

Chapter 4
Structural Analysis And Design

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4.1 Introduction:

Many structures are built of reinforced concrete: bridges, buildings, retaining walls, tunnels, and others.

Reinforced concrete is logical union of two materials: plain concrete, which possesses high compressive strength but little tensile strength, and steel bars embedded in the concrete, which can provide the needed strength in tension.

Plain concrete is made by mixing cement, fine aggregate, coarse aggregate, water, and frequently admixtures.

Understanding of reinforced concrete behavior is still far from complete, building codes and specifications that give design procedures are continually changing to reflect latest knowledge.

Structural concrete can be classified into:

- Lightweight concrete with unit weight from about 1350 to 1850 kg/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

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✓ **Material :**

Concrete: B300.... $F_{cu} = 30 \text{ N/mm}^2 \text{ (MPa)}$ 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 NONPRESTRESSED BEAMS OR ONE-WAY SLABS
UNLESS DEFLECTIONS ARE CALCULATED. (ACI 318M-11)

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

FOR RIB :

$$h_{\min} \text{for (one end)} = L/18.5 = 6.4/18.5 = 35 \text{ cm}$$

$$h_{\min} \text{for (both end continuous)} = L/21 = 5.6/21 = 27 \text{ cm}$$

FOR BEAM :

$$h_{\min} \text{for (both end continuous)} = L/21 = 6.5/21 = 31 \text{ cm}$$

$$h_{\min} \text{for (Cantilever)} = L/8 = 1.5/8 = 27 \text{ cm}$$

$$h_{\min} \text{for (one end)} = L/18.5 = 5.8/18.5 = 32 \text{ cm}$$

FOR SOLID SLABS :

h_{\min} for (one end continuous) = $L/24 = 8/24 = 34\text{cm}$

h_{\min} for (both end continuous) = $L/28 = 7.2/28 = 26\text{cm}$

The minimum thickness will be $h_{\min} = 35\text{cm}$

select 35cm for rib slab with hidden beam

$h = 35\text{cm}$ (27 cm Hollow Block + 8 cm Topping)

select (35cm) for solid slab.

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.

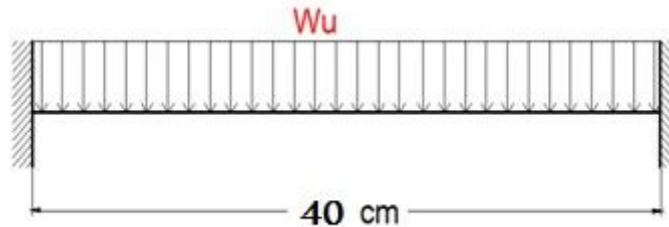


Fig 4.1: topping load.

✓ **Load calculations:**

Dead load calculations:

Dead load from:	$\delta \times \gamma \times 1$	KN/m
Tiles	$0.03 \times 23 \times 1$	0.69
Mortar	0.03×22	0.66
Coarse sand	$0.07 \times 17 \times 1$	1.19
Topping	$0.08 \times 25 \times 1$	2
Interior partitions	2.3	2.3
	Σ	6.84

Table (4.2) : Dead load calculation Topping

Live load :

$$L_L = 4 \text{ KN/m}^2$$

$$L_L = 4 \text{ KN/m}^2 \times 1\text{m} = 4 \text{ KN/m}$$

Factored load :

$$W_U = 1.2 \times 6.84 + 1.6 \times 4 = 14.61 \text{ KN/m.}$$

$$M_u = \frac{w_u L^2}{12}$$

$$M_u = \frac{14.61 \times 0.4^2}{12} = 0.195 \text{ KN.m of strip width}$$

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

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

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

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

$$\phi M_n \gg M_u = 0.195 \text{ KN.m}$$

No reinforcement is required by analysis. According **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. 450 mm.
3. $S = 380 \left(\frac{280}{f_s} \right) - 2.5 C_c = 380 \left(\frac{280}{\frac{2}{3} \cdot 420} \right) - 2.5 \cdot 20 = 330 \text{ mm}$

but

$$S \leq 300 \left(\frac{280}{f_s} \right) = 300 \left(\frac{280}{\frac{2}{3} \cdot 420} \right) = 300 \text{ mm ACI 10.6.4}$$

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

4.5 Design of One-Way Ribbed Slab(RG14) :

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

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

Select $bw=12\text{cm}$

$h \leq 3.5*bw$ ACI(8.13.2)

Select $h=32\text{cm} < 3.5*12=42\text{ cm}$

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

Select $tf=8\text{cm}$

✓ **Statically system and Dimensions**

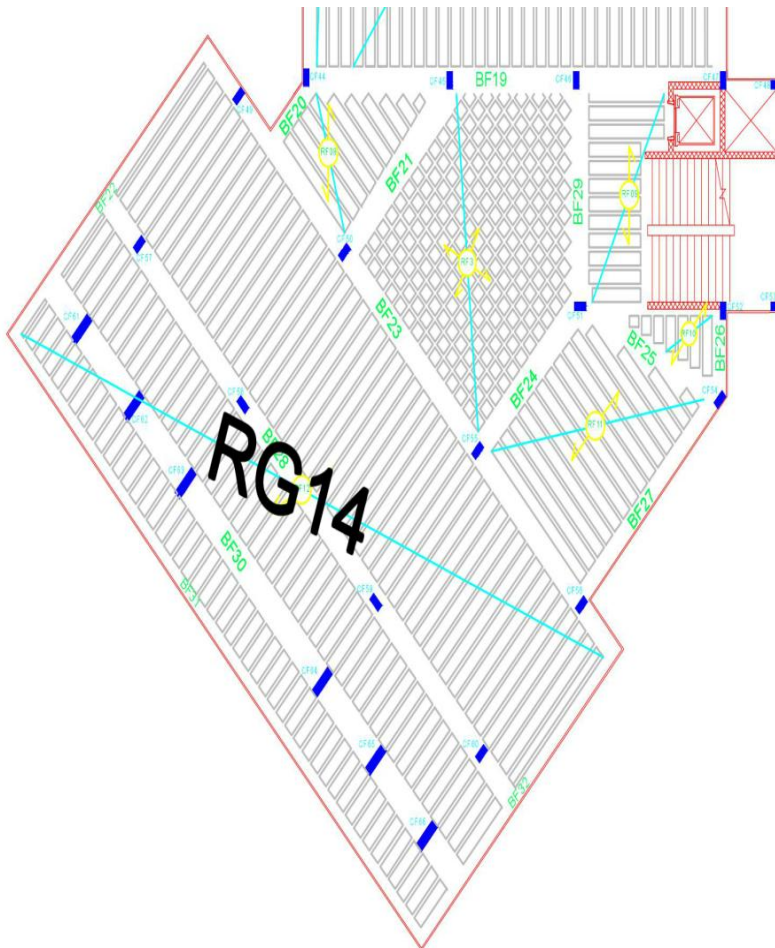


Fig 4.2: One Way Rib slab (RG14)

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Load calculations :

*Dead load:

Material	$W = h \times \gamma \times b \text{ (KN/M)}$
Tiles	$0.03 \times 23 \times 0.52 = 0.359$
Mortar	$0.03 \times 22 \times 0.52 = 0.343$
Course sand	$0.07 \times 17 \times 0.52 = 0.619$
Topping	$0.08 \times 25 \times 0.52 = 1.04$
R.c rib	$0.27 \times 25 \times 0.12 = 0.81$
Hollow block	$0.27 \times 10 \times 0.4 = 1.08$
Plaster	$0.03 \times 22 \times 0.52 = 0.343$
Interior partition	$2.3 \times 0.52 = 1.196$
Total dead load	5.79 KN/m

Table (4.3): Dead load calculation Topping of rib 14

Dead load /rib = 5.80KN/m

Live load = 4KN/M²

Live load /rib = $4 \text{ KN/m}^2 \times 0.52 \text{ m} = 2.08 \text{ KN/m}$.

*The effective flange (be) :

1) $be \leq \frac{L}{4} = \frac{3500}{4} = 875 \text{ mm}$

2) $be \leq b_w + 16hf = 120 + 16 \times 80 = 1400 \text{ mm}$

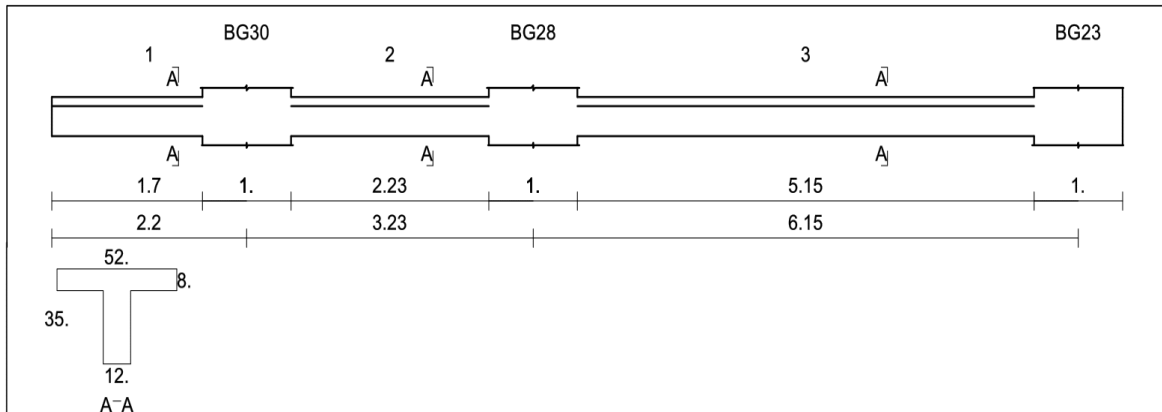
3) $be \leq \text{center to center spacing between adjacent beam} = \frac{400}{2} + \frac{400}{2} + 120 = 520 \text{ mm}$

Take be=520 mm

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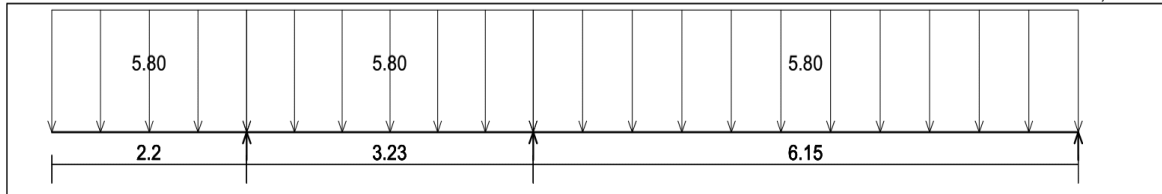
Geometry Units: meter, cm



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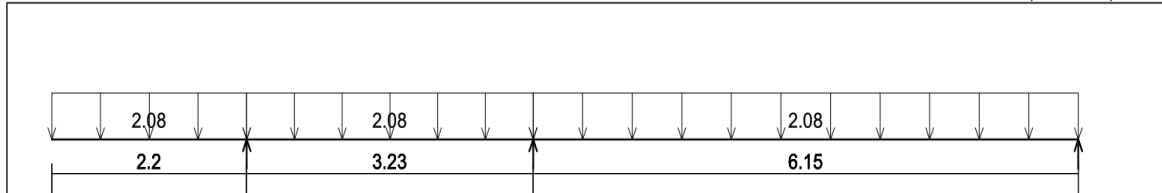
load group no. 1
Dead load - Service

Units: kN, meter



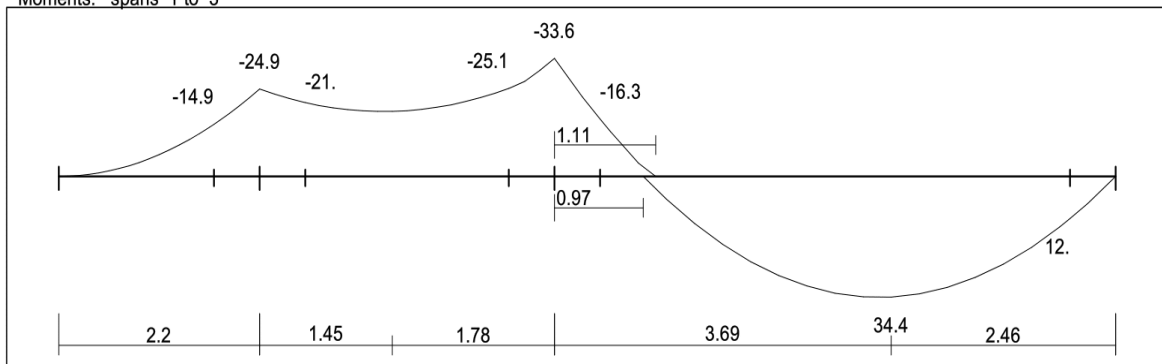
Live load - Service

Load factors: 1.20, 1.20/1.60, 0.00



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

Moments: spans 1 to 3



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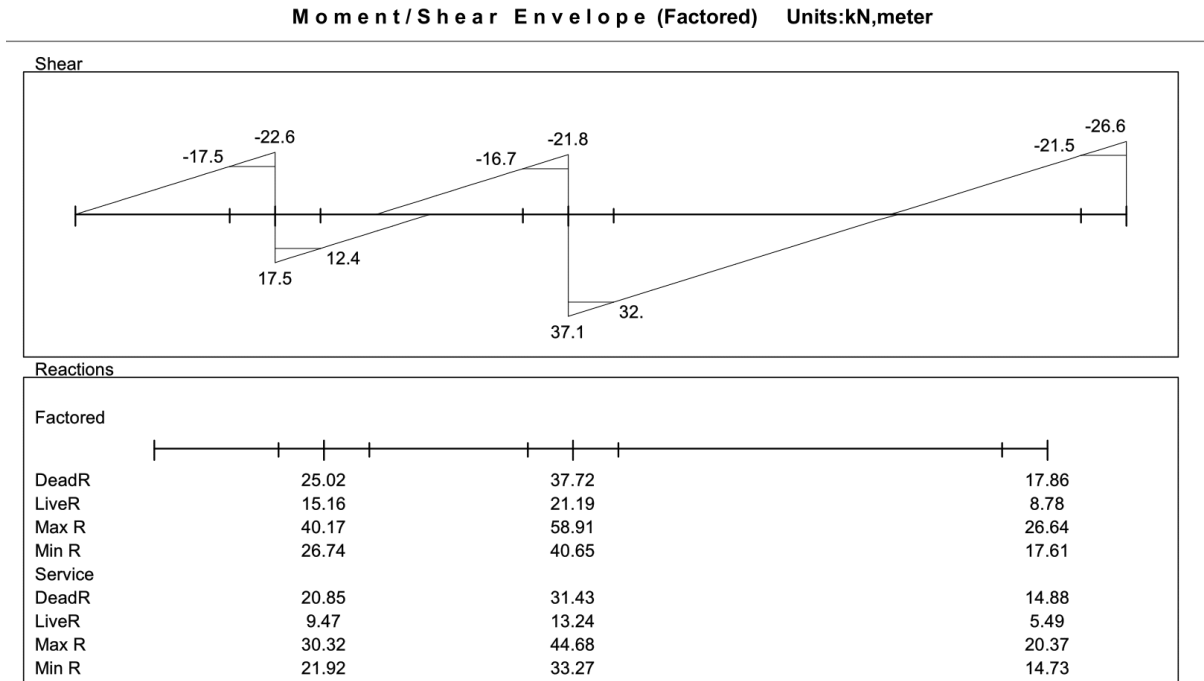


Fig 4.3: Shear & Moment Envelope Diagram (RG14)

***Design of positive moment:**

$$M_u = 34.4 \text{ kN.m.}$$

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

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

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

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

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

$$M_{nf} \gg \frac{M_u}{\phi} = \frac{34.4}{0.9} = 38.22 \text{ kN.m}, \text{ the section will be designed as } \mathbf{rectangular \text{ section}}$$

$$b_e = 520 \text{ mm.}$$

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$$R_n = \frac{M_u}{\phi b d^2} = \frac{34.4 \times 10^6}{0.9 \times 520 \times 316^2} = 0.736 \text{ Mpa.}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{420}} \right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.736}{420}} \right) = 0.00178$$

$$A_{s, \text{req}} = \rho \cdot b \cdot d = 0.00178 \times 520 \times 316 = 293.3 \text{ mm}^2$$

*Check for $A_{s, \text{min}}$.

