0.15 \times 0.26 \times 0.5 \times 0.4 = 0.0078$
8	Plaster	22	$0.02 \times 0.64 \times 0.54 \times 22 = 0.1521$
9	partition		$0.54 \times 0.64 \times 1.5 = 0.5184$
			3.4096 kN/m

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

Nominal Total Dead Load = 3.4096 kN/rib

$WuD = 1.2 \times 3.4096 / (0.54 \times 0.64) = 11.84 \text{ KN}$

$WuL = 1.6 \times 5 = 8 \text{ kN/m}^2$

4.5 Design of Topping

Dead load calculation:

No.	Material	Calculation
1	Tile	$0.03 \times 23 \times 1 = 0.69 \text{ KN/m}$
2	mortar	$0.03 \times 22 \times 1 = 0.66 \text{ KN/m}$
3	Coarse sand	$0.07 \times 17 \times 1 = 1.19 \text{ KN/m}$
4	topping	$0.08 \times 25 \times 1 = 2.0 \text{ KN/m}$
5	Interior partitions	$2 \times 1 = 2 \text{ KN/m}$
Sum		6.84 kN/m

Table (4 – 4) Calculation of the total dead load on topping

Live load = $5 \times 1 \text{ KN/m}$

$$W_u = 1.2DL + 1.6L = (1.2 * 6.84) + (1.6 * 5) = 16.808 \text{ KN/m. (Total Factored load)}$$

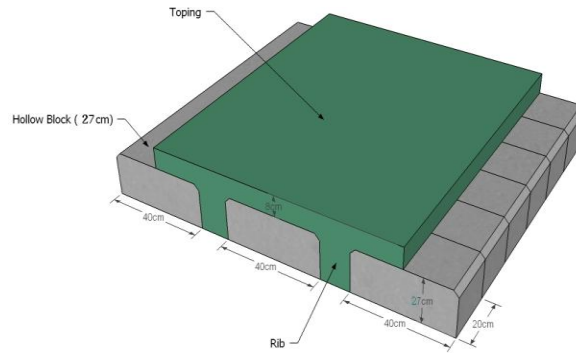


Fig. (4-2) Topping of slab

Assume slab fixed at supported points (ribs):

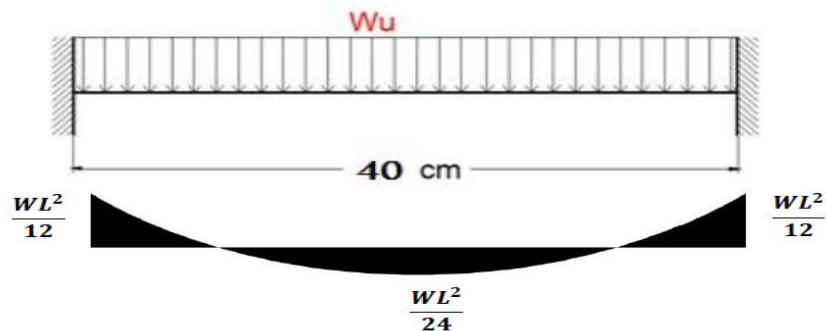


Fig 4.3: Topping load and moment diagram.

$$M_u = \frac{W_u * l^2}{12}$$

$$= \frac{16.208 * 0.4^2}{12} = 0.216 \text{ KN.m /m of strip width .}$$

$(\Phi M)_n > M_u$ [Strength Condition, where $\Phi = 0.55$] for plane concrete

$$M_n = 0.42 \times \sqrt{f_c'} S m \quad \text{ACI-318-14 (22-5.1)}$$

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

$$M_n = 0.42 \times \sqrt{24} * 1066666.67 = 2.194 \text{ KN.m}$$

$$\Phi M_n = 0.55 * 2.194 = 1.2 \text{ KN.m}$$

$$\Phi M_n = 1.2 \text{ KN.m} > M_u = 0.216 \text{ KN.m}$$

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

According to ACI 7.12.2.1, ρ shrinkage = 0.0018

$$A_{s_{\min}} = \rho * b * h = 0.0018 * 1000 * 80 = 144 \text{ mm}^2 / 1\text{m}$$

Try bars $\Phi 8$ with $A_s = 50.26 \text{ mm}^2$

$$\text{No. of } \Phi 8 = \frac{A_{s_{\text{req}}}}{A_{\text{bar}}} = \frac{144}{50.26} = 2.86 \rightarrow \text{Spacing}(S) = \frac{1}{2.86} = 0.348 \text{ m} = 348 \text{ mm}$$

$$\leq 380 \left(\frac{280}{f_s} \right) - 2.5 * C_c \leq 300 \left(\frac{280}{f_s} \right)$$

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

$$S = 300 * \left(\frac{280}{\frac{2}{3} * 420} \right) = 300 \text{ mm}$$

Not more than: $S_{\max} = 450 \text{ mm}$

Use $\Phi 8 / 20 \text{ cm}$, with $A_s_{\text{provided}} = 251 \text{ mm}^2/\text{m}$ both directions.

4.6 Design of One Way Rib Slab

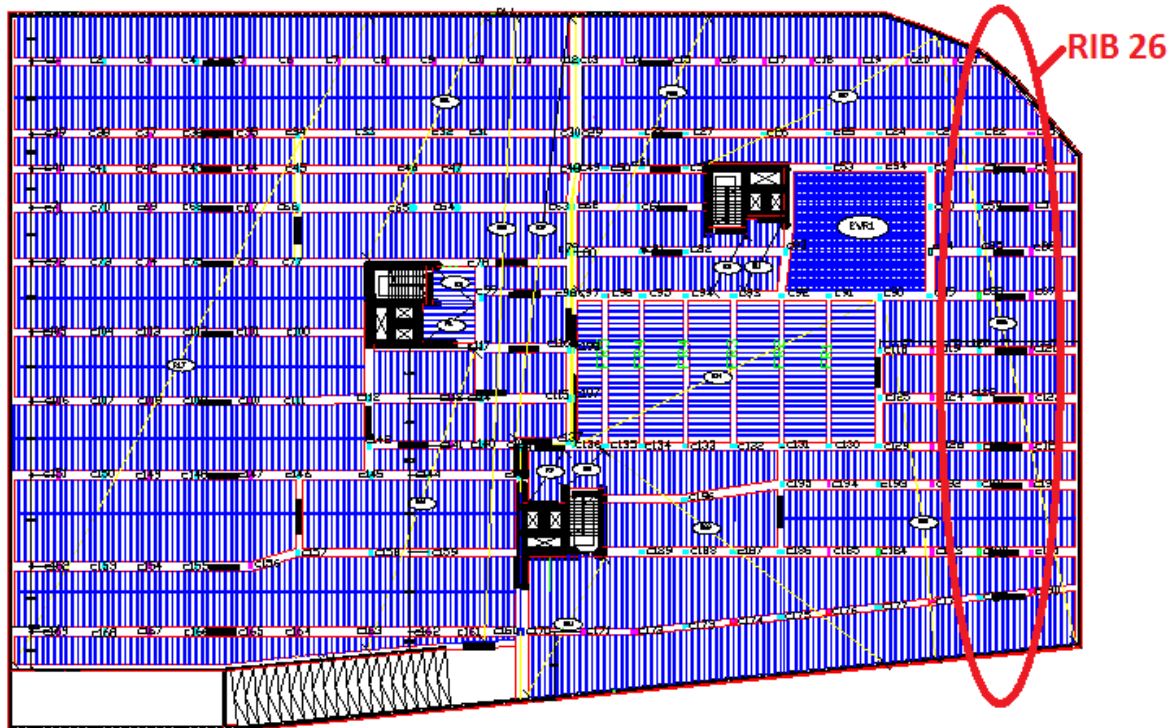


Fig. (4 – 4) Rib 26

❖ /Material :-

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

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

Geometry Units: meter, cm

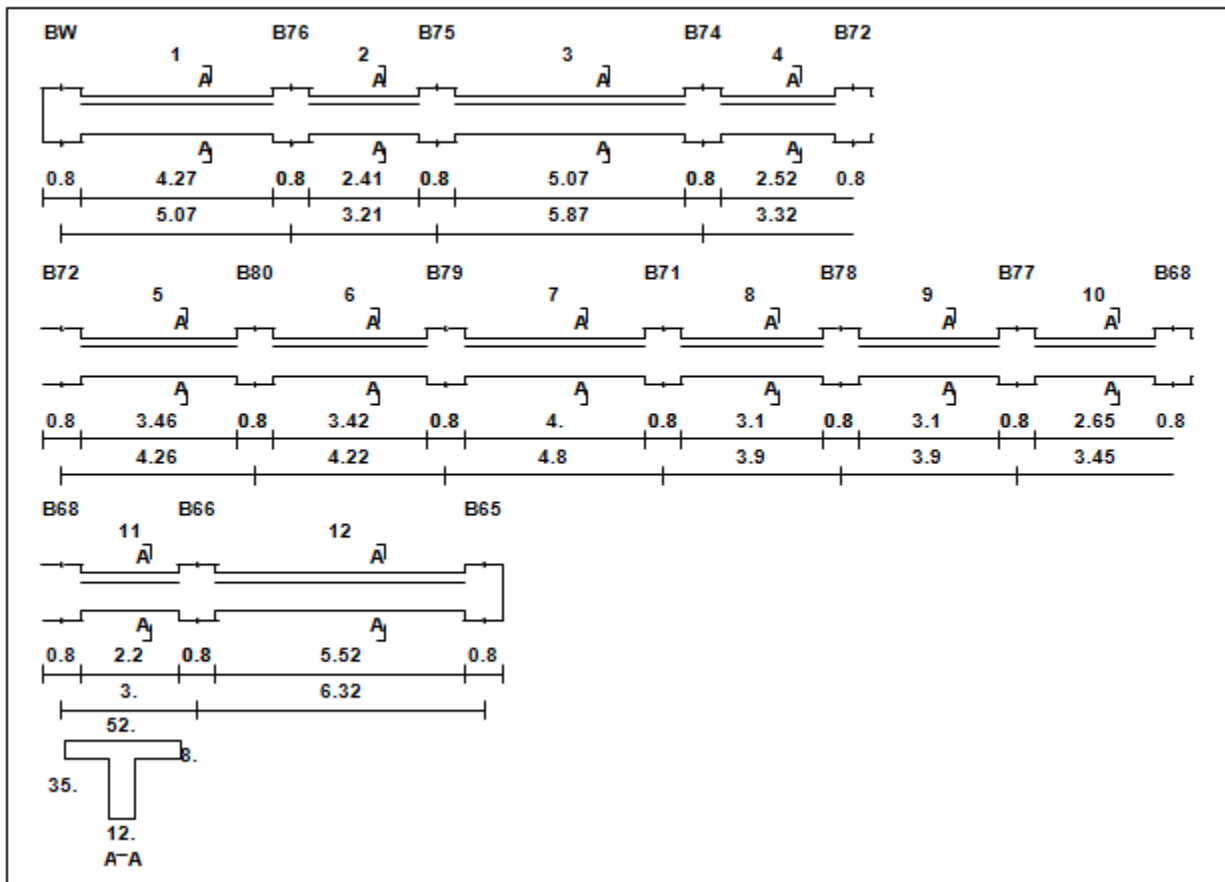


Figure (4-5): Rib geometry.

