Kingdom of Saudi Arabia
University of Tabuk
Faculty of Engineering
Department of Civil Engineering

المملكة العربية السعودية
جامعة تبوك-كلية الهندسة
قسم الهندسة المدنية

Course: CE 452

Reinforced Concrete II

Group 2, Semester 2: 1446

Mini Project Title:

Analysis and design of Two-way slabs

Group No	Student Name	Student ID
2		421004527
		421004832
		422010968

Instructor: Prof. Basuony El-Garhy

Contents

1. Introduction

The foundation is one of the most important parts of any building because it transfers the loads safely to the soil. If the foundation is not designed properly, the building may face problems such as settlement or even failure.

In this mini-project, we are asked to design isolated footings for a multistory building. The loads on the columns are calculated first using the building dimensions and the given uniform loads. Then, the soil bearing capacity is checked using Terzaghi's equation with a factor of safety. According to these values, the required size of the footings under each column is determined.

The project also includes checking the settlement of the footings to make sure it is within safe limits. After that, the structural design of the footing reinforcement is carried out, and detailed drawings (plan and section) are prepared.

The objectives of this project are:

1. To practice the design of isolated footings in real conditions.
2. To check both soil bearing capacity and settlement requirements.
3. To provide reinforcement details for safe construction.
4. To prepare a short technical report and presentation.

2. Statement of the problem

The given multistory building is supported on three columns (C1, C2, and C3) of different sizes. Each column carries different loads that must be transferred safely to the soil. The soil profile shows varying properties with depth, which affects both the bearing capacity and the settlement of the foundations.

The problem is to design safe and economical isolated footings for these columns by:

- Calculating column loads.
- Checking the allowable soil bearing capacity.
- Determining footing dimensions that prevent failure.
- Ensuring that settlement is within acceptable limits.
- Providing reinforcement details for construction.

