DESIGN OF FOUNDATIONS

Dr. Izni Syahrizal bin Ibrahim
Faculty of Civil Engineering
Universiti Teknologi Malaysia
Email: iznisyahrizal@utm.my
• **Foundation** – Part of structure which **transmits load** from the structure to the underlying soil or rock
• All soils compress noticeably when loaded causing structure to settle
• Requirements in the design of foundations:

(i) Total settlement of the structure to be limited to a tolerably small amount
(ii) Differential settlement of various parts of structure shall be eliminated
• To limit settlement, it is necessary to transmit the structure load to a soil stratum of sufficient strength.
• Spread the structure load over a sufficiently large area of stratum to minimize bearing pressure.

• **Satisfactory soil:** Use footings
• **Adequate soil:** Use deep foundations i.e. piles
• Pressure distribution under a footing

- Uniform distributed
- Cohesive soil
- Cohesionless soil
Types of Foundation

Pad Footings

- Transmit load from piers and columns
- Simplest and cheapest type
- Use when soil is relatively strong or when column loads are relatively light
- Normally square or rectangular shape in plan
- Has uniform thickness
**Types of Foundation**

**Combine Footings**

- Use when two columns are closed together
- Combine the footing to form a continuous base
- Base to be arranged so that its centreline coincides with the centre of gravity of the load – provide uniform pressure on the soil
Types of Foundation

Strap Footings

- Use where the base for an exterior column must not project beyond the property line
- Strap beam is constructed between exterior footing & adjacent interior footing
- Purpose of strap – to restrain overturning forces due to load eccentricity on the exterior footing
Types of Foundation

Strap Footings (continued)

- Base area of the footings are proportioned to the bearing pressure
- Resultant of the loads on the two footings should pass through the centroid of the area of the two bases
- Strap beam between the two footings should NOT bear against the soil
Types of Foundation

Strip Footings

- Use for foundations to load-bearing wall
- Also use when pad footings for number of columns are closely spaced
- Also use on weak ground to increase foundation bearing area
Types of Foundation

Raft Foundations

- Combine footing which covers the whole building
- Support all walls & columns
- Useful where column loads are heavy or bearing capacity is low – need large base
- Also used where soil mass contains compressible layers or soil is variable – differential settlement difficult to control
Types of Foundation

Pile Foundations

- More economic to be used when solid bearing stratum i.e. rock is deeper than about 3 m
- Pile loads can either be transmitted to a stiff bearing layer (some distance below surface) or by friction along the length of pile
- Pile types – precast (driven into the soil) or cast in-situ (bored)
- Soil survey is important to provide guide on the length of pile and safe load capacity of the pile
Types of Foundation

Pile Foundations

Load from Structure

Pile Cap

Lower Density

Medium Density

High Density
Design of Pad Footing

Thickness and Size of Footing

Area of pad:

\[ A = \frac{G_k + Q_k + W}{\text{Soil bearing capacity}} \]

Minimum effective depth of pad:

\[ d = \frac{N_{Ed}}{\nu_{rd,\text{max}} \cdot u_o} \]

- \( N_{Ed} = \) Ultimate vertical load = 1.35\( G_k \) + 1.5\( Q_k \)
- \( \nu_{rd,\text{max}} = 0.5 \nu f_{cd} = 0.5 \left[ 0.6 \left( 1 - \frac{f_{ck}}{250} \right) \right] \left( \frac{f_{ck}}{1.5} \right) \)
- \( u_o = \) Column perimeter
Design for Flexure

- Critical section for bending – At the face of the column
- Moment is taken on a section passing completely across the footing and due to ultimate load on one side of the section
- Moment & shear is assessed using **STR (Structure)** combination

**STR Combination 1:**

\[ N = 1.35G_k + 1.5Q_k \]
Design of Pad Footing

Check for Shear

- May fail in shear as vertical shear or punching shear

![Diagram showing vertical shear sections and punching shear perimeters]