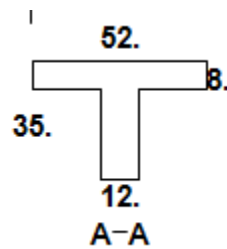


Figure (4-6) : Rib Section

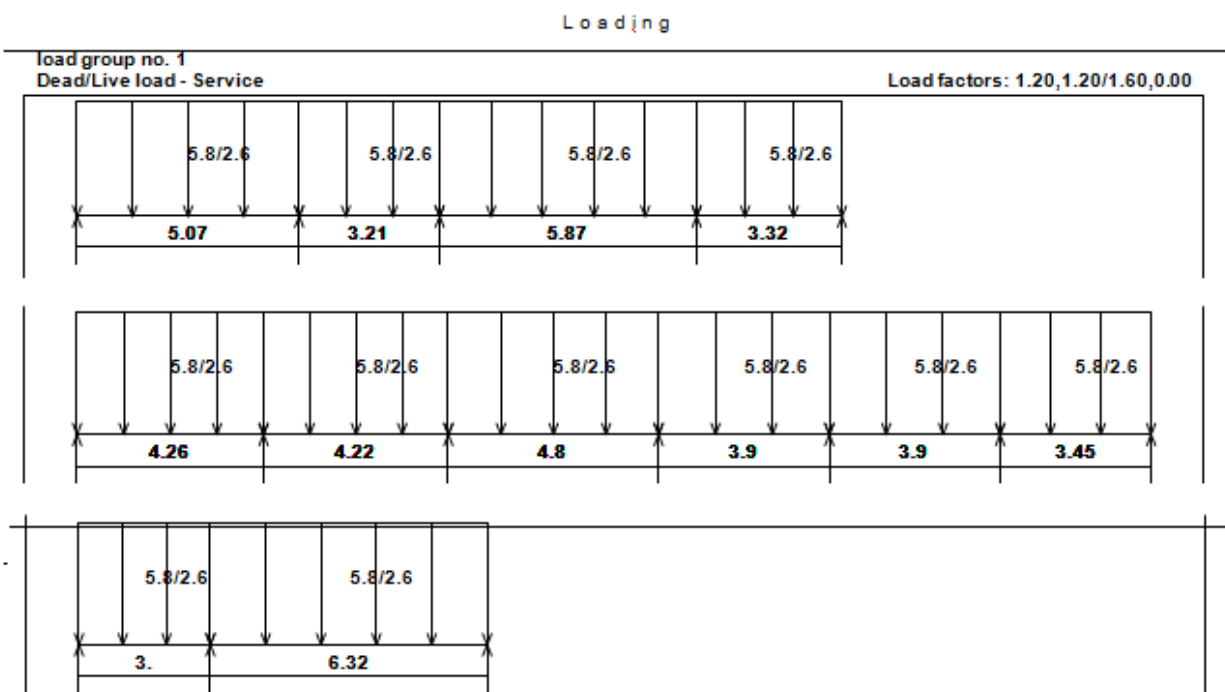


Figure (4-7) : loading of Rib 26

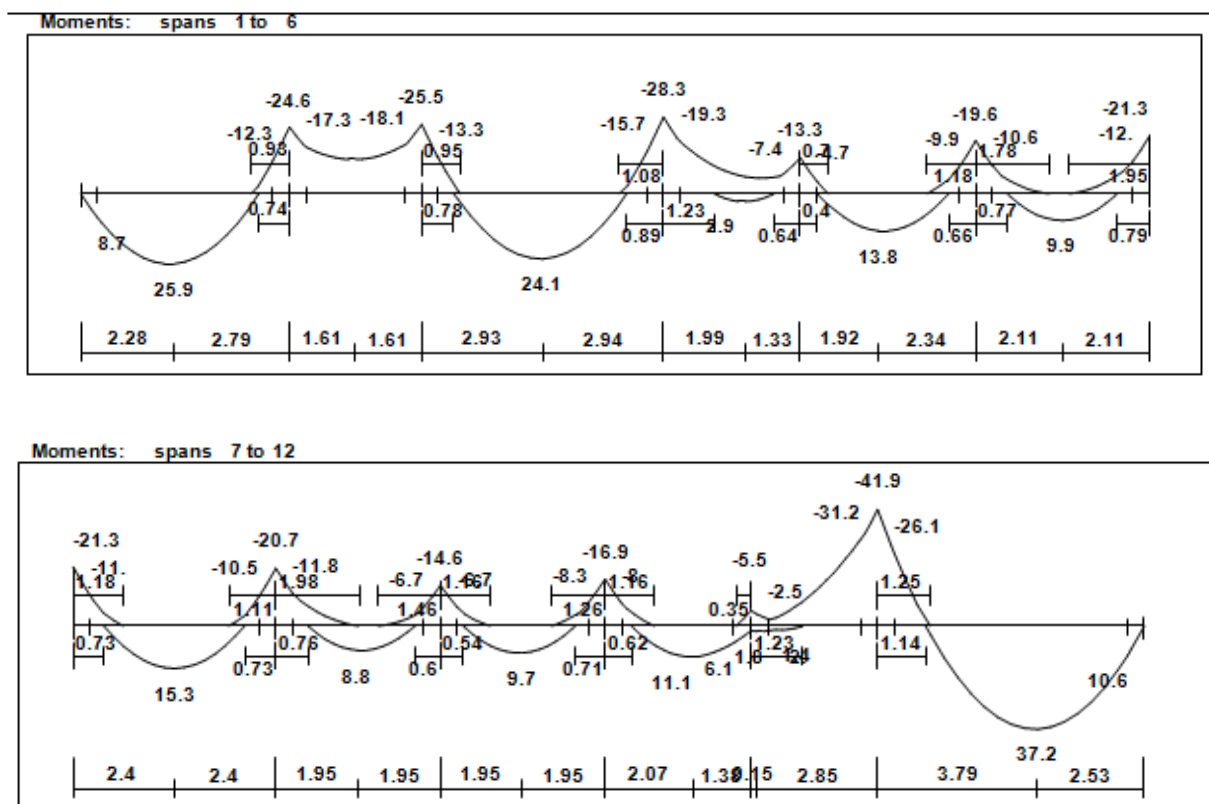


Figure (4-8) : Moment Envelop of rib26.

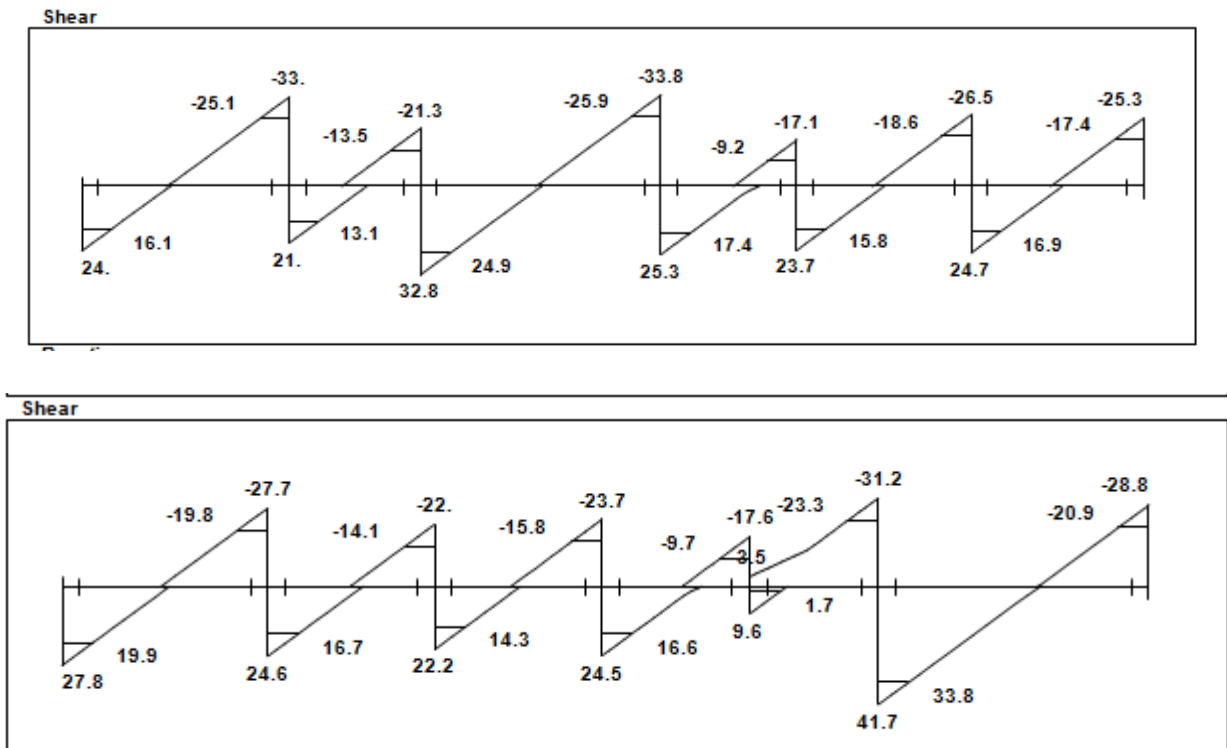


Figure (4-9) : Shear Envelop of rib26.

Design of flexure :-

$$d = h - \text{cover} - (db/2) = 350 - 20 - 8 = 322 \text{ mm}$$

$$Mu(+) = 37.2 \text{ kN.m}$$

Check if $a > hf$

$$Mnf = 0.85 f_c' b h_f (d - hf/2) = 0.85 * 24 * 520 * 80 (322 - 80/2) * 10^{-6} = 239.3 \text{ KN.m}$$

$$\phi * Mn > mu$$

$$215.37 > 37.2$$

→ The section will be designed as rectangular section with $b = 520 \text{ mm}$.