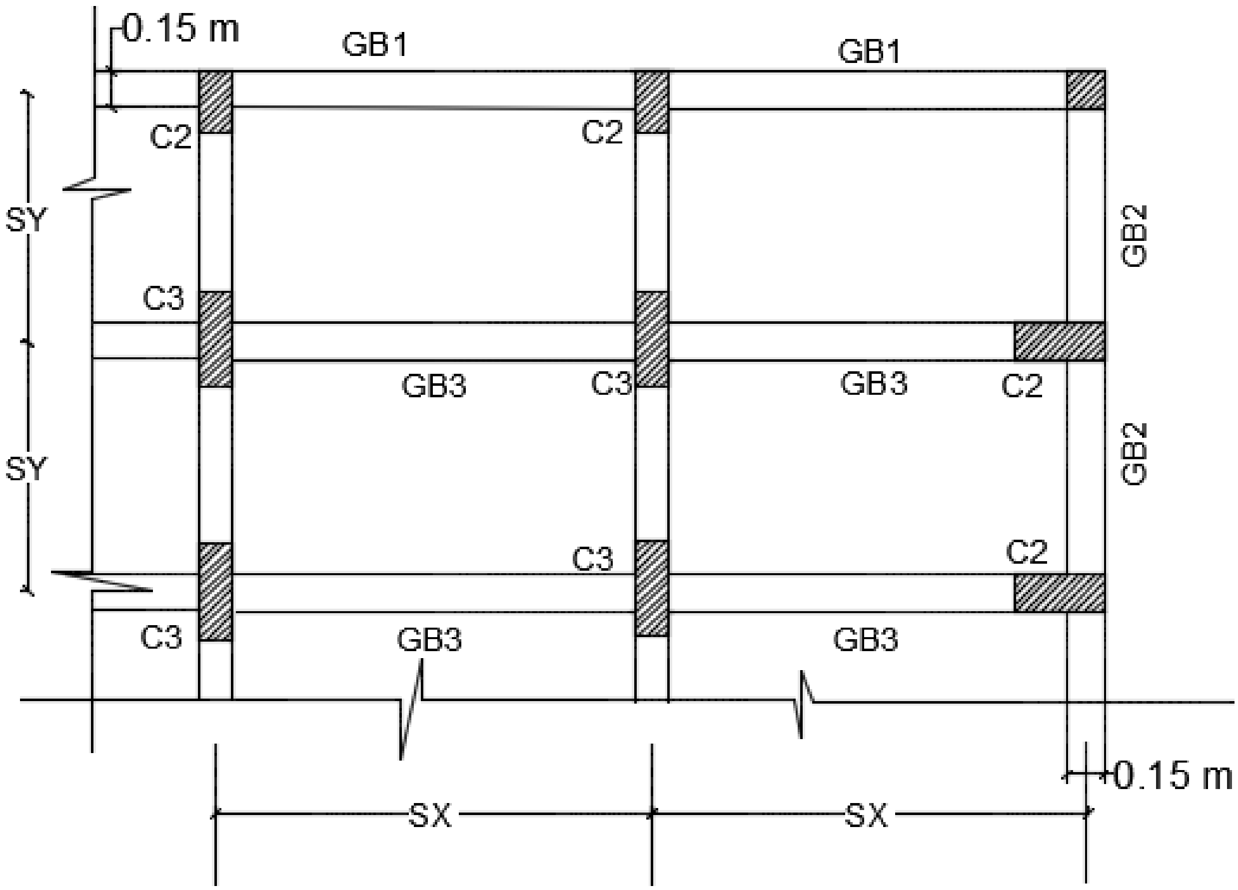


Fig (1) Plan Of administrative building

3. Requirements

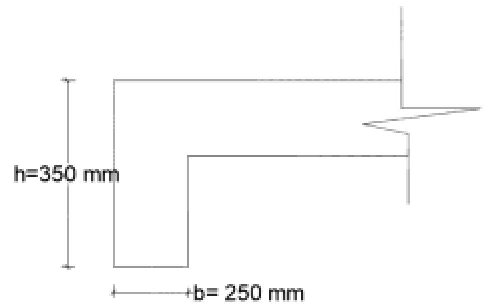
1. Calculation of column loads (working and ultimate).
2. Determination of the allowable soil bearing capacity at different depths.
3. Design of isolated footings under columns (C1, C2, and C3).
4. Calculation of footing settlements and check for serviceability.
5. Structural design of reinforcement for the footings.
6. Preparation of foundation layout and reinforcement detailing (plan and section).
7. Presentation of results in a technical report and PowerPoint presentation.

4. Analysis and design

Assume:

Beam dimensions = $b = 250\ mm,\ h = 350\ mm$

For edge beams:



$$I_e = \frac{1}{12} \times b \times h^3 \times 1.5$$
$$I_e = \frac{1}{12} \times 250 \times (350)^3 \times 1.5$$
$$I_e = 1.34 \times 10^9\ mm^4$$

For interior beams:

$$I_i = \frac{1}{12} \times b \times h^3 \times 2$$
$$I_i = \frac{1}{12} \times 250 \times (350)^3 \times 2$$
$$I_i = 1.79 \times 10^9\ mm^4$$

Assume:

- Slab thickness $h = 180\ mm$
- Cover = $35\ mm$
- Span (long) = $5.0\ m$, Span (short) = $3.5\ m$
- Beam dimensions = $b = 250\ mm, h = 350\ mm$
- Steel yield stress $f_y = 420\ MPa$

1. Moment of Inertia of Slab (Is):

For long span (5.0 m):

$$I_{s,long} = \frac{b \times h^3}{12} = \frac{5000 \times 180^3}{12} = 2.43 \times 10^9 \; mm^4$$

For short span (3.5 m):

$$I_{s,short} = \frac{3500 \times 180^3}{12} = 1.70 \times 10^9 \; mm^4$$

2. Moment of Inertia of Beams (Ib):

For edge beams:

$$I_e = \frac{1}{12} \times b \times h^3 \times 1.5 = 1.34 \times 10^9 \; mm^4$$

For interior beams:

$$I_i = \frac{1}{12} \times b \times h^3 \times 2 = 1.79 \times 10^9 \; mm^4$$

3. Stiffness Ratio (α):

For long span:

$$\alpha_{long} = \frac{I_b}{I_{s,long}} = \frac{1.79 \times 10^9}{2.43 \times 10^9} = 0.74$$

For short span:

$$\alpha_{short} = \frac{I_b}{I_{s,short}} = \frac{1.79 \times 10^9}{1.70 \times 10^9} = 1.05$$

Average value:

$$\alpha_{fn} \approx 0.9 \; (< 2) \Rightarrow \text{One-way slab}$$

4. Span Ratio (β):

$$\beta = \frac{L_{long}}{L_{short}} = \frac{5.0}{3.5} = 1.43$$

5. Minimum Thickness from Code:

Effective span:

$$L_n = 5000 - 2 \times 35 = 4930\,mm$$

Formula:

$$h = \frac{L_n \times (0.8 + \frac{f_y}{1400})}{36 + 9\beta}$$
$$h = \frac{4930 \times (0.8 + 0.3)}{36 + 12.87} = \frac{4930 \times 1.1}{48.87}$$
$$h \approx 111\,mm$$

Required thickness = **111 mm**

Assumed thickness = **180 mm**

the assumed thickness is Ok.