Bends may be required

Vertical shear sections

Punching shear perimeters

$2d$

$d$

$h$

$d$
Check for Shear

(i) Vertical Shear

- Critical section at distance $d$ from the face of column
- Vertical shear force $= \sum$ Load acting outside the section
- If $V_{Ed} < V_{Rd,c}$ = No shear reinforcement is required
Check for Shear
(ii) **Punching Shear**

**Axial Force Only**

- Critical section at a perimeter $2d$ from the face of the column
- Punching shear force = $\sum$ Load outside the critical perimeter
- Shear stress, $v_{Ed} = \frac{V_{Ed}}{u.d}$ where $u$ = Critical perimeter
- If $v_{Ed} < v_{Rd,c}$ = No shear reinforcement is required
- Also ensure that $V_{Ed} < V_{Rd,max}$
Check for Shear
(ii) **Punching Shear (continued)**

**Axial Force & Bending Moment**

- Punching shear resistance can be significantly reduced of a co-existing bending, $M_{Ed}$
- However, adverse effect of the moment will give rise to a non-uniform shear distribution around the control perimeter
- Refer to Cl. 6.4.3(3) of EC2
Check for Shear

(ii) **Punching Shear (continued)**

Shear stress, \( v_{Ed} = \frac{\beta V_{Ed}}{u_1 \cdot d} \)

where;

\( \beta = \) factor used to include effect of eccentric load & bending moment =

\[ 1 + k \left( \frac{M_{Ed}}{V_{Ed}} \right) \left( \frac{u_1}{W_1} \right) \]

\( k = \) coefficient depending on the ratio between column dimension \( c_1 \) & \( c_2 \)

<table>
<thead>
<tr>
<th>( c_1/c_2 )</th>
<th>( \leq 0.5 )</th>
<th>1.0</th>
<th>2.0</th>
<th>( \geq 3.0 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( k )</td>
<td>0.45</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
</tr>
</tbody>
</table>

\( u_1 = \) length of basic control perimeter

\( W_1 = \) function of basic control perimeter corresponds to the distribution of shear = \( 0.5c_1^2 + c_1 c_2 + 4c_2 d + 16d^2 + 2\pi dc_1 \)
Check for Shear

(ii) **Punching Shear (continued)**

![Diagram of shear forces and punching shear in a pad footing.](image-url)
Cracking & Detailing Requirements

- All reinforcements should extend the full length of the footing.
- If $L_x > 1.5(c_x + 3d)$, at least two-thirds of the reinforcement parallel to $L_y$ should be concentrated in a band width $(c_x + 3d)$ centred at column where $L_x$ & $L_y$ and $c_x$ & $c_y$ are the footing and column dimension in $x$ and $y$ directions.
- Reinforcements should be anchored each side of all critical sections for bending. Usually possible to achieve using straight bar.
- Spacing between centre of reinforcements $< 20$ mm for $f_{yk} = 500$ N/mm$^2$.
- Reinforcements normally not provided in the side face nor in the top face (except for balanced & combined foundation).
- Starter bar should terminate in a 90° bend tied to the bottom reinforcement, or in the case of unrefined footing spaced 75 mm off the building.
Example 1

PAD FOOTING

(AXIAL LOAD ONLY)
Example 1: Pad Footing (Axial Load)

Axial Force, $N$:

\[ G_k = 600 \text{kN} \]
\[ Q_k = 400 \text{kN} \]

Column size: 300 $\times$ 300 mm

- $f_{ck} = 25 \text{ N/mm}^2$
- $f_{yk} = 500 \text{ N/mm}^2$
- $\gamma_{soil} = 150 \text{ N/mm}^2$
- Unit weight of concrete = 25 kN/m$^3$
- Design life = 50 years
- Exposure Class = XC2
- Assumed $\phi_{bar} = 12 \text{ mm}$
Durability & Bond Requirements

Min cover regards to bond, $c_{min,b} = 12$ mm
Min cover regards to durability, $c_{min,dur} = 25$ mm
Allowance in design for deviation, $\Delta c_{dev} = 10$ mm