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

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

$$1. A_{s, \text{min}} = 0.25 \frac{\sqrt{24}}{420} 120 \times 316 = 110.6 \text{ mm}^2$$

$$2. A_{s, \text{min}} = \frac{1.4}{420} 120 \times 316 = 126.4 \text{ mm}^2 \text{ Control}$$

$$A_s = 293.3 \text{ mm}^2 \geq A_{s, \text{min}} = 126.4 \text{ mm}^2$$

Use 2Ø14, $A_{s, \text{provided}} = 307.9 \text{ mm}^2 > A_{s, \text{required}} = 293.3 \text{ mm}^2$ **Ok**

Check for strain:

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{307.9 \times 420}{0.85 \times 520 \times 24} = 12.19 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{9.66}{0.85} = 14.34 \text{ mm}$$

$$\epsilon_s = 0.003 \left(\frac{d - c}{c} \right) = 0.003 \left(\frac{316 - 14.34}{14.34} \right) = 0.063 > 0.005 \quad \text{Ok}$$

***Design of negative moment:**

$$M_u = -25.1 \text{ KN.m.}$$

Assume bar diameter Ø 12 for main positive reinforcement.

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

$$R_n = \frac{M_u}{\phi b d^2} = \frac{25.1 \times 10^6}{0.9 \times 120 \times 316^2} = 2.32 \text{ Mpa.}$$

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

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$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{420}} \right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 2.32}{420}} \right) = 0.00588$$

$$A_{s, \text{req}} = \rho \cdot b \cdot d = 0.00588 \times 120 \times 316 = 223 \text{ mm}^2$$

*Check for $A_{s, \text{min}}$.

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

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

$$1. \quad A_{s, \text{min}} = 0.25 \frac{\sqrt{24}}{420} 120 \times 316 = 110.6 \text{ mm}^2$$

$$2. \quad A_{s, \text{min}} = \frac{1.4}{420} 120 \times 316 = 126.4 \text{ mm}^2 \text{ Control}$$

$$A_{s, \text{required}} = 223 \text{ mm}^2.$$

Use 2 $\phi 14$, $A_{s, \text{provided}} = 307.9 \text{ mm}^2 > A_{s, \text{required}} = 223 \text{ mm}^2$. **Ok**

Check for strain:

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{307.9 \times 420}{0.85 \times 120 \times 24} = 52.8 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{52.8}{0.85} = 66 \text{ mm}$$

$$\epsilon_s = 0.003 \left(\frac{d - c}{c} \right) = 0.003 \left(\frac{316 - 66}{66} \right) = 0.011 > 0.005 \quad \text{Ok}$$

✓ Shear Design for (RG14):

V_u at distance d from support = **32 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**).

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

$$\phi V_c = 0.75 \times 34.05 = 25.55 \text{ KN}.$$

$$0.5 \phi V_c = 0.5 \times 25.55 = 12.78 \text{ KN}$$

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$$0.5 \phi V_c < V_u < \phi V_c \dots\dots\dots \text{NO}$$

So

$$v_s = \frac{vu}{\phi} - vc = \frac{32}{0.75} - 34.05 = 8.61 \text{ KN}$$

$$v_{s,min} = \frac{1}{16} \sqrt{f'c} b_w d = \frac{1}{16} \sqrt{24} * 120 * 316 = 11.65$$

$$v_{s,min} = \frac{1}{3} b_w d = \frac{1}{3} 120 * 316 = 12.64 \text{ KN} \dots\dots\text{control}$$

$$v_{s,min} = 12.64 \text{ KN} > v_s = 8.61 \text{ KN}$$

shear reinforcement are required.

$$\text{Take } v_s = v_{s,min} = 12.64 \text{ KN}$$

Use 2 leg $\Phi 8$.

$$A_v = 100.5 \text{ mm}^2$$

$$s = \frac{A_v * d * f_y}{v_s}$$

$$s = \frac{100.5 * 316 * 420}{12.64 * 10^3} = 1055.25 \text{ mm}$$

$$s_{max} \leq \frac{d}{2} = \frac{316}{2} = 158 \text{ mm} \quad \text{or} \quad s_{max} \leq 600 \text{ mm}$$

Use 2 leg $\Phi 8$ @ 150 mm.

4.6 Design of Beam(BG28) :

✓ **Load calculations:**

Load calculations for BG28:

Table Dead Load Calculations for Beam(BG28)(4.4):-

Type	b γ h	KN/m
Tiles	0.03*1*23	0.69
Mortar	0.03*1*22	0.66
Sand	0.07*1*17	1.19
Reinforced concrete	0.35*1*25	8.75
Plaster	0.02*1*22	0.44
Sum		11.7

The distributed Dead and Live loads acting upon BG28 can be defined from the support reactions of the RG14.

1. From RG14

The maximum support reaction (factored) from Dead Loads for RG14 upon BG28 is 30.43 KN . The distributed Dead Load from the RG14 on BG28:

$$DL = 30.43 / 0.52 = 60.45 \text{ KN/m}$$

Live Load calculations: The maximum support reaction (factored) from Live Loads for RG14 upon BG28 is 19.2 KN .

The distributed Live Load from the RG14 on BG28:

$$LL = 19.2 / 0.52 = 36.92 \text{ KN/m}$$

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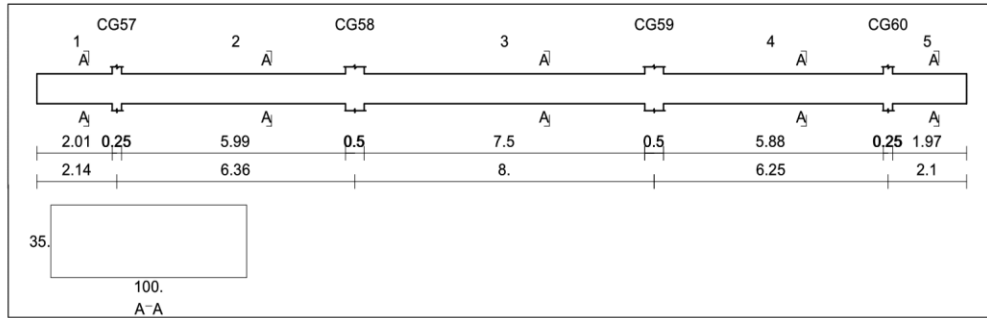


Fig 4.4: BG28

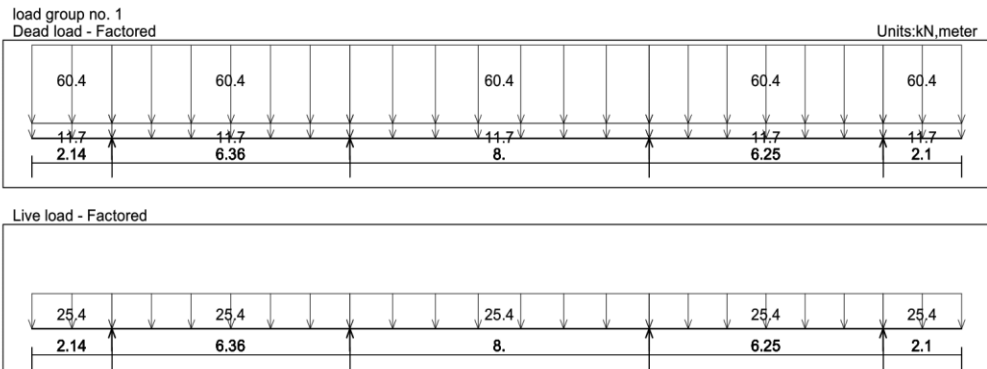
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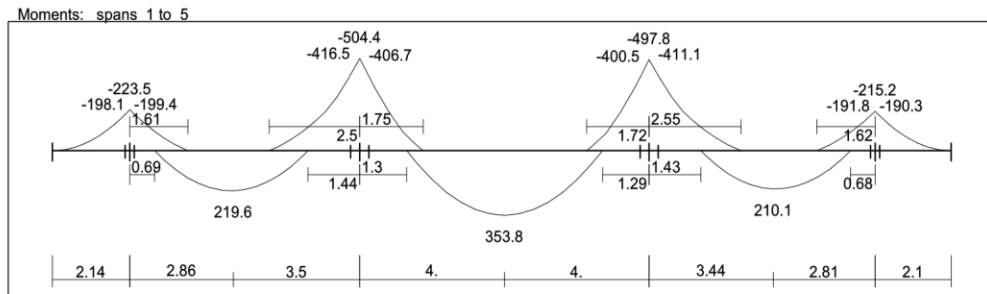
Geometry Units: meter, cm



Loading



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



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

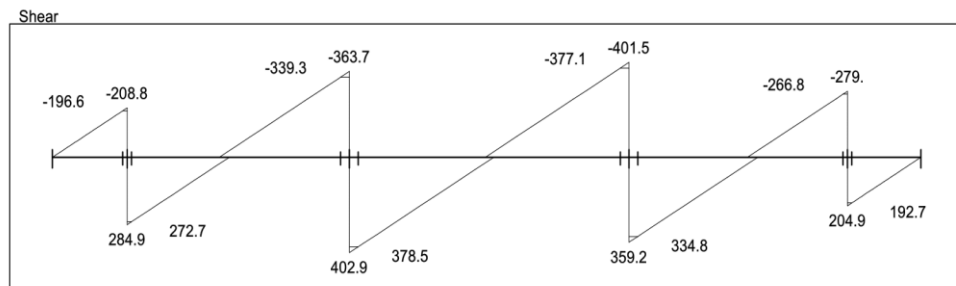


Fig 4.5 Loading and Moment /Shear Envelope.

✓ **Flexural Design for (BG28) :**

Determine of $M_{n,max}$:

$$d = 350 - 40 - 8 - \frac{18}{2} = 293mm$$

$$c = \frac{3}{7}d = \frac{3}{7} * 293 = 125.57 mm$$

$$a = \beta_1 c = 125.57 * 0.85 = 106.73mm$$

$$M_{n,max} = 0.85f'_c b a \left(d - \frac{a}{2} \right) = 0.85 * 24 * 106.73 * 1000 * (293 - 106.73/2) * 10^{-6} = 521.75 \text{ KN.m}$$

$$\phi M_{n,max} = 0.82 * 521.75 = 427.84 \text{ KN.m} < -416.5$$

Design as singly reinforcement

Design for positive moment :

$$1) M_u = 353.5 \text{ KN}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{353.8 \times 10^6}{0.9 \times 1000 \times 293^2} = 4.58 \text{ Mpa.}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2mR_n}{420}} \right) = \frac{1}{20.58} \left(1 - \sqrt{1 - \frac{2 \times 20.58 \times 4.58}{420}} \right) = 0.01251$$

$$A_s = \rho \cdot b \cdot d = 0.01251 \times 1000 \times 293 = 3667.46 \text{ mm}^2.$$

Check for $A_{s,min}$.

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

$$A_{s,min} = 0.25 \frac{\sqrt{24}}{420} 1000 \times 293 = 854.4 \text{ mm}^2$$

$$A_{s,min} = \frac{1.4}{420} 1000 \times 293 = 976.6 \text{ mm}^2 \text{ Control.}$$

$$A_{s,min} = 976.6 \text{ mm}^2 < A_s = 3667.46 \text{ mm}^2$$

Use 15Ø 18 Bottom, $A_{s,provided} = 3817.5 \text{ mm}^2 > A_{s,required} = 3667.46 \text{ mm}^2$ Ok

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Check spacing :

$$S = \frac{1000 - 40 \times 2 - 10 \times 2 - (15 \times 18)}{14} = 45 \text{ mm} > 25 \quad \dots \quad OK$$

Check for strain:

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{3817.5 \times 420}{0.85 \times 1000 \times 24} = 78.6 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{78.6}{0.85} = 92.51 \text{ mm}$$

$$\epsilon_s = 0.003 \left(\frac{d - c}{c} \right) = 0.003 \left(\frac{293 - 92.5}{92.5} \right) = 0.0065 > 0.005 \quad Ok$$

2) $M_u = 210.1 \text{ kN.m}$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{210.1 \times 10^6}{0.9 \times 1000 \times 293^2} = 2.72 \text{ Mpa.}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 m R_n}{420}} \right) = \frac{1}{20.58} \left(1 - \sqrt{1 - \frac{2 \times 20.58 \times 2.72}{420}} \right) = 0.00697$$

$$A_s = \rho \cdot b \cdot d = 0.00697 \times 1000 \times 293 = 2044.3 \text{ mm}^2.$$

Check for $A_{s, \min}$.

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

$$A_{s, \min} = 0.25 \frac{\sqrt{24}}{420} 1000 \times 293 = 854.4 \text{ mm}^2$$

$$A_{s, \min} = \frac{1.4}{420} 1000 \times 293 = 976.6 \text{ mm}^2 \text{ Control.}$$

$$A_{s, \min} = 2044.3 \text{ mm}^2 > A_s = 976.6 \text{ mm}^2$$

Use **9# 18 Bottom**, $A_{s, \text{provided}} = 2290.5 \text{ mm}^2 > A_{s, \text{required}} = 2044.3 \text{ mm}^2$ Ok

Check spacing :

$$S = \frac{1000 - 40 \times 2 - 2 \times 10 - (9 \times 18)}{8} = 92.25 \text{ mm} > 25 \quad \dots \quad OK$$

Check for strain:

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{2290.5 \times 420}{0.85 \times 1000 \times 24} = 47.15 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{47.15}{0.85} = 55.48 \text{ mm}$$

$$\epsilon_s = 0.003 \left(\frac{d - c}{c} \right) = 0.003 \left(\frac{293 - 55.48}{55.48} \right) = 0.0128 > 0.005 \quad \text{Ok}$$

3) $M_u = 219.6 \text{ KN.m}$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{219.6 \times 10^6}{0.9 \times 1000 \times 293^2} = 2.84 \text{ Mpa.}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 m R_n}{420}} \right) = \frac{1}{20.58} \left(1 - \sqrt{1 - \frac{2 \times 20.58 \times 2.84}{420}} \right) = 0.00731$$

$$A_s = \rho \cdot b \cdot d = 0.00731 \times 1000 \times 293 = 2142.44 \text{ mm}^2.$$

Check for $A_{s,\min}$.

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

$$A_{s,\min} = 0.25 \frac{\sqrt{24}}{420} 1000 \times 293 = 854.4$$

$$A_{s,\min} = \frac{1.4}{420} 1000 \times 293 = 976.6 \text{ mm}^2 \text{ Control.}$$

$$A_{s,\min} = 976.6 \text{ mm}^2 < A_s = 2142.44 \text{ mm}^2$$

Use 9Ø18 Bottom. $A_{s,\text{provided}} = 2290.5 \text{ mm}^2 > A_{s,\text{required}} = 2142.44 \text{ mm}^2$. Ok

Check spacing :

$$S = \frac{1000 - 40 \cdot 2 - 2 \cdot 10 - (9 \times 18)}{8} = 92.25 \text{ mm} > 25 \text{ OK}$$

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Check for strain:

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{2290.5 \times 420}{0.85 \times 1000 \times 24} = 47.15 \text{ mm}$$

$$c = \frac{a}{B_1} = \frac{47.15}{0.85} = 55.48 \text{ mm}$$

$$\epsilon_s = 0.003 \left(\frac{d - c}{c} \right) = 0.003 \left(\frac{293 - 55.48}{55.48} \right) = 0.0128 > 0.005 \quad \text{Ok}$$

Design for Negative moment :

$$1) M_u = -416.5 \text{ KN.m}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{416.5 \times 10^6}{0.9 \times 1000 \times 293^2} = 5.39 \text{ Mpa.}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 m R_n}{420}} \right) = \frac{1}{20.58} \left(1 - \sqrt{1 - \frac{2 \times 20.58 \times 5.39}{420}} \right) = 0.0152$$

$$A_s = \rho \cdot b \cdot d = 0.0152 \times 1000 \times 293 = 4458.2 \text{ mm}^2.$$

Check for $A_{s,min}$.