Beam Design Calculation

Materials & factors

- Concrete $f_c' = 25$MPa, steel $f_y = 420$MPa
- Strength reduction factors: $\phi_{flex} = 0.90$, $\phi_{shear} = 0.75$
- Slab thickness $h_s = 180$mm → slab self-weight $= 0.18 \times 25 = 4.5$kN/m²
- Member width for design strip $b = 1000$mm (for As calculations in beams we use actual beam width)
- Beam widths $b = 250$mm

- For $L = 5.0$m: total depth $h = 500$mm → effective depth $d \approx 457$mm
- For $L = 3.5$m: total depth $h = 450$mm → effective depth $d \approx 407$mm

- Concrete unit weight = 24kN/m³
- Tributary widths (from your plan):
  - 5 m beam: interior = 3.5 m, edge = 1.75 m
  - 3.5 m beam: interior = 5.0 m, edge = 2.5 m

Line loads

$$\begin{aligned} w_{DL} &= (12.0\,\text{kN/m}^2) \times (\text{trib. width}) + w_{\text{beam}} \\ w_{LL} &= (5.0\,\text{kN/m}^2) \times (\text{trib. width}) \\ w_u &= 1.2\,w_{DL} + 1.6\,w_{LL} \end{aligned}$$

Beam self-weight w_{beam}:

- $0.25 \times 0.50 \times 24 = 3.0$kN/m (for $h = 500$)
- $0.25 \times 0.45 \times 24 = 2.7$kN/m (for $h = 450$)

A) 5.0 m Interior Beam (trib. width = 3.5 m, $h = 500$→ $d \approx 457$mm)

Loads

$$\begin{aligned} w_{DL} &= 12.0 \times 3.5 + 3.0 = 42.0 + 3.0 = 45.0\text{ kN/m} \\ w_{LL} &= 5.0 \times 3.5 = 17.5\text{ kN/m} \\ w_u &= 1.2(45.0) + 1.6(17.5) = 54.0 + 28.0 = 82.0\text{ kN/m} \end{aligned}$$

Bending moment

$$M_u = \frac{82.0 \times 5^2}{8} = \frac{82.0 \times 25}{8} = 256.25\text{ kN.m}$$

Flexural steel

$$\frac{M_u}{\phi} = \frac{256.25 \times 10^6}{0.9} = 2.847 \times 10^8\text{ N\cdotpmm}$$

Solve $A_s \cdot 420(457 - \frac{A_s \cdot 420}{2 \cdot 0.85 \cdot 25 \cdot 250}) = 2.847 \times 10^8$.
This gives $A_s \approx 1{,}750\ \text{mm}^2$.

Provide: 4Ø25 ($4 \times 491 = 1964\ \text{mm}^2$) bottom at mid-span, and 4Ø25 top continuous over supports.

Shear

$$V_u = \frac{82.0 \times 5}{2} = 205\ \text{kN}$$

$$V_c = 0.17\sqrt{25}\,(250)(457) = 0.85 \times 114{,}250 = 97{,}113\ \text{N} = 97.1\ \text{kN}$$

$$\phi V_c = 0.75 \times 97.1 = 72.8\ \text{kN}$$

$$V_s \text{ needed} \geq \frac{V_u}{\phi} - V_c = \frac{205}{0.75} - 97.1 = 176.2\ \text{kN}$$

Stirrups capacity per spacing s: $V_s = \frac{A_v f_y d}{s} = \frac{100.5 \times 420 \times 457}{s} = \frac{19{,}290}{s}$ kN.

$$\frac{19{,}290}{s} \geq 176.2 \Rightarrow s \leq 110\ \text{mm}$$

Provide: Ø8 @ 100 mm within ≈$2d$ of supports, then Ø8 @ 150 mm toward mid-span .

<u>B) 5.0 m Edge Beam (trib. width = 1.75 m, $h = 500 \rightarrow d \approx 457$mm)</u>

Loads

$$w_{DL} = 12.0 \times 1.75 + 3.0 = 21.0 + 3.0 = 24.0\ \text{kN/m}$$

$$w_{LL} = 5.0 \times 1.75 = 8.75\ \text{kN/m}$$

$$w_u = 1.2(24.0) + 1.6(8.75) = 28.8 + 14.0 = 42.8\ \text{kN/m}$$

Bending moment

$$M_u = \frac{42.8 \times 25}{8} = 133.75\ \text{KN.m}$$

Flexural steel

$$\frac{M_u}{\phi} = \frac{133.75 \times 10^6}{0.9} = 1.486 \times 10^8\ \text{N\textbackslash cdotpmm}$$

Solve → $A_s \approx 835 \text{ mm}^2$.

Provide: 3Ø20 ($3 \times 314 = 942 \text{ mm}^2$) bottom mid-span, and 3Ø20 top continuous.

Shear

$$\begin{aligned} V_u &= \frac{42.8 \times 5}{2} = 107.0 \text{ kN} \\ \phi V_c &= 72.8 \text{ kN(same } d, b_w) \\ V_s \text{ needed} &\geq \frac{107.0}{0.75} - 97.1 = 45.6 \text{ kN} \\ \frac{19{,}290}{s} &\geq 45.6 \Rightarrow s \leq 423 \text{ mm} \end{aligned}$$

Code spacing limit near supports $\leq d/2 \approx 230\text{mm}$ →
Provide: Ø8 @ 200 mm near supports; may increase to Ø8 @ 300 mm at mid-span.