Nominal cover, $c_{nom} = c_{min} + \Delta c_{dev} = 25 + 10 = 35$ mm

$\therefore c_{nom} = 35$ mm
Example 1: Pad Footing (Axial Load)

Size

Service load, \( N \) = 1000 kN
Assumed selfweight 10% of service load, \( W \) = 100 kN

Area of footing required = \( \frac{(N+W)}{\gamma_{soil}} = \frac{(1000+100)}{150} = 7.33 \text{ m}^2 \)

\[ \therefore \text{ Try footing size, } B \times H \times h = 3 \text{ m} \times 3 \text{ m} \times 0.45 \text{ m} \]

Area, \( A = 9 \text{ m}^2 \)
Selfweight, \( W = 9 \times 0.45 \times 25 = 101 \text{ kN} \)

Check Service Soil Bearing Capacity = \( \frac{(N+W)}{A} = \frac{(1000+101)}{9} \)
= 122 \text{ kN/m}^2 \leq 150 \text{ kN/m}^2 \\Rightarrow \text{ OK} \]
Example 1: Pad Footing (Axial Load)

**Analysis**

Ultimate axial force, $N_{Ed}$

\[ N_{Ed} = 1.35G_k + 1.5Q_k \]
\[ = 1.35 \times 600 + 1.5 \times 400 = 1410 \text{ kN} \]

Soil pressure at ultimate load, $P = \frac{N_{Ed}}{A} = \frac{1410}{9} = 157 \text{ kN/m}^2$

Soil pressure per m length, $w = 157 \times 3 \text{ m} = 470 \text{ kN/m}$

![Diagram of pad footing](Diagram)
Example 1: Pad Footing (Axial Load)

Analysis

Ultimate axial force, $N_{Ed}$

$$N_{Ed} = 1.35G_k + 1.5Q_k$$

$$= 1.35 (600) + 1.5 (400) = 1410 \text{ kN}$$

Soil pressure at ultimate load, $P = \frac{N_{Ed}}{A} = \frac{1410}{9} = 157 \text{ kN/m}^2$

Soil pressure per m length, $w = 157 \times 3 \text{ m} = 470 \text{ kN/m}$

$$M_{Ed} = 470 \times 1.35 \times \frac{1.35}{2}$$

$$= 428 \text{ kNm}$$
**Example 1: Pad Footing (Axial Load)**

### Main Reinforcement

Effective depth, \( d = h - c - 1.5\phi_{bar} = 450 - 35 - (1.5 \times 12) = 397 \text{ mm} \)

\[
K = \frac{M_{Ed}}{f_{ck}bd^2} = \frac{428 \times 10^6}{25 \times 3000 \times 397^2} = 0.036 < K_{bal} = 0.167
\]

∴ Compression reinforcement is **NOT** required

\[
z = d \left[ 0.25 - \left( \frac{K}{1.134} \right) \right] = 0.97d > 0.95d
\]

\[
A_{s,req} = \frac{M_{Ed}}{0.87f_{yk}z} = \frac{428 \times 10^6}{0.87 \times 500 \times 0.95 \times 397} = 2611 \text{ mm}^2
\]
Example 1: Pad Footing (Axial Load)

Minimum & Maximum Area of Reinforcement

\[ A_{s,\text{min}} = 0.26 \left( \frac{f_{ctm}}{f_y} \right) bd = 0.26 \left( \frac{2.56}{500} \right) 0.0013bd \geq 0.0013bd \]

\[ \therefore A_{s,\text{min}} = 0.0013bd = 0.0013 \times 3000 \times 397 = 1589 \text{ mm}^2 \]

\[ A_{s,\text{max}} = 0.04A_c = 0.04bh = 0.04 \times 3000 \times 397 = 54000 \text{ mm}^2 \]

Provide 24H12 (\( A_{s,\text{prov}} = 2715 \text{ mm}^2 \))
Example 1: Pad Footing (Axial Load)

(i) Vertical Shear

Critical shear at **1.0d** from face of column:

\[ V_{Ed} = 470 \times 0.953 = 448 \text{ kN} \]
Example 1: Pad Footing (Axial Load)