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

$$A_{s,min} = 0.25 \frac{\sqrt{24}}{420} 1000 \times 293 = 854.4$$

$$A_{s,min} = \frac{1.4}{420} 1000 \times 293 = 976.6 \text{ mm}^2 \text{ Control.}$$

$$A_{s,min} = 976.6 \text{ mm}^2 < A_s = 4458.2 \text{ mm}^2$$

$$\text{Use } 18 \text{ } \phi 18 \text{ Top, } A_{s,provided} = 4581 \text{ mm}^2 > A_{s,required} = 4458.2 \text{ mm}^2. \quad \text{Ok}$$

Check spacing :

$$S = \frac{1000 - 40 \cdot 2 - 2 \cdot 10 - (18 \times 18)}{17} = 33.9 \text{ mm} > 25 \quad \dots \quad \text{OK}$$

Check for strain:

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{4581 \times 420}{0.85 \times 1000 \times 24} = 94.31 \text{ mm}$$

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$$c = \frac{a}{\beta_1} = \frac{45.75}{0.85} = 53.83 \text{ mm}$$

$$\epsilon_s = 0.003 \left(\frac{d - c}{c} \right) = 0.003 \left(\frac{293 - 94.31}{94.31} \right) = 0.0063 > 0.005 \quad \text{Ok}$$

Design for negative moment in cantilever:

$$2) M_u = -199.4 \text{ KN.m}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{199.4 \times 10^6}{0.9 \times 1000 \times 293^2} = 2.58 \text{ Mpa.}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{420}} \right) = \frac{1}{20.58} \left(1 - \sqrt{1 - \frac{2 \times 20.58 \times 2.58}{420}} \right) = 0.00659$$

$$A_s = \rho \cdot b \cdot d = 0.00659 \times 1000 \times 293 = 1930.78 \text{ mm}^2.$$

Check for $A_{s,\min}$.

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

$$A_{s,\min} = 0.25 \frac{\sqrt{24}}{420} 1000 \times 293 = 854.4$$

$$A_{s,\min} = \frac{1.4}{420} 1000 \times 293 = 976.6 \text{ mm}^2 \text{ Control.}$$

$$A_{s,\min} = 976.6 \text{ mm}^2 < A_s = 1930.78 \text{ mm}^2$$

Use 8Ø 18 **Top**, $A_{s,\text{provided}} = 2036 \text{ mm}^2 > A_{s,\text{required}} = 1930.78 \text{ mm}^2$ Ok

Check spacing :

$$S = \frac{1000 - 40 \cdot 2 - 2 \cdot 10 - (8 \times 18)}{7} = 108 \text{ mm} > 25 \dots \text{OK}$$

Check for strain:

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{2036 \times 420}{0.85 \times 1000 \times 24} = 41.91 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{41.9}{0.85} = 49.31 \text{ mm}$$

$$\varepsilon_s = 0.003 \left(\frac{d - c}{c} \right) = 0.003 \left(\frac{293 - 49.31}{49.31} \right) = 0.0148 > 0.005 \quad \text{ok}$$

Design positive moment in cantilever:

In this area we will put minimum steel in order to catch concrete.

Use 2 Φ 18 .

✓ **Shear Design for (BG28):**

1. $V_u = 378.5 \text{ KN}$

$$V_c = \frac{1}{6} \sqrt{f'c} b_w d = \frac{1}{6} \sqrt{24} * 1000 * 293 * 10^{-3} = 239.22 \text{ KN}$$

$$\Phi V_c = 0.75 * 239.22 = 179.42 \text{ KN}$$

$$v_{s,min} = \frac{1}{3} b_w d = \frac{1}{3} 1000 * 293 * 10^{-3} = 97.66 \text{ KN control}$$

$$v_{s,min} = \frac{1}{16} \sqrt{f'c} b_w d = \frac{1}{16} * \sqrt{24} * 1000 * 293 * 10^{-3} = 89.7 \text{ KN}$$

$$v_{s'} = \frac{1}{3} \sqrt{f'c} b_w d = \frac{1}{3} \sqrt{24} * 1000 * 293 * 10^{-3} = 478.46$$

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

$$0.75(239.22 + 97.66) < 378.5 < 0.75(239.22 + 478.46)$$

$$252.66 < 378.5 < 538.26 \text{ ...ok}$$

shear reinforcement are required .

Use 2 leg Φ 10 .

$$A_v = 157.1 \text{ mm}^2 .$$

$$V_s = V_u - V_c = \frac{378.5}{0.75} - 239.22 = 265.45 \text{ KN}$$

$$S = \frac{A_v f_{yt} d}{v_s} = \frac{157.1 * 420 * 293}{265.45 * 1000} = 72.8 \text{ mm} \quad (\text{control})$$

$$s_{max} \leq \frac{d}{2} = \frac{293}{2} = 146.5 \text{ mm} \quad \text{or } s_{max} \leq 600 \text{ mm}$$

Use 2 leg $\Phi 10$ @75 mm .

2. $V_u = 339.3 \text{ KN}$

$$V_c = \frac{1}{6} \sqrt{f_c'} b_w d = \frac{1}{6} \sqrt{24} * 1000 * 293 * 10^{-3} = 239.22 \text{ KN}$$

$$\Phi V_c = 0.75 * 239.22 = 179.42 \text{ KN}$$

$$v_{s,min} = \frac{1}{3} b_w d = \frac{1}{3} 1000 * 293 * 10^{-3} = 97.66 \text{ KN control}$$

$$v_{s,min} = \frac{1}{16} \sqrt{f_c'} b_w d = \frac{1}{16} * \sqrt{24} * 1000 * 293 * 10^{-3} = 89.7 \text{ KN}$$

$$v_{s'} = \frac{1}{3} \sqrt{f_c'} b_w d = \frac{1}{3} \sqrt{24} * 1000 * 293 * 10^{-3} = 478.46$$

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

$$0.75(239.22+97.66) < 339.3 < 0.75(239.22+478.46)$$

$$252.66 < 339.3 < 538.26 \text{ ...ok}$$

shear reinforcement are required .

Use 2 leg $\Phi 10$.

$$A_v = 157.1 \text{ mm}^2 .$$

$$V_s = V_u - V_c = \frac{339.3}{0.75} - 239.22 = 213.18 \text{ KN}$$

$$S = \frac{A_v f_{yt} d}{v_s} = \frac{157.1 * 420 * 293}{213.18 * 1000} = 90.7 \text{ mm} \quad (\text{control})$$

$$s_{max} \leq \frac{d}{2} = \frac{293}{2} = 146.5 \text{ mm} \quad \text{or } s_{max} \leq 600 \text{ mm}$$

Use 2 leg $\Phi 10$ @100 mm .

3. $V_u = 272.7 \text{ KN}$

$$V_c = \frac{1}{6} \sqrt{f_c'} b_w d = \frac{1}{6} \sqrt{24} * 1000 * 293 * 10^{-3} = 239.22 \text{ KN}$$

$$\Phi V_c = 0.75 * 239.22 = 179.42 \text{ KN}$$

$$v_{s,min} = \frac{1}{3} b_w d = \frac{1}{3} 1000 * 293 * 10^{-3} = 97.66 \text{ KN control}$$

$$v_{s,min} = \frac{1}{16} \sqrt{f_c'} b_w d = \frac{1}{16} * \sqrt{24} * 1000 * 293 * 10^{-3} = 89.7 \text{ KN}$$

$$v_{s'} = \frac{1}{3} \sqrt{f_c'} b_w d = \frac{1}{3} \sqrt{24} * 1000 * 293 * 10^{-3} = 478.46$$

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

$$0.75(239.22 + 97.66) < 272.7 < 0.75(239.22 + 478.46)$$

$$252.66 < 272.7 < 538.26 \dots \text{ok}$$

shear reinforcement are required .

Use 2 leg $\Phi 10$.

$$A_v = 157.1 \text{ mm}^2 .$$

$$V_s = V_n - V_c = \frac{272.7}{0.75} - 239.22 = 124.4 \text{ KN}$$

$$s = \frac{A_v f_{yt} d}{v_s} = \frac{157.1 * 420 * 293}{124.4 * 1000} = 155.4 \text{ mm} \quad (\text{control})$$

$$s_{max} \leq \frac{d}{2} = \frac{293}{2} = 146.5 \text{ mm} \quad (\text{control}) \quad \text{or } s_{max} \leq 600 \text{ mm}$$

Use 2 leg $\Phi 10$ @ 150 mm .

4.7 Design Two Way Ribbed Slab

✓ **Determination of Thickness for Two Way Ribbed Slab:**

Assume $H = 35\text{cm}$

$$I_{b1} = \frac{100 \cdot 35^3}{12} = 357291.66 \text{ cm}^4$$

$$I_{b2} = \frac{80 \cdot 35^3}{12} = 285833.3 \text{ cm}^4$$

$$I_{b3} = \frac{100 \cdot 35^3}{12} = 357291.66 \text{ cm}^4$$

$$I_{b4} = \frac{80 \cdot 35^3}{12} = 285833.3 \text{ cm}^4$$

$$Y_c = \frac{40 \cdot 8 \cdot 4 + 35 \cdot 12 \cdot 16}{40 \cdot 8 + 35 \cdot 12} = 10.81 \text{ cm}$$

$$I_r = \frac{52 \cdot 10.81^3}{3} - \frac{40 \cdot 2.81^3}{3} + \frac{21 \cdot 24.19^3}{3} = 78219.6 \text{ cm}^4$$

$$I_{s1} = \frac{78219.6 \cdot \left(\frac{760}{2} + 80\right)}{52} = 722027.07 \text{ cm}^4$$

$$I_{s2} = \frac{78219.6 \cdot \left(\frac{760}{2} + 80\right)}{52} = 691942.6 \text{ cm}^4$$

$$I_{s3} = \frac{78219.6 \cdot \left(\frac{880}{2} + 100\right)}{52} = 812280.4615 \text{ cm}^4$$

$$I_{s4} = \frac{78219.6 \cdot \left(\frac{880}{2} + 100\right)}{52} = 782196 \text{ cm}^4$$

$$\alpha_1 = \frac{285833.3}{691942.6} = 0.41$$

$$\alpha_2 = \frac{357291.66}{812280.46} = 0.43$$

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$$\alpha_3 = \frac{337291.66}{722027.07} = 0.46$$

$$\alpha_4 = \frac{285833.3}{812280.46} = 0.36$$

$$\alpha_{fm} = \frac{0.41+0.46+0.43+0.36}{4} = 0.415 > 0.2$$

$$\beta = \frac{8.8}{7.6} = 1.12$$

$$h_{min} = \frac{8800 * (0.8 + (\frac{420}{1400}))}{36 + 1.15 * 5 * (0.415 - 0.2)} = 26.06 \text{ cm}$$

$$h = 35 \text{ cm} > h_{min} = 26.06 \text{ cm}$$

✓ **Load Calculation:**

Table 4.5: table of two way rib calculation.

No.	Parts of Flight	Calculation
1	Tiles	$22 * 0.03 * 0.52 * 0.52 = 0.178 \text{ KN}$
2	Mortar	$22 * 0.02 * 0.52 * 0.52 = 0.119 \text{ KN}$
3	Sand	$16 * 0.07 * 0.52 * 0.52 = 0.303 \text{ KN}$
4	Topping	$25 * 0.08 * 0.52 * 0.52 = 0.541 \text{ KN}$
5	Rib	$25 * 0.27 * 0.12 * (0.52 + 0.4) = 0.745 \text{ KN}$
6	Block	$9 * 0.27 * 0.4 * 0.4 = 0.389 \text{ KN}$
7	Plaster	$22 * 0.02 * 0.52 * 0.52 = 0.119 \text{ KN}$
Sum		2.80 KN

Dead Load of slab:

$$DL = \frac{2.80}{0.52 \times 0.52} = 10.36 \text{ KN/m}^2$$

$$W_D = 1.2 \times 10.36 = 12.432 \text{ KN/m}^2$$

$$LL = 3 \text{ KN/m}^2$$

$$W_L = 1.6 \times 3 = 4.8 \text{ KN/m}^2$$

$$W = 12.432 + 4.8 = 17.232 \text{ KN/m}^2$$

✓ **Moments Calculations:**

From tables use Case (1): C neg = 0.0

$$C_{a,dL} = 0.049 \text{ , , } C_{b,dL} = 0.032$$

$$C_{a,LL} = 0.049 \text{ , , } C_{b,LL} = 0.032$$

$$M_{a,d}^+ = C_{a,dL} \times W_{ud} \times (L_a)^2 = 0.049 \times 12.432 \times (7.6)^2 \times 0.52 = 18.3 \text{ KN.m / rib.}$$

$$M_{a,L}^+ = C_{a,LL} \times W_{ul} \times (L_a)^2 = 0.049 \times 4.8 \times (7.6)^2 \times 0.52 = 7.06 \text{ KN.m / rib.}$$

$$M_{a \text{ positive}} = 18.3 + 7.06 = 25.36 \text{ KN.m / rib.}$$

Negative moment at discontinuous edges = 1/3
positive

$$M_{a \text{ neg}} = (1/3) \times 25.36 = 8.45 \text{ KN.m / rib.}$$

$$M_{b,d}^+ = C_{b,dL} \times W_{ud} \times (L_b)^2 = 0.032 \times 12.432 \times (8.8)^2 \times 0.52 = 16.01 \text{ KN.m / rib.}$$

$$M_{b,L}^+ = C_{b,LL} \times W_{ul} \times (L_b)^2 = 0.032 \times 4.8 \times (8.8)^2 \times 0.52 = 6.18 \text{ KN.m / rib.}$$

$$M_{b \text{ positive}} = 16.01 + 6.18 = 22.19 \text{ KN.m / rib.}$$

Negative moment at discontinuous edges = 1/3
positive.

$$M_b \text{ neg} = (1/3) \times 22.19 = 5.96 \text{ KN.m / rib.}$$

✓ **design of positive moments:**

Design of Positive Moment :- (Ma=25.36KN.m)

Assume bar diameter ϕ 14 for main positive reinforcement

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

$$R_n = \frac{M_u}{\phi b d^2} = \frac{25.36 \times 10^6}{0.9 \times 520 \times 315^2} = 0.54 \text{ Mpa}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{420}} \right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.54}{420}} \right) = 0.00130$$

$$A_{s, \text{req}} = \rho \cdot b \cdot d = 0.00130 \times 520 \times 315 = 212.94 \text{ mm}^2$$

Check for As min:-

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

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

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

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

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$$A_{s_{req}} = 212.94 \text{ mm}^2 > A_{s_{min}} = 126 \text{ mm}^2 \text{ OK}$$

Use 2 ø12, A_s , provided = 226.2 mm² > $A_{s, required}$ = 212.94 mm²... Ok

Check for strain:-

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{226.2 \times 420}{0.85 \times 520 \times 24} = 8.96 \text{ mm}$$

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

$$\varepsilon_s = 0.003 \left(\frac{d - x}{x} \right) = 0.003 \left(\frac{315 - 10.54}{10.54} \right) = 0.086 > 0.005 \quad \text{Ok}$$

Design of Positive Moment:- ($M_b = 22.19 \text{ kN.m}$)

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

$$R_n = \frac{M_u}{\phi b d^2} = \frac{22.19 \times 10^6}{0.9 \times 520 \times 315^2} = 0.47 \text{ Mpa}$$

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

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

$$A_{s, req} = \rho \cdot b \cdot d = 0.001132 \times 520 \times 315 = 185.46 \text{ mm}^2$$

Check for A_s min:-

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

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

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

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

$$A_{s\text{req}} = 185.46 \text{ mm}^2 > A_{s\min} = 126 \text{ mm}^2 \text{ OK}$$

Use 2 ø12, As, provided= 226.2 mm² > A_{s, required}= 185.46 mm²... Ok

Check for strain:-

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{226.2 \times 420}{0.85 \times 520 \times 24} = 8.96 \text{ mm}$$

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

$$\epsilon_s = 0.003 \left(\frac{d - x}{x} \right) = 0.003 \left(\frac{315 - 10.54}{10.54} \right) = 0.086 > 0.005 \quad \text{Ok}$$

✓ **design of negative moments:**

Design of Negative Moment :- (Ma=-8.43KN.m)

Assume bar diameter ø 12 for negative reinforcement

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

$$R_n = \frac{M_u}{\phi b d^2} = \frac{8.43 \times 10^6}{0.9 \times 120 \times 316^2} = 0.78 \text{ Mpa}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2mR_n}{420}} \right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.78}{420}} \right) = 0.002$$

$$A_{s, \text{req}} = \rho \cdot b \cdot d = 0.002 \times 120 \times 316 = 75.84 \text{ mm}^2$$