C) 3.5 m Interior Beam (trib. width = 5.0 m, $h = 450$→ $d \approx 407$mm)

Loads

$$\begin{aligned} w_{DL} &= 12.0 \times 5.0 + 2.7 = 60.0 + 2.7 = 62.7 \text{ kN/m} \\ w_{LL} &= 5.0 \times 5.0 = 25.0 \text{ kN/m} \\ w_u &= 1.2(62.7) + 1.6(25.0) = 75.24 + 40.0 = 115.24 \text{ kN/m} \end{aligned}$$

Bending moment

$$M_u = \frac{115.24 \times 3.5^2}{8} = \frac{115.24 \times 12.25}{8} = 176.46 \text{ kN} \cdot \text{m}$$

Flexural steel

$$\frac{M_u}{\phi} = \frac{176.46 \times 10^6}{0.9} = 1.961 \times 10^8 \text{ N\textbackslash cdotpmm}$$

Solve with $d = 407\text{mm}$ → $A_s \approx 1{,}314 \text{ mm}^2$.

Provide: 4Ø22 ($4 \times 380 = 1520 \text{ mm}^2$) bottom mid-span, and 4Ø22 top over supports.

Shear

$$V_u = \frac{115.24 \times 3.5}{2} = 201.7 \text{ kN}$$
$$V_c = 0.17\sqrt{25}\,(250)(407) = 0.85 \times 101{,}750 = 86{,}488 \text{ N} = 86.5 \text{ kN}$$
$$\phi V_c = 0.75 \times 86.5 = 64.9 \text{ kN}$$
$$V_s \text{ needed} \geq \frac{201.7}{0.75} - 86.5 = 182.4 \text{ kN}$$
$$\frac{A_v f_y d}{s} = \frac{100.5 \times 420 \times 407}{s} = \frac{17{,}179}{s} \text{ kN} \Rightarrow s \leq 94 \text{ mm}$$

Provide: Ø8 @ 90–100 mm within ≈$2d$ of supports, then Ø8 @ 150–200 mm toward mid-span.

D) 3.5 m Edge Beam (trib. width = 2.5 m, $h = 450$→ $d \approx 407$mm)

Loads

$$w_{DL} = 12.0 \times 2.5 + 2.7 = 30.0 + 2.7 = 32.7 \text{ kN/m}$$
$$w_{LL} = 5.0 \times 2.5 = 12.5 \text{ kN/m}$$
$$w_u = 1.2(32.7) + 1.6(12.5) = 39.24 + 20.0 = 59.24 \text{ kN/m}$$

Bending moment

$$M_u = \frac{59.24 \times 3.5^2}{8} = \frac{59.24 \times 12.25}{8} = 90.71 \text{ kN} \cdot \text{m}$$

Flexural steel

$$\frac{M_u}{\phi} = \frac{90.71 \times 10^6}{0.9} = 1.008 \times 10^8 \text{ N\textbackslash cdotpmm}$$

Solve with $d = 407$mm → $A_s \approx 628 \text{ mm}^2$.

Provide: 3Ø18 ($3 \times 254 = 762 \text{ mm}^2$) bottom mid-span, top over supports 3Ø18 continuous
(or bottom 2Ø20 = 628 mm² with heavier top steel at supports).

Shear

$$V_u = \frac{59.24 \times 3.5}{2} = 103.7 \text{ kN}$$
$$\phi V_c = 64.9 \text{ kN}$$
$$V_s \text{ needed} \geq \frac{103.7}{0.75} - 86.5 = 51.7 \text{ kN}$$
$$\frac{17{,}179}{s} \geq 51.7 \Rightarrow s \leq 332 \text{ mm}$$

Spacing limit near supports $\leq d/2 \approx 200$mm →
Provide: Ø8 @ 200 mm near supports, increase to Ø8 @ 300 mm toward mid-span.

Column Design

Given (from the project)

- Stories = 11; story height (assumed) = 3.0 m
- Loads per floor area: DL = 12.0 kN/m² (includes slab self-weight), LL = 5.0 kN/m²
- Grid spans: 5.0 m × 3.5 m → tributary areas:
  - Corner (C1) = $2.5 \times 1.75 = 4.375$m²/floor
  - Edge (C2) = 8.75m²/floor
  - Interior (C3) = 17.5m²/floor
- Concrete unit weight ≈ 25 kN/m³
- Concrete strength $f_c' = 25$MPa, steel $f_y = 420$MPa
- Strength reduction for compression (tied columns) $\phi = 0.65$
- Column sizes (Table 2 used throughout):
  - C1: 300×400mm → $A_g = 0.3 \times 0.4 = 0.12\text{m}^2 = 120{,}000\ \text{mm}^2$
  - C2: 300×600mm → $A_g = 180{,}000\ \text{mm}^2$
  - C3: 300×800mm → $A_g = 240{,}000\ \text{mm}^2$