Design of Negative moment of rib 18:

Maximum negative moment $M_u = 31.2 \text{ kN.m}$

$$M_n = 31.2 / 0.9 = 34.7 \text{ kN.m}$$

$$m = \frac{f_y}{0.85 * f_c'} = \frac{412}{0.85 * 24} = 20.2$$

$$R_n = \frac{M_n}{b * d^2} = \frac{34.7 * 10^{-3}}{0.12 * (0.322)^2} = 2.79 \text{ Mpa}$$

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

$$\rho = \frac{1}{20.2} \left(1 - \sqrt{1 - \frac{2(20.2)(2.79)}{412}} \right) = 0.007312$$

$$A_s = 0.007312 (120) (322) = 282.5 \text{ mm}^2$$

$$A_{s_{\min}} = \frac{\sqrt{f_c'}}{4(f_y)} (b_w)(d) \geq \frac{1.4}{f_y} (b_w)(d) \dots\dots\dots (ACI - 10.5.1)$$

$$A_{s_{\min}} = \frac{\sqrt{24}}{4(412)} (120)(322) \geq \frac{1.4}{412} (120)(322)$$

$$A_{s_{\min}} = 114.86 < 131.3 \dots\dots\dots \text{the larger is control}$$

$$A_{s_{\min}} = 131.30 \text{ mm}^2$$

$$282.5 \text{ mm}^2 > A_{s_{\min}} = 131.30 \text{ mm}^2$$

$$\# \text{ of bars} = A_s / A_{s_{\text{bar}}} = 282.5 / 153.93 = 2 \text{ bars}$$

$$* \text{ Note } A_{\Phi 14} = 153.93 \text{ mm}^2$$

$$A_s \text{ provided} = 307.87 \text{ mm}^2$$

Select 2 Φ 14mm .

- *Check for yielding*

Tension = compression

$$A_s * f_y = 0.85 * f_c * b * a$$

$$307.87 * 412 = 0.85 * 120 * 24 * a$$

$$a = 51.8 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{51.8}{0.85} = 60.95 \text{ mm} \quad \text{OK}$$

$$\varepsilon_s = \frac{322 - 60.95}{60.95} \times 0.003$$

$$\varepsilon_s = 0.0128 > 0.005$$

Design of Positive moment of rib 18

Maximum positive moment is $M_u = 37.2 \text{ kN.m}$

$$M_n = 37.2 / 0.9 = 41.3 \text{ kN.m}$$

$$m = \frac{f_y}{0.85 * f_c'} = \frac{412}{0.85 * 24} = 20.2$$

$$R_n = \frac{M_n}{b * d^2} = \frac{41.3 * 10^{-3}}{0.52 * (0.322)^2} = 0.766 \text{ Mpa}$$

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

$$\rho = \frac{1}{20.2} \left(1 - \sqrt{1 - \frac{2(0.766)(20.2)}{412}} \right) = 0.0019$$

$$A_s = 0.0019 (520) (322) = 317.38 \text{ mm}^2$$

$$A_{s_{\min}} = \frac{\sqrt{f_c'}}{4(f_y)} (b_w)(d) \geq \frac{1.4}{f_y} (b_w)(d) \dots\dots\dots (ACI - 10.5.1)$$

$$A_{s_{\min}} = \frac{\sqrt{24}}{4(412)} (120)(322) \geq \frac{1.4}{412} (120)(322)$$

$$A_{s_{\min}} = 114.86 < 131.30$$

$$A_{s_{\min}} = 131.30 \text{ mm}^2$$

$$317.38 \text{ mm}^2 > A_{s_{\min}} = 131.30 \text{ mm}^2$$

$$\# \text{ of bars} = A_s / A_{s_{\text{bar}}} = 317.38 / 201.06 = 2 \text{ bars}$$

$$* \text{ Note } A_{\Phi 16} = 201.06 \text{ mm}^2$$

$$A_s \text{ providing} = 402.12 \text{ mm}^2$$

Select 2 Φ 12mm .

- **Check for yielding**

Tension = compression

$$A_s * f_y = 0.85 * b * a$$

$$402.12 * 412 = 0.85 * 520 * 24 * a$$

$$a = 15.62 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{15.62}{0.85} = 18.38 \text{ mm}$$

$$\epsilon_s = \frac{322 - 18.38}{18.38} \times 0.003$$

$$\epsilon_s = 0.0496 > 0.005 \text{ OK}$$

Design of shear:

$$V_u = 25.9$$

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

$$= 0.75 * \frac{\sqrt{24}}{6} 0.12 * 0.322 * 1000$$

$$= 23.66 \text{ KN}$$

$$1.1 * \Phi V_c = 1.1 * 23.66 = 26.026 \text{ KN.}$$

Check for items:-

$$1/ \quad V_u \leq \Phi V_c / 2$$

$$25.9 \leq 13.013 \quad (\text{ X })$$

$$2/ \quad \Phi V_c / 2 \leq V_u \leq \Phi V_c$$

$$13.013 \leq 25.9 \leq 23.66 \quad (\text{ X })$$

$$3/ \quad \Phi V_c \leq V_u \leq \Phi V_c + \Phi V_{smin}$$

$$23.66 \leq 25.9 \leq 33.32 \quad (\checkmark)$$

$$\Phi V_{smin} \geq 0.75 \left(\frac{1}{3} \right) * b_w * d = 0.75 * \left(\frac{1}{3} \right) * 0.12 * 0.322 * 1000 = \mathbf{9.66 \text{ KN.}} \quad (\text{control})$$

$$\geq 0.75 \left(\frac{\sqrt{24}}{16} \right) * b_w * d = 0.75 * \frac{\sqrt{24}}{16} * 0.12 * 0.322 * 1000 = 8.87 \text{ kn.}$$

$$\Phi V_{smin} = 9.4 \text{ KN.}$$

So Case (3) satisfy

$$\text{Take } A_v = 2 \Phi 8 = 2 * 50$$

$$A_v / s = V_s / f_y * d$$

$$2 * 50 / s = 2.98 / 322 * 412 \quad \rightarrow s = 445 \text{ mm}$$

$$S \leq d / 2 = 161 \text{ mm}$$

$$\leq 600 \text{ m.}$$

Use $\Phi 8 @ 15 \text{ cm c/c}$

4-7 - Design of Two way rib slab :-

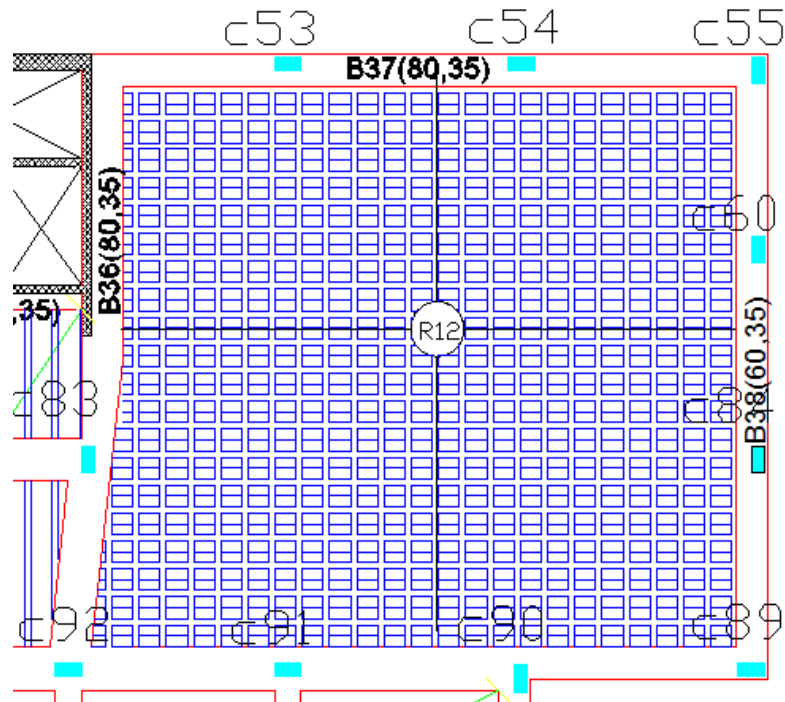


Figure (4-10): Two way ribbed slab

❖ Material :-

- ⇒ concrete B300 $F_c' = 24 \text{ N/mm}^2$
- ⇒ Reinforcement Steel $f_y = 420 \text{ N/mm}^2$
- ⇒ **Dead Load** $DL = 3.4096 \text{ kN/rib} = 9.9 \text{ KN/m}^2$ $WD = 1.2 \times 9.9 = 11.88 \text{ KN/m}^2$
- ⇒ **Live load** $LL = 5 \text{ KN/m}^2$ $WL = 1.6 \times 5 = 8 \text{ KN/m}^2$
- ⇒ $W = DL + LL = 11.88 + 8 = 19.88 \text{ KN/m}^2$.

Design of two way ribbed slab:

Design for shear :

Case 1 : discontinuous

La = 10.4 m – short direction .

Lb = 11.7 m – long direction .

$$m = l_a / l_b = 10.4 / 11.7 = 0.89$$

$$W_a = 0.612 - \max$$

$$W_b = 0.388$$

$$\text{Total Load} = 10.4 \times 11.7 \times 19.88 = 2419 \text{ KN.}$$

$$\text{Load at short beam} = \frac{0.612 \times 2419 \times 0.52}{2 \times 10.4} = 37.0 \text{ KN}$$

$$V_{ud} = 37.0 - 19.88 \times 0.52 \times 0.362 = 33.26 \text{ KN}$$

$$\phi V_c = 1.1 \times \frac{0.75}{6} \times \sqrt{24} \times 140 \times 362 \times 10^{-3} = 34.14$$

$$\frac{1}{2} \phi V_c = 17.1 < V_u < \phi V_c$$

No Need Shear reinforcement .

Design for positive moment :

Case 1

$$L_a / L_b = 10.4 / 11.7 = 0.89$$

$$C_{aD} = 0.0498$$

$$C_{aL} = 0.0498$$

$$C_{bD} = 0.0284$$

$$C_b L = 0.0284$$

$$M_{a+ve} = [C_{adl} \cdot W \cdot L \cdot a^2]$$

At short direction:

$$M_{a+ve} = [0.0498 \times 11.88 \times 10.4^2 + 0.0498 \times 8 \times 10.4^2] \times 0.54 = 57.82 \text{ KN.m}$$

At Long direction:

$$M_{b+ve} = [0.0284 \times 11.88 \times 11.7^2 + 0.0284 \times 8 \times 11.7^2] \times 0.64 = 49.46 \text{ KN.m}$$

Design of positive moment at short direction:

$$d = 400 - 20 - 8 - 20/2 = 362 \text{ mm}$$

$$M_u = 57.82 \text{ Kn.m}$$

$$R_n = \frac{M_u}{\phi \times b \times d^2} = \frac{57.82}{0.9 \times 540 \times 362^2} = 0.91$$

$$M = \frac{412}{0.85 \times 24} = 20.20$$

$$\rho = \frac{1}{20.20} \times \left(1 - \sqrt{1 - \frac{2 \times 0.91 \times 20.20}{412}} \right) = 2.26 \times 10^{-3}$$

$$A_s = 540 \times 362 \times 2.26 \times 10^{-3} = 441.8 \text{ mm}^2$$

So use 2Φ20.