Check for As min:-

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

$$A_{s \text{ min}} = \frac{\sqrt{24}}{4(420)} (120)(316) = 110.5 \text{ mm}^2$$

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

$$A_{s \text{ min}} = \frac{1.4}{420} (120)(316) = 126.4 \text{ mm}^2 \text{ **controls**}$$

$$A_{s \text{ req}} = 75.84 < A_{s \text{ min}} = 126.4 \text{ mm}^2 \text{ **OK**}$$

Use 2 $\phi 10$, $A_{s, \text{provided}} = 157.08 \text{ mm}^2 > A_{s, \text{required}} = 126.4 \text{ mm}^2 \dots \text{Ok}$

Check for strain:-

$$a = \frac{A_s f_y}{0.85 b f_c'} = \frac{157.08 \times 420}{0.85 \times 120 \times 24} = 26.95 \text{ mm}$$

$$x = \frac{a}{\beta_1} = \frac{26.95}{0.85} = 31.7 \text{ mm}$$

$$\epsilon_s = 0.003 \left(\frac{d - x}{x} \right) = 0.003 \left(\frac{316 - 31.7}{31.7} \right) = 0.026 > 0.005 \quad \text{Ok}$$

Design of Negative Moment :- (Mb=-7.4KN.m)

Assume bar diameter ϕ 12 for negative reinforcement

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

$$R_n = \frac{M_u}{\phi b d^2} = \frac{7.4 \times 10^6}{0.9 \times 120 \times 316^2} = 0.68 \text{ Mpa}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{420}} \right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.68}{420}} \right) = 0.00164$$

$$A_{s, \text{req}} = \rho \cdot b \cdot d = 0.00164 \times 120 \times 316 = 62.18 \text{ mm}^2$$

Check for As min:-

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

$$A_{s \text{ min}} = \frac{\sqrt{24}}{4(420)} (120)(316) = 110.57 \text{ mm}^2$$

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

$$A_{s \text{ min}} = \frac{1.4}{420} (120)(316) = 126.4 \text{ mm}^2 \text{ **controls**}$$

$$A_{s \text{ req}} = 62.18 < A_{s \text{ min}} = 126.4 \text{ mm}^2 \text{ **OK**}$$

Use 2 ϕ 10, $A_{s, \text{provided}} = 157.08 \text{ mm}^2 > A_{s, \text{required}} = 126.4 \text{ mm}^2 \dots \text{ Ok}$

Check for strain:-

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{157.08 \times 420}{0.85 \times 120 \times 24} = 26.95 \text{ mm}$$

$$x = \frac{a}{\beta_1} = \frac{26.95}{0.85} = 31.7 \text{ mm}$$

$$\varepsilon_s = 0.003 \left(\frac{d - x}{x} \right) = 0.003 \left(\frac{316 - 31.7}{31.7} \right) = 0.026 > 0.005 \quad \text{Ok}$$

✓ **design of shear for rib:**

Maximum shear coefficient in short direction as in case (1) $W_a @ m = 0.863$

$$W_a = 0.64$$

The total load on the panel = $7.6 \times 8.8 \times 17.232 = 1152.4 \text{ KN}$

The load per rib at the face of long beam = $0.64 \times 1152.4 \times 0.52 / (2 \times 8.8) = 21.79 \text{ KN}$

$$V_{ud} = V_{uface} - W_u \cdot b_f \cdot d = 21.79 - 17.232 \times 0.52 \times 0.316 = 18.95 \text{ KN}$$

The shear strength of one rib:

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

$$\phi V_c = 0.75 \times 34.05 = 25.54 \text{ KN}$$

$$0.5 \phi V_c = 0.5 \times 25.54 = 12.77 \text{ KN}$$

$$0.5 \phi V_c < V_{ud} < \phi V_c$$

$$12.77 < 18.95 < 25.54$$

Minimum shear reinforcement is required except for joist construction.

$$V_{ud} = 17.232 \times 0.52 \times (3.8 - 0.316) = 31.3 \text{ KN}$$

$$\phi V_c = 25.54 < V_{ud} = 31.3 \text{ KN}$$

$$\Phi V_{smin} \geq 0.75 \left(\frac{1}{3} \right) * b_w * d = 0.75 * \left(\frac{1}{3} \right) * 120 * 316 * 10^{-3} = 9.48 \text{ KN Controls}$$

$$\Phi V_{smin} \geq 0.75 \left(\frac{\sqrt{f_c'}}{16} \right) * b_w * d = 0.75 * \left(\frac{\sqrt{24}}{16} \right) * 120 * 316 * 10^{-3} = 8.7 \text{ KN}$$

$$\Phi (V_c + V_{smin}) = 35.021 > V_u = 31.3 > \Phi V_c = 25.54$$

$$V_s' = \frac{1}{3} \sqrt{f_c'} * b_w * d = 61.9 \text{ KN}$$

$$V_s = \frac{V_u}{\Phi} - V_c = 12.92 \text{ KN}$$

$$V_{smin} = 11.4 < V_s = 12.92 < V_s' = 55.85 \dots\dots\dots \text{Case IV}$$

Use 2 leg $\Phi 8$

$$A_{vmin} = 100 \text{ mm}^2$$

$$\frac{A_{vmin}}{s} = \frac{v_s}{df_{yt}}$$

$$s = \frac{A_{vmin}}{v_s} df_{yt}$$

$$s = 926.5 \text{ mm}$$

$$s_{max} \leq \frac{d}{2} = \frac{285}{2} = 142.5 \text{ mm} \quad \text{or } s_{max} \leq 600 \text{ mm}$$

Use 4 leg $\Phi 8 @ 125 \text{ mm}$

4.8 Design of One Way Solid Slab

✓ Material:-

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

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

✓ Slab Thickness Calculation:-

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

Min H (deflection requirement):-

-For one end continuous:-

$$\frac{L}{24} = \frac{4.25}{24} = 0.17$$

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

✓ 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)

Table 4.6: Dead Load Calculation of Solid Slab.

#	material	calculation
1	Tiles	0.03*22=0.66
2	mortar	0.03*22=0.66
3	Coarse sand	0.07*16=1.12
4	RC concrete	0.30*25=7.5
5	plaster	0.02*22=0.44
	Sum	10.38

Live load =5 KN/m

✓ **Design of Positive Moment :**

Design of Positive Moment :-(Mu=25.3 KN.m)

Assume bar diameter Φ14 for main reinforcement

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

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

$$R_n = \frac{25.3 * 10^6 / 0.9}{1000 * (173)^2} = 0.939(\text{Mpa})$$

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

$$\rho = \frac{1}{20.59} \left(1 - \sqrt{1 - \frac{2(20.59)(0.939)}{420}} \right) = 0.00228$$

$$A_s = \rho * b * d = 0.00228 * 100 * 143 = 3.3 \text{ cm}^2$$

Check for As min:-

$$A_s \text{ min} = \rho_{\text{min}} * b * h = 0.0018 * 100 * 20 = 3.6 \text{ cm}^2$$

$$A_{s\text{req}} = 3.3 \text{ cm}^2 > A_{s\text{min}} = 3.6 \text{ cm}^2 \quad \text{NOT OK}$$

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

Design of Positive Moment :-(Mu=27.6KN.m)

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

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

$$R_n = \frac{27.6 * 10^6 / 0.9}{1000 * (173)^2} = 1.02 \text{ (Mpa)}$$

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

$$\rho = \frac{1}{20.59} \left(1 - \sqrt{1 - \frac{2(20.59)(1.02)}{420}} \right) = 0.00245$$

$$A_s = \rho * b * d = 0.00245 * 100 * 143 = 3.56 \text{ cm}^2$$

Check for As min:-

$$A_s \text{ min} = \rho_{\text{min}} * b * h = 0.0018 * 100 * 20 = 3.6 \text{ cm}^2$$

$$A_{s\text{req}} = 3.56 \text{ cm}^2 > A_{s\text{min}} = 3.6 \text{ cm}^2 \quad \text{Not OK}$$

Use ϕ 12/20cm , $A_{s,\text{provided}} = 5.65 \text{ cm}^2 \geq A_{s,\text{required}} = 3.6 \text{ cm}^2$ Ok

✓ **Design of Negative Moment:**

Design of Negative Moment:- (Mu=23.4 KN.m)

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

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

$$R_n = \frac{23.4 * 10^6 / 0.9}{1000 * (173)^2} = 0.868(\text{Mpa})$$

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

$$\rho = \frac{1}{20.59} \left(1 - \sqrt{1 - \frac{2(20.59)(0.868)}{420}} \right) = 0.0021$$

$$A_s = \rho * b * d = 0.0021 * 100 * 173 = 3.65 \text{ cm}^2$$

Check for As min:-

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

$$A_{s \text{ req}} = 3.65 \text{ cm}^2 > A_{s \text{ min}} = 3.6 \text{ cm}^2 \quad \text{OK}$$

Use ϕ 12/20cm , $A_{s \text{ provided}} = 5.65 \text{ cm}^2 \geq A_{s \text{ required}} = 3.6 \text{ cm}^2$ Ok

Shrinkage and Temperature:-

$$\rightarrow \rho = 0.0018$$

$$A_{s \text{ min}} = \rho_{\text{min}} * b * h = 0.0018 * 100 * 20 = 3.6 \text{ cm}^2 \quad (\text{control})$$

Use $\Phi 10$ @ 200 mm

✓ **Shear Design:-**

Check Whether Thickness Is Adequate For Shear:-

$$V_{u,\max} = 47.4 \text{ KN/ 1m strip}$$

$$d = h - 20 - db = 200 - 20 - (14 / 2) = 173 \text{ mm}$$

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

$$= \frac{1}{6} * 0.75 * \sqrt{24} * 1000 * 173 = 105.9 \text{ KN / 1 m strip}$$

$$\Phi V_c = 79.42 \text{ KN} > V_{u,\max} = 47.4 \text{ KN/ 1m strip}$$

The thickness of the slab is adequate enough.

4.9 Design of Two Way Solid Slab:

✓ Calculate the minimum thickness slab :

$$h_{min} = 22 \text{ cm}$$

$$y(CB6) = \frac{20 * 120 * 30 + 80 * 200}{20 * (80 + 20) + 80 * 20} = 21.1 \text{ cm}$$

$$Ib(CB6) = \frac{120 * 18.9^3}{3} + \frac{80 * 20^3}{3} = 225575.6 \text{ cm}^4$$

$$y(CB5) = \frac{20(80 + 40) * 30 + 80 * 30}{20 * (80 + 40) + 20 * 80} = 22 \text{ cm}$$

$$Ib(CB5) = \frac{120 * 18^3}{3} + \frac{80 * 20^3}{3} = 225575.6 \text{ cm}^4$$

$$Is1 = Is2 = \frac{(380 + 35) * 20^3}{12} = 275000 \text{ cm}^4$$

$$Is3 = \frac{(325 + 35) * 20^3}{12} = 240000 \text{ cm}^4$$

$$Is4 = \frac{(655 + 35) * 20^3}{12} = 460000 \text{ cm}^4$$

$$\alpha f1 = \alpha f2 = \frac{Ib66}{Is2} = \frac{Ib56}{Is1} = \frac{1160412.5}{275000} = 4.264$$

$$\alpha f3 = \frac{Ib55}{Is3} = \frac{1172751}{240000} = 4.264$$

$$\alpha f4 = \frac{Ib53}{Is4} = \frac{1160412.5}{460000} = 2.522$$

$$\alpha fm = \frac{\epsilon \alpha}{4} = \frac{2 * 2.522 + 2 * 4.264}{4} = 3.393$$

$$\text{for } \alpha fm \leq 0.2\beta = \frac{I_{nlong}}{I_{nshort}} = 6.8/6.3 = 1.07$$

$$h_{min} = \frac{ln * (0.8 + Fy/1400)}{36 + 9B} = \frac{780 * (0.8 + 420/1400)}{36 + 9 * 1.07} = 16.3 \text{ cm}$$

but we will select 20cm slab thickness.

✓ **Dead load calculations:**

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

Dead load from:	$\delta \times \gamma$	KN/m
Tiles	$0.03 \times 23 \times 1$	0.69
Mortar	$0.02 \times 22 \times 1$	0.44
Coarse sand	$0.07 \times 16 \times 1$	1.12
Slab	$0.20 \times 25 \times 1$	5
Plaster	$0.02 \times 22 \times 1$	0.44
Partitions	2×1	2
		8.75

Dead load = 8.75 KN/m².

Live load = 5 KN/m².

WuD = 1.2 * Dead load = 1.2 * 8.75 = 10.5 KN/m².

WuL = 1.6 * live load = 1.6 * 5 = 8 KN/m².

Wu = 10.5 + 8 = 18.5 KN/m²

✓ **Shear Design :**

$l_a/l_b = 0.85$

$W_b = 0.79$

$W_a = 0.21$

- The total load on the panel being ($7.6 * 6.5 * 19.604$) = 968.487 KN
- The load at face of the long beam is ($0.79 \times 1614 / (2 * 6.5)$) = 58.86 KN

Assume the Φ 16

$$d = 200 - 20 - 12 \div 2 = 174 \text{ mm}$$

- $V_c = (\sqrt{28} * 1000 * 174 * 10^{-3}) \div 6 = 153.45 \text{ KN}$

$$\phi V_c = 0.75 \times 181.26 = 115.1 \text{ KN}$$

$$V_u < \phi V_c.$$

The thickness of the slab is adequate enough

✓ **Flexural Design:**

$$(I_a/I_b = 0.92)$$

Positive moments :

$$C_a D = 0.0318$$

$$C_a L = 0.037$$

$$C_b D = 0.0228$$

$$C_b L = 0.0272$$

$$M_{a+ve, D} = C_a * W * L_a^2 = 0.0318 * 10.5 * 6.3^2 = 13.25 \text{ KN.m/m}$$

$$M_{a+ve, L} = C_a * W * L_a^2 = 0.0374 * 8 * 6.3^2 = 11.806 \text{ KN.m/m}$$

$$\underline{M_{a+ve} = M_{a+ve, L} + M_{a+ve, D} = 25.05 \text{ KN.m/m}}$$

$$M_{b+ve, D} = C_b * W * L_b^2 = 0.0228 * 10.5 * 6.8^2 = 11.06 \text{ KN.m/m}$$

$$M_{b+ve, L} = C_b * W * L_b^2 = 0.0228 * 8 * 6.8^2 = 8.43 \text{ KN.m/m}$$

$$\underline{M_{b+ve} = M_{b+ve, L} + M_{b+ve, D} = 16.83 \text{ KN.m/m}}$$

✓ **Positive Moment:**

***Mub = 16.83KN.m/m**

Assume the d_{Bar} = 12 mm

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

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

$$= \frac{16.83 * 10^6 / 0.9}{1000 * 176^2} = 0.60 MPa .$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2mK_n}{f_y}} \right) = \frac{1}{20.59} \left(1 - \sqrt{1 - \frac{2 * 20.59 * 0.6}{420}} \right) = 0.003$$

$$A_{s_{req}} = 0.003 * 1000 * 176 = 534 \text{ mm}^2/\text{m}$$

$$A_{s_{min}} = 0.0018 * b * h = 0.0018 * 1000 * 200 = 360 \text{ mm}^2 / \text{m}$$

$$A_s = 534 \text{ mm}^2 \geq A_{s_{min}} = 360 \text{ mm}^2/\text{m}$$

Use Φ 14 \ 20cm with As=770mm²/m

***Mua = 25.05KN.m/m**

Assume the d_{Bar} = 12 mm

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

$$K_n = \frac{Mu}{b \cdot d^2} = \frac{24.28 * 10^6 / 0.9}{1000 * 176^2} = 0.85 MPa .$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2mK_n}{f_y}} \right) = \frac{1}{20.59} \left(1 - \sqrt{1 - \frac{2 * 20.59 * 0.85}{420}} \right) = 0.002$$

$$A_{s_{req}} = 0.002 * 1000 * 174 = 348 \text{ mm}^2/\text{m}$$

$$A_{s_{\min}} = 0.0018 * b * h = 0.0018 * 1000 * 200 = 360 \text{ mm}^2 / \text{m}$$

$$A_s = 385 \text{ mm}^2 \geq A_{s_{\min}} = 360 \text{ mm}^2 / \text{m}$$

Use Φ 14 \ 20cm with $A_s=770 \text{ mm}^2/\text{m}$

✓ **Negative Moment:**

$$\mathbf{M_{ua} = 42.5 \text{ N.m/m}}$$

Assume the $d_{\text{Bar}}=12 \text{ mm}$

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

$$K_n = \frac{Mu}{b \cdot d^2} = \frac{49.7 * 10^6 / 0.9}{1000 * 176^2} = 1.768 \text{ MPa} .$$

$$m = \frac{f_y}{0.85 \times f_c'} = \frac{420}{0.85 \times 28} = 17.64$$

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2mK_n}{f_y}} \right) = \frac{1}{17.64} \left(1 - \sqrt{1 - \frac{2 * 17.64 * 1.768}{420}} \right) = 0.00438$$

$$A_{s_{\text{req}}} = 0.00438 * 1000 * 174 = 770 \text{ mm}^2 / \text{m}$$

$$A_{s_{\min}} = 0.0018 * b * h = 0.0018 * 1000 * 200 = 360 \text{ mm}^2 / \text{m}$$

$$A_s = 770 \text{ mm}^2 \geq A_{s_{\min}} = 360 \text{ mm}^2 / \text{m}$$

Use Φ 16 \ 20cm with $A_s=1004.8 \text{ mm}^2/\text{m}$

Note: other moments requires areinforcement less than minimum, Use Φ 12 \ 15cm with $A_s=678 \text{ mm}^2/\text{m}$