Factored axial capacity model

$$\phi P_n = \phi[0.85 f_c'(A_g - A_{st}) + f_y A_{st}]\ \text{(N)}$$

Design condition: $\phi P_n \geq P_u$

Factored axial load per column
Separate DL and LL to apply load factors:

$$\begin{aligned} P_{DL} &= (\text{Trib.Area}) \times DL \times \text{stories} \\ P_{LL} &= (\text{Trib.Area}) \times LL \times \text{stories} \\ P_{u,\text{floor loads}} &= 1.2\, P_{DL} + 1.6\, P_{LL} \end{aligned}$$

Add column self-weight (factored):
$W_{col} = A_g\, h_{\text{story}}\, \gamma \times$ stories, then $1.2\, W_{col}$.

1) Corner Column (C1) — 300×400mm

Tributary area: $A_t = 4.375$m²/floor

Floor loads

$$P_{DL} = 4.375 \times 12.0 \times 11 = 577.5 \text{ kN}$$
$$P_{LL} = 4.375 \times 5.0 \times 11 = 240.625 \text{ kN}$$
$$P_{u,\text{floor}} = 1.2(577.5) + 1.6(240.625) = 1078.0 \text{ kN}$$

Column self-weight
Area $= 0.12$ m², height per story = 3.0m, $\gamma = 25$ kN/m³

$$W_{col} = 0.12 \times 3.0 \times 25 \times 11 = 99.0 \text{ kN}, 1.2W_{col} = 118.8 \text{ kN}$$

Total factored axial load

$$P_u = 1078.0 + 118.8 = 1196.8 \text{ kN}$$

Capacity & required steel
$A_g = 120{,}000$ mm². Solve for A_{st} in

$$\phi[0.85(25)(A_g - A_{st}) + 420\, A_{st}] \ \geq \ 1196.8 \times 10^3$$

→ The equation gives $A_{st,\text{req}} < 0$(concrete alone suffices), so min steel governs.

Minimum steel (tied column): $A_{st,\min} = 1\%\ A_g = 1200$ mm²

Provide (example): 4Ø20 ($4 \times 314 = 1256$ mm²)
Ties: Ø10 closed ties @ 150 mm (max per code limits)

2) Edge Column (C2) — 300 × 600mm

Tributary area: $A_t = 8.75$m²/floor

Floor loads

$$P_{DL} = 8.75 \times 12.0 \times 11 = 1155.0 \text{ kN}$$
$$P_{LL} = 8.75 \times 5.0 \times 11 = 481.25 \text{ kN}$$
$$P_{u,\text{floor}} = 1.2(1155.0) + 1.6(481.25) = 2156.0 \text{ kN}$$

Column self-weight
Area $= 0.18\ \text{m}^2$

$$W_{col} = 0.18 \times 3.0 \times 25 \times 11 = 148.5\ \text{kN}, 1.2W_{col} = 178.2\ \text{kN}$$

Total factored axial load

$$P_u = 2156.0 + 178.2 = 2334.2\ \text{kN}$$

Capacity & required steel
$A_g = 180{,}000\ \text{mm}^2$:

$$\phi[0.85(25)(A_g - A_{st}) + 420\ A_{st}]\ \geq\ 2334.2 \times 10^3$$

→ Solution gives $A_{st,\text{req}} < 0$→ min steel governs.

Minimum steel: $A_{st,min} = 1\%$

$$A_g = 1800\ \text{m}^2$$

Provide (example): 6Ø20 ($6 \times 314 = 1884\ \text{mm}^2$)
Ties: Ø10 @ 150 mm (reduce to 100 mm near beam-column joints if required by detailing rules)

3) Interior Column (C3) — 300×800mm

Tributary area: $A_t = 17.5$m²/floor

Floor loads

$$\begin{aligned} P_{DL} &= 17.5 \times 12.0 \times 11 = 2310.0\ \text{kN} \\ P_{LL} &= 17.5 \times 5.0 \times 11 = 962.5\ \text{kN} \\ P_{u,\text{floor}} &= 1.2(2310.0) + 1.6(962.5) = 4312.0\ \text{kN} \end{aligned}$$

Column self-weight
Area $= 0.24\ \text{m}^2$

$$W_{col} = 0.24 \times 3.0 \times 25 \times 11 = 198.0\ \text{kN}, 1.2W_{col} = 237.6\ \text{kN}$$

Total factored axial load

$$P_u = 4312.0 + 237.6 = 4549.6 \text{ kN}$$

Required steel area
$A_g = 240{,}000 \text{ mm}^2$.

