Chapter 4

Structural Analysis & Design

City Center "A"-

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City Center "B"-

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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.

4.1.1 Design method and requirements:

The design strength provided by a member is calculated in accordance with the requirements and assumptions of ACI code (318M_14).

4.1.2 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 occur.

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,

Strength provided ≥ strength required to carry factored loads.
4.2 Factored loads: -

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

\[ W_u = 1.4 D_{LACI-code-318-14}(9.2.1). \]

\[ W_u = 1.2 D_L + 1.6 L_{LACI-code-318-14}(9.2.2). \]

**Materials:-**

Concrete B300, \( Fc' = 0.8 \times 30 = 24 \text{ N/mm}^2 = 24 \text{Mpa} \)

Reinforcement Steel, \( f_y = 420 \text{ N/mm}^2 = 420 \text{ Mpa} \)

\( f_{yt} = 420 \text{ Mpa} \), will be used in design and calculations.

4.3 Slabs Thickness calculation:-

According to ACI-Code-318-14 table 9.5(a), the minimum thickness of non-prestressed beams or one way, slabs unless deflections are computed for one end continuous for one-way rib slabb given as following:

**Fig (4-1):Rib(R2-(17)) at the First floor**

**Fig (4-2): spans of rib (R2-(17))**
H_{\text{min}} \text{ for two end continuous beam}

\[ H_{\text{min}} = \frac{L}{21} \text{ longest two end continuous supported is } 6.78\text{m} \]

\[ H_{\text{min}} = \frac{6780}{21} = 322.85\text{ mm} \]

For First floor slab, use thickness of slab 35cm.

**4.4 Load Calculation:**

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

![Typical section in ribbed slab](image)
4.5 Design of Topping:-

4.5.1 Calculation of Dead load For 1m strip

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit weight(kN/m3)</th>
<th>Load (KN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>tile</td>
<td>23</td>
<td>1</td>
</tr>
<tr>
<td>mortar</td>
<td>22</td>
<td>2</td>
</tr>
<tr>
<td>sand</td>
<td>16</td>
<td>3</td>
</tr>
<tr>
<td>topping</td>
<td>25</td>
<td>4</td>
</tr>
<tr>
<td>block</td>
<td>10</td>
<td>5</td>
</tr>
<tr>
<td>rib</td>
<td>25</td>
<td>6</td>
</tr>
<tr>
<td>plaster</td>
<td>22</td>
<td>7</td>
</tr>
<tr>
<td>partition</td>
<td>2.3(KN/m2)</td>
<td>8</td>
</tr>
</tbody>
</table>

Table (4-2) calculation of dead load for topping.
4.5.2 Calculation of live load

From Jordan's Code

L.L\_total = 4KN/m

Wu = 1.2D.L + 1.6L.L

= 1.2*6.55 + 1.6*4 = 14.3 KN/m

**Design of shear :-**

Used \( f_y = 420 \text{ MPa} \) & \( f'c = 24\text{MPa} \)

\[
\phi \times V_c = 0.75 \times \sqrt{24} \times \frac{1}{6} \times 1000 \times 80 \times 0.001 = 49 \text{KN} > 2.86 \text{KN} \times \phi
\]

No shear reinforcement is required.

**Check** \( \Phi M_n > M_u \)

\[
M_u = \frac{w_u \times I^2}{12} = \frac{14.3 \times 0.4^2}{12} = 0.19 \text{ kN.m}
\]

\[
M_n = 0.42 \sqrt{f'c} \times s
\]

\[
S = \frac{bh^2}{6}
\]

\[
M_n = 0.42 \sqrt{f'c} \times \frac{bh^2}{6}
\]

\[
M_n = 0.42 \times \sqrt{24} \times \frac{1000 \times 80^2}{6} \times 10^{-6} = 2.19 \text{ kN.m}
\]

\( \phi = 0.55 \) for plain concrete

\[
\phi \times M_n = 0.55 \times 2.19 = 1.205 \text{ kN.m}
\]

\[
\phi \times M_n = 1.204 \text{ kN.m} > M_u = 0.195 \text{ kN.m}
\]
No reinforcement is required according to ACI-Code-318M-14, so \( A_s \) min for slabs as Shrinkage and temperature reinforcement.

**Shrinkage and temperature reinforcement must be provided.**

For the shrinkage and temperature reinforcement:

\[ \rho = 0.0018 \]

\textit{ACI-318-14 (7.12.2)}

\[ A_s = \rho \times b \times h = 0.0018 \times 1000 \times 80 = 144 \text{ mm}^2/\text{m} \]

\( A_s (\varnothing 8) = 50.27 \text{ mm}^2 \)

So number of bars = \( 144/50.27 = 2.86 \)

\( 1/N = 350 \text{ mm} \)

The step is the smallest of:

1. \( S=3 \times h = 240 \text{ mm} \)
2. \( S = 380 \left( \frac{280}{f_s^2} \right) 2.5Cc = 380 \left( \frac{280}{(2/3) \times 420} \right) - 2.5 \times 20 \)

\[ = 330 \]

select mesh \( \varnothing 8/20 \text{ cm} \), \( A_s \) prov = 2.51 \( \text{ cm}^2/\text{m} \) > \( A_s \) min = 1.44 \( \text{ cm}^2/\text{m} \)

**Then use \( \varnothing 8 @ 20 \text{ cm} \) for practical purposes in both directions.**

From practical consideration, the secondary reinforcement parallel to the rib shall be placed in the slab and spaced at distance not more than half of the spacings between ribs (usually two bars upon each 40 cm width block).
4.6 Design of Rib (R1-(09)):-

Fig (4-5): Rib(R1-(09)) at the First floor

4.6.1 Design constant:-
- \( b_E \) For T- section is the smallest of the following:

\[
b_E = \frac{L_n}{4} = \frac{4.83}{4} = 1.21 \text{m}
\]

\[
b_E = b_w + 16 \frac{t_f}{16} = 12 + 16 (8) = 1.4 \text{ m}
\]

\[
b_E = c/c \text{ spacing between adjacent ribs } = 0.52 \text{ m}
\]

Control ... 52cm

- Requirements for Slab Floor According to \textit{ACI (318M-14)}.

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

Select \( b_w = 12 \text{cm} \)

\( h \leq 3.5 \times b_w \) .................................................. ACI (8.13.2)

Select \( h = 35 \text{cm} < 3.5 \times 12 = 42 \text{cm} \)

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

Select \( t_f = 8 \text{cm} \)
4.6.2 Calculation of Dead load:-

<table>
<thead>
<tr>
<th>Material</th>
<th>Calculation</th>
<th>Load (KN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tiles</td>
<td>23<em>0.03</em>0.52</td>
<td>0.3588</td>
</tr>
<tr>
<td>Mortar</td>
<td>22<em>0.02</em>0.52</td>
<td>0.2288</td>
</tr>
<tr>
<td>Sand</td>
<td>16<em>0.07</em>0.52</td>
<td>0.5824</td>
</tr>
<tr>
<td>Topping</td>
<td>25<em>0.08</em>0.52</td>
<td>1.04</td>
</tr>
<tr>
<td>Block</td>
<td>10<em>0.27</em>0.4</td>
<td>1.08</td>
</tr>
<tr>
<td>Rib</td>
<td>25<em>0.27</em>0.12</td>
<td>0.81</td>
</tr>
<tr>
<td>Plastering</td>
<td>22<em>0.02</em>0.52</td>
<td>0.2288</td>
</tr>
<tr>
<td>Partition</td>
<td>2.3*0.52</td>
<td>1.196</td>
</tr>
</tbody>
</table>

Table (4-3) calculation of the total load for (R1-(09)).

**Total dead load = 5.584 KN/m/rib**

4.6.3 Calculation of Live load:-

From Jordanian live loads table live load for malls is 4 KN/m²

Total live load = 4*0.52 = 2.08 KN/m/rib

Material :-

- concrete B300 \( Fc' = 24 \text{ N/mm}^2 \)
- Reinforcement Steel \( fy = 420 \text{ N/mm}^2 \)

**Section :-**

\[ b = 12 \text{cm} \quad bf = 52 \text{ cm} \]

\[ h = 35 \text{cm} \quad Tf = 8 \text{ cm} \]
Fig. (4-6) Geometry of Rib (R1-(09)).