4.10 Design of Stair:

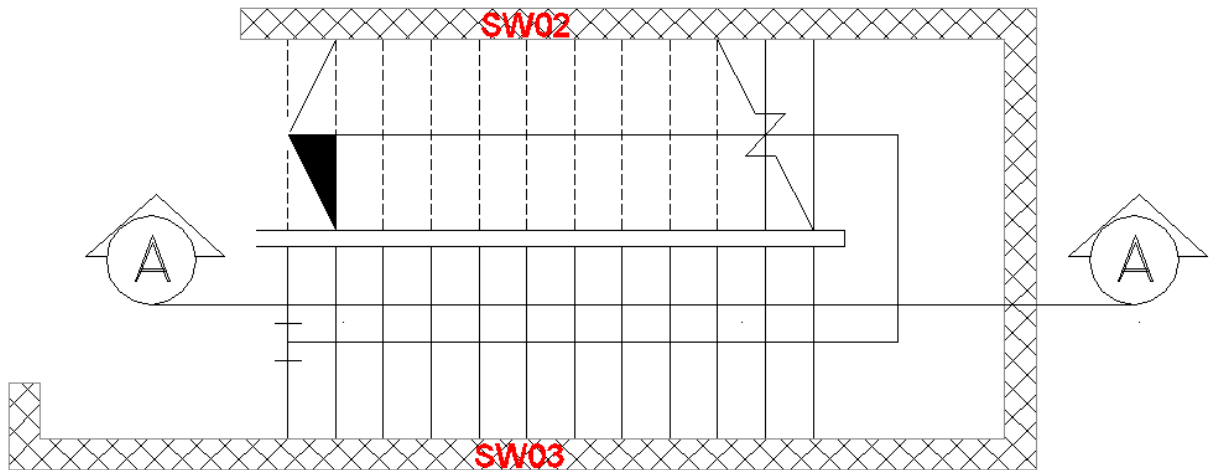


Figure4.6: Stair Plan.

✓ Material :-

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

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

✓ Design of Flight :-

✓ Determination of Thickness:-

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

$$h_{\min} = 5.8/20 = 29 \text{ cm}$$

Take $h = 30 \text{ cm}$

The Stair Slope by $\theta = \tan^{-1}(15/30) = 26.56^\circ$

✓ **Load Calculation:-**

Dead Load For Flight For 1m Strip:-

Table 1-8: Dead Load Calculation of Flight.

No.	Parts of Flight	Calculation
1	Tiles	$27 \times 0.03 \times 1 \times (0.35 + 0.15/0.3) = 1.35 \text{ KN/m}$
2	Mortar	$22 \times 0.02 \times 1 \times (0.3 + 0.15/0.3) = 0.66 \text{ KN/m}$
3	Stair	$25 \times 1 \times (0.3 + 0.15/2) / 0.3 = 1.875 \text{ KN/m}$
4	Slab	$25 \times 0.25 \times 1 / \cos 26.56 = 6.99 \text{ KN/m}$
5	Plaster	$22 \times 0.03 \times 1 / \cos 26.56^\circ = 0.738 \text{ KN/m}$
Sum		13 KN/m

Live Load For Landing For 1m Strip = $4 \times 1 = 4 \text{ KN/m}$

Factored Load For Flight :-

$$W_U = 1.2 \times 13 + 1.6 \times 4 = 23.6 \text{ KN/m}$$

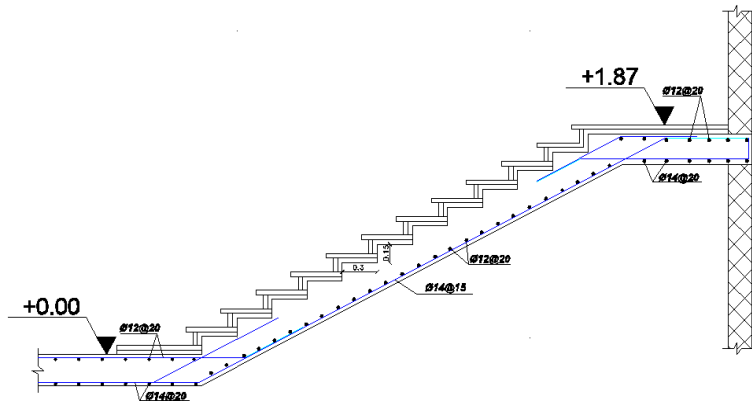


Fig 4.7: Stair Section.

$$R = (W \cdot L) / 2 = 23.6 \cdot 3.31 / 2 = 42.3 \text{ KN}$$

1- Design of Shear for Flight :- ($V_u = 27.45 \text{ KN}$)

Assume bar diameter $\phi 14$ for main reinforcement

$$d = h - \text{cover} - \frac{d_b}{2} = 300 - 20 - \frac{14}{2} = 273 \text{ mm}$$

$$V_u = 42.3 \text{ KN}$$

$$V_c = \frac{1}{6} \sqrt{f'c} b_w d = \frac{1}{6} \sqrt{24} \cdot 1000 \cdot 273 = 222.9 \text{ KN/m}$$

$$\Phi V_c = 0.75 \cdot 222.9 = 167.2 \text{ KN/m}$$

$$V_u = 42.3 < \Phi V_c = 167.2 \text{ KN/m}$$

The thickness is enough .

2- Design of Bending Moment for Flight :- ($M_u = 80.25 \text{ KN.m}$)

$$M_u = 80.25 \cdot (1.5 + 1.6) - \frac{23.6 \cdot 1.35^2}{2} = 89.1 \text{ m KN}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{89.1 \times 10^6}{0.9 \times 1000 \times 273^2} = 1.2 \text{ Mpa}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{420}} \right) = \frac{1}{20.59} \left(1 - \sqrt{1 - \frac{2 \times 20.59 \times 1.2}{420}} \right) = 0.00294$$

$$A_{s,req} = \rho \cdot b \cdot d = 0.00294 \times 1000 \times 273 = 804.2 \text{ mm}^2$$

$$A_{s,min} = 0.0018 \times 1000 \times 300 = 540 \text{ mm}^2$$

$$A_{s,req} > A_{s,min} = 540 \text{ mm}^2$$

$$A_{s,req} = 804.2 \text{ mm}^2$$

Check for Spacing :-

$$1) S = 3h = 3 \times 300 = 900 \text{ mm}$$

$$2) S = 380 \times (280 / (2/3 \times 420)) - 2.5 \times 20 = 330 \leq S = 300 \times (280 / (2/3 \times 420)) = 250 \text{ mm}$$

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

$S = 250 \text{ mm}$ is control

Use $\phi 14$ @ 250 mm

3- Lateral or Secondary Reinforcement For Flight :-

$$A_{s,req} = A_{s,min} = 0.0018 \times 1000 \times 300 = 540 \text{ mm}^2$$

Use $\phi 14$ @ 300 mm , $A_{s,provided} = 461.7 \text{ mm}^2 > A_{s,required} = 540 \text{ mm}^2$... Ok

✓ Design of Landing :

✓ Load Calculation:-

Dead Load For Landing For 1m Strip:-

Live Load For Landing For 1m Strip = $4 \times 1 = 4 \text{ kN/m}$

Factored Load For Landing :-

$$W_U = 1.2 \times 9.26 + 1.6 \times 4 = 19.3 \text{ kN/m}$$

✓ **System of Landing :-**

$$R = \frac{19.3 \times 1.7}{2} + 21.15 \times 1.9 = 75.45 \text{ KN}$$

1- Design of Shear:- (Vu=75.45 KN)

Assume bar diameter ϕ 14 for main reinforcement

$$d = h - \text{cover} - \frac{d_b}{2} = 300 - 20 - \frac{14}{2} = 273 \text{ mm}$$

$$V_c = \frac{1}{6} \sqrt{f'c} b_w d = \frac{1}{6} \sqrt{24} \times 1000 \times 273 = 222.9 \text{ KN}$$

$\Phi * V_c = 0.75 \times 222.9 = 167.2 \text{ KN} > V_u = 75.45 \text{ KN} \dots \text{Thickness of slab is enough}$

2- Design of Bending Moment :- (Mu=70.6KN.m)

Assume bar diameter ϕ 14 for main reinforcement

$$d = h - \text{cover} - \frac{d_b}{2} = 250 - 20 - \frac{14}{2} = 213 \text{ mm}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{70.6 \times 10^6}{0.9 \times 1000 \times 213^2} = 1.052 \text{ Mpa}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 m R_n}{420}} \right) = \frac{1}{20.59} \left(1 - \sqrt{1 - \frac{2 \times 20.59 \times 1.052}{420}} \right) = 0.002575$$

$$A_{s, \text{req}} = \rho \cdot b \cdot d = 0.002575 \times 1000 \times 213 = 548 \text{ mm}^2$$

$$A_{s, \text{min}} = 0.0018 \times 1000 \times 213 = 383 \text{ mm}^2$$

$$A_{s, \text{req}} = 548 \text{ mm}^2 \dots \text{is control}$$

Check for Spacing:-

4) $S = 3h = 3 \times 300 = 900 \text{ mm}$

5) $S = 380 \times (280 / (2/3 \times 420)) - 2.5 \times 20 = 330 \leq S = 300 \times (280 / (2/3 \times 420)) = 200 \text{ mm}$

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

$S = 200 \text{ mm}$ is control

Use $\phi 14 @ 200 \text{ mm}$

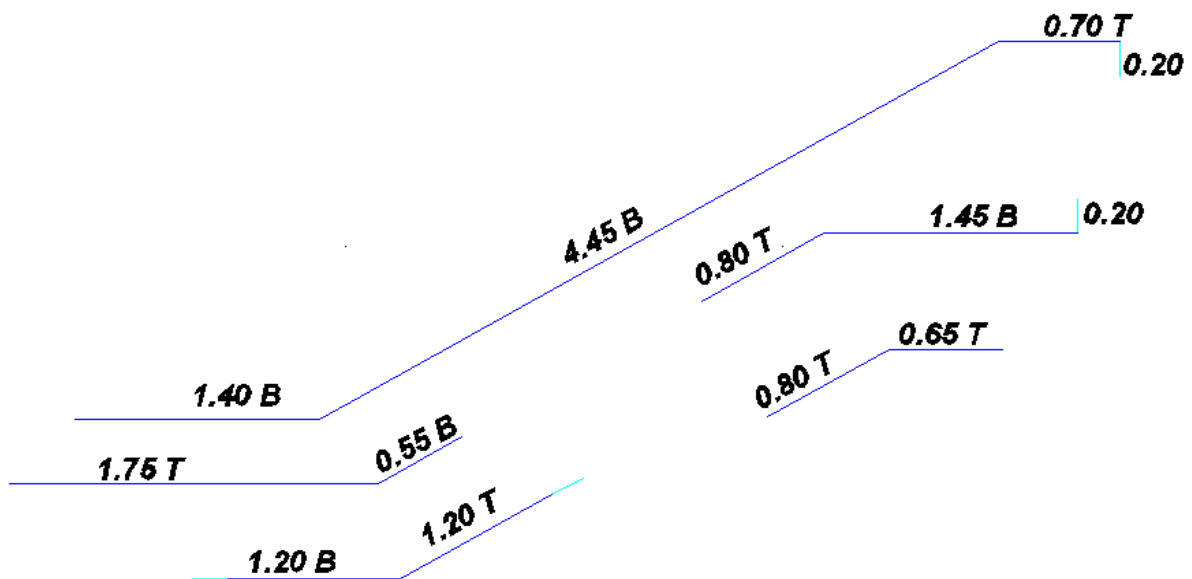


Fig 4.7: Stair Reinforcement.

4.11 Design of Column

✓ Material :-

⇒ concrete B350 $F_c' = 24 \text{ N/mm}^2$

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

✓ Load Calculation:-

Service Load:-

Dead Load = 2400KN

Live Load = 700 KN

Factored Load:-

$$P_U = 1.2 \times 2400 + 1.6 \times 700 = 4000 \text{ KN}$$

✓ Dimensions of Column:-

Assume $\rho_g = 0.01$

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

$$4000 * 1000 = 0.65 \times 0.8 \times A_g \{0.85 * 24 (1 - 0.01) + 0.01 * 420\}$$

$$A_g = 315457.4 \text{ mm}^2$$

Assume Rectangular Section

Try $h = 600 \text{ mm}$

$$b = 400$$

Selecting Longitudinal Bars:

$$4000 * 1000 = 0.65 \times 0.8 \times A_g \{0.85 * 24 (240000 - A_{st}) + A_{st} * 420\}$$

$$A_{st} = 6980 \text{ mm}^2$$

Use 12 ϕ 28, $A_{st,prov} = 7308 \text{ mm}^2 > A_{st} = 6980 \text{ mm}^2$

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

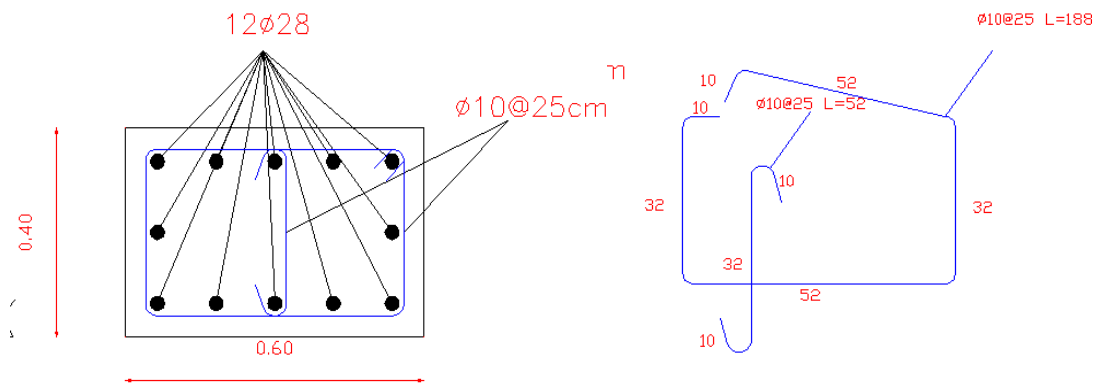


Fig 4.9:Column section and reinforcement.