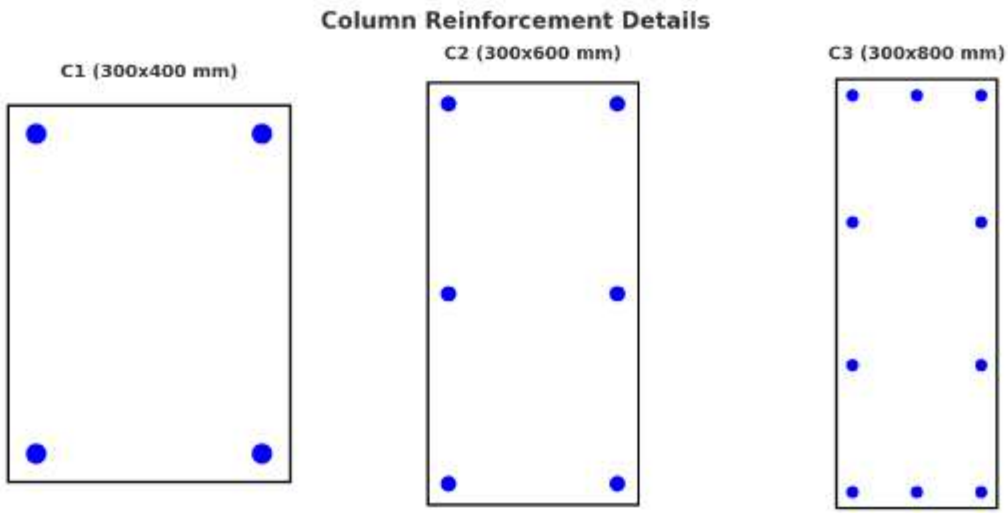
Solve

$$\phi[0.85(25)(240000 - A_{st}) + 420\, A_{st}] \ \geq\ 4549.6 \times 10^3$$
$$0.65[21.25(240000 - A_{st}) + 420 A_{st}] \ \geq\ 4.5496 \times 10^6 \text{ N}$$
$$\Rightarrow A_{st,\text{req}} \approx 4763 \text{ mm}^2$$

- 10Ø25 → $10 \times 491 = 4910 \text{ m}^2$
- (meets > 4763)
- (or 12Ø22 = 4560 mm² , insufficient; 12Ø25 = 5892 mm² ,but heavier)

Ties: Ø10 closed ties @ 150 mm (may reduce to 100 mm within the joint regions).
Cover: ≥ 40 mm at edges exposed to weather; ≥ 25–30 mm for interior exposure (use your course requirement).

C1 (300×400): longitudinal **4Ø20**, ties Ø10 @ 150 mm (100 mm at joints).
C2 (300×600): longitudinal **6Ø20**, ties Ø10 @ 150 mm (100 mm at joints).
C3 (300×800): longitudinal **10Ø25**, ties Ø10 @ 150 mm (100 mm at joints).



Slab Design

From the building plan (Group 2), the floor system is divided by beams into 3 spans in X-direction and 3 spans in Y-direction, so we have:

$$\mathbf{3 \times 3 = 9\ \boldsymbol{panels}}$$

Each panel must be checked separately using the Moment Coefficient Method.

<u>Moment Coefficient Method for Two-Way Slab</u>

Given data

- Short span: $L_x = 3.5\ m$

- Long span: $L_y = 5.0\ m$
- Dead + Live load = 12.5 kN/m^2
- Self-weight of slab:

$$w_{self} = 0.18 \times 25 = 4.5\ kN/m^2$$

- Total load:

$$w = 12.5 + 4.5 = 17.0\ kN/m^2$$

Step 1. Span ratio

$$\frac{L_x}{L_y} = \frac{3.5}{5.0} = 0.70$$
$$\frac{L_y}{L_x} = \frac{5.0}{3.5} = 1.43$$

Step 2. Moment coefficients

For interior panel, continuous slab:

- Short span:
  - Negative coefficient $C_{x,neg} = 0.086$
  - Positive coefficient $C_{x,pos} = 0.059$
- Long span:
  - Negative coefficient $C_{y,neg} = 0.065$
  - Positive coefficient $C_{y,pos} = 0.046$

Step 3. Design moments

Formula:

$$M = C \times w \times L^2$$

(a) Short span ($L_x = 3.5\ m$)

- Negative moment:

$$M_{x,neg} = 0.086 \times 17.0 \times (3.5^2) = 17.9\, kN \cdot m/m$$

- Positive moment:

$$M_{x,pos} = 0.059 \times 17.0 \times (3.5^2) = 12.3\, kN \cdot m/m$$

(b) Long span ($L_y = 5.0\ m$)

- Negative moment:

$$M_{y,neg} = 0.065 \times 17.0 \times (5.0^2) = 27.6\, kN \cdot m/m$$

- Positive moment:

$$M_{y,pos} = 0.046 \times 17.0 \times (5.0^2) = 19.6\, kN \cdot m/m$$

Panel (1)