Fig. (4-7) Service load of Rib (R1-(09))

Fig. (4-8) Rib Envelope (R1-(09))
4.6.4 Design of flexure:-

4.6.4.1 Design of Negative moment of rib (R1-09):

1) Maximum negative moment $Mu = 17.4 KN.m$.

\[ d = depth - cover - diameter of stirrups - (diameter of bar/2) \]

\[ = 350 - 20 - 10 \times \frac{12}{2} = 315 \text{ mm.} \]

\[ Mn = Mu / \phi = 17.4 / 0.9 = 19.33 \text{ KN.m} \]

\[ m = \frac{f_y}{0.85f_c} = \frac{420}{0.85 \times 24} = 20.6 \]

\[ K_n = \frac{M_n}{b \times d^2} = \frac{19.33 \times 10^{-3}}{0.12 \times (0.315)^2} = 1.623 \text{ MPa} \]

\[ \rho = \frac{1}{m} \left( 1 - \sqrt{1 - \frac{2K_n \rho m}{f_y}} \right) \]

\[ = \frac{1}{20.6} \left[ 1 - \sqrt{1 - \frac{2 \times 1.623 \times 20.6}{420}} \right] = 0.00403 \]

\[ \rightarrow A_s = \rho \times b \times d = 0.00403 \times 120 \times 315 = 152.334 \text{ mm}^2. \]
\[ A_{\text{min}} = \frac{f_c}{4f_y} \cdot \hat{b}_w \cdot a \geq \frac{14}{420} \cdot \hat{b}_w \cdot a \quad \text{(ACI-10.5.1)} \]

\[ = \sqrt{\frac{24}{420} \cdot 120 \cdot 315} \geq \frac{14}{420} \cdot 120 \cdot 315 \]

\[ = 110.23 \text{ mm}^2 < 126 \text{ mm}^2 \quad \text{Larger value is control.} \]

\[ \rightarrow A_{\text{min}} = 126 \text{ mm}^2 < A_{\text{req}} = 152.334 \text{ mm}^2. \]

\[ \therefore A_s = 152.334 \text{ mm}^2. \]

\[ 2 \quad 10 = 157.08 \text{ mm}^2 > A_{\text{req}} = 152.334 \text{ mm}^2. \text{ OK.} \]

\[ \therefore \text{Use } 2 \quad 10 \]

\[ \rightarrow \text{Check for strain: } (\varepsilon_s \geq 0.005) \]

Tension = Compression

\[ A_s \cdot f_y = 0.85 \cdot f_y^t \cdot b \cdot a \]

\[ 157.08 \cdot 420 = 0.85 \cdot 24 \cdot 120 \cdot a \]

\[ a = 26.95 \text{ mm.} \]

\[ \hat{c} = \frac{a}{\beta_1} = \frac{26.95}{0.85} = 31.7 \text{ mm.} \quad \text{* Note: } f_y^t = 24 \text{MPa} < 28 \text{ MPa} \rightarrow \beta_1 = 0.85 \]

\[ \varepsilon_s = \frac{d-\hat{c}}{\hat{c}} \cdot 0.003 \]

\[ = \frac{315-31.7}{31.7} \cdot 0.003 = 0.027 > 0.005 \quad \therefore =0.9 \text{ OK} \]

4.6.4.2 Design of Positive moment of rib (R1-(09))

\[ d = \text{depth - cover} - \text{diameter of stirrups} - (\text{diameter of bar/ 2}) \]

\[ = 350 - 20 - 10 \cdot \frac{10}{2} = 315 \text{ mm.} \]

\[ \rightarrow M_{\text{max}} = 18.5 \text{KN.m} \]

\[ b_E \leq \text{Distance center to center between ribs} = 520 \text{ mm.} \quad \text{Controlled.} \]

\[ \leq \text{Span/4} = 4830/4 = 1207.5 \text{ mm.} \]

\[ \leq (16\times t_o) + b_w = (16\times 80) + 120 = 1400 \text{ mm.} \]
→ $b_E = 520 \text{ mm}$.

→ $M_{nf} = 0.85 f'_c * b_E * t_f * d - \frac{t_f}{2}$

$$= 0.85 * 24 * 0.52 * 0.08 * 0.315 - \frac{0.08}{2} * 10^3 = 233.37 \text{ KN.m}$$

$M_{nf} = 0.9 * 233.37 = 210.0 \text{ KN.m}$

→ $M_{nf} = 210.0 \text{ KN.m} > M_{u, max} = 18.5 \text{ KN.m}$.

:. Design as rectangular section.

1) Maximum positive moment $M_u^{(+) } = 18.5 \text{ KN.m}$

$M_n = M_u / \phi = 18.5 / 0.9 = 20.56 \text{ KN.m}$

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

$K_n = \frac{M_n}{b_d^2} = \frac{20.56 * 10^{-3}}{0.52^2 * (0.315)^2} = 0.398 \text{ MPa}$

$$\rho = \frac{1}{m} \left(1 - \frac{2 + K_n m}{f_y}ight)$$

$$= \frac{1}{20.6} \left(1 - \frac{2 + 0.398 * 20.6}{420}\right) = 0.00096$$

→ $A_s = \rho * b * d = 0.00096 * 520 * 315 = 157.248 \text{ mm}^2$.

$$A_{s, min} = \frac{f'_c}{4 (f_y)} * b_w * d \geq \frac{1.4 f_y}{f_y} * b_w * d \quad \text{(ACI-10.5.1)}$$

$$= \frac{\sqrt{24}}{4 * 420} * 120 * 315 \geq \frac{1.4 * 420}{420} * 120 * 315$$

$$= 110.22 \text{ mm}^2 < 126 \text{ mm}^2 \quad \text{ Larger value is control.}$$

→ $A_{s, min} = 126 \text{ mm}^2 > A_{s, req} = 157 \text{ mm}^2$.

:. $A_s = 157.248 \text{ mm}^2$.

2) $10 = 157.1 \text{ mm}^2 > A_{s, req} = 157 \text{ mm}^2$. OK.

:. Use $2 \Phi 10$
\[ \text{Tension} = \text{Compression} \]

\[ A_s \times f_y = 0.85 \times f'_c \times b \times a \]

157.1 \times 420 = 0.85 \times 24 \times 520 \times a

\[ a = 6.22 \text{ mm.} \]

\[ \varepsilon = \frac{a}{\beta_1} = \frac{6.22}{0.85} = 7.3 \text{ mm.} \]

\[ \varepsilon_s = \frac{d-c}{\varepsilon} \times 0.003 \]

\[ = \frac{315-73}{7.3} \times 0.003 = 0.126 > 0.005 \quad \therefore = 0.9 \text{ OK} \]

2) Maximum positive moment \( M_u^{(+)} = 17.1 \text{KN.m} \)

\[ M_n = M_u / \phi = 17.1 / 0.9 = 19 \text{KN.m} \]

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

\[ K_m = \frac{M_n}{b \times d^2} = \frac{17.1 \times 10^{-3}}{0.52 \times (0.315)^2} = 0.33 \text{MPa} \]

\[ \rho = \frac{1}{m} \left( 1 - \frac{2K_m M_n}{f_y} \right) \]

\[ = \frac{1}{20.6} \left( 1 - \frac{2 \times 0.33 \times 19}{420} \right) = 0.00079 \]

\[ \rightarrow A_s = \rho \times b \times d = 0.00079 \times 120 \times 315 = 29.86 \text{mm}^2. \]

\[ A_{s_{\text{min}}} = \frac{f'_c}{4(f_y)} \times b_a \times d \geq \frac{1}{4} \times f_y \times b_a \times d \quad \text{...........(ACI-10.5.1)} \]

\[ = \frac{120 \times 315}{420} \geq \frac{1.4}{420} \times 120 \times 315 \]

\[ = 110.2 \text{mm}^2 \geq 126 \text{mm}^2 \quad \text{............. Larger value is control.} \]

\[ \rightarrow A_{s_{\text{min}}} = 126 \text{mm}^2 > A_{s_{\text{req}}} = 29.86 \text{mm}^2. \]

\[ \therefore A_s = 126 \text{mm}^2. \]

2 \quad 10 = 157 \text{mm}^2 > A_{s_{\text{req}}} = 126 \text{mm}^2. \text{ OK.} \]

\[ \therefore \text{Use } 2 \quad 10 \]

\[ \rightarrow \text{Check for strain:} \quad (\varepsilon_s \geq 0.005) \]

\[ \text{Tension} = \text{Compression} \]

\[ A_s \times f_y = 0.85 \times f'_c \times b \times a \]
157*420 = 0.85 * 24 * 520* a

a = 6.216mm.

c = \frac{a}{\beta_1} = \frac{6.216}{0.85} = 7.313 \text{ mm.}

* Note: $f'_c = 24 \text{ MPa} < 28 \text{ MPa} \Rightarrow \beta_1 = 0.85$

$\varepsilon_s = \frac{d-c}{c} * 0.003

= \frac{315-7.313}{7.313} * 0.003 = 0.126 > 0.005 \therefore =0.9 \text{ OK}$

4.6.4.3 Design of shear of rib (R1-(09))

1) $V_u = 13.7 \text{ KN.}$

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

$= 0.75 * \frac{\sqrt{24}}{6} * 0.12 * 0.315 * 10^3 = 23.1 \text{ KN.}$

1.1 * $V_c = 1.1 * 23.1 = 25.6 \text{ KN.}$

→ Check for items:

1- Item 1 : $V_u \leq \frac{V_c}{2}.$

$13.7 \leq \frac{25.6}{2} = 12.8 \ldots \ldots \text{Not satisfy}$

2- Item 2 : $\frac{V_c}{2} < V_u \leq V_c$

$12.8 \leq 13.7 \leq 600 \text{ mm.}$

*: Item (2) is satisfy → minimum shear reinforcement is required.