✓ **Design of the tie reinforcement :**

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

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

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

spacing $\leq 16 \times d_b = 16 \times 2.8 = 44.8 \text{ cm} \dots$

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

spacing $\leq \text{least.dim} = 40 \text{ cm control}$

Use $\phi 10 @ 25 \text{ cm}$

4.12 Design of shear wall:

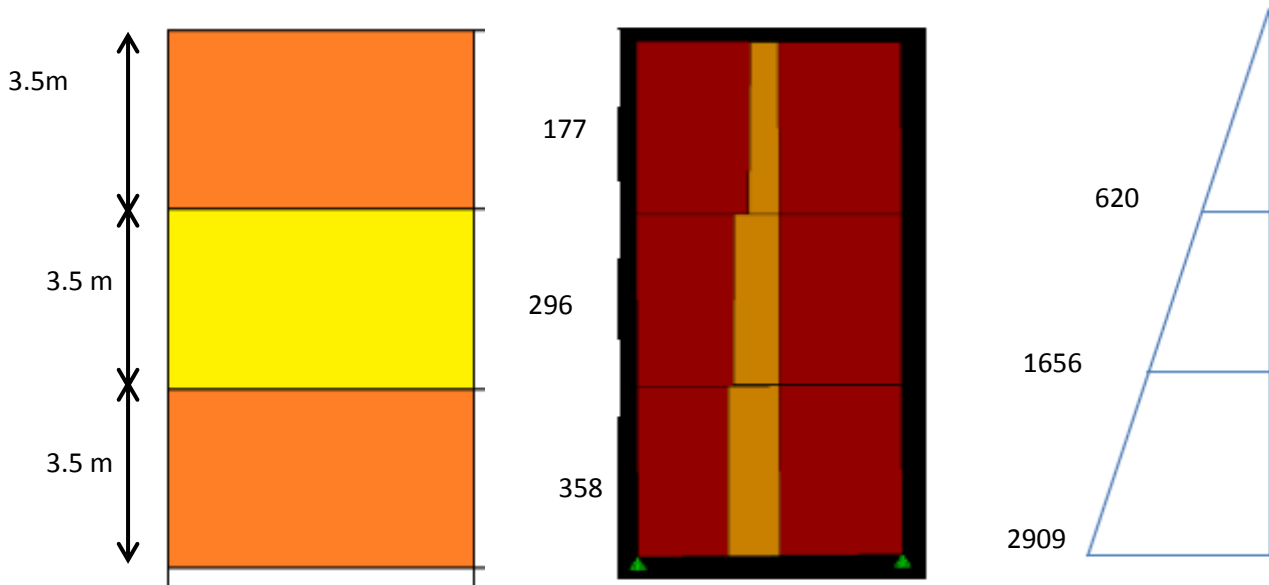
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$$h_w = 10.5 \text{ m} , L_w = 6.5 \text{ m}$$

$$d \leq 0.8 * L_w = 0.8 * 6.9 = 5.2 \text{ m} \dots \text{control}$$

$$d \leq 0.8 * h_w = 0.8 * 10.5 = 8.4 \text{ m}$$



4.10 Shear force and moment on the wall from ETABS

$$L_w / 2 = 3.25 \text{ m} \dots\dots \text{control}$$

$$h_w / 2 = 5.25 \text{ m}$$

✓ Design horizontal reinforcement :

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

$$V_{c1} = \frac{\sqrt{24}}{6} \times 200 \times 5200 = 901.4 \text{ KN (control)}$$

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

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

$$V_{c2} = \frac{\sqrt{24} \times 200 \times 5200}{4} + 0.0 = 1273.72 \text{ KN}$$

$$Mu(1) = 1656 + 358 * (3.5 - 3.25) = 1745.5 \text{ kN.m}$$

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

$$V_{c3} = \left[\frac{\sqrt{24}}{2} + \frac{3.25(\sqrt{24} + 0.0)}{\left\langle 358 - \frac{6.5}{2} \right\rangle} \right] \times \frac{200 \times 5200}{10} = 2542.67 \text{ KN}$$

So thickness of wall is safe.

✓ **Design for horizontal reinforcement :**

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

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

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

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

$$S_{\max} \leq L_w / 5 = 6500 / 5 = 1300 \text{ mm}$$

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

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

Select $\Phi 10 @ 20 \text{ cm}$

✓ **Design for Vertical reinforcement:-**

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

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

$$A_{vv} = \left\{ 0.0025 + 0.5 \left(2.5 - \frac{10.5}{6.5} \right) * \left(\frac{2 * 79}{300 * 200} - 0.0025 \right) \right\} * s * 200$$

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

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

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

$$S = 298 \text{ mm}$$

$$S_{\max} \leq L_w / 3 = 6500 / 3 = 2166 \text{ mm}$$

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

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

$$\text{Take } s = 250 \text{ mm} < s_{\max}$$

Select $\Phi 12$ -20 cm

✓ **Design of bending moment:**

$$C > \left(\frac{L_w}{0.007 * 600} \right) = \frac{6500}{4.2} = 1547.6 \text{ mm}$$

$$\text{length of boundary element} = C - 0.1 \times L_w$$

$$\text{length of boundary element} = 1547.6 - 0.1 \times 6500 = 897.6 \text{ mm}$$

$$C_w = \frac{C}{2.0} = \frac{1547.6}{2.0} = 773.8 \text{ mm}$$

Select the boundary element = 960 mm

$$A_{sv} = \frac{L_w}{s_l} \times A_{s_v} \longrightarrow = \frac{2 * 79}{250} \times 6500 = 4108 \text{ mm}^2$$

$$\frac{Z}{L_w} = \frac{1}{2 + 0.85 * \beta * f_c * L_w * h / (A_s * F_y)}$$

$$\frac{Z}{L_w} = \frac{1}{2 + 0.85 \times 0.85 \times 24 \times 6500 \times 200 / (4108 \times 420)} = 0.076$$

$$M_{uv} = 0.9 \times F_y \times 0.5 \times A_s \times L_w \times \left(1 - \left(\frac{Z}{L_w} \right) \right)$$

$$M_{uv} = 0.9 * 420 * 0.5 * 4108 \times 6500 * (1 - (0.076 / 2)) = 4854.9 \text{ KN.m}$$

$$M_{uv} > M_u$$

So, Boundary is not required.

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✓ **Material :-**

\Rightarrow concrete B350 $f_c' = 24 \text{ N/mm}^2$

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

✓ Load Calculations :-

Dead Load = 800 Kn , Live Load = 160 Kn

$$\text{Total services load} = 800 + 160 = 960 \text{ Kn}$$

Total Factored load = $1.2 \cdot 800 + 1.6 \cdot 160 = 1216 \text{ Kn}$

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

Soil density = 20 Kg/cm³

Allowable Bearing Capacity = 350 Kn/m²

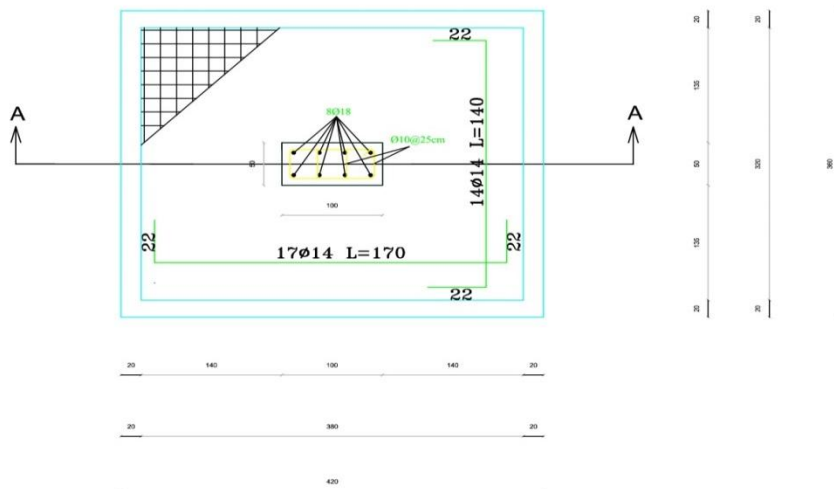


Fig 4.11 : Footing plan.

Assume $h = 60\text{cm}$

$$q_{\text{net-allow}} = 350 - 25*0.6 - 20*0.4 - 25*0.7 = 309.5\text{kn/m}^2$$

✓ **Area of Footing :-**

$$A = \frac{Pt}{q_{\text{net-allow}}} = \frac{960}{309.3} = 3.2 \text{ m}^2$$

Assume Square Footing

B required = 1.79 m

Select B = 1.9 m

✓ **Bearing Pressure :-**

$$q_u = 1216/1.9*1.9 = 336.8 \text{ Kn/m}^2$$

✓ **Design of Footing :-**

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

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

Assume $h = 60\text{cm}$, bar diameter $\phi 14$ for main reinforcement and 7.5 cm Cover

$$d = 600 - 75 - 14 = 511 \text{ mm}$$

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

$$V_u = 336.8 * \left(\frac{1.9-0.35}{2} - 0.511 \right) * 1.9 = 168.93\text{Kn}$$

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

$$\phi.V_c = 0.75 * \frac{1}{6} * \sqrt{24} * 1900 * 511 = 581.64 Kn$$

$$\phi.V_c = 581.64 Kn > V_u = 168.93 Kn$$

∴ Safe

✓ Design of Two Way Shear Strength :-

$$V_u = P_u - FR_b$$

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

$$V_u = 1216 - 336.8[(0.5 + 0.511) * (0.25 + 0.511)] = 1172.5 Kn$$

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

$$b_o = 2 * (51.1 + 50) + 2 * (51.1 + 25) = 354.0 cm$$

$\alpha_s = 40$ for interior column

$$\phi.V_c = \phi \cdot \frac{1}{6} \left(1 + \frac{2}{\beta_c} \right) \sqrt{f_c'} b_o d = \frac{0.75}{6} * \left(1 + \frac{2}{2} \right) * \sqrt{24} * 3540 * 511 = 2215.4 Kn$$

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$$\phi V_c = \phi \cdot \frac{1}{12} \left(\frac{\alpha_s}{b_o/d} + 2 \right) \sqrt{f'_c} b_o d = \frac{0.75}{12} * \left(\frac{40 * 511}{3540} + 2 \right) * \sqrt{24} * 3540 * 511 = 1107 \text{Kn}$$

$$\phi V_c = \phi \cdot \frac{1}{3} \sqrt{f'_c} b_o d = \frac{0.75}{3} * \sqrt{24} * 3540 * 511 = 2215 \text{Kn}$$

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

✓ Design of Bending Moment :-

Critical Section at the Face of Column

$$F_R = q_u * \left(\frac{B-a}{2} \right) * L = 336.8 * \left(\frac{1.9-0.25}{2} \right) * 1.9 = 527.9 \text{Kn}$$

$$M_u = 527.9 * 0.465 = 245.7 \text{Kn.m}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{245.7 \times 10^6}{0.9 \times 1900 \times 511^2} = 0.54 \text{Mpa}$$

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

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

$$A_{s, \text{req}} = \rho \cdot b \cdot d = 0.00152 \times 1900 \times 511 = 1480 \text{mm}^2$$

$$A_{s, \text{min}} = 0.0018 * 1900 * 600 = 2052 \text{ mm}^2$$

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

Check for Spacing :-

$$S = 3h = 3 * 60 = 180 \text{cm}$$

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

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

Use 17ø14, $A_{s, \text{provided}} = 2618 \text{ mm}^2 > A_{s, \text{required}} = 2052 \text{ mm}^2 \dots$ Ok

And In Another Direction Use 14 ø14 $A_{s, \text{prov}} = 2156 \text{ mm}^2$

✓ **Development Length In Footing :-**

Tension Development Length In Footing :-

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

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

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

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

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

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

$$L_{d_{available}} = \frac{1900 - 500}{2} - 75 = 625 \text{ mm}$$

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

Compression Development Length In Footing :-

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

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

$$L_{d_{Creq}} = 288.05\text{mm}$$

$$L_{d_{available}} = 600 - 75 - 14 - 14 = 497\text{mm} > L_{d_{Creq}} = 288.0 \text{ mm} \dots\dots \text{Ok}$$

Lap Splice of Dowels In Column :-

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

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

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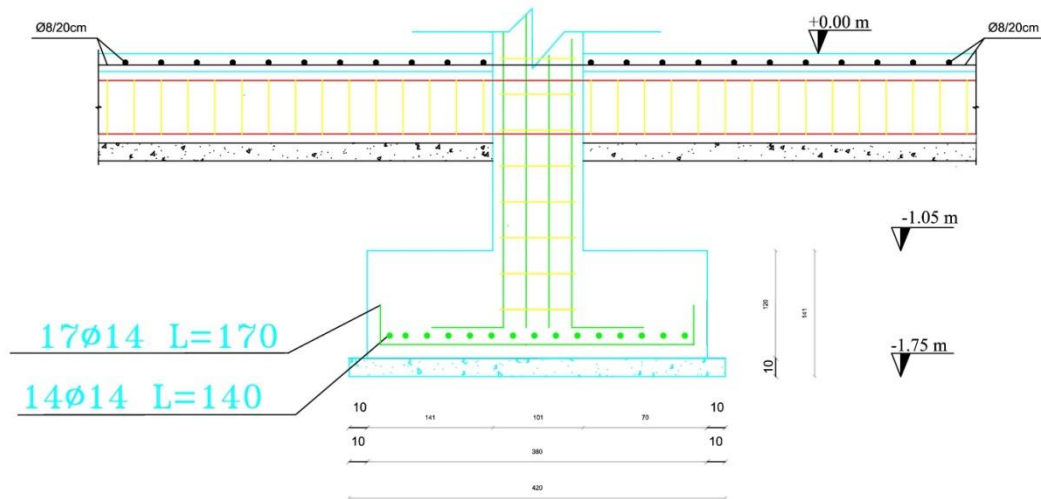


Fig 4.12 :Footing Reinforcement Details.

4.14 Design Of Steel Truss

✓ **Pos./TS/: Design Steel Truss.**

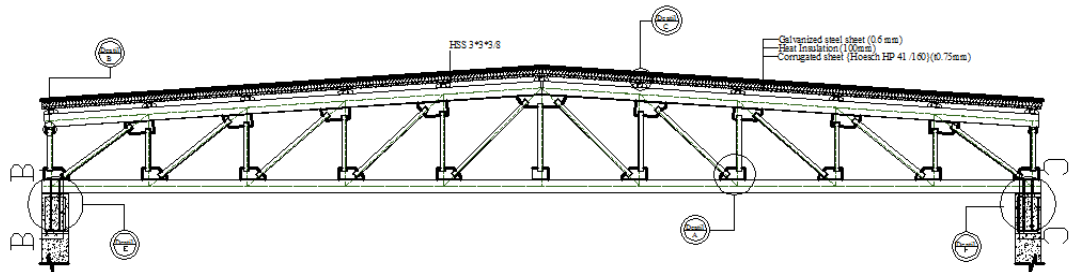
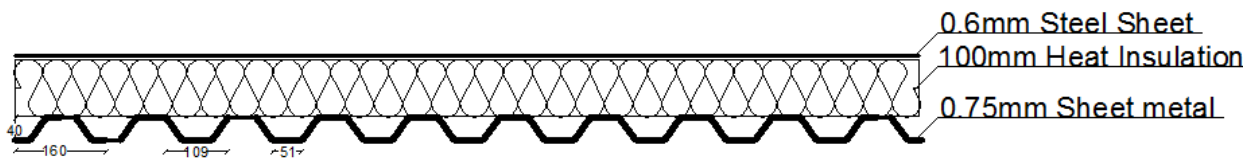


Figure 4-13 shows a gross section of the components that is located over the purlin , and it is as the following :

1. Surface layer of galvanized steel sheet with thickness of 0.6 mm.
2. Specific kind of duct, its dimension taken from some tables depending on number of spans and the dead and live loads that can it supports.