(i) Vertical Shear

\[ k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{397}} = 1.71 < 2.0 \]

**Note:**

Bar extend beyond critical section at \( l_{bd} = 953 - 35 = 918 \) mm

\( (l_{bd} + d) = 40\Ø + d = (40 \times 12) + 397 = 877 \) mm

\( \therefore A_{sl} = 2715 \text{ mm}^2 \)

\[ \rho_l = \frac{A_{sl}}{bd} = \frac{2715}{3000 \times 397} = 0.0023 \leq 0.02 \]
(i) Vertical Shear

\[
V_{Rd,c} = \left[0.12k(100\rho_l f_{ck})^{1/3}\right]bd \\
= [0.12 \times 1.71(100 \times 0.0023 \times 25)^{1/3}]3000 \times 397 = 436463 \text{ N} = 436 \text{ kN}
\]

\[
V_{\text{min}} = \left[0.035k^{3/2}\sqrt{f_{ck}}\right]bd \\
= [0.035 \times 1.71^{3/2}\sqrt{25}]3000 \times 397 = 465970 \text{ N} = 466 \text{ kN}
\]

\[V_{Ed} \text{ (448 kN)} < V_{\text{min}} \text{ (466 kN)} \quad \Rightarrow \quad \text{OK}\]
(ii) Punching Shear

Critical shear at \(2.0d\) from face of column:

Average \(d = 450 - 35 - 12 = 403\) mm

\(\therefore 2d = 806\) mm

Control perimeter, \(u = (4 \times 300) + (2\pi \times 806) = 6265\) mm

Area within perimeter, \(A = (0.30 \times 0.30) + (4 \times 0.30 \times 0.806) + (\pi \times 0.806^2) = 3.10\) m\(^2\)

< \((l_{bd} + d) = 40\varnothing + d = (40 \times 12) + 397 = 877\) mm

\(\therefore\) Reinforcement NOT contributed to punching resistance
Example 1: Pad Footing (Axial Load)

(ii) Punching Shear

Punching shear force:
\[ V_{Ed} = 157 \times (3^2 - 3.10) = 925 \text{ kN} \]

Punching shear resistance:
\[ V_{Rd,c} = V_{min} = \left[ 0.035 k^{3/2} f_{ck}^{1/2} \right] ud \]
\[ = \left[ 0.035 (1.71)^{3/2} (25)^{1/2} \right] (6265 \times 403) \]
\[ = 983199 \text{ N} = 983 \text{ kN} > V_{Ed} (925 \text{ kN}) \]

Soil pressure = 157 kN/m²
(iii) Maximum Punching Shear at Column Perimeter

Maximum punching shear force:

\[ V_{Ed,\text{max}} = 157 \times (3^2 - 0.09) = 1400 \text{ kN} \]

Maximum shear resistance:

\[ V_{Rd,\text{max}} = 0.5ud \left[ 0.6 \left( 1 - \frac{f_{ck}}{250} \right) \right] \left( \frac{f_{ck}}{1.5} \right) \]

\[ = 0.5(4 \times 300) \times 403 \left[ 0.6 \left( 1 - \frac{25}{250} \right) \right] \left( \frac{25}{1.5} \right) \]

\[ = 2176 \text{ kN} > V_{Ed,\text{max}} \]

\[ \Rightarrow \text{OK} \]

Soil pressure = 157 kN/m²
**Example 1: Pad Footing (Axial Load)**

### Cracking

**Cracking**

\( h = 450 \text{ mm} > 200 \text{ mm} \)

Steel stress, \( f_S = \left[ \frac{(G_k+0.3Q_k)}{(1.35G_k+1.5Q_k)} \right] \left( \frac{A_{s,req}}{A_{s,prov}} \right) \left( \frac{f_{yk}}{1.15} \right) \)

\[
= \left[ \frac{(600+0.3\times400)}{(1.35\times600+1.5\times400)} \right] \left( \frac{2611}{2715} \right) \left( \frac{500}{1.15} \right) = 213 \text{ N/mm}^2
\]