\[
\left(\frac{d_h}{s}\right)_{\text{min}} \geq \frac{1}{16} * \frac{f'_c}{f_{yt}} * b_w = \frac{1}{16} * \frac{\sqrt{24}}{420} * 0.12 = 8.75 \times 10^{-5}.
\]

\[
\geq \frac{1}{3} * \frac{b_w}{f_{yt}} = \frac{1}{3} * \frac{0.12}{420} = 9.52 \times 10^{-5} \ldots \ldots \text{Control.}
\]

Try 8 (2 Legs):

\[
\frac{2 \times 50 \times 10^{-6}}{S} = 9.52 \times 10^{-5} \Rightarrow S = 1.05 \text{ m}
\]

$S \leq \frac{d}{2} = \frac{315}{2} = 157.5 \text{ mm.}$

\[
\leq 600 \text{ mm.}
\]

*: Use 8 @ 10 Cm
2) \( V_u = 23 \) KN.

\[
V_c = \frac{f'_c}{6} b_w d
\]

\[= 0.75 \times \frac{\sqrt[3]{24}}{6} \times 0.12 \times 0.315 \times 10^3 = 23.15 \text{ KN.}\]

\[1.1 \times V_c = 1.1 \times 23.15 = 25.6 \text{ KN.}\]

→ Check for items:

1- Item 1 : \( V_u \leq \frac{V_c}{2} \).

\[23 \leq \frac{25.6}{2} = 12.8 \ldots \text{Not satisfy}\]

2- Item 2 : \( \frac{V_c}{2} < V_u \leq V_c \)

\[12.8 \leq 23 \leq 25.6 \ldots \text{ Satisfy.}\]

.: Item (2) is satisfy → minimum shear reinforcement is required.

\[
\left(\frac{A_s}{S}\right)_{\text{min}} \geq \frac{1}{16} \times \frac{f'_c}{f_y} b_w = \frac{1}{16} \times \frac{\sqrt[3]{24}}{420} \times 0.12 = 8.75 \times 10^{-5}.
\]

\[
\geq \frac{1}{3} \times \frac{b_w}{f_y} = \frac{1}{3} \times \frac{0.12}{420} = 9.52 \times 10^{-5} \ldots \ldots \text{Control.}\]

Try 8 (2 Legs):

\[
\frac{2 \times 50 \times 10^{-6}}{S} = 9.52 \times 10^{-5} \rightarrow S = 1.05 \text{ m}
\]

\[S \leq \frac{d}{2} = \frac{315}{2} = 157.5 \text{ mm.}\]

\[\leq 600 \text{ mm}\]

Use 8 @ 10 Cm
4.7 Design of Beam (B1-(17)):

Fig (4-10): Reinforcement of Rib(R1-(09))

Fig (4-11) Location of beam (B1-(17))
Material:
- Concrete B300 $F_{c'} = 24\, \text{N/mm}^2$
- Reinforcement Steel $f_y = 420\, \text{N/mm}^2$

Section:
- $B = 50\, \text{cm}$
- $h = 35\, \text{cm}$

Fig (4-12) : Beam Geometry (B1-(17)).

Fig (4-13) : Service Load of Beam (B1-(17)).
Fig (4-14) : Beam Envelop (B1-(17)).
4.7.1 Check whether the section will be act as singly or doubly reinforcement section:

$\rightarrow \ Mu_{\text{max}} = 28.6 \text{KN.m}$.

$b_w = 50 \text{Cm.} \ , \ h = 35 \text{ Cm.}$

$d = \text{depth - cover – diameter of stirrups – (diameter of bar/2)}$

$= 350 - 40 - 10 - \frac{12}{2} = 294 \text{ mm.}$

$C_{\text{max}} = \frac{3}{7} \times d = \frac{3}{7} \times 294 = 126 \text{ mm.}$

$a_{\text{max}} = \beta_1 \times C_{\text{max}} = 0.85 \times 126 = 107.1 \text{ mm.} \quad * \text{Note: } f'_c = 24 \text{MPa}< 28 \text{ MPa} \rightarrow \beta_1 = 0.85$

$\ Mn_{\text{max}} = 0.85 \times f'_c \times b \times a \times (d - \frac{d}{2})$

$= 0.85 \times 24 \times 0.5 \times 0.1071 \times (0.294 - \frac{0.1071}{2}) \times 10^3$

$= 262.67 \text{KN.m}$

$\frac{250}{3} \times (0.004-0.002) = 0.816$

$\rightarrow \ Mn_{\text{max}} = 0.82 \times 262.67 = 215.39 \text{KN.m} \quad * \text{Note: } \varepsilon_s = 0.004 \rightarrow \quad = 0.82$

$\rightarrow \ Mn_{\text{max}} = 215.39 \text{KN.m} \rightarrow \ Mu = 28.6 \text{KN.m}$.

$\therefore \text{Singly reinforced concrete section.}$

4.7.2 Flexure design:

4.7.2.1 Design of Positive moment:-

1) Maximum negative moment $Mu^{(-)} = 28.6 \text{KN.m}$.

$Mn_{\text{max}} = 215.39 \text{KN.m} \rightarrow \ Mn = 28.6 \text{KN.m} \rightarrow \text{Singly reinforced concrete section}$

$Mn = \frac{\text{Mu}}{f'_c} = 28.9 / 0.9 = 32.11 \text{KN.m}$.

$m = \frac{f'_c}{0.85f'_c} = \frac{420}{0.85\times24} = 20.6$

$K_n = \frac{Mn}{b + d^2} = \frac{321 \times 10^{-3}}{0.294^2} = 0.74 \text{MPa}$.

$\rho = \frac{1}{m} (1 - \frac{2 + K_n \times m}{f'_c})$
\[ \frac{1}{20.6} = 1 - \frac{2 + 0.74 \times 20.6}{420} = 0.0017 \]

→ \( A_s = \rho \times b_w \times d = 0.0017 \times 500 \times 294 = 249.9 \text{ mm}^2 \).

\[ A_{\text{min}} = \frac{f'_{\text{c}}}{4 \left( f_y \right)} \times b_w \times d \geq \frac{14}{f_y} \times b_w \times d \quad \text{.........(ACI-10.5.1)} \]

\[ = \frac{\sqrt{24}}{4 \times 420} \times 500 \times 294 \geq \frac{14}{420} \times 500 \times 294 \]

\[ = 428.66 \text{ mm}^2 < 490 \text{ mm}^2 \quad \text{......... Larger value is control.} \]

→ \( A_{\text{min}} = 490 \text{ mm}^2 > A_{\text{req}} = 249.9 \text{ mm}^2 \).

\[ \therefore A_s = 490 \text{ mm}^2. \]

\[ \# \text{ of bars} = \frac{A_{\text{req}}}{A_{\text{bar}}} = \frac{490}{113.09} = 4.3 \rightarrow \# \text{ of bars} = 5 \text{ bars.} \]

\[ \therefore \text{ Use } 4 \times 12 \rightarrow A_s = 490 \times 5 = 113.09 \text{ mm}^2 > A_{\text{req}} = 490 \text{ mm}^2. \]

→ Check for strain:- \( (\varepsilon_s \geq 0.005) \)

Tension = Compression

\[ A_s \times f_y = 0.85 \times f_{c'} \times b \times a \]

\[ 565.45 \times 420 = 0.85 \times 24 \times 500 \times a \]

\[ a = 23.3 \text{ mm.} \]

\[ \varepsilon = \frac{a}{\beta_1} = \frac{23.3}{0.85} = 27.4 \text{ mm.} \]

* Note: \( f_{c'} = 24 \text{ MPa} < 28 \text{ MPa} \rightarrow \beta_1 = 0.85 \)

\[ \varepsilon_s = \frac{d - c}{c} \times 0.003 \]

\[ = \frac{294 - 27.4}{27.4} \times 0.003 = 0.029 > 0.005 \quad \therefore = 0.9 \text{ OK} \]

: Use 4 \( 12. \)

### 4.7.2.3 Design of shear:-

1) \( Vu = 30.7 \text{ KN} \).

\[ V_c = \frac{f'_{c}}{6} \times b_w \times d \]

\[ = 0.75 \times \frac{\sqrt{24}}{6} \times 0.5 \times 0.294 \times 10^3 = 90 \text{ KN}. \]
Check for section dimensions:

\[ V_c + \left( \frac{2}{3} \cdot f_c' \cdot b_w \cdot d \right) = 119.4 + \left( \frac{2}{3} \cdot 0.75 \cdot \sqrt{24} \cdot 0.5 \cdot 0.294 \cdot 10^3 \right) \]

\[ = 119.4 + 360.1 = 479.47 \text{ KN} \]

\[ \gg \gg V_u = 30.7 \text{ KN}. \]

\( \therefore \) Dimension is big enough.