Design of positive moment at long direction:

$$d = 400 - 20 - 8 - 20/2 = 362 \text{ mm}$$

$$M_u = 49.46 \text{ Kn.m}$$

$$R_n = \frac{M_u}{\phi \times b \times d^2} = \frac{49.46}{0.9 \times 640 \times 362^2} = 0.66$$

$$M = \frac{412}{0.85 \times 24} = 20.20$$

$$\rho = \frac{1}{20.20} \times \left(1 - \sqrt{1 - \frac{2 \times 0.66 \times 20.20}{412}} \right) = 1.6 \times 10^{-3}$$

$$A_s = 640 \times 362 \times 1.6 \times 10^{-3} = 370.688 \text{ mm}^2$$

So use 2Φ18.

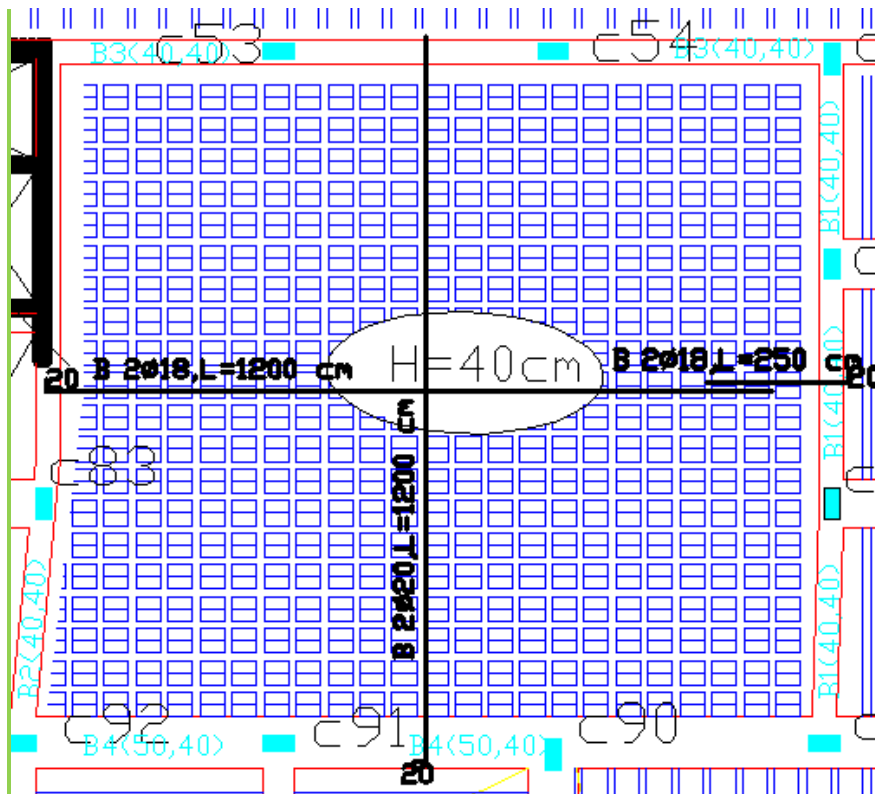


Figure (4-11): Two way ribbed slab reinforcement

4.8 Design of Beam

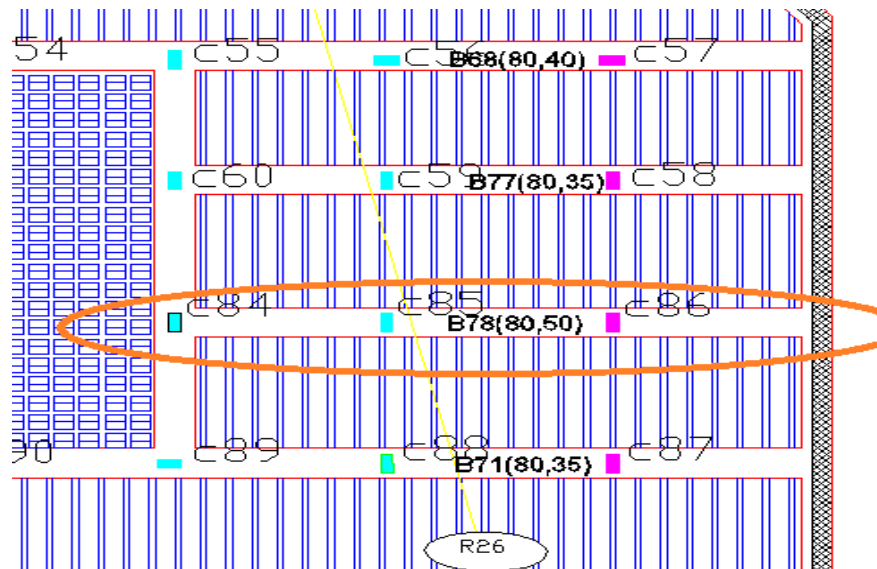


Fig. (4 – 12) Beam 78 .

❖ Material :-

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

✓ Load Calculations:-

Dead Load Calculations for Beam(B 78):-

The distributed Dead and Live loads acting upon B0-3 can be defined from the support reactions of the R26.

The maximum support reaction from Dead Loads for R26 upon B0-3 is 21.33 KN,

The distributed Dead Load from the R26 on B78

$$DL = (21.33 / 0.52) = 41.02 \text{ KN / m}$$

$$\text{Self weight of beam} = 6.72 \text{ KN / m}$$

Live Load calculations for Beam (B78):-

The maximum support reaction from Live Loads for R26 upon B 78 is 11.61KN .

The distributed Live Load from the Rib 26 on B78.