(Figure 4- 14:Cross section of sheet metal)

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Table4-9:sheet metal for 2&3spanes

Zwischenauflegerbreite ≥ 60 mm

Dicke mm	Gewicht kN/m ²	I _{eff} cm ⁴ /m	Zulässige, gleichmäßig verteilte Belastung in kN/m ² bei einer Stützweite l in m (inkl. Eigengewicht)															
			1,00	1,25	1,50	1,75	2,00	2,25	2,50	2,75	3,00	3,25	3,50	3,75	4,00	4,25	4,50	4,75
0,63	0,066	16,5	1 7,98	5,64	3,98	2,92	2,24	1,77	1,43	1,18	0,99	0,85	0,73	0,64	0,56	0,50	0,44	0,40
			2 7,98	5,64	3,98	2,92	2,24	1,77	1,43	1,18	0,99	0,85	0,73	0,61	0,50	0,42	0,35	0,30
			3 7,98	5,64	3,98	2,92	2,24	1,77	1,43	1,18	0,99	0,85	0,73	0,61	0,50	0,42	0,35	0,30
0,75	0,078	20,8	1 10,70	7,44	5,16	3,79	2,90	2,30	1,86	1,54	1,29	1,10	0,95	0,83	0,73	0,64	0,57	0,51
			2 10,70	7,44	5,16	3,79	2,90	2,30	1,86	1,54	1,29	1,10	0,94	0,77	0,63	0,53	0,44	0,38
			3 10,70	7,44	5,16	3,79	2,90	2,30	1,73	1,30	1,00	0,79	0,63	0,51	0,42	0,35	0,30	0,25
0,88	0,092	25,8	1 13,94	9,44	6,56	4,82	3,69	2,91	2,36	1,95	1,64	1,40	1,20	1,05	0,92	0,82	0,73	0,65
			2 13,94	9,44	6,56	4,82	3,69	2,91	2,36	1,95	1,64	1,40	1,17	0,95	0,78	0,65	0,55	0,47
			3 13,94	9,44	6,56	4,82	3,69	2,91	2,13	1,60	1,23	0,97	0,78	0,63	0,52	0,43	0,37	0,31
1,00	0,104	30,4	1 17,17	11,41	7,92	5,82	4,46	3,52	2,85	2,36	1,98	1,69	1,46	1,27	1,11	0,99	0,88	0,79
			2 17,17	11,41	7,92	5,82	4,46	3,52	2,85	2,36	1,98	1,69	1,38	1,12	0,92	0,77	0,65	0,55
			3 17,17	11,41	7,92	5,82	4,46	3,46	2,52	1,89	1,46	1,15	0,92	0,75	0,62	0,51	0,43	0,37
1,25	0,130	39,4	1 24,56	15,78	10,96	8,05	6,16	4,87	3,94	3,26	2,74	2,33	2,01	1,75	1,54	1,36	1,22	1,09
			2 24,56	15,78	10,96	8,05	6,16	4,87	3,94	3,26	2,74	2,23	1,78	1,45	1,19	1,00	0,84	0,71
			3 24,56	15,78	10,96	8,05	6,16	4,48	3,26	2,45	1,89	1,48	1,19	0,97	0,80	0,66	0,56	0,48
1,50	0,156	47,5	1 31,80	20,35	14,13	10,38	7,95	6,28	5,09	4,20	3,53	3,01	2,60	2,26	1,99	1,76	1,57	1,41
			2 31,80	20,35	14,13	10,38	7,95	6,28	5,09	4,20	3,42	2,69	2,15	1,75	1,44	1,20	1,01	0,86
			3 31,80	20,35	14,13	10,38	7,69	5,40	3,94	2,96	2,28	1,79	1,43	1,17	0,96	0,80	0,67	0,57
1,50	0,156	47,5	2 31,80	20,80	15,19	11,55	9,04	6,35	4,63	3,48	2,68	2,11	1,69	1,37	1,13	0,94	0,79	0,67
			3 31,80	20,80	14,28	9,00	6,03	4,23	3,09	2,32	1,79	1,40	1,12	0,91	0,75	0,63	0,53	0,45

Table 4-10 shows the values (type, weight ,support load) for duct that will be used to carry the live and the dead loads , consult the number and the length of spans.

Zwischenauflegerbreite ≥ 60 mm

Dicke mm	Gewicht kN/m ²	I _{eff} cm ⁴ /m	Zulässige, gleichmäßig verteilte Belastung in kN/m ² bei einer Stützweite l in m (inkl. Eigengewicht)															
			1,00	1,25	1,50	1,75	2,00	2,25	2,50	2,75	3,00	3,25	3,50	3,75	4,00	4,25	4,50	4,75
0,63	0,066	16,5	1 8,95	5,73	3,98	2,99	2,38	1,93	1,60	1,35	1,15	0,99	0,86	0,75	0,67	0,59	0,53	0,48
			2 8,95	5,73	3,98	2,99	2,38	1,93	1,60	1,21	0,93	0,73	0,59	0,48	0,39	0,33	0,28	0,23
			3 8,95	5,73	3,98	2,99	2,09	1,47	1,07	0,81	0,62	0,49	0,39	0,32	0,26	0,22	0,18	0,16
0,75	0,078	20,8	1 11,62	7,44	5,16	3,96	3,14	2,55	2,10	1,77	1,50	1,29	1,12	0,98	0,87	0,77	0,69	0,62
			2 11,62	7,44	5,16	3,96	3,14	2,55	2,03	1,53	1,18	0,92	0,74	0,60	0,50	0,41	0,35	0,30
			3 11,62	7,44	5,16	3,95	2,64	1,86	1,35	1,02	0,78	0,62	0,49	0,40	0,33	0,28	0,23	0,20
0,88	0,092	25,8	1 14,75	9,44	6,55	5,11	4,04	3,27	2,70	2,26	1,92	1,65	1,43	1,26	1,11	0,99	0,88	0,80
			2 14,75	9,44	6,55	5,11	4,04	3,27	2,51	1,88	1,45	1,14	0,91	0,74	0,61	0,51	0,43	0,37
			3 14,75	9,44	6,55	4,88	3,27	2,29	1,67	1,26	0,97	0,76	0,61	0,50	0,41	0,34	0,29	0,24
1,00	0,104	30,4	1 17,82	11,41	8,15	6,25	4,93	3,98	3,28	2,75	2,33	2,00	1,74	1,52	1,34	1,20	1,07	0,96
			2 17,82	11,41	8,15	6,25	4,93	3,98	2,96	2,23	1,72	1,35	1,08	0,88	0,72	0,60	0,51	0,43
			3 17,82	11,41	8,15	5,76	3,86	2,71	1,98	1,48	1,14	0,90	0,72	0,59	0,48	0,40	0,34	0,29
1,25	0,130	39,4	1 24,65	15,78	11,55	8,81	6,93	5,58	4,59	3,84	3,25	2,79	2,42	2,12	1,87	1,66	1,49	1,34
			2 24,65	15,78	11,55	8,81	6,93	5,26	3,84	2,88	2,22	1,75	1,40	1,14	0,94	0,78	0,66	0,56
			3 24,65	15,78	11,55	7,45	4,99	3,51	2,56	1,92	1,48	1,16	0,93	0,76	0,62	0,52	0,44	0,37
1,50	0,156	47,5	1 31,80	20,80	15,19	11,55	9,06	7,28	5,98	4,99	4,23	3,62	3,14	2,75	2,42	2,15	1,92	1,73
			2 31,80	20,80	15,19	11,55	9,04	6,35	4,63	3,48	2,68	2,11	1,69	1,37	1,13	0,94	0,79	0,67
			3 31,80	20,80	14,28	9,00	6,03	4,23	3,09	2,32	1,79	1,40	1,12	0,91	0,75	0,63	0,53	0,45

✓ Load calculation :

1-Dead load :

$$1\text{-load of galvanized steel sheet} = \frac{0.6}{1000} * 75 = 0.05 \text{ kN/m}^2$$

$$2\text{-Dead load of heat insulation} = 0.1 \text{ m} * 1 \text{ kN/m}^3 = 0.1 \text{ kN/m}^2$$

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3-Sheet metal with thickness of 0.75mm = 0.078KN/m²
 → Total dead load = 0.228KN/m²

2-Snow load :

Depending on the table of the snow load which it depends on the height of the building over the sea level which is 1005m, the snow load is :

$$S_t = \frac{h-400}{400}$$

$$S_t = \frac{1005-400}{400} = 1.5 \text{ KN/m}^2$$

→ Total snow load = 1.5KN/m²

→ Total load ($q_t = \text{dead load} + \text{snow load} = 0.228 + 1.5 = 1.728 \text{ KN/m}^2$)

From the table (4-8) the bearing load of sheet metal is 5.16 KN/m²

$Q_u = 5.16 \text{ KN/m}^2 > q_t = 1.728 \text{ KN/m}^2$ ok

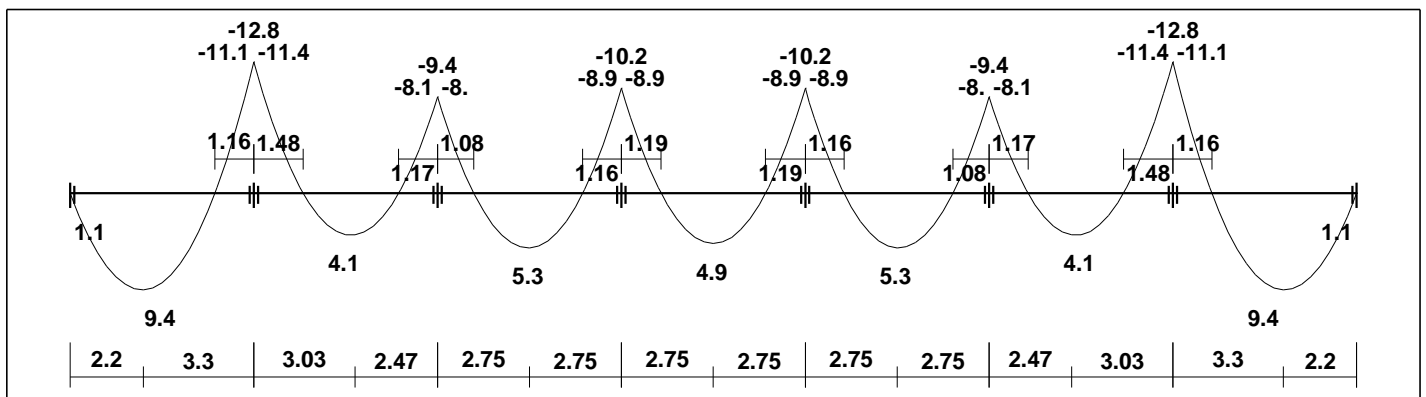
Note: the members are A36 ($F_y = 36 \text{ ksi}$ and $F_u = 58 \text{ ksi}$)

By using the previous dead and snow loads determine the max reaction on the purlins , and then apply these values as a linear on the length of the purlin.

✓ Design of purlins:

$$Q_u = 1.5(1.2D.L + 1.6S) = 1.5(1.2 \cdot 0.228 + 1.6 \cdot 1.5) = \boxed{4 \text{ KN/m}^2}$$

-Design of moment



(Figure 4-15: Moment envelope for purlins)

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$$M_U = 11.4 \text{ kn.m} = \frac{11.4}{4.448} * 1000 * \frac{1}{25.4} = 101 \text{ kip.in}$$

$$\phi_b M_b \geq M_u$$

$$0.9 * 36 * Z_x = 101 \text{ kip.in} \quad Z_x = 3.1 \text{ in}^3$$

$$\text{Select HSS } 3 \times 3 \times \frac{3}{8} Z_x = 3.25 \text{ in}^3$$

$$\frac{b}{t} = 5.6, \frac{h}{t} = 5.6$$

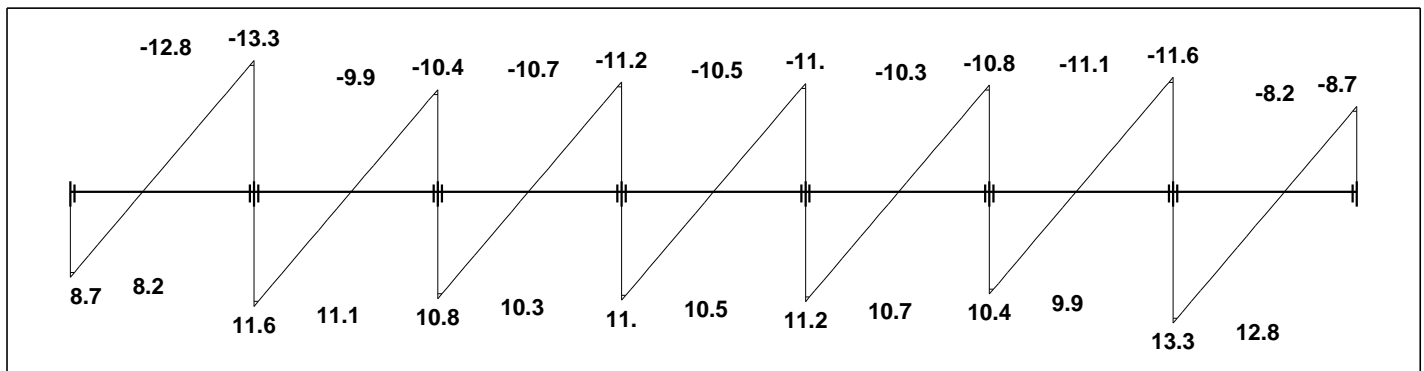
Check compact:

$$\lambda_p = 1.12 \sqrt{\frac{E}{F_y}} = 1.12 \sqrt{\frac{28000}{36}} = 31.3$$

$$\lambda_r = 1.40 \sqrt{\frac{E}{F_y}} = 1.40 \sqrt{\frac{28000}{36}} = 39.1$$

$$\frac{b}{t} = 5.6 < \lambda_p = 31.3 \quad , \dots \text{compact section}$$

✓ -Design of shear:



(Figure 4-16: Shear envelop for purlins)

$$v_U = 12.8 \text{ kn} = \frac{12.8}{4.448} * 1000 = 2.87 \text{ kip}$$

$$\phi_v v_b \geq v_u$$

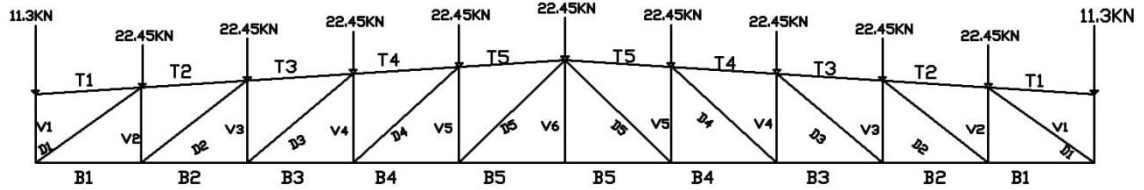
$$0.9 * 0.6 * f_y * d * t_w \geq 2.87 \text{ kip}$$

$$0.9 * 0.6 * 36 * 3 * \frac{3}{8} = 21.87 \text{ kip} > 2.87 \text{ kip} \dots \text{OK}$$

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✓ *Design the member of the truss :*



(Figure 4- 17:Truss system)

✓ **The truss consist of four types of members :**

1) The vertical member (v)

These member are under compression Force :

NO .of member	Value of compression force	
	Kn	Kips
V1	19.37	4.355
V2(max)	91.90	20.662
V3	55.44	12.46
V4	20.90	4.699
V5	10.46	2.352
V6	0.91	0.20

Table4-11:Vertical member forces

2) The Diagonal member (D)

These member are under tension force :

NO .of member	Value of tension force	
	Kn	Kips
D1(max)	227.14	51.067
D2	150.37	33.807
D3	85.77	19.284
D4	31.64	7.114
D5	14.0	3.148

Table4-12:diagonal member forces

3) The top member (T)

These member are under compression Force :

NO .of member	Value of compression force	
	Kn	Kips
T1	4.90	1.102
T2	193.68	43.545
T3	313.88	70.569
T4	379.76	85.382
T5(max)	402.69	90.537

Table4-13:Top member forces

4) The bottom member (B)

These member are under Tension Force :

NO .of member	Value of tension force	
	Kn	Kips
B1	187.85	42.235
B2	310.55	69.821
B3	377.95	84.974
B4(max)	402.02	90.386
B5	391.94	88.119

Table4-13:bottom member forces

Design of tension member:

1st) Diagonal members :

Max. value of tension =227.14 Kn (51.067 Kips)

Check :

- **Tensile yielding:**

$$P_u = \Phi * F_Y * A_g$$

$$A_g = 51.067 / 0.9 * 36 = 1.58 \text{ in}^2$$

Try L3*2*3/8 with $A_g = 1.73 \text{ in}^2$

- **Tensile rupture strength :**

$$\phi_t \times P_n = 0.75 \times f_u \times A_e$$

$$\phi_t \times P_n = 0.75 \times 58 \times (0.85 \times 1.73) = 63.97 \text{ Kips} > 51.067 \text{ Kips} \dots \text{ok}$$

2nd) Bottom members :

Max. value of tension =402.02 Kn (90.386 Kips)

- **Tensile yielding:**

$$P_u = \Phi * F_Y * A_g$$

$$A_g = 90.386 / 0.9 * 36 = 2.79 \text{ in}^2$$

Try W6*12 with $A_g = 3.55 \text{ in}^2$

- **Tensile rupture strength :**

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$$\phi_t P_n = 0.75 \times f_u \times A_e$$

$$\phi_t P_n = 0.75 \times 58 \times (0.85 \times 3.55) = 131.3 \text{ Kips} > 90.386 \text{ Kips} \dots \text{ok}$$