4.7.2.4 Check for the case of shear:

1- Item 1: \( V_u \leq \frac{V_c}{2} \).

\[ 30.7 \leq \frac{90}{2} = 45 \ldots \text{ satisfy.} \]

\( \therefore \) Item (1) is satisfy \( \rightarrow \) minimum shear reinforcement is required.

\[ \left( \frac{A_{S}}{s} \right)_{\text{min}} \geq \frac{1}{16} \cdot \frac{f_y'}{f_{y_t}} \cdot b_w = \frac{1}{16} \cdot \frac{124}{420} \cdot 0.12 = 8.75 \cdot 10^{-5}. \]

\[ \geq \frac{1}{3} \cdot \frac{b_w}{f_{y_t}} = \frac{3}{3} \cdot \frac{0.12}{420} = 9.52 \cdot 10^{-5} \ldots \text{Control.} \]

Try \( \phi \) (2 Legs):

\[ \frac{2.50 \cdot 10^{-6}}{s} = 9.52 \cdot 10^{-5} \rightarrow s = 1.05 \text{ m} \]

\[ S \leq \frac{d}{2} = \frac{294}{2} = 147 \text{ mm} \leq 600 \text{ mm}. \]

\( \therefore \) Use \( \phi \) @ 10 Cm

\[ \text{Fig. (4-15) Detail of Beam and section(B1-17).} \]

4.8 Design of Column(C32):-
4.8.1 Load calculation:

\[
DL = 2933.68 \text{ KN} \quad LL = 993.66 \text{ KN}
\]

\[
P_u = 5110.275 \text{ KN} \quad P_{n,req} = 5110.275 / 0.65 = 7862 \text{ KN}
\]

Assume rectangular section with \( \rho = 2.38\% \)

\[
P_u = 0.8 \times A_g \times (0.85 \times f_c' \gamma + \rho g (f_y - 0.85 f_c'))
\]

\[
7862 = 0.8 \times A_g \times (0.85 \times 24 + 0.0238 \times (420 - 0.85 \times 24))
\]

\[
A_g = 3285.6 \text{ cm}^2
\]

Use 60*55 cm with \( A_g = 3300 \text{ cm}^2 > A_g,req = 3285.6 \text{ cm}^2 \)

4.8.2 Check slenderness effect:

Lu: Actual unsupported (unbraced) length.

K: effective length factor \((K = 1 \text{ for braced frame})\).

R: radius of gyration \(= \sqrt{I/A} \approx 0.3 h \)

\[
Lu = 2.76 \text{ m}
\]

\[
M1/M2 = 1
\]

In 60 cm -Direction

\[
Kl_{u/r} < 34 (M1/M2) < 40
\]

\[
(1 \times 2.76) / (0.3 \times 0.6) = 15.33 < 22 \quad \Rightarrow \text{Short}
\]

In 55 cm -Direction

\[
Kl_{u/r} < 34 (M1/M2)
\]

\[
(1 \times 3.78) / (0.3 \times 0.55) = 16.73 > 22 \quad \Rightarrow \text{Short}
\]

Short in Both Direction

\( \Rightarrow \text{Here we can solve this column as short tied column} \)
\[ P_n = 0.8 \times Ag \times (0.85 \times f_c' + \rho \times (f_y - 0.85 \times f_c')) \]

\[ P_n = 0.8 \times 600 \times 550 \times (0.85 \times 24 + 0.0238 \times (420 - 0.85 \times 24)) \]

\[ = 7896.4 \text{ KN} > P_{n,\text{req}} = 7862 \text{ KN} \ldots \ldots \text{OK} \]

### 4.8.4 Design of the tie reinforcement:

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

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

\[ S \leq \text{Least dimension.} \]

\[ \text{Spacing} \leq 16 \times d_b = 16 \times 2.5 = 40 \text{ cm} \ldots \text{control} \]

\[ \text{Spacing} \leq 48 \times d_t = 48 \times 1.0 = 48 \text{ cm} \]

\[ \text{Spacing} \leq \text{least.dim} = 55 \text{ cm} \]

**Use** \( \phi 10@20 \text{ cm} \)

**For Using SbCoulmn We have using** **16v25.**

---

**60*55**

---

**16v25.**

**Fig. (4-16) Detail of Reinforcement of Coulmn (C32)**

### 4.9 Design of Stair.
4.9.1 Minimum slab:

\[ h_{\text{min}} = \frac{L}{20} = \frac{410}{20} = 20.5 \text{cm thickness for deflection} \] (for simply supported one way solid)

Take \( h_{\text{min}} = 250 \text{mm} \).

4.9.2 Loads Calculation of stair case (1):

\[ \begin{align*}
\text{Qu} &= 21.41 \text{KN.m} \\
\text{RA} &= 35.32 \\
\text{RB} &= 35.32 \\
0.4 & \quad 3.3 & \quad 0.4 \\
4.10 \text{m} & 
\end{align*} \]

Figure 4-18: loads of the flight.
Flight Dead Load computations:

\[ Y = \tan^{-1}\left(\frac{\text{rise}}{\text{run}}\right) \]

<table>
<thead>
<tr>
<th>material</th>
<th>Quality Density</th>
<th>( W \text{ kN/m} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tiles</td>
<td>23</td>
<td>( 27\left(\frac{0.17+0.35}{0.3}\right) \times 0.03 \times 1 ) = 1.403</td>
</tr>
<tr>
<td>Mortar</td>
<td>22</td>
<td>( 22\left(\frac{0.17+0.3}{0.3}\right) \times 0.02 \times 1 ) = 1.034</td>
</tr>
<tr>
<td>Stair steps</td>
<td>25</td>
<td>( \frac{25\left(\frac{0.17+0.3}{2}\right)}{0.3} ) \times 1 = 2.125</td>
</tr>
<tr>
<td>R.C solid slab</td>
<td>25</td>
<td>( \frac{25 \times 0.25 \times 1}{\cos 29.54} ) = 7.184</td>
</tr>
<tr>
<td>Plaster</td>
<td>22</td>
<td>( \frac{22 \times 0.03 \times 1}{\cos 29.54} ) = 0.76</td>
</tr>
<tr>
<td>Total Dead Load</td>
<td>( \Sigma )</td>
<td>12.506 kN</td>
</tr>
</tbody>
</table>

Table 4-4: Dead load calculation for flight of stair.

Landing Dead load computation:
### Material Quality Density $W\text{ KN/m}^3$

<table>
<thead>
<tr>
<th>Material</th>
<th>Quality Density</th>
<th>$W\text{ KN/m}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tiles</td>
<td>23</td>
<td>$23 \times 0.03 \times 1 = 0.69$</td>
</tr>
<tr>
<td>Mortar</td>
<td>22</td>
<td>$22 \times 0.02 \times 1 = 0.44$</td>
</tr>
<tr>
<td>R.C solid slab</td>
<td>25</td>
<td>$25 \times 0.25 \times 1 = 6.25$</td>
</tr>
<tr>
<td>Plaster</td>
<td>22</td>
<td>$22 \times 0.03 \times 1 = 0.66$</td>
</tr>
<tr>
<td>Total Dead load</td>
<td>$\Sigma$</td>
<td>8.04</td>
</tr>
</tbody>
</table>

Table 4-5: Dead load calculation for landing of stair.

*live load = LL = 4KN/m$^2$

Total factored load: $w = 1.2D + 1.6L$

*for flight* $w = 1.2 \times 12.506 + 1.6 \times 4 = 21.41\text{ KN/m}$

*for landing* $w = 1.2 \times 8.04 + 1.6 \times 4 = 16.05\text{ N/m}$

#### 4-9-3Design of flight (Slab S1 is supported at the centerline of beam and L1).

The reaction at point A:

$$R_B = R_A = \frac{21.41 \times 3.3}{2} = 35.32\text{ KN}$$

- Check for shear strength:

Assume bar diameter $\ø14$ for main reinforcement.
\[ d = h - 20 - \frac{\bar{d}}{2} = 250 - 20 - \frac{14}{2} = 223 \text{mm} \]

Take the maximum shear as the support reaction
\[ V_u = 35.32 \times \cos 29.54 = 28.27 \text{KN} \]

\[ V_c = \frac{f_c^2}{6} b_w d \]
\[ = \frac{\sqrt{24}}{6} \times 1000 \times 223 \times 10^{-3} = 182.1 \text{KN}. \]
\[ \times V_c = 0.75 \times 182.1 = 136.55 \text{KN/1m strip} \]
\[ V_{u,max} = 28.27 < \frac{1}{2} \times V_c = 68.27 \text{KN} ...... \textbf{The thickness of the slab is enough}. \]