$$LL = 11.61 / 0.52 = 22.33 \text{ KN/m.}$$

⇒

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

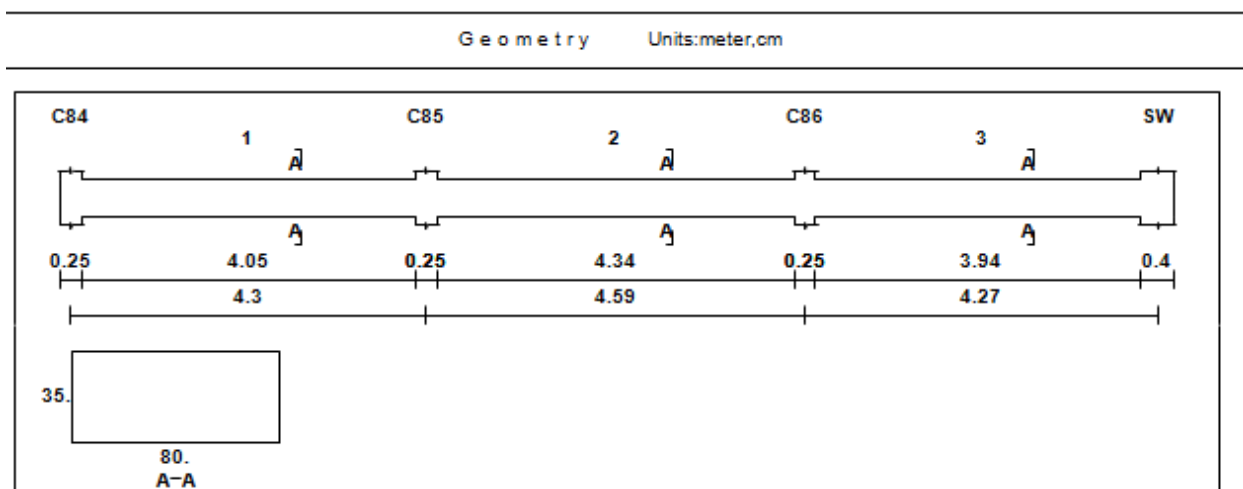


Figure (4-13) : Beam Geometry

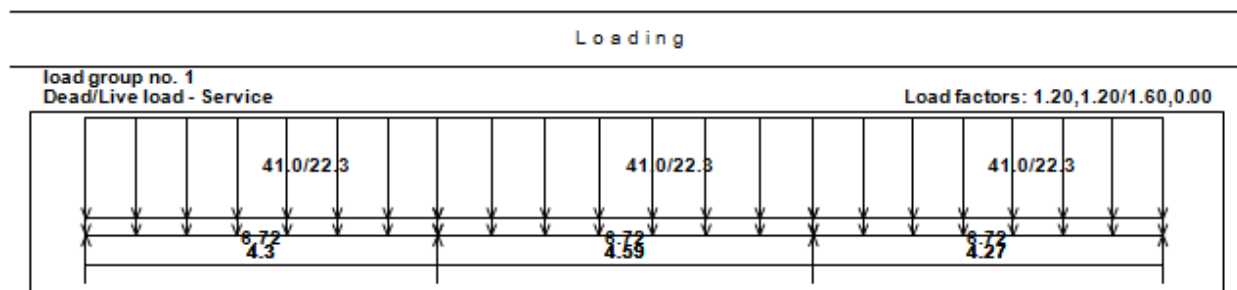


Figure (4-14) : Load of beam

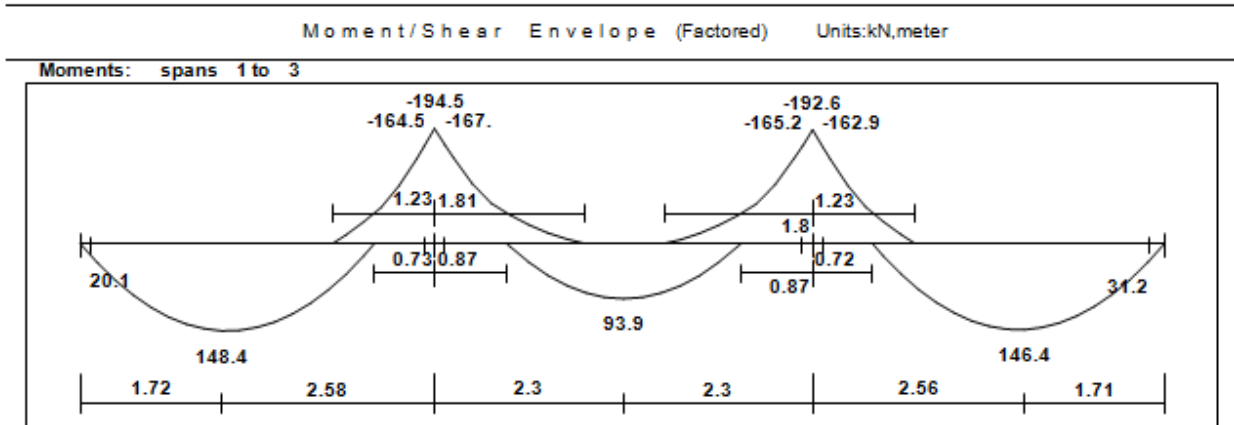


Figure (4-15) : Moment Envelop for Beam

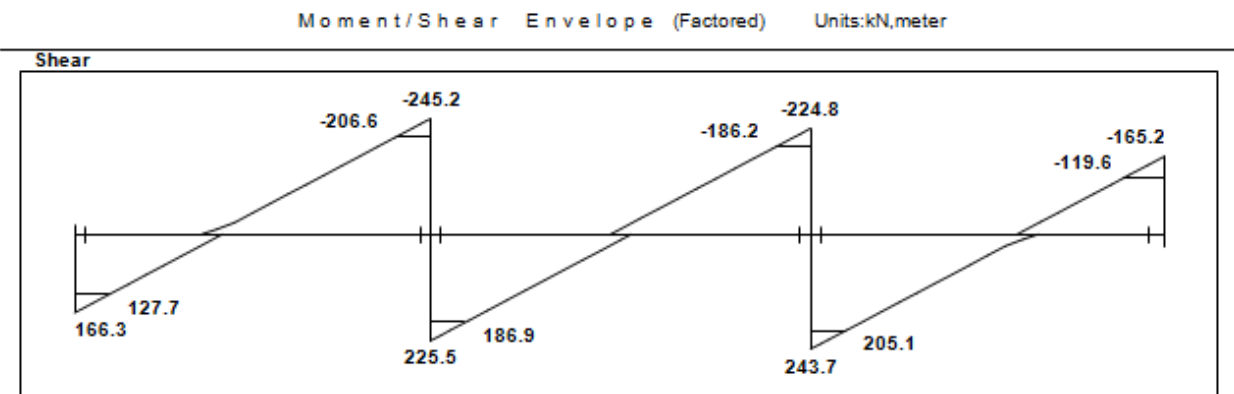


Figure (4-16) : Shear Envelop for Beam

Design of flexure:-**4.8.1. Design of Positive moment:-**

$$b_w = 80\text{cm}, h = 35\text{cm}$$

$$d = 350 - 40 - 10 - 10 = 290\text{mm}$$

$$M_u = 148.4 \text{ KN} \cdot \text{m}$$

$$C = \frac{3}{7} d = \frac{3}{7} * 290 = 124.29 \text{ mm} .$$

$$a = B * c = 0.85 * 124.29 = 105.65 \text{ mm} .$$

$$M_{n\max} = 0.85 * f_c * a * b * (d - a/2)$$

$$= 0.85 * 24 * 0.10565 * 0.8 * (0.290 - 0.10565/2) = 409.0 \text{ kn.m}$$

$$\Phi M_n = 0.82 * 409.0 = 335.38 \text{ kn.m}$$

$$\Phi M_n \geq 148.4 \text{ kn.m}$$

The section is singly

$$M_n = 148.4 / 0.9 = 164.9 \text{ kN.m}$$

$$m = \frac{f_y}{0.85 * f_c} = \frac{412}{0.85 * 24} = 20.2$$

$$R_n = \frac{M_n}{b * d^2} = \frac{164.9 * 10^{-3}}{0.8 * (0.290)^2} = 2.45 \text{ Mpa}$$

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

$$\rho = \frac{1}{20.2} \left(1 - \sqrt{1 - \frac{2(2.45)(20.2)}{412}} \right) = 0.00635$$

$$A_s = 0.00635 (800) (290) = 1474.23 \text{ mm}^2$$

$$A_{s_{\min}} = \frac{\sqrt{f_c'}}{4(f_y)} (b_w)(d) \geq \frac{1.4}{f_y} (b_w)(d) \dots\dots\dots (ACI - 10.5.1)$$

$$A_{s_{\min}} = \frac{\sqrt{24}}{4(412)} (800)(290) \geq \frac{1.4}{412} (800)(290)$$

$$A_{s_{\min}} = 689.7 < 788.35$$

$$A_{s_{\min}} = 788.35 \text{ mm}^2$$

$$1474.23 \text{ mm}^2 > A_{s_{\min}} = 788.35 \text{ mm}^2$$

$$\# \text{ of bars} = A_s / A_{s_{\text{bar}}} = 1474.23 / 254.5 = 6 \text{ bars}$$

$$* \text{ Note } A_{\Phi 18} = 254.5 \text{ mm}^2$$

As providing = 1527 mm²

Select 6Φ 18mm .

• Check for yielding

Tension = compression

$$A_s * f_y = 0.85 * b * a$$

$$1527 * 412 = 0.85 * 800 * 24 * a$$

$$a = 38.55 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{38.55}{0.85} = 35.45 \text{ mm}$$

$$\varepsilon_s = \frac{290 - 35.45}{35.45} \times 0.003$$

$$\varepsilon_s = 0.0215 > 0.005$$

ok

4.8.2. Design of negative moment:-

$$b_w = 80 \text{ cm}, h = 35 \text{ cm}$$

$$d = 350 - 40 - 10 - 10 = 290 \text{ mm}$$

$$M_u = 167 \text{ kN.m}$$

$$M_n = 167 / 0.9 = 185.6 \text{ kN.m}$$

$$m = \frac{f_y}{0.85 * f_c'} = \frac{412}{0.85 * 24} = 20.2$$

$$R_n = \frac{M_n}{b * d^2} = \frac{185.6 * 10^{-3}}{0.8 * (0.290)^2} = 2.76 \text{ Mpa}$$

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

$$\rho = \frac{1}{20.2} \left(1 - \sqrt{1 - \frac{2(2.76)(20.2)}{412}} \right) = 0.00723$$

$$A_s = 0.00723(800)(290) = 1676.5 \text{ mm}^2$$

$$A_{s_{\min}} = \frac{\sqrt{f_c'}}{4(f_y)} (b_w)(d) \geq \frac{1.4}{f_y} (b_w)(d) \dots\dots\dots (ACI - 10.5.1)$$

$$A_{s_{\min}} = \frac{\sqrt{24}}{4(412)} (800)(290) \geq \frac{1.4}{412} (800)(290)$$

$$A_{s_{\min}} = 689.7 < 788.35$$

$$A_{s_{\min}} = 788.35 \text{ mm}^2$$

$$1676.5 \text{ mm}^2 > A_{s_{\min}} = 788.35 \text{ mm}^2$$

$$\# \text{ of bars} = A_s / A_{s_{\text{bar}}} = 1676.5 / 254.5 = 7 \text{ bars}$$

$$* \text{ Note } A_{\Phi 18} = 254.5 \text{ mm}^2$$

$$A_s \text{ providing} = 1781.5 \text{ mm}^2$$

Select 7 Φ 18 mm .

- **Check for yielding**

Tension = compression

$$A_s * f_y = 0.85 * b * a$$

$$1781.5 * 412 = 0.85 * 800 * 24 * a$$

$$a = 44.97 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{44.97}{0.85} = 52.91 \text{ mm}$$

$$\epsilon_s = \frac{290 - 52.91}{52.91} \times 0.003$$

$$\epsilon_s = 0.0134 > 0.005$$

Ok

4.8.3.Design of shear

$$V_u = 206.6 \text{ KN}$$

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

$$= 0.75 * \frac{\sqrt{24}}{6} 1 * 800 * 290$$

$$= 142.1 \text{ KN}$$

$$\Phi V_c + (2/3) \Phi * \frac{\sqrt{f_c'}}{6} b_w * d = 142.1 + 94.7 = 236.8 \text{ KN} .$$

$236.8 > v_u = 206.6 \text{ KN} . \rightarrow$ the dimension is big enough .

Check for items:-

$$1/ \quad V_u \leq \Phi V_c / 2 \quad (\text{ X })$$

$$2/ \quad \Phi V_c / 2 \leq V_u \leq \Phi V_c \quad (\text{ X })$$

$$3/ \quad \Phi V_c \leq V_u \leq \Phi V_c + \Phi V_{smin} \quad (\checkmark)$$

$$\Phi V_{smin} \geq 0.75 \left(\frac{1}{3} \right) * b_w * d = 0.75 * \left(\frac{1}{3} \right) * 800 * 290 = 58 \text{ KN} . \quad (\text{control})$$

$$\geq 0.75 \left(\frac{\sqrt{24}}{16} \right) * b_w * d = 0.75 * \frac{\sqrt{24}}{16} * 290 * 800 = 53.3 \text{ kn} .$$

$$\Phi V_{smin} = 58 \text{ KN} .$$

So item (3) satisfy

$$\text{Take } A_v = 4 \Phi 10 = 4 * 78.5$$

$$A_v/s = V_s/f_y * d$$

$$4*78.5/s = 58/290*412$$

$$S \leq d/2 = 145 \text{ mm}$$

$$\leq 600 \text{ mm.}$$

Select $S = 10.0 \text{ cm}$

Use $\Phi 10$ (4legs) @ 10.0 cm c/c

4.9 Design of column

Select column (C85) for design in basement floor.

$$p_u = 470.66 \text{ KN}$$

$$p_{nreq} = \frac{470.66}{0.65} = 724.1 \text{ KN}$$

$$\text{Use } \rho = \rho_g = 1\%$$

$$P_n = 0.8 * A_g \{0.85 * f_c' + \rho_g (f_y - 0.85 f_c')\}$$

$$0.7241 = 0.8 * A_g [0.85 * 24 + 0.01 * (412 - 0.85 * 24)]$$

$$A_g = 0.0372 \text{ m}^2$$

Use $0.25 \times 0.5 \text{ m}$ with $A_g = 0.125 \text{ m}^2 > A_{greq} = 0.0372 \text{ m}^2$

Check Slenderness Effect:

- In 0.25 m -Direction

$$\frac{klu}{r} < 34 - 12 \frac{M_1}{M_2} \quad \dots\dots\dots \text{ACI} - (10.12.2)$$

l_u : Actual unsupported (un braced) length.