✓ *Design of compression member:*

1st) vertical members :

Max. value of compression = 91.90 Kn (20.662 Kips)

Take section member

(Try L3*3*3/8 with $A_g = 2.11 \text{ in}^2$)

$$A_g = 2.11 \text{ in}^2, r_x = 0.91 \text{ in}, r_y = 0.91 \text{ in}.$$

$$\text{Length of the member} = 1.5 \text{ m} (4.92 \text{ ft})$$

-Determine of the reduction factor for slender "Unstiffened element ":

$$\lambda = \frac{b}{t} = \frac{3}{3/8} = 8$$

$$0.45 \sqrt{\frac{E}{F_y}} = 0.75 \sqrt{\frac{29000}{36}} = 12.77$$

$$\frac{b}{t} = \frac{3}{3/8} = 8 < 12.77 \dots Q_s = 1.0$$

$$Q = Q_a \times Q_s = 1.0$$

$$\frac{L}{r_x} = \frac{4.92 \times 12}{0.91} = 64.88$$

$$0 < \frac{L}{r_x} < 80$$

$$\left(\frac{k \times l}{r} \right) = 72 + 0.75 \times \frac{L}{r_x} = 72 + 0.75 \times \left(\frac{12 \times 4.92}{0.91} \right) = 120.66$$

$$4.71 \sqrt{\frac{E}{Q_s \times f_y}} = 4.71 \sqrt{\frac{29000}{1 \times 36}} = 133.68 > 120.66$$

$$f_e = \frac{\pi^2 \times E}{\left(\frac{kl}{r} \right)^2} = \frac{\pi^2 \times 29000}{(120.66)^2} = 19.66 \text{ ksi}$$

$$F_{cr} = Q \times 0.658^{(Q f_y / f_e)} \times f_y$$

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$$F_{cr} = 1.0 \times 0.658^{(36/19.66)} \times 36 = 16.73 \text{ kips}$$

$$\phi P_n = \phi \times f_{cr} \times A_g$$

$$\phi P_n = 0.9 \times 16.73 \times 2.11 = 31.8 \text{ kips} > 20.662 \text{ kips} \dots \text{ok}$$

*Note : use L3*3*3/8 for both of diagonal and vertical members .*

2nd) top members :

Max. value of compression = 402.69 kn (90.537 kips)

$$\text{Assume } \frac{Kl}{r} = 75$$

$$4.71 \sqrt{\frac{29000}{36}} = 133.68$$

$$f_e = \frac{\pi^2 \times E}{(kl/r)^2} = \frac{\pi^2 \times 29000}{(75)^2} = 50.88 \text{ ksi}$$

$$F_{cr} = Q \times 0.658^{(Q f_y / f_e)} \times f_y$$

$$F_{cr} = 1 \times 0.658^{(1 \times 36 / 50.88)} \times 36 = 26.77 \text{ kips}$$

$$\frac{Kl}{r} = \frac{1 \times 12 \times 4.92}{r} = 75 \rightarrow r = 0.787$$

$$A_g = \frac{pu}{\phi F_{cr}} = \frac{90.537}{0.9 \times 26.77} = 3.76 \text{ in}^2$$

Use W8*15 with $A_g = 4.44 \text{ in}^2$

✓ Design of weld:

The calculation of weld based on the following :

- 1) Fillet weld is used.
- 2) The plates are A36 ($f_y = 36 \text{ ksi}$, $F_u = 58 \text{ ksi}$)
- 3) The plate thickness is ($t = 0.5 \text{ in}$)
- 4) The electrodes having $F_{EXX} = 70 \text{ ksi}$
- 5) The shielded metal arc welding (SMAW) is used.

1st) Design of weld between the vertical member and the Gusset plate in the corners of the truss:

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The section of the vertical member is angle (L3*3*3/8), $A_g=2.11 \text{ in}^2$, $y=0.884$.

The value of Max. compression in the vertical member is $V_u=20.662 \text{ Kips}$.

$$\text{Max. weld size } (a_{\max}) = t - \frac{1}{16} = \frac{3}{8} - \frac{1}{16} = \frac{5}{16} \text{ in}$$

$$\text{Min. Weld size } (a_{\min}) = \frac{3}{16} \text{ in}$$

$$\text{Use weld size } (a) = \frac{1}{4} \text{ in}$$

- Design strength of weld :

$$\phi \times R_{nw} = \phi \times t_e \times 0.6 \times F_{Exx}$$

$$\phi \times R_{nw} = 0.75 \times \left(0.707 \times \frac{1}{4}\right) \times 0.6 \times 70 = 5.57 \text{ kips}$$

- Design strength of base material :

$$\phi \times R_n = \phi \times (0.6 \times f_y) \times t = 1.0 \times 0.6 \times 36 \times \frac{3}{8} = 8.1 \text{ kips} > 5.57 \text{ kips} \dots \text{ok}$$

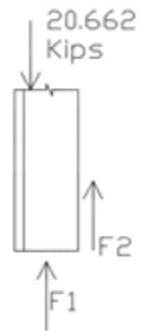
Or

$$\phi \times R_n = \phi \times (0.6 \times f_u) \times t = 0.75 \times 0.6 \times 58 \times \frac{3}{8} = 9.79 \text{ kips} > 5.57 \text{ kips} \dots \text{ok}$$

$$f_1 = 5.57 \times 3 = 16.71 \text{ kips}$$

$$f_2 = 20.662 - 16.71 = 3.952 \text{ kips}$$

$$l_{w2} = \frac{f_2}{\phi \times R_{nw}} = \frac{3.952}{5.57} = 0.71 \text{ in} \dots \dots \text{use } 1.0 \text{ in}$$



(Figure 4-18:) weld between vertical member and gusset plate)

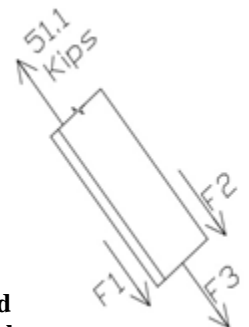
2nd) Design of weld between the diagonal member and the gusset plate:

- The section of the diagonal member is angel (L3*3*3/8)
- For the vertical member use the same size and dimension of weld for the previous vertical member.

The value if Max. Tension in the diagonal member is $T_u = 51.1 \text{ kip}$.

$$\text{Max. weld size } (a_{\max}) = t - \frac{1}{16} = \frac{3}{8} - \frac{1}{16} = \frac{5}{16} \text{ in}$$

$$\text{Min = Weld size } (a_{\min}) = \frac{3}{16} \text{ in}$$



(Figure 4- 1:weld between diagonal member and gusset plate)

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Use weld size (a) = $\frac{1}{4}$ in

✓ **Design strength of weld :**

$$\phi \times R_{nw} = \phi \times t_e \times 0.6 \times F_{Exx}$$

$$\phi \times R_{nw} = 0.75 \times \left(0.707 \times \frac{1}{4}\right) \times 0.6 \times 70 = 5.57 \text{ kips}$$

- Design strength of base material :

$$\phi \times R_n = \phi \times (0.6 \times f_y) \times t = 1.0 \times 0.6 \times 36 \times \frac{3}{8} = 8.1 \text{ kip} > 5.57 \text{ kip} \dots \text{ok}$$

Or

$$\phi \times R_n = \phi \times (0.6 \times f_u) \times t = 0.75 \times 0.6 \times 58 \times \frac{3}{8} = 9.79 \text{ kip} > 5.57 \text{ kip} \dots \text{ok}$$

$$F_3 = 3 \times 5.57 = 16.71 \text{ kips}$$

$$\sum M \text{ at } F_1 = 0$$

$$= 16.71 \times 1.5 + F_2 \times 3 - 51.1 \times (3 - 0.884) = 0$$

$$F_2 = 27.69 \text{ kips}$$

$$F_1 = 51.1 - 16.71 - 27.69 = 6.7 \text{ kips}$$

$$l_{w1} = \frac{f_1}{\phi \times R_{nw}} = \frac{6.7}{5.57} = 1.21 \text{ in} \dots \dots \text{use } 1.5 \text{ in}$$

$$l_{w2} = \frac{f_2}{\phi \times R_{nw}} = \frac{27.69}{5.57} = 4.97 \text{ in} \dots \dots \text{use } 5 \text{ in}$$

Check for rupture

$$L = \frac{(5 + 1.5)}{2} = 3.25$$

$$U = 1 - \frac{x}{l} = 1 - \frac{0.884}{3.25} = 0.728$$

$$\phi t P_n = 0.75 \times f_u \times A_e$$

$$\phi t P_n = 0.75 \times 58 \times (0.728 \times 2.11) = 66.82 \text{ kips} > 51.1 \text{ kips} \dots \dots \text{ok}$$

3rd) Design of weld between the bottom member and the gusset plate:

The section of the bottom member is angel (W6*12)

$$11 / 2.54 = 4.33 \text{ in}$$

$$R_u = \sqrt{(R_v + R_y)^2 + (R_h + R_x)^2}$$

$$R_v = \frac{P_y}{L} = 0$$

$$R_h = \frac{P_x}{L} = \frac{20.662}{14.76 * 2} = 0.7 \text{ kip/in}$$

$$I_p = 2 * \frac{14.76^2}{12} = 535.93 \text{ in}^3$$

$$R_x = \frac{M * Y}{I_p} = 0 \dots y = 0$$

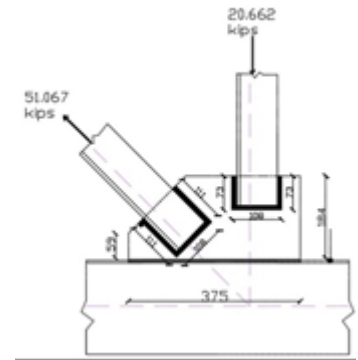
$$R_y = \frac{M * x}{I_p} = \frac{(20.662) * \left(\frac{4.33}{2}\right)}{535.93} = 0.1$$

$$R_u = \sqrt{(0 + 0.1)^2 + (0.7 + 0)^2} = 0.71 \text{ kip/in}$$

$$\phi * R_{nw} = R_u$$

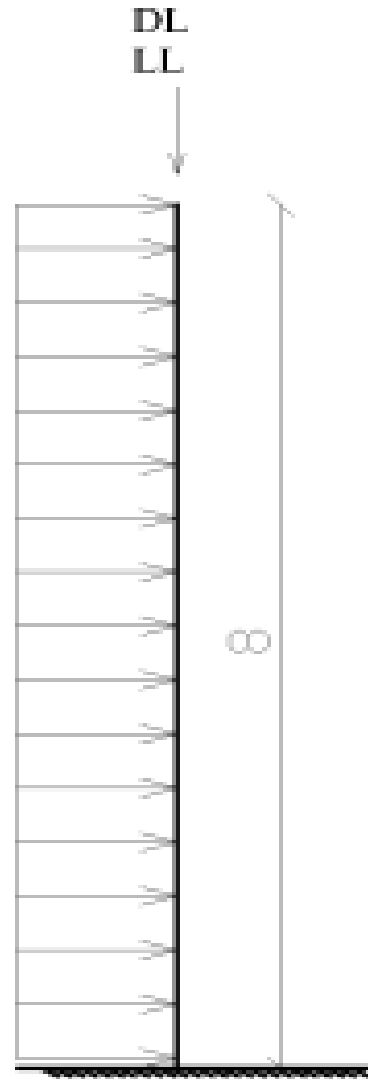
$$0.75 * (0.707a) * 0.6 * 70 = 0.71 \dots a = 0.032 \text{ in}$$

$$\text{Take } a = \frac{2}{16} \text{ in}$$



(Figure 4- 20:weld between gusset plate and bottom member)

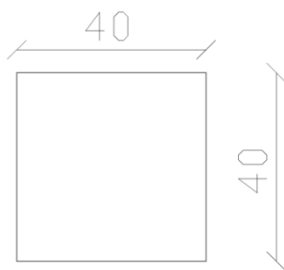
✓ **Pos./CS/: Design of Steel Truss Column.**



(4-21)System and Loading:

Section:

we choose square section for this column and its dimension is 40*40 cm



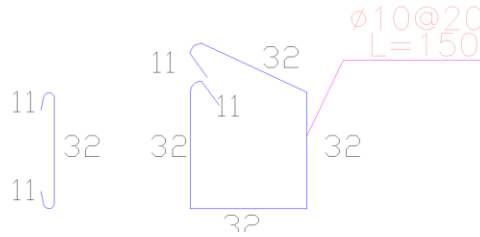
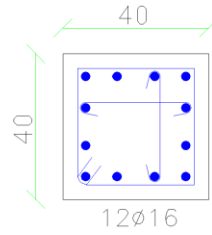
(Figure 4- 2: Steel truss Column System)

✓ **Design:**

By using ETABS program design we define the section and enter the load and get the design.

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We use 12 Ø16 .

(Figure 4- 23:Steel Truss Column Design)

✓ **Pos./F/: Design of Foundation.**

✓ **materials:**

$$f'_c = 24MPa$$

$$f_y = 420MPa$$

✓ **loading:**

-vertical load from truss:

$$P_u = 150KN$$

-Lateral load (wind load):

$$\text{Wind load} = 2.04kn/m$$

The value of moment from wind load :

$$P_{w,L} = 5.1 * 0.8 * 0.5 = 2.04KN/m \text{ at the column}$$

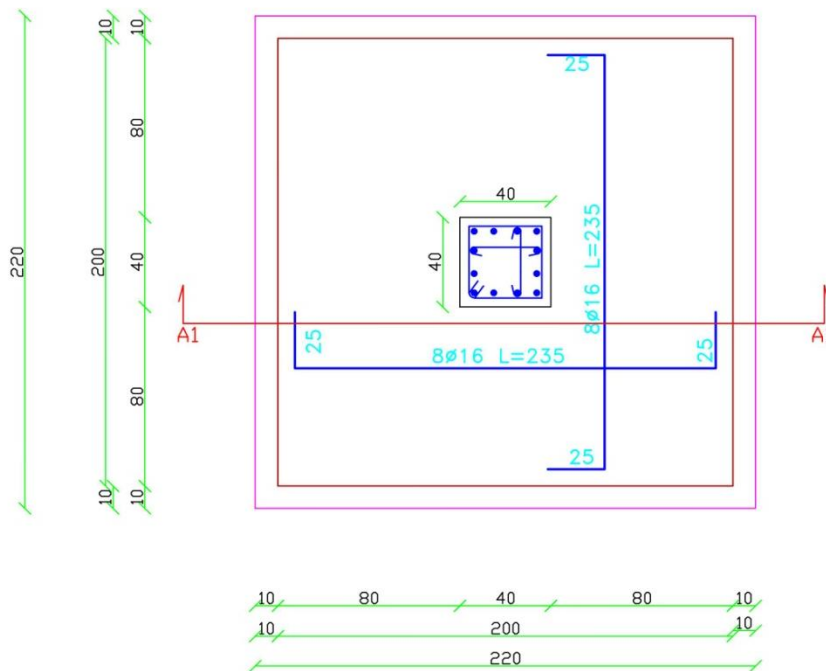
$$M = 2.04 * 8 * 4 = 65.3 \text{ kN.m}$$

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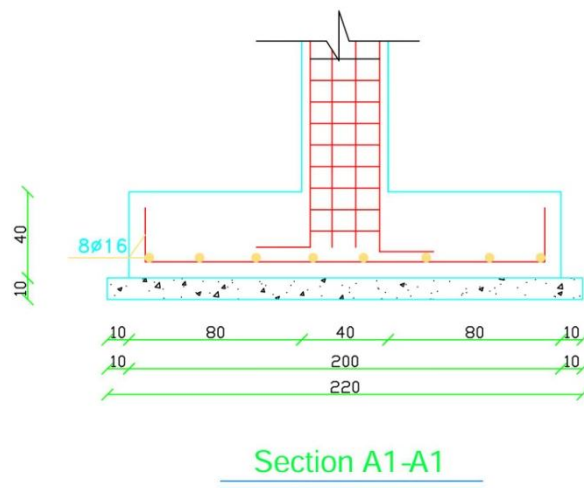
Structural Analysis And Design

✓ Design:

By using Atir program design we define the section and enter the load and get the design.



(Figure 4-24 :Steel foundation plan)



(Figure 4-25:Steel foundation reinforcement)