Calculate the maximum bending moment and steel reinforcement:
\[ M_u = 35.32 \times 2.05 - 21.41 \times 1.65 \times \frac{1.65}{2} = 43.26 \text{KN.m} \]
\[ M_n = M_u / = 43.26 / 0.9 = 48.067 \text{KN.m} \]

\[ d = \text{depth} - \text{cover} - \text{diameter of stirrups} - \text{diameter of } \frac{\text{bar}}{2} \]
\[ 300 - 20 - \frac{14}{2} = 223 \text{mm}. \]
\[ m = \frac{f_y}{0.85 f_c} = \frac{420}{0.85 \times 24} = 20.6 \]
\[ R_n = \frac{M_n}{b \times d^2} = \frac{48.067 \times 10^6}{1000 \times 223^2} = 0.97 \text{MPa} \]
\[ \rho = \frac{1}{m} (1 - \frac{2R_n m}{f_y}) \]
\[ = \frac{1}{20.6} (1 - \frac{2 \times 0.97 \times 20.6}{420}) = 0.0024 \]
\[ A_s = \rho \times b \times d = 0.0024 \times 1000 \times 223 = 535.2 \text{mm}^2. \]
\[ A_{s,min} = \rho \times b \times h = 0.0018 \times 1000 \times 250 = 450 \text{mm}^2 \]
\[ A_s = 535.2 \text{mm}^2 > A_{s,min} = 450 \text{mm}^2 \]

use \( \phi 14 @ 20 \) then
\[ n = \frac{A_s}{A_{s,min}} = \frac{535.2}{15393} = 3.47 = 4, \quad s = \frac{1}{n} = 4 = 0.250 \text{m}. \]
Step (S) is smallest of:

1. \( 3h = 3 \times 300 = 900\, mm \)
2. \( 450\, mm \)
3. \[ s = 380 \left( \frac{280}{f_s} \right) - 2.5C_c = 380 \left( \frac{280}{3420} \right) - 2.5 \times 20 = 330\, mm \]

\[ s \leq 300 \left( \frac{280}{f_s} \right) = 300 \left( \frac{280}{3420} \right) = 300\, mm - control \]

\[ s = 200\, mm \leq s_{\text{max}} = 300\, mm - OK \]

Select \( s = 300\, mm \)

Temperature and shrinkage reinforcement.

\[ A_s \, \text{Temperature and shrinkage} = 0.0018 \times 1000 \times 250 = 450\, mm^2 \]

use \( \varnothing 10@15 \) then

\[ n = \frac{A_s}{A_s \varnothing 10} = \frac{450}{78.5} = 5.7 \approx 6 \quad s = \frac{1}{n} = \frac{1}{6} = 0.16\, m \]

Take 150 mm

Step (S — for Temperature and shrinkage reinforcement) is the smallest of:

1. \( 5h = 5 \times 250 = 1250\, mm \)
2. \( 450\, mm - control \)

\[ s = 150\, mm < s_{\text{max}} = 450\, mm - OK \]

Select \( s = 450\, mm \)

4-9-4 Design of slab L1 (landing):

\[ w_R = q_u + \text{support of flight} = 16.05 + 23.32 = 51.37\, KN/m \]

The reaction at each end
Check for shear strength:
Assume bar diameter Ø14 for main reinforcement.

\[ d = h - 20 - \frac{d_p}{2} = 25 - 20 - \frac{14}{2} = 223\,mm \]

Take the maximum shear as the support reaction \[ V_{sl} = 88.61 - 51.37 \times 0.323 = 72.07\,KN \]

\[ V_c = \frac{f_c'}{6} b_w d \]

\[ = \frac{\sqrt{24}}{6} \times 1000 \times 223 \times 10^{-3} = 182.1\,KN. \]

\[ \times V_c = 0.75 \times 182.1 = 136.56\,KN/1m\,strip \]

\[ V_c = 136.56\,KN > V_{sl,max} = 72.07 \]

**The thickness of the slab is enough.**

use \( h = 25 \,cm \)

Calculate the maximum bending moment and steel reinforcement:

\[ M_d = \frac{51.37 \times 3.45^2}{8} = 76.43\,KN.m \]

\[ M_n = Mu / \times 76.43 / 0.9 = 84.92\,KN.m \]

\[ d = depth - cover - diameter\,of\,stirrups - (diameter\,of\,bar/2 \]

\[ = 250 - 20 - \frac{14}{2} = 223\,mm. \]

\[ m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 \times 24} = 20.6 \]
\[ R_n = \frac{M_n}{b \cdot d^2} = \frac{84.92 \times 10^6}{1000 \times 22^3} = 1.71 \text{ MPa} \]

\[ \rho = \frac{1}{m} \left( 1 - \frac{1 - 2 \cdot R_n \cdot m}{f_y} \right) \]

\[ = \frac{1}{20.6} \left( 1 - \frac{2 \cdot 1.71 \times 20.6}{420} \right) = 0.00426 \]

\[ A_s = \rho \cdot b \cdot d = 0.00426 \cdot 1000 \cdot 223 = 950 \text{ mm}^2. \]

\[ A_{s,\text{min}} = \rho \cdot b \cdot h = 0.0018 \cdot 1000 \cdot 250 = 450 \text{ mm}^2 \]

\[ A_s = 950 \text{ mm}^2 > A_{s,\text{min}} = 450 \text{ mm}^2 \]

use Ø14 then

\[ n = \frac{A_s}{A_{s,\text{Ø14}}} = \frac{950}{153.93} = 6.2 \quad s = \frac{1}{n} = 0.16 \]

Step (S) is smallest of:

1. \( 3h = 3 \times 300 = 900 \text{mm} \)

2. \( 450 \text{mm} \)

3. \( s = 380 \cdot \frac{280}{f_s} - 2.5 \bar{C}_e = 380 \cdot \frac{280}{3 \times 420} - 2.5 \cdot 20 = 330 \text{mm} \)

\[ s \leq 300 \cdot \frac{280}{f_s} = 300 \cdot \frac{280}{3 \times 420} = 300 \text{mm} - \text{control} \]

\[ s = 150 \text{mm} < s_{\text{max}} = 300 \text{mm} - \text{OK} \]

- Temperature and shrinkage reinforcement.

\[ A_s = \text{Temperature and shrinkage} = 0.0018 \cdot 1000 \cdot 250 = 450 \text{mm}^2 \]

\[ n = \frac{A_s}{A_{s,\text{Ø14}}} = \frac{450}{153.93} = 2.9, \quad s = \frac{1}{n} = \frac{1}{3} = 0.333 \text{ m} = .300 \]

Step (S - for Temperature and shrinkage reinforcement) is the smallest of:

1. \( 5h = 5 \times 250 = 1250 \text{mm} \)

2. \( 450 \text{mm} - \text{control} \)

\[ s = 300 \text{mm} < s_{\text{max}} = 450 \text{mm} - \text{OK} \]

Select \( s = 450 \text{mm} \)
4.10 Design of basement wall

4.10.1 Load Calculation:

\( f'_c = 24 \text{MPa} \)

\( f_y = 420 \text{MPa} \)

\( \gamma = 18 \text{KN/m}^3 \)

Figure 4-20: Basement wall

\( \varnothing = 30^\circ \)

\( LL = 4 \text{KN/m}^2 \)

**Thickness** = \( h = 20 \text{cm}, \) **cover** = \( 4 \text{cm} \)

The design will be for 1m width

- **Analysis:**
- **Loads**

Neglect the axial load, since its low value.

\( e_1 = K_o \cdot \gamma \cdot h \)

\( e_2 = K_o \cdot LL \)

\( K_o = 1 - \sin \varnothing \)

So,

\( K_o = 1 - \sin 30 = 1 - 0.5 = 0.5 \)

\( e_o = 0.5 \cdot 18 \cdot 2.935 = 29.14 \text{KN/m}^2 \)

\( e_L = 0.5 \cdot 4 = 2 \text{KN/m}^2 \)

\( E_L = 2 \cdot 2.935 = 5.87 \text{KN/m}^2 \)

Support reactions:

\( B_x = 21.025 \text{KN} \)
\[ A_x = 38.625 \text{ KN} \]

\[ V = 0 \quad \text{at} \quad y = ? \]

\[ 21.025 - P \cdot y \cdot \frac{y}{2} - 2 \cdot y = 0 \]

\[ \frac{P \cdot y}{y} = \frac{26.415}{2.935} = 9 \]

\[ 21.025 - 9 \cdot y \cdot \frac{y}{2} - 2 \cdot y = 0 \]

\[ 4.5y^2 + 2y - 21.025 = 0 \]

\[ y = 2 m \]

\[ M_{u,max} = 21.025 \cdot 2 - 9 \cdot 2 \cdot \frac{2}{3} \cdot \frac{2}{2} - 2 \cdot 2 \cdot \frac{2}{2} = 26.05 \text{ KN.m} \]

**Factored internal forces**

\[ V_u = 1.6 \cdot V_{\text{max}} = 1.6 \cdot 38.625 = 61.8 \text{ KN} \]

\[ M_u = 1.6 \cdot M_{\text{max}} = 1.6 \cdot 26.05 = 41.68 \text{ KN} \]

**- Design**

**Design of shear**

\[ d = 200 - 40 - 8 = 152 \text{ mm} \]

\[ V_u = 61.8 \text{ KN} \]

\[ V_c = 0.75 \cdot \frac{f'_c}{6} b_d d = V_c = 0.75 \cdot \frac{\sqrt{24}}{6} \cdot 1000 \cdot 152 = 93 \text{ KN} > V_u = 61.8 \text{ KN} \]

The thickness of Wall is Adequate Enough

**Design of flexure**
Vertical reinforcement of Tension face

\[ M_{ub} = 41.68 \text{ KN.m} \]

\[ M_n = \frac{M_{ub}}{0.9} = 46.31 \text{ KN.m} \]