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

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

$$L_u = 3.8 \text{ m}$$

$$M_1/M_2 = 1$$

$K=1$, According to ACI 318-02 (10.10.6.3) The effective length factor, k , shall be permitted to be taken as 1.0.

$$\frac{klu}{r} < 34 - 12 \frac{M_1}{M_2} \quad \dots\dots\dots ACI - (10.12.2)$$

$$\frac{1 \times 3.80}{0.3 \times 0.25} = 50.67 < 22$$

\therefore Long Column in 0.25m:direction

• In 0.5m-Dirction

$$\frac{klu}{r} < 34 - 12 \frac{M_1}{M_2} \quad \dots\dots\dots ACI - (10.12.2)$$

$$\frac{1 \times 3.80}{0.3 \times 0.5} = 25.33 < 22$$

\therefore long Column in 0.5m:direction

$$EI = 0.4 \frac{E_c I_g}{1 + \beta_d} \dots\dots\dots [ACI 318 - 2002(Eq. 10 - 15)]$$

$$E_c = 4700 \sqrt{f'c'} = 4700 \sqrt{24} = 23025.20 \text{ Mpa}$$

$$\beta_d = \frac{1.2DL}{P_u} = \frac{281.22}{470.66} = .598$$

$$I_g = \frac{b \times h^3}{12} = \frac{0.5 \times 0.25^3}{12} = 0.000651$$

$$EI = \frac{0.4 \times 23025.20 \times 0.000651}{1 + .598} = 3.75 \text{ Mpa}$$

$$P_{cr} = \frac{\pi^2 EI}{(KLu)^2} = \frac{3.14^2 \times 3.75}{(1 \times 3.8)^2} = 2.56 \text{ KN}$$

$$\delta_{ns} = \frac{Cm}{1 - \frac{Pu}{0.75P_c}} = \frac{1}{1 - \frac{470.66}{0.75 \times 2.56 \times 1000}} = 1.32 > 1$$

$$e_{\min} = 15 + 0.03 \times h = 15 + 0.03 \times 250 = 22.5 \text{ mm} = 0.0225 \text{ m}$$

$$e = e_{\min} \times \delta_{ns} = 0.0225 \times 1.32 = 0.0297 \text{ mm}$$

$$\frac{e}{h} = \frac{0.0297}{0.25} = 0.1188$$

From Interaction Diagram

$$\frac{\phi P_n}{A_g} = \frac{470.66}{0.25 \times 0.5} \times \frac{145}{1000} = 0.55 \text{ Ksi}$$

$$\rho_g = 0.01$$

$$A_s = \rho \times A_g = 0.01 \times 500 \times 250 = 1250 \text{ mm}^2$$

\therefore use 12 ϕ 12

Design of the Reinforcement:

$S \leq 16 \text{ db}$ (longitudinal bar diameter).....ACI - 7.10.5.2

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

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

$$\text{spacing} \leq 16 \times d_b = 16 \times 1.6 = 25.6 \text{ cm}$$

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

$$\text{spacing} \leq \text{least.dim.} = 30 \text{ cm}$$

Use ϕ 10 @ 30 cm

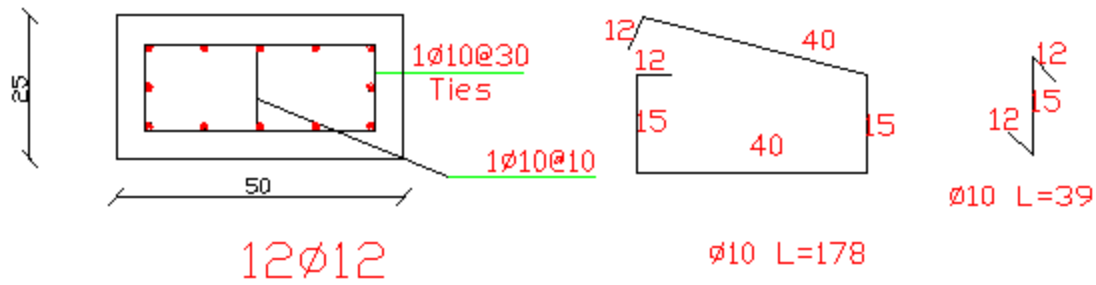


Figure (4-17) : columns section

4.10 Design of Footing

❖ Material :-

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

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

✓ Load Calculations :- (From Column Group 2)

Dead Load = 234.35Kn , Live Load = 118.4Kn

Total services load = 234.35+118.4 = 352.75Kn

Total Factored load = $1.2 \times 234.35 + 1.6 \times 118.4 = 470.66\text{Kn}$

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

Soil density = 18 Kg/cm³

Allowable Bearing Capacity = 400Kn/m²

Assume $h = 50\text{cm}$

$$q_{net-allow} = 400 - 18*0.40 - 25*0.5 - 5 = 375.3\text{kn/m}^2$$

✓ Area of Footing :-

$$A = \frac{Pt}{q_{net-allow}} = \frac{352.75}{375.3} = 0.94 \text{ m}^2$$

Assume Square Footing

Select $B = 1.10\text{m}$

✓ Bearing Pressure :-

$$q_u = 470.66/1.10*1.10 = 389\text{Kn/m}^2$$

✓ Design of Footing :-

1- Design of One Way Shear Strength :-

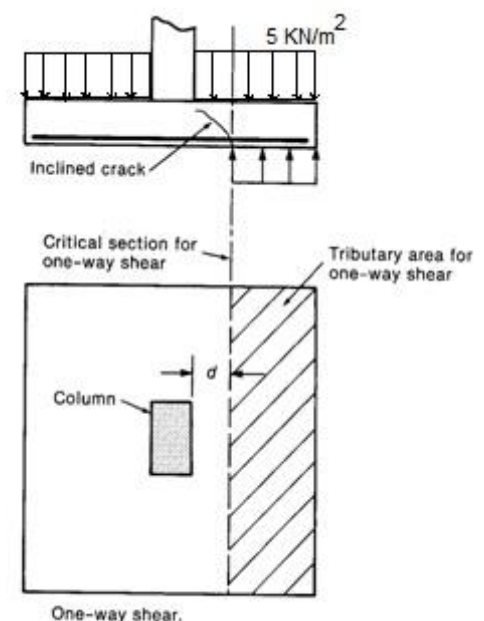
Critical Section at Distance (d) From The Face of Column

Assume $h = 50\text{cm}$, bar diameter $\phi 14$ for main reinforcement and 7.5 cm Cover

$$d = 500 - 75 - 20 = 405 \text{ mm}$$

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

$$V_u = 389 * \left(\frac{1.1-0.25}{2} - 0.405 \right) * 1.1 = 8.558\text{Kn}$$



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

$$\phi.V_c = 0.75 * \frac{1}{6} * \sqrt{24} * 1100 * 405 = 272.8Kn$$

$$\phi.V_c = 272.8Kn > V_u = 8.558Kn$$

\therefore Safe

2- Design of Two Way Shear Strength :-

$$V_u = P_u - FR_b$$

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

$$V_u = 389[(0.0.25 + 0.405) * (0.5 + 0.405)] = 462.7Kn$$

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

$$b_o = 2 * (0.25 + 0.405) + 2 * (0.5 + 0.405) = 3.12m$$

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

$$\phi V_c = \phi \cdot \frac{1}{6} \left(1 + \frac{2}{\beta_c} \right) \sqrt{f'_c} b_o d = \frac{0.75}{6} * \left(1 + \frac{2}{2} \right) * \sqrt{24} * 3120 * 405 = 1547.6 Kn$$

$$\phi V_c = \phi \cdot \frac{1}{12} \left(\frac{\alpha_s}{b_o / d} + 2 \right) \sqrt{f'_c} b_o d = \frac{0.75}{12} * \left(\frac{40 * 405}{3120} + 2 \right) * \sqrt{24} * 3120 * 405 = 2782.7 Kn$$

$$\phi V_c = \phi \cdot \frac{1}{3} \sqrt{f'_c} b_o d = \frac{0.75}{3} * \sqrt{24} * 3120 * 405 = 1547.6 Kn$$

$$\Phi V_c = 1547.6 \text{ KN} > V_u = 462.7 \text{ KN}$$

3- Design of Bending Moment :-

Critical Section at the Face of Column

$$M_u = 389 * 1.1 * 0.425 * 0.425 / 2 = 38.6 \text{ Kn.m}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{38.6 \times 10^6}{0.9 \times 1100 \times 405^2} = 0.24 \text{ Mpa}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 m R_n}{420}} \right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.24}{420}} \right) = 0.00057$$

$$A_{s, \text{req}} = \rho \cdot b \cdot d = 0.00057 \times 1100 \times 405 = 256.1 \text{ mm}^2$$

$$A_{s, \text{min}} = 0.0018 * 1100 * 500 = 990 \text{ mm}^2$$

$$A_{s, \text{req}} = A_{s, \text{min}} = 990 \text{ mm}^2 \text{ is control}$$

Check for Spacing :-

$$S = 3h = 3 * 500 = 1500 \text{ mm}$$

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

$$S = 500 \text{ mm}$$

$$S = 500 \text{ mm} \text{ is control}$$

Use 7Ø14 in Both Direction, $A_{s, \text{provided}} = 1077.6 \text{ mm}^2 > A_{s, \text{required}} = 990 \text{ mm}^2 \dots$ Ok

Check for strain:-

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{1077.6 \times 420}{0.85 \times 1100 \times 24} = 20.17 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{20.17}{0.85} = 23.73 \text{ mm}$$

$$\epsilon_s = 0.003 \left(\frac{d - c}{c} \right) = 0.003 \left(\frac{405 - 23.73}{23.73} \right) = 0.048 > 0.005 \dots \dots \mathbf{Ok}$$

4- Design of Dowels :-

Load Transfer In Footing :-

$$\Phi P_{nb} = \Phi (0.85 f'_c A_1 \times \sqrt{\frac{A_2}{A_1}})$$

$$A_1 = 0.5 \times 0.25 = 0.125 \text{ m}^2$$

$$A_2 = 1.1 \times 1.1 = 1.21 \text{ m}^2$$

$$\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{1.21}{0.125}} = 9.68 > 2 \dots \dots \dots \sqrt{\frac{A_2}{A_1}} = 2$$

$$\Phi P_{nb} = 0.65 \times (0.85 \times 24 \times 110 \times 2) = 2917.2 \text{ Kn}$$

$$\Phi P_n = 2917.2 > P_u = 470.66 \dots \dots \dots \mathbf{ok}$$

No Need For Dowels

$$A_{s,min} = 0.005 * A_c = 0.005 * 500 * 250 = 625 \text{ mm}^2$$

$$\mathbf{Use \ 8\phi 20, \ A_{s,provided} = 2512 \text{ mm}^2 > A_{s,required} = 600 \text{ mm}^2 \dots \mathbf{Ok}}$$