- **Short span (3.5 m):**
  - Negative moment = 17.9 kN·m/m
  - Positive moment = 12.3 kN·m/m
- **Long span (5.0 m):**
  - Negative moment = 27.6 kN·m/m
  - Positive moment = 19.6 kN·m/m

Panel 2

- $L_x = 3.5\, m,\ L_y = 3.5\, m$
- Ratio = 1.00
- Coefficients: short span (neg = 0.075, pos = 0.056), long span (neg = 0.075, pos = 0.056)
- $M_{x,neg} = 15.6\, kN \cdot m/m,\ M_{x,pos} = 11.6\, kN \cdot m/m$

- $M_{y,neg} = 15.6\, kN \cdot m/m$, $M_{y,pos} = 11.6\, kN \cdot m/m$

Panel 3

- $L_x = 3.5\, m$, $L_y = 5.0\, m$
- Ratio = 0.70 (same as Panel 1)
- $M_{x,neg} = 17.9\, kN \cdot m/m$, $M_{x,pos} = 12.3\, kN \cdot m/m$
- $M_{y,neg} = 27.6\, kN \cdot m/m$, $M_{y,pos} = 19.6\, kN \cdot m/m$

Panel 4

- $L_x = 5.0\, m$, $L_y = 3.5\, m$
- Ratio = 1.43
- Coefficients: short span (neg = 0.090, pos = 0.065), long span (neg = 0.045, pos = 0.033)
- $M_{x,neg} = 38.3\, kN \cdot m/m$, $M_{x,pos} = 27.6\, kN \cdot m/m$
- $M_{y,neg} = 13.4\, kN \cdot m/m$, $M_{y,pos} = 9.8\, kN \cdot m/m$

Panel 5

- $L_x = 5.0\, m$, $L_y = 5.0\, m$
- Ratio = 1.00
- Coefficients: short span (neg = 0.075, pos = 0.056), long span (neg = 0.075, pos = 0.056)
- $M_{x,neg} = 31.9\, kN \cdot m/m$, $M_{x,pos} = 23.8\, kN \cdot m/m$
- $M_{y,neg} = 31.9\, kN \cdot m/m$, $M_{y,pos} = 23.8\, kN \cdot m/m$

Panel 6

- $L_x = 5.0\, m$, $L_y = 3.5\, m$

- Ratio = 1.43 (same as Panel 4)
- $M_{x,neg} = 38.3\ kN \cdot m/m$, $M_{x,pos} = 27.6\ kN \cdot m/m$
- $M_{y,neg} = 13.4\ kN \cdot m/m$, $M_{y,pos} = 9.8\ kN \cdot m/m$

Panel 7

- $L_x = 3.5\ m$, $L_y = 5.0\ m$
- Ratio = 0.70 (same as Panel 1)
- $M_{x,neg} = 17.9\ kN \cdot m/m$, $M_{x,pos} = 12.3\ kN \cdot m/m$
- $M_{y,neg} = 27.6\ kN \cdot m/m$, $M_{y,pos} = 19.6\ kN \cdot m/m$

Panel 8

- $L_x = 3.5\ m$, $L_y = 3.5\ m$
- Ratio = 1.00 (same as Panel 2)
- $M_{x,neg} = 15.6\ kN \cdot m/m$, $M_{x,pos} = 11.6\ kN \cdot m/m$
- $M_{y,neg} = 15.6\ kN \cdot m/m$, $M_{y,pos} = 11.6\ kN \cdot m/m$

Panel 9

- $L_x = 3.5\ m$, $L_y = 5.0\ m$
- Ratio = 0.70 (same as Panel 1)
- $M_{x,neg} = 17.9\ kN \cdot m/m$, $M_{x,pos} = 12.3\ kN \cdot m/m$
- $M_{y,neg} = 27.6\ kN \cdot m/m$, $M_{y,pos} = 19.6\ kN \cdot m/m$

Slab reinforcement design (per 1-m strip)

- Concrete $f_c' = 25$MPa, steel $f_y = 420$MPa, $\phi = 0.9$
- Slab thickness $h = 180$mm → effective depth $d \approx h - \text{cover} - \phi/2 = 180 - 25 - 5 = 150$mm
- Strip width $b = 1000$mm
- Minimum steel for slabs (ACI for $f_y = 420$): $A_{s,\min} = 0.0018\, b\, h = 0.0018(1000)(180) = 324\ \text{mm}^2$

Design moments

- Short span (x): $M_{x,neg} = 17.9$, $M_{x,pos} = 12.3$kN·m/m
- Long span (y): $M_{y,neg} = 27.6$, $M_{y,pos} = 19.6$kN·m/m

1) Required steel areas

Use

$$R_n = \frac{M_u}{\phi\, b\, d^2}, \rho = \frac{0.85 f_c'}{f_y}(1 - \sqrt{1 - \frac{2R_n}{0.85 f_c'}}), A_s = \rho\, b\, d$$