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

\[ R_n = \frac{M_n}{b \cdot d^2} = \frac{46.31 \times 10^6}{1000 \times (152)^2} = 2.0 \text{ MPa} \]

\[ \rho = \frac{1}{m} \left( 1 - \frac{1 - 2 \cdot R_n \cdot m}{f_y} \right) \]

\[ = \frac{1}{20.6} \left( 1 - \frac{1 - 2 \cdot 2 \cdot 20.6}{420} \right) = 0.005 \]

\[ A_{s,req} = \rho b d = 0.005 \times 1000 \times 152 = 760 \text{ mm}^2 \]

\[ A_{s,min} = 0.0012 \times 1000 \times 200 = 240 \text{ mm} \]

\[ A_{s,req} = 760 \text{ mm}^2 > A_{s,min} = 240 \text{ mm}^2 \ldots \text{OK} \]

\[ \therefore A_{s,req} = 760 \text{ mm}^2 \]

Select 7012 with \( A_{s,pro} = 791.68 \text{ mm}^2 > A_{s,req} = 760 \text{ mm}^2 \ldots \text{OK} \]

Vertical reinforcement of Compression face

\[ A_{s,min} \text{ for flexure} = 0.25 \times \frac{f'c^d}{f_y} \times b_w \times d = 0.25 \times \frac{\sqrt{24}}{420} \times 1000 \times 152 = 443 \text{ mm}^2 / \text{m} \]

\[ A_{s,min} \text{ for flexure} = 1.4 \frac{f_y}{f_y} \times b_w \times d = 1.4 \times 420 \times 1000 \times 152 = 506.67 \text{ mm}^2 / \text{m} \]

Select 5012 with \( A_{s,pro} = 565.5 \text{ mm}^2 > A_{s,min} = 506.67 \text{ mm}^2 / \text{m} \)

For inside wall \( \phi 12@15 \text{ cm} = 7.91 \text{ cm}^2 > 7.60 \text{ cm}^2 \)

For outside wall \( \phi 12@20 \text{ cm} = 5.65 \text{ cm}^2 > 5.1 \text{ cm}^2 \)
Horizontal Reinforcement due to Cracking:

\[ A_{req} h = 0.002 \times b \times h = 0.002 \times 100 \times 20 = 4 \text{ cm}^2/\text{m} \]

For one side \( A_s = 2 \text{ cm}^2/\text{m} \)

Select for one side horizontal reinforcement \( \phi 10@20 = 3.93 \text{ cm}^2 > 2 \text{ cm}^2 \)

Figure 4-21: reinforcement of Basement wall
4.11 Design of Isolated Footing (F5 C50).

Fig. (4-22) : Footing geometry
From column group 5:

\[ DL = 1823.96 \text{ KN} \]

\[ LL = 818.37 \text{ KN} \]

Factored load = 3498.14 kN.

Soil weight = 18 kN/m3.

Allowable soil pressure = 400 kN/m2.

\[ F_c' = 24 \text{ Mpa} \]

\[ F_y = 420 \text{ Mpa} \]

Cover = 7.5 cm

### 4.11.1 Determine the net soil pressure:

use steel bar 14

Assume \( h = 70 \text{ cm} \); \( d = 700-75-14 = 611 \text{ mm} \)

Weight of footing = 0.7*25 = 17.5 Kn/m^2

Weight of soil = 1*18 = 18 Kn/m^2

Total surcharge load foundation:

\[ W = 17.5 + 18 = 35.5 \text{ KN/m}^2 \]

\[ q_{all.net} = 400 - 35.5 = 364.5 \text{ KN/m}^2 \]

### 4.11.2: Design of the footing area:

\[ A = \frac{P_n}{q_{all.net}} = \frac{2642.33}{364.5} = 7.25 \text{ m}^2 \]

\[ A = b*l \]

Take \( b = 2.80 \text{ m} \)

\( l = 7.25/2.80 = 2.6 \text{, take } l = 2.80 \text{m} \)

\[ q_{u} = \frac{3498}{(2.80 * 2.80)} = 446.2 \text{ KN/m} \]
4.11.3 Check for one way shear:

For X-direction:

\[ Vu = ((2.80 - 0.50) \times 0.5 - 0.611) \times 446.2 \times 2.80 \]

\[ Vu = 673.4 \text{ KN} \]

For Y-direction:

\[ Vu = ((L - a) \times 0.5 - d) \times qu \times b \]

\[ Vu = ((2.80 - 0.5) \times 0.5 - 0.611) \times 446.2 \times 2.80 \]

\[ Vu = 673.4 \text{ KN} \]

\[ \phi V_{c,x} = \phi \left( \sqrt{f'c} \right) \times bw \times d / 6 \]

\[ = 0.75 \times \sqrt{24} \times 2800 \times 611 \times 10^{-3} / 6 \]

\[ = 1047.6 \text{ KN} > V_{ux} = 673.4 \text{ KN} \Rightarrow \text{OK} \]

\[ \phi V_{c,y} = \phi \left( \sqrt{f'c} \right) \times bw \times d / 6 \]

\[ = 0.75 \times \sqrt{24} \times 2800 \times 611 \times 10^{-3} / 6 \]

\[ = 1047.6 \text{ KN} > V_{uy} = 673.4 \text{ KN} \Rightarrow \text{OK} \]

4.11.4 Check for two way shear:

\[ Vu,x = qu \times (b \times l - (a+d) \times (c+d)) \]

\[ = 446.2 \times (2.80 \times 2.80 - (0.5+0.611) \times (0.5 + 0.611)) \]

\[ = 2506.7 \text{ KN.} \]

\[ s = 40 \text{ for interior column} \]

\[ \beta = 50/(50) = 1.0 \]

\[ bo = \text{Perimeter of critical section taken at (d/2) from the loaded area} \]

\[ bo = 2 \times (a+d+c+d) \]

\[ = 2 \times (0.50+0.611 \times 2+0.5) \]

\[ = 4.444 \text{ m} \]
Vc the smallest of:

\[ V_c = 1/6 \times (1 + 2/B) \sqrt{(fc') \times b \times d} \]

where \( 1/6 \times (1 + 2/1.0) = 0.50 \)

\[ V_c = 1/12((\alpha sd)/b + 2) \sqrt{(fc') \times b \times d} \]

where \( 1/12((40 \times 0.611)/4.444 + 2) = 0.625 \)

\[ V_{c1} = 1/3 \times \sqrt{(fc') \times b \times d} \]

where \( 1/3 = 0.333 \ldots \) control

Take \( V_{c1} = 1/3 \times \sqrt{(fc') \times b \times d} = 1/3 \times \sqrt{24 \times 4444 \times 611 \times [10] \times (-3)} = 4434.04 \text{ KN} \)

\[ \bar{V}_{c1} = 0.75 \times 7057.8 = 3325.5 \text{ KN} \]

\[ \bar{V}_{c1} = 3325.5 > V_u = 2506.7 \text{ KN} \ldots \ldots \text{ ok} \]

### 4.11.5 Design for bending moment:

#### 4.11.5.1 Design flexure for long and short direction:

use steel bar 14

\( b = 2.8 \text{ m}, \ h = 700 \text{ mm}, \ d = 611 \text{ mm} \)

\[ M_u = 446.2 \times 2.80 \times (0.5)^2/2 = 156.17 \text{ KN.m} \]

\[ m = f_y/(0.85 \times fc') = 420/(0.85 \times 24) = 20.59. \]

\[ R_n = M_u/(\varnothing b \times d^2) = (156.17 \times [10]^6)/(0.9 \times 2800 \times ((611)^2) = 0.17 \text{ MPa}. \]

\[ \rho = 1/m(1 - \sqrt{(1 - (2 \times R_n \times m)/f_y)}) \]

\[ = 1/20.59(1 - \sqrt{(1 - (2 \times 20.59 \times 0.17)/420)}) = 0.00041 \]

\[ A_s = * b \times d = 0.00041 \times 2800 \times 611 = 701.428 \text{ mm}^2. \]

\[ A_{s_min} = 0.0018 \times b \times h = 0.0018 \times 2800 \times 700 = 3528 \text{ mm}^2. \]

\[ A_{s_min} = 3528 \text{ mm}^2 \geq A_{req} = 701.428 \text{ mm}^2. \]

\[ \therefore A_s = A_{s_min} = 3528 \text{ mm}^2. \]

\[ n = A_{req}/(A_{bar\varnothing 14}) = (3528)/153.94 = 25.2 \]

\[ \therefore \text{ Use 26 } 14 \]

\[ S = (2800 - 75 \times 2 - 26 \times 14)/25 = 91.44 \text{ mm} \]

Step S is the smallest of
$3h = 3 \times 700 = 2100\text{mm}$

450........control

$S = 91.44 < S_{\text{max}} = 450$ ...... ok
4.12 Design of Flat slab:

The design done by using SAFE program.

4.12.1 Load calculation:

Assume slab thickness 30cm.