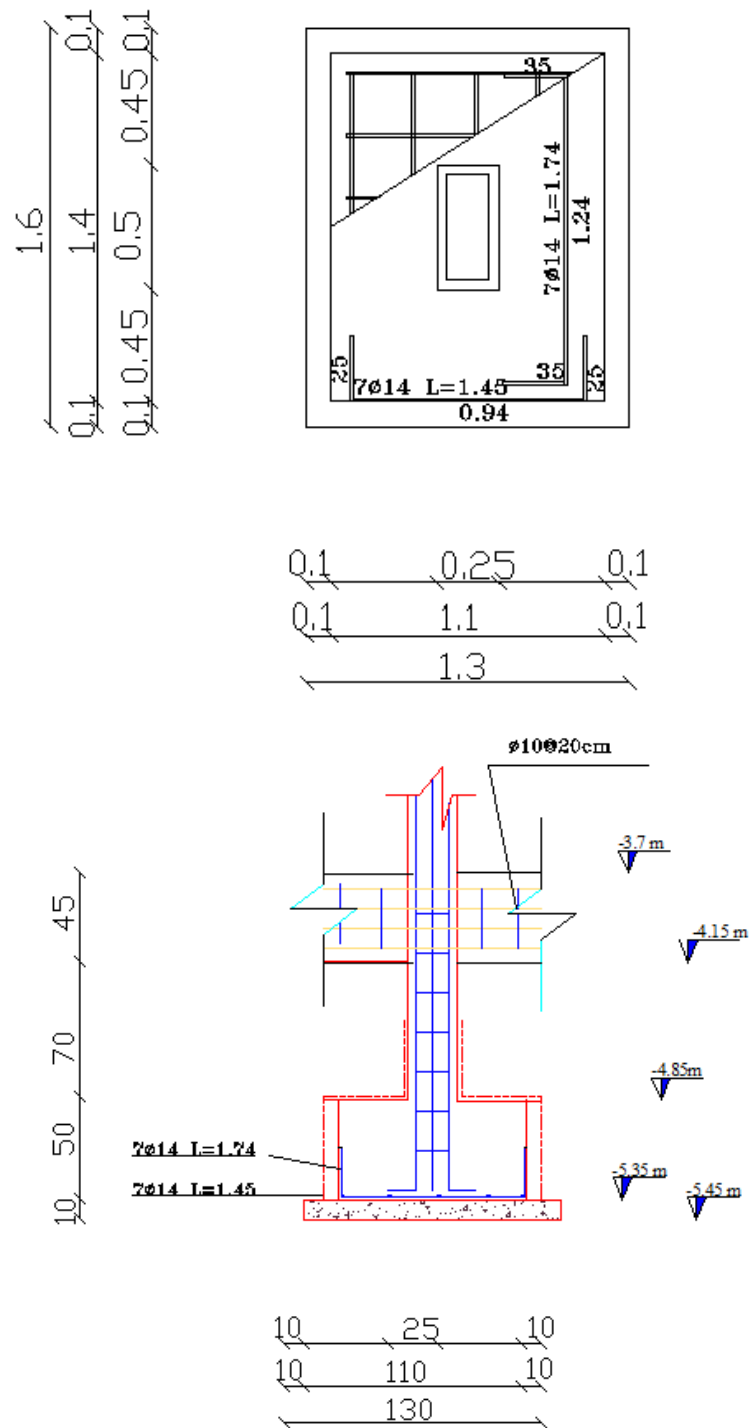


Fig 4.18 : Foundation Reinforcement.

4.11 Design of Stair

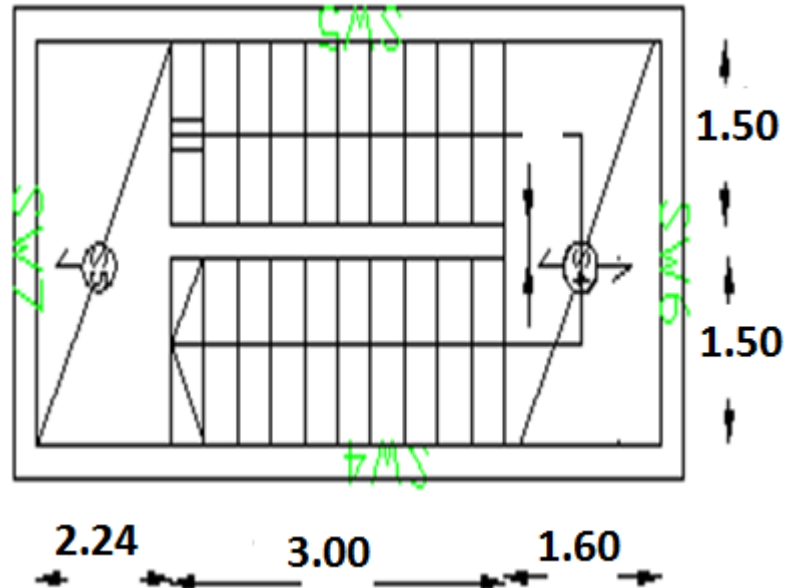


Fig 4.19: Stair Plan.

❖ Material :-

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

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

Load Calculation & Design of stair :-

$$h_{\min} = L/28$$

$$h_{\min} = 6.84/28 = 24.4 \text{ cm}$$

Take $h = 25 \text{ cm}$

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

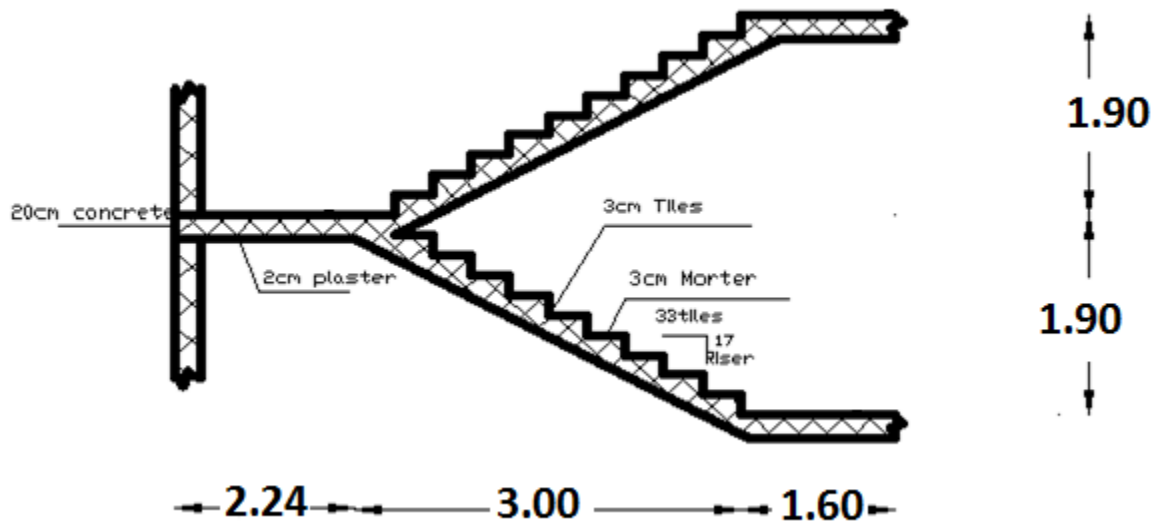


Fig 4.20: Stair Section.

Dead Load For Flight For 1m Strip:-

No.	Parts of Flight	Calculation
1	Tiles	$23 \times 0.03 \times 1 \times (0.33 + 0.17/0.3) = 1.15 \text{ KN/m}$
2	Mortar	$22 \times 0.03 \times 1 \times (0.3 + 0.17/0.3) = 1.04 \text{ KN/m}$
3	Stair	$25 \times 0.5 \times 0.17 \times 1 = 2.13 \text{ KN/m}$
4	R.C	$25 \times 0.25 \times 1 / \cos 29.56 = 7.19 \text{ KN/m}$
5	Plaster	$22 \times 0.02 \times 1 / \cos 29.56^\circ = 0.51 \text{ KN/m}$
Sum		12.02 KN/m

Table (4.5): Dead Load Calculation of Flight.

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

Factored Load For Flight :-

$$W_U = 1.2 \times 12.02 + 1.6 \times 5 = 22.42 \text{ KN/m}$$

Dead Load For landing For 1m Strip:-

No.	Parts of Landing	Calculation
1	Tiles	$22 \times 0.03 \times 1 = 0.66 \text{ KN/m}$
2	Mortar	$22 \times 0.03 \times 1 = 0.66 \text{ KN/m}$
4	R.C	$25 \times 0.25 \times 1 = 6.25 \text{ KN/m}$
5	Plaster	$22 \times 0.02 \times 1 = 0.44 \text{ KN/m}$
Sum		8.01KN/m

Table (4.6): Dead Load Calculation of landing.