- Short span – top (negative):

  $A_s = 323\ \text{mm}^2$ ⇒governs by $A_{s,\min} = 324\, mm^2$

- Short span – bottom (positive):

  $A_s = 220\, mm^2$ ⇒governs by $A_{s,\min} = 324\, mm^2$

- Long span – top (negative):

$$A_s = 503\ \text{mm}^2$$

- Long span – bottom (positive):

$$A_s = 354\ \text{mm}^2$$

2) Provide bars & spacing (use Ø10 or Ø12)

(Bar areas: Ø10 → 78.5 mm², Ø12 → 113 mm²)

- **Short span (x-direction)**
  - Top over supports (negative): need ≥324 → Ø10 @ 200 mm (provides 392 mm² ≥ 324)
  - Bottom at mid-span (positive): need ≥324 → Ø10 @ 200–225 mm (at 200 mm provides 392 mm²; at 225 mm provides 349 mm²)
- **Long span (y-direction)**
  - Top over supports (negative): need 503 → Ø10 @ 150 mm (≈ 523 mm²) *or* Ø12 @ 200 mm (≈ 565 mm²)

- Bottom at mid-span (positive): need 354 → Ø10 @ 200 mm (≈ 392 mm²) *or* Ø12 @ 300 mm (≈ 377 mm²)

Spacing limits are satisfied (typical max for slab main steel ≈ $3h = 540$mm; you're below that).

Isolated Footing Design

Step 1. Column loads (factored)

(from previous column design):

- Corner column (C1): $P_u = 1197$ kN
- Edge column (C2): $P_u = 2334$ kN
- Interior column (C3): $P_u = 4550$ kN

Step 2. Soil bearing capacity

Assume allowable soil pressure $q_{allow} = 250$ kN/m^2
(use actual geotechnical data if given).

Service load is found by dividing factored load by 1.5:

$$P_{serv} = \frac{P_u}{1.5}$$

Step 3. Required footing area

$$A_{req} = \frac{P_{serv}}{q_{allow}}$$

- C1: $P_{serv} = 1197/1.5 = 798\ kN$→ $A = 798/250 = 3.19\ m^2$→ use 1.8 × 1.8 m footing

- C2: $P_{serv} = 2334/1.5 = 1556\ kN$→ $A = 1556/250 = 6.22\ m^2$→ use 2.5 × 2.5 m footing
- **C3:** $P_{serv} = 4550/1.5 = 3033\ kN$→ $A = 3033/250 = 12.13\ m^2$→ use **3.5 × 3.5 m** footing

Step 4. Thickness of footing

Check for one-way shear and punching shear.

- Assume trial depth $h = 800\ mm$for C3 (largest).
- Effective depth $d = h - cover - \phi/2 \approx 750\ mm.$

Punching shear check (C3):
Critical section at $d/2$from column face.

$$V_u = \frac{P_u}{Area_{footing}} = \frac{4550}{12.25} = 371\ kN/m^2$$

Shear perimeter: $u_0 = 2(c_x + c_y + 2d)$
(Here $c_x = 0.3, c_y = 0.8$m column dims).
You check:

$$\phi V_c = 0.75 \times 0.17\sqrt{f_c'}\, u_0\, d$$

If $\phi V_c > V_u \times (footing\ area)$

, thickness is safe.

(Detailed substitution can be written if needed.)

Step 5. Flexural reinforcement

Maximum moment at face of column:

$$M = q_{net}\frac{L^2}{8}$$

where $q_{net} = P_{serv}/A$.

- For C3 footing:
  $q_{net} = 3033/12.25 = 248\ kN/m^2$.
  Span = half footing size = 1.75 m.

$$M = 248 \times 1.75^2/2 \approx 380\ kN \cdot m/m$$

Design steel area:

$$A_s = \frac{M}{\phi f_y j d}$$

(j ≈ 0.9 d).

You then provide bars in both directions, usually same size and spacing (since footing is square).

Step 6. reinforcement

- C1 footing (1.8 × 1.8 × 0.6 m): use Ø16 @ 200 mm each way bottom.
- C2 footing (2.5 × 2.5 × 0.7 m): use Ø18 @ 200 mm each way bottom.
- C3 footing (3.5 × 3.5 × 0.8 m): use Ø20 @ 200 mm each way bottom.

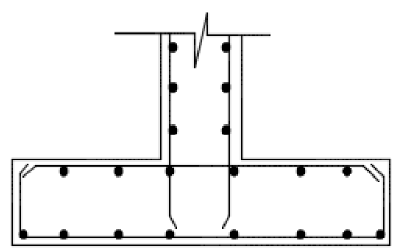


Fig (4) **Footing RFT Details**
3.5*3.5*0.8

5. References

1. Saudi Building Code National Committee. (2018). Saudi Building Code (SBC 304) – Concrete Structures.