<table>
<thead>
<tr>
<th>No.</th>
<th>Material</th>
<th>Thickness cm</th>
<th>Quality Density KN/m³</th>
<th>Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Slab</td>
<td>30</td>
<td>25</td>
<td>0.30×25 = 7.5</td>
</tr>
<tr>
<td>2</td>
<td>Sand</td>
<td>7</td>
<td>17</td>
<td>0.07×17 = 1.19</td>
</tr>
<tr>
<td>3</td>
<td>Mortar</td>
<td>2</td>
<td>22</td>
<td>0.02×22 = 0.44</td>
</tr>
<tr>
<td>4</td>
<td>Tile</td>
<td>3</td>
<td>23</td>
<td>0.03×23 = 0.69</td>
</tr>
<tr>
<td>5</td>
<td>Plaster</td>
<td>2</td>
<td>22</td>
<td>0.02×22 = 0.44</td>
</tr>
<tr>
<td>6</td>
<td>Partitions</td>
<td></td>
<td></td>
<td>2.38</td>
</tr>
</tbody>
</table>

\[ \sum = 12.64 \text{ KN/m}^2 \]

Table (4 – 6) Calculation of the total dead load for flat slab.
4.12.2 Check for punching shear:

![Diagram](image)

**Figure (4-23):** Punching Shear Capacity Ratios / Shear Reinforcement for flat slab

As shown all ratios less than 1, so we don’t have punching reinforcement.
4.12.3 Design for bending moment:

*Figure (4-24):* moment distribution in x-direction
The design of flat slab done by using Finite Element method.
Selected Ø14/15cm in both direction for top reinforcement
Selected Ø14/15cm in both direction for bottom reinforcement

**Figure (4-25):** moment distribution in y-direction
4.13 Design of column (C44):

4.13.1 Load calculation:

DL = 845 KN  
LL = 200 KN  

\[ P_u = 1333 \text{ KN} \quad P_{n,req} = \frac{1333}{0.65} = 2050.76 \text{ KN} \]

Assume rectangular section with \( \rho = 1.67\% > 1\% \)

\[ P_n = 0.8 \times A_g \times (0.85 \times f_c' + \rho_g \times (f_y - 0.85 \times f_c')) \]

\[ 2050.76 = 0.8 \times A_g \times (0.85 \times 28 + 0.0167 \times (420 - 0.85 \times 28)) \]

\[ A_g = 842.78 \text{ cm}^2 \]
Use 25×60 cm with $A_g = 1500 \text{ cm}^2 > A_{g,\text{req}} = 842.78 \text{ cm}^2$

### 4.13.2 Check slenderness effect:

Lu: Actual unsupported (unbraced) length.

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

R: radius of gyration $= \sqrt(l/A) = 0.3\ h$

$Lu = 4.3\ m$

$M1/M2 = 1$

**In 25 cm - Direction**

$$Klu/r < 34\ 12 \ (M1/M2) < 40$$

$$(1 \times 4.3 \ )/(0.3 \times 0.25) = 57.33 > 22 \Rightarrow \text{long}$$

**In 60cm - Direction**

$$Klu/r < 34\ 12 \ (M1/M2)$$

$$(1 \times 4.3 \ )/(0.3 \times 0.6) = 23.89 > 22 \Rightarrow \text{Long}$$

Long in x direction

Long in y direction

### 4.13.3 Calculation for reinforcement:

**In 25 cm - Direction**

$$E_c = 4700 \times \sqrt{28} = 24870.1\text{MPa}$$
\[ \delta_{\text{shear}} = \frac{(1.2 \times D \times \text{sustained})}{P_e} = \frac{(1.2 \times 845)}{1333} = 0.76 \]

\[ l_b = b \times \left( \frac{h}{12} \right)^3 = 60 \times (25)^3 / 12 = 0.00781 \text{ m}^4 \]

\[ E I = \left( 0.4 \times E_c \times I \right) / (1 + \beta \times \text{dns}) = \left( 0.4 \times 24870.1 \times 0.00781 \right) / (1 + 0.76) = 44.14 \text{ MN.m}^2 \]

\[ P_c = \frac{\pi^2 \times E I}{(K I u)^2} \]

\[ = \frac{(\pi^2 \times 44.1)}{(1.0 \times 4.3)^2} \]

\[ = 23.6 \text{ MN} \]

\[ C_m = 0.6 + 0.4 \times \frac{M1}{M2} = 1 \]

\[ \delta_{\text{eq}} = C m / (1 - (P_e / (0.75 \times P_c))) = 1 / (1 - (1333 / (0.75 \times 23.6 \times 1000))) = 1.1 < 1.4 \]

\[ e_{\min} = 15 + 0.03 \times h = 15 + 0.03 \times 250 = 22.5 \text{ mm} \]

\[ e = e_{\min} \times \delta_{\text{eq}} = 22.5 \times 1.15 = 25.875 \text{ mm} \]

\[ e / h = 25.875 / 250 = 0.099 < 0.1 \ldots \ldots \text{(e = 0.082h < 0.1h)} \]

**In 60 cm - Direction**

\[ E_c = 4700 \times \sqrt{28} = 24870.1 \text{ MPa} \]

\[ \delta_{\text{shear}} = \frac{(1.2 \times D \times \text{sustained})}{P_e} = \frac{(1.2 \times 845)}{1333} = 0.76 \]

\[ l_b = b \times \left( \frac{h}{12} \right)^3 = 25 \times (60)^3 / 12 = 0.0045 \text{ m}^4 \]

\[ E I = \left( 0.4 \times E_c \times I \right) / (1 + \beta \times \text{dns}) = \left( 0.4 \times 24870.1 \times 0.0045 \right) / (1 + 0.76) = 25.44 \text{ MN.m}^2 \]

\[ P_c = \frac{\pi^2 \times E I}{(K I u)^2} \]

\[ = \frac{(\pi^2 \times 25.44)}{(1.0 \times 4.3)^2} \]

\[ = 13.57 \text{ MN} \]
Structural analysis & design

\[ C_m = 0.6 + 0.4 \times (M1/M2) = 1 \]

\[ \delta_{eq} = C_m/(1 - (P_u)/(0.75 P_c)) = 1/(1 - (1333)/(0.75 \times 13.57 \times 1000)) = 1.15 < 1.4 \]

\[ e_{min} = 15 + 0.03 \times h = 15 + 0.03 \times 600 = 33 \text{ mm} \]

\[ e = e_{min} \times \delta_{eq} = 33 \times 1.15 = 37.95 \text{ mm} \]

\[ e/h = 37.95/600 = 0.063 < 0.1 \ldots \ldots (e = 0.082h < 0.1h) \]

→ Here we can solve this column as short tied column

\[ P_n = 0.8 \times A_g \times (0.85 \times f_{c} + \psi (f_y - 0.85 f_c)) \]

\[ P_n = 0.8 \times 250 \times 600 \times (0.85 \times 28 + 0.0167 \times (420 - 0.85 \times 28)) \]

\[ = 3649.9 \text{ KN} > P_{n, req} = 2050.76 \text{ KN} \ldots \ldots \text{OK} \]

4.13.4 Design of the tie reinforcement:

\[ S \leq 6 \text{ db (longitudinal bar diameter)} \]

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

\[ S \leq \text{ Least dimension.} \]

\[ \text{ spacing } \leq 6 \times d_b = 16 \times 2.0 = 32 \text{ cm} \]

\[ \text{ spacing } \leq 8 \times d_t = 48 \times 1.0 = 48 \text{ cm} \]

\[ \text{ spacing } \leq \text{ least.dim} = 25 \text{ cm} \ldots \text{ control} \]

\[ 20 \text{ cm } \leq 25 \text{ cm} \ldots \text{ ok} \]

Use \( \phi 10@20 \text{ cm} \)

For column 5(c44)
Fig. 4-27: Reinforcement of column 44

4.14: Design of Shear wall (sw15)

(Figure 4-28: Moment and shear diagram)
Fc = 28MPa  
Fy = 420 MPa  
t = 25cm, shear wall thickness  
Lw = 6.20m, shear wall width  
Hw₁ for one wall = 3.00 m  
Hw₂ for one wall = 5.1 m story height  
Hw₃ for one wall = 3.3 m story height

### 4.14.1: Design of shear

\[ \sum F_x = V_u = 374 KN \]