✓ Load Calculation:-

Dead Load For Landing For 1m Strip = 8.01 KN/m

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

Stair reinforcement Design of one meter strip :- (for flight)

The value of V_u at the center of support = $(22.42 \times 3)/2 = 33.63 \text{ KN}$.

$$d = 250 - 20 - 7 = 223 \text{ mm} .$$

$$\phi V_c = (0.75/6) * \sqrt{24} * 1000 * 223 = 136.56 \text{ KN} > 33.63 \text{ KN} \dots\dots \text{ok} .$$

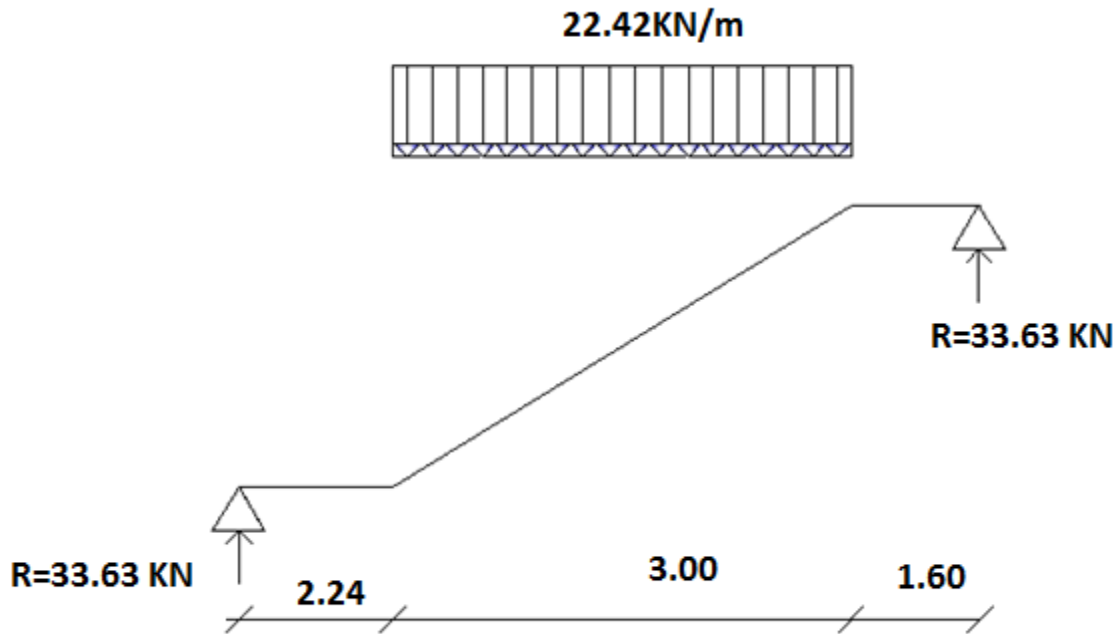


Figure (4-21): Load distribution on stairs

$$M_{u \max} = (33.63 * (6.84/2)) - (8.81 * 1.6 * (1.6 + 3/2)) - (22.42 * 3/2 * 3/4) = 57.37 \text{ KN.m}$$

$$d = 250 - 20 - 7 = 223 \text{ mm}$$

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

$$R_n = \frac{M_n}{b d^2} = \frac{57.37 * (10)^6}{(0.9)(1000)(223)^2} = 1.28 \text{ Mpa}$$

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2mR_n}{f_y}} \right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 * 20.6 * 1.28}{420}} \right) = 0.00315$$

$$A_s = 0.00315 * (100) * (22.3) = 7.024 \text{ cm}^2$$

$$\text{Use } 5\Phi 14 @ 25 \text{ cm c/c with } A_s = 7.696 \text{ cm}^2 > 7.024 \text{ cm}^2$$

Min reinforcement :-

$$A_{s \min} = 0.0018 * 1000 * 250 = 4.5 \text{ cm}^2 \quad \text{Use } 4\Phi 12 @ 20 \text{ cm c/c}$$

$$A_s = 7.648 > 4.5 \text{ cm}^2$$

Design of landing:-

Design of S2:-

$$M_u = (18.5 * (3.6/2)) - (8.81 * 1.5 / (1.5 + 0.6/2)) 17.61 * 0.6/2 * 0.6/4 = 18.63 \text{ KN.m}$$

$$d = 250 - 20 - 7 = 223 \text{ mm}$$

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

$$R_n = \frac{M_n}{b d^2} = \frac{18.63 * (10)^6}{(0.9)(1000)(0.223)^2} = 0.416 \text{ Mpa}$$

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2mR_n}{f_y}} \right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 * 20.6 * 0.416}{420}} \right) = 0.001$$

$$A_s = 0.001 * (100) * (22.3) = 2.23 \text{ cm}^2$$

Use 4Φ 14 @ 25 cm c/c with $A_s = 6.16 \text{ cm}^2 > 4.91 \text{ cm}^2$

Min reinforcement :-

$$A_{s \text{ min}} = 0.0018 * 1000 * 250 = 4.5 \text{ cm}^2 \quad \text{Use 4 } \Phi 12 \text{ @ 25 cm c/c}$$

$$A_s 4.91 > 4.5 \text{ cm}^2$$

4-12-Design of solid slab of the stair roof :-**Determination of load:**

$$\text{Dead load} = 0.2 \times 25 \times 1 = 5 \text{ KN/m}^2$$

$$\text{Live load} = 1.5 \text{ KN/m}^2$$

$$q_u = (1.2 \times 5) + (1.6 \times 1.5) = 8.4 \text{ KN/m}$$

$$h = 20 \text{ cm}$$

$$d = h - 2 \times 0.5 = 20 - 2 \times 0.5 = 17.5 \text{ cm}$$

$$M_u = (q_u \times l^2) / 8 = 8.4 \times 6.2^2 / 8 = 40.36 \text{ KN.m}$$

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

$$R_n = \frac{M_n}{b d^2} = \frac{40.36 \times (10)^{-3}}{(0.9)(1)(0.175)^2} = 1.46 \text{ Mpa}$$

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2mR_n}{f_y}} \right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 1.46}{420}} \right) = 0.0036$$

$$A_s = 0.0036 \times (100) \times (17.5) = 6.3 \text{ cm}^2$$

Min reinforcement:

$$A_{s \text{ min}} = 0.0018 \times b \times h = 0.0018 \times 100 \times 20 = 3.6 \text{ cm}^2$$

$$\text{Select } A_s = 3.6 \text{ cm}^2$$

$$\text{Use } \Phi 14 @ 20 \text{ cm c/c with } A_s = 7.7 \text{ cm}^2 > 6.3 \text{ cm}^2$$

Longelir reinforcement for one meter strip:

$$A_s = 0.0018 * 100 * 20 = 3.6 \text{ cm}^2$$

Use $\Phi 12 @ 25 \text{ cm}$ with $A_s = 4.52 \text{ cm}^2 > 3.6 \text{ cm}^2$

Top reinforcement :

Use $\Phi 8 @ 15 \text{ cm}$ in both direction

Design of shear reinforcement :

$$V_u \text{ max} = q_u * L / 2 = (8.4 * 6.2) / 2 = 26.04 \text{ KN}$$

$$\Phi V_c \geq V_u \text{ max}$$

$$\Phi V_c = \frac{0.75 \sqrt{f_c'}}{6} (b_w)(d)$$

$$\Phi V_c = \frac{0.75 \sqrt{24}}{6} (1000)(175) = 107.165 \text{ KN}$$

$$107.1 > 26.04 \text{ KN/m}$$

No Shear reinforcement is required .

4.13 Design of Shear Wall

❖ Material and Sections:- (From Shear Wall 1)

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

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

⇒ Shear Wall Thickness $h = 15 \text{ cm}$

⇒ Shear Wall Width $L_w = 3.6 \text{ m}$

⇒ Shear Wall Height $H_w = 26.4 \text{ m}$

⇒

✓ Design of Horizontal Reinforcement:-

$$\sum F_x = V_u = 700 \text{ KN}$$

The critical Section is the smaller of:

$$\frac{l_w}{2} = \frac{3.6}{2} = 1.8 \text{ m}$$

$$\frac{h_w}{2} = \frac{26.4}{2} = 13.2 \text{ m}$$

storey height (H_w) = 3.4 m.....Control

$$d = 0.8 \times L_w = 0.8 \times 3.6 = 2.88 \text{ m}$$

$$\begin{aligned} \phi V_{nmax} &= \phi \frac{5}{6} \sqrt{f_c'} h d \\ &= 0.75 * 0.83 * \sqrt{24} * 150 * 2880 = 1317.4 \text{ KN} > V_u = 700 \text{ KN} \end{aligned}$$

V_c is the smallest of :

$$1 - V_c = \frac{1}{6} \sqrt{f'_c} h d = \frac{1}{6} \sqrt{24} * 150 * 2880 = 352.7 \text{ KN} \dots\dots \text{Control}$$

$$2 - V_c = 0.27 \sqrt{f'_c} h d + \frac{N_u d}{4 l_w} = 0.27 \sqrt{24} * 150 * 2880 + 0 = 571.4 \text{ KN}$$

$$3 - V_c = \left[0.05 \sqrt{f'_c} + \frac{l_w \left(0.1 \sqrt{f'_c} + 0.2 \frac{N_u}{l_w h} \right)}{\frac{M_u}{V_u} - \frac{l_w}{2}} \right] h d$$

$$\frac{21099.6 - 16339.4}{3.95} = \frac{M_u - 16339.4}{3.95 - 3.4} \Rightarrow M_u = 16.881 \text{ KN.m}$$

$$\frac{M_u}{V_u} - \frac{l_w}{2} = \frac{16.88}{1205.1} - \frac{3.6}{2} = -1.78 \leq 0.00 \dots\dots \text{Neglected}$$

$$V_u = 700 \text{ KN}$$

$$\phi * V_c = 0.75 * 352.7 = 264.5 \text{ KN}$$

$$V_{s,min} = \frac{\sqrt{f'_c}}{16} * b_w * d = \frac{\sqrt{24}}{16} * 150 * 2880 = 232.3 \text{ KN} \dots\dots\dots \text{Control}$$

$$V_{s,min} = \frac{1}{3} * b_w * d = \frac{1}{3} * 150 * 2880 = 244 \text{ KN}$$

$$\phi * (V_c + V_{s,min}) = 0.75 * (352.7 + 232.3) = 740.75 \text{ KN}$$

$$A_{vh} = \frac{2 * \pi * 10^2}{4} = 157.1 \text{ mm}^2$$

$$\frac{157.1}{S_h} = 0.75$$

$$S_h = 200 \text{ mm}$$

- Maximum spacing is the least of :

$$\frac{L_w}{5} = 3600/5 = 720 \text{ mm}$$

$$3 * h = 3 * 150 = 450 \text{ mm}$$

$$360 \text{ mm} \dots\dots \text{Control}$$

Use $\phi 10/150 \text{ mm}$ for two layers

✓ Design of Vertical Reinforcement:-

$$\frac{A_{vv}}{S_v} = \left[0.0025 + 0.5 \left(2.5 - \frac{h_w}{Lw} \right) \left(\frac{A_{vh}}{S_h * h} - 0.0025 \right) \right] * 150$$

$$\frac{A_{vv}}{S_v} = \left[0.0025 + 0.5 \left(2.5 - \frac{26.4}{3.6} \right) \left(\frac{157.1}{150 * 150} - 0.0025 \right) \right] * 150$$

$$\frac{A_{vv}}{S_v} = 0.350$$

Select Ø 16 in Two Layer

$$A_{vh} = \frac{2 * \pi * 16^2}{4} = 402 \text{ mm}^2$$

$$\frac{402}{S_v} = 0.350$$

$$S_v = 401 \text{ mm}$$

- Maximum spacing is the least of :

$$\frac{Lw}{3} = 3600/3 = 1200 \text{ mm}$$

$$3 * h = 3 * 150 = 450 \text{ mm}$$

450 mm Control

Use Ø 16/150 mm for two layers