### 4.14.2: Design of the Horizontal reinforcement:

The critical Section is the smaller of:

\[ \frac{L_w}{2} = \frac{6.20}{2} = 3.10 \text{ m} \]
\[ \frac{H_w}{2} = \frac{21.30}{2} = 10.65 \text{ m} \]

storyheight \( H_w \) = 3.30 m \ldots control

\[ d = 0.8 \times L_w = 0.8 \times 6.20 = 4.96 \text{ m} \]

\[ \phi V_{\text{max}} = \phi \frac{5}{6} f_c' h_d \]
\[ = 0.75 \times 0.83 \times \sqrt{28} \times 250 \times 4960 \times 10^{-3} = 4084.5 KN > V_u \]

\[ V_c \] is the smallest of:

\[ 1 - V_c = \frac{1}{6} f_c' h_d = \frac{1}{6} \sqrt{28} \times 250 \times 4960 \times 10^{-3} = 1093.58 KN \]

\[ 2 - V_c = 0.27 f_c' h_d + \frac{N_u d}{4 L_w} = 0.27 \sqrt{28} \times 250 \times 4960 \times 10^{-3} + 0 = 1771.6 KN \]

\[ 3 - V_c = 0.05 f_c' + \frac{0.1}{V_u} \frac{L_w}{2} \frac{N_u}{2} h_d \]
\[ = 0.05 \sqrt{28} + \frac{6.20 \times 0.1 \sqrt{28} + 0}{6.70} \frac{250 \times 4960}{250 \times 4960} = 935.25 KN \ldots \text{cont} \]
\[
\frac{M_{ud}}{V_{tu}} = \frac{l_w}{2} = \frac{2308.8}{374} - \frac{6.20}{2} = 3.07
\]
\[
\frac{1}{V_u} = 374 \text{KN} < \frac{2}{0.75} \times 935.25 = 380.8 \text{ KN} \quad \text{No need reinforcement}
\]

- **Minimum shear reinforcement is required:**
  Take \( \rho = 0.0025 \)

- **Maximum spacing is the least of:**
  \[
  \frac{L_w}{5} = \frac{6200}{5} = 1240 \text{ mm}
  \]
  \[3* h = 3 \times 250 = 750 \text{ mm}
  \]
  450 mm …… Control

**Try \( \phi 12 \) (As = 113.1 mm²) for two layers**

\[
\rho = \frac{h * S^2}{A_{vh}} = \frac{2 \times 113.1}{250 \times S^2} = 0.0025
\]
\[S^2 = 455.1 \text{ mm}^2 \quad , \quad \phi 12 @ 250 \text{ mm}
\]

→ use \( \phi 12 @ 250 \text{ mm} \) in two layer

**4.14.3: Design for Vertical reinforcement:**

\[
\frac{h_w}{L_w} = \frac{21.30}{6.20} = 3430 \text{ mm}
\]
\[
\frac{L_w}{3} = \frac{6200}{3} = 2066.67 \text{ mm}
\]
450 mm …… Control

\[3* h = 3 \times 250 = 750 \text{ mm}
\]
\[A_{nv} = 0.0025 * S * h
\]

**Try \( 12 \) (As = 113.1 mm²)**

113.1*2=0.0025*S*250
\[S = 452.4 \]

Select \( 12 @ 250 \text{ mm} \) In tow layer.
4.14.4: Design of bending moment (uniformly distribution flexural reinforcement):

\[ A_{st} = \frac{6200}{250} \times 2 \times 113.1 = 5609.76 mm^2 \]

\[ w = \frac{A_{st} f_y}{L_w h f_c} = \frac{5609.7}{6200 \times 250} \times \frac{420}{28} = 0.05 \]

\[ \alpha = \frac{P_u}{L_w h f_c'} = 0 \]

\[ C = \frac{w + \alpha}{2w + 0.85\beta_1} = \frac{0.05 + 0}{2 \times 0.05 + 0.85 \times 0.85} = 0.06 \]

\[ \phi M_n = 0.5 A_{st} f_y l_w (1 + \frac{P_u}{A_{st} f_y})(1 - \frac{c}{l_w}) = 0.9 \times 0.5 \times 5609.7 \times 420 \times 6200 (1 + 0)(1 - 0.0806) = 6043 KN.m > Mu \]

Select 12 @250mm for vertical reinforcement.

4.15 Design of the Mat Foudation reinforcement:

Design done by using SAFE.

4.15.1 Load calculation:

Density of soil = 18KN/m³
Allowable soil pressure = 400KN/m²
Fc' = 28Mpa
Fy = 420 Mpa
Cover = 7.5 cm

Take the reaction of columns and walls from ETABS.

4.15.2 Determine the soil pressure:

Subgrade Modulus of soil = 120*400 = 48000KN/m³
Max pressure = 300 KN/m² < 400 KN/m²

4.15.3 Check for punching shear:

As shown all ratios less than 1, so we don't have punching reinforcement.
4.15.4 Design for bending moment:

*Figure (4-31):* moment distribution in x-direction
Figure (4-32): moment distribution in y-direction

Figure (4-33): reinforcement of mat foundation.
The design of mat foundation by using Finite Element method

Selected basic mesh  16/10cm for top reinforcement

Selected basic mesh  16/10cm for bottom reinforcement
4.16.4 : Design of weld:

The calculation of weld based on the following:

1) Fillet weld is used.
2) The plates are A36(fy=36 ksi,Fu=58 ksi)
3) The plat thickness is (t=0.5 in)
4) The electrodes having F_{Exx}=70 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:
The section of the vertical member is angle (L3*3*3/8) , Ag=2.11 in², y=0.884.
The value of Max. compression in the vertical member is V_u=20.662 Kips.

Max. weld size (a_{max})= \frac{t - \frac{1}{16}}{1} = \frac{3}{8} - \frac{1}{16} = \frac{5}{16} in
Min. weld size (a_{min})= \frac{3}{16} in
Use weld size (a)= \frac{1}{4} in

- Design strength of weld:
  \( \phi \times R_{nw} = \phi \times t \times 0.6 \times F_{Exx} \)
  \( \phi \times R_{nw} = 0.75 \times (0.707 \times \frac{1}{4}) \times 0.6 \times 70 = 5.57 \) kips

- Design strength of base material:
  \( \phi \times R_{n} = \phi \times (0.6 \times f_{y}) \times t = 1.0 \times 0.6 \times 36 \times \frac{3}{8} = 8.1 \) kips >5.57 kips….ok

  Or

  \( \phi \times R_{n} = \phi \times (0.6 \times f_{u}) \times t = 0.75 \times 0.6 \times 58 \times \frac{3}{8} = 9.79 \) kips >5.57 kips….ok

  \( f_{1}=5.57 \times 3=16.71 \) kips
  \( f_{2}=20.662-16.71=3.952 \) kips

  \( \frac{f_{2}}{0 \times R_{nw}} = \frac{3.952}{5.57} = 0.71 \) in  ... use 1.0 in

2nd) Design of weld between the diagonal member and the gusset plate:
- The section of the diagonal member is angle (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 \( T_u = 51.1 \) kip.

Max. weld size (a_{max})= \frac{t - \frac{1}{16}}{1} = \frac{3}{8} - \frac{1}{16} = \frac{5}{16} in
Min = Weld size (a_{min})= \frac{3}{16} in
Use weld size (a)= \frac{1}{4} in
- Design strength of weld:
  \[ \varnothing \times R_{nw} = \varnothing \times (0.6 \times \varnothing) \times (0.6 \times 70) = 5.57 \text{ kips} \]

- Design strength of base material:
  \[ F_3 = 3 \times 5.57 = 16.71 \text{ kips} \]
  \[ \Sigma M \text{ at } F_1 = 0 \]
  \[ = 16.71 \times 1.5 + F_2 \times 3 - 51.1 \times (3 - 0.884) = 0 \]

  \[ F_2 = 27.69 \text{ kips} \]
  \[ F_1 = 51.1 - 16.71 - 27.69 = 6.7 \text{ kips} \]

\[ lw_1 = \frac{f_1}{\varnothing \times R_{nw}} = \frac{6.7}{5.57} = 1.21 \text{ in} \quad \text{...use 1.5 in} \]

\[ lw_2 = \frac{f_2}{\varnothing \times R_{nw}} = \frac{27.69}{5.57} = 4.97 \text{ in} \quad \text{...use 5 in} \]

Check for rupture

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

\[ U = 1 - \frac{x}{L} = 1 - \frac{0.884}{3.25} = 0.728 \]

\[ \varnothing tPn = 0.75 \times f_u \times A_e \]

\[ \varnothing tPn = 0.75 \times 58 \times 0.728 \times 2.11 = 66.82 \text{ kips} > 51.1 \text{ kips} \quad \text{.... ok} \]

3rd) Design of weld between the bottom member and the gusset plate:
The section of the bottom member is angle (W6\text{*}12)

\[ 11/2.54 = 4.33 \text{ in} \]

\[ Ru = \frac{(R_v + R_y)^2 \times (R_h + R_X)^2}{Py} \]

\[ R_v = \frac{P_y}{L} = 0 \]

\[ R_h = \frac{P_x}{L} = \frac{20.662}{14.76 \times 2} = 0.7 \text{ kip/in} \]

\[ Ip = 2 \times \frac{14.76^2}{12} = 535.93 \text{ in}^3 \]
\[ Rx = \frac{M \times y}{I_p} = 0 \quad \text{...} \quad y = 0 \]

\[ Ry = \frac{M \times x}{I_p} = \frac{20.662 \times \left( \frac{433}{2} \right)}{535.93} = 0.1 \]

\[ Ru = (0 + 0.1)^2 + (0.7 + 0)^2 = 0.71 \text{ kip/in} \]

\[ 0 \times R_{nw} = Ru \]

0.75 \times (0.707a) \times 0.6 \times 70 = 0.71 \quad \text{...} \quad a = 0.032 \text{ in} \]

\[ Take \ a = \frac{2}{16} \text{ in} \]