For design crack width 0.3 mm:

**Maximum allowable bar spacing = 200 mm**

**Actual bar spacing = \( \frac{[3000-2(35)-12]}{23} \) = 127 mm < 200 mm**

Cracking OK
Example 1: Pad Footing (Axial Load)

**Detailing**

**Plan View**

**Section View**

Dimensions:
- 3000
- 24H12
- 450
- 3000
- 24H12

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Example 2

PAD FOOTING

(AXIAL LOAD & MOMENT)
Example 2: Pad Footing (Axial Load & Moment)

Axial Force, $N = 1500$ kN
Moment = 50 kNm

- Design Life = 50 years (Table 2.1: EN 1990)
- Exposure Class = XC3
- $f_{ck} = 30$ N/mm$^2$
- $f_{yk} = 500$ N/mm$^2$
- $\gamma_{soil} = 150$ N/mm$^2$
- Unit weight of concrete = 25 kN/m$^3$
- Assumed $\phi_{bar} = 12$ mm

Column size: 250 x 350 mm
Durability & Bond Requirements

Min cover regards to bond, $c_{min,b} = 12$ mm
Min cover regards to durability, $c_{min,dur} = 25$ mm
Allowance in design for deviation, $\Delta c_{dev} = 10$ mm

Nominal cover, $c_{nom} = c_{min} + \Delta c_{dev} = 25 + 10 = 35$ mm

$\therefore c_{nom} = 35$ mm
Example 2: Pad Footing (Axial Load & Moment)

Size

Service axial, $N$ = 1500 kN / 1.40 = 1071 kN
Service moment, $M$ = 50 kNm / 1.40 = 36.1 kNm
Assumed selfweight 10% of service load, $W$ = 100 kN

Area of footing required = $\frac{(N+W)}{\gamma_{soil}} = \frac{(1071+107.1)}{150} = 7.85 \ m^2$

\[ \therefore \ Try \ footing \ size, \ B \times H \times h = 2.80 \ m \times 3.50 \ m \times 0.65 \ m \]

Area, $A = 9.80 \ m^2$
Selfweight, $W = 9.80 \times 0.65 \times 25 = 159 \ kN$
Example 2: Pad Footing (Axial Load & Moment)

Size (Continued)

\[ I_{xx} = \frac{BH^3}{12} = \frac{2.8 \times 3.5^3}{12} = 10.0 \text{ m}^4 \]
\[ y = \frac{H}{2} = \frac{3.5}{2} = 1.75 \text{ m} \]

Maximum soil pressure,
\[ P = \frac{(N+W)}{A} + \frac{My}{I} = \frac{(1071+159)}{9.80} + \frac{50 \times 1.75}{10.0} \]
\[ = 132 \text{ kN/m}^2 \leq 150 \text{ kN/m}^2 \quad \Rightarrow \text{OK} \]
Example 2: Pad Footing (Axial Load & Moment)

Analysis

Ultimate soil pressure, \( P = \frac{N}{A} \pm \frac{My}{I} = \frac{1500}{9.80} \pm \frac{50 \times 1.75}{10.0} = 153 \pm 8.7 \text{ kN/m}^2 \)

\[ \therefore P_{\text{min}} = 144 \text{ kN/m}^2 \text{ and } P_{\text{max}} = 162 \text{ kN/m}^2 \]
Example 2: Pad Footing (Axial Load & Moment)

Analysis (Continued)

\[ M_{xx} = \left( 154 \times \frac{1.575^2}{2} \right) + \left[ (162 - 154) \left( \frac{1.575}{2} \right) \times \left( 1.575 \times \frac{2}{3} \right) \right] = 197 \text{ kNm/m} \times 2.80 \text{ m} = 553 \text{ kNm} \]

\[ M_{yy} = 153 \times \frac{1.275^2}{2} = 124 \text{ kNm/m} \times 3.50 \text{ m} = 435 \text{ kNm} \]
Effective Depth

\[ d_x = h - c - 0.5 \phi_{bar} = 650 - 35 - (0.5 \times 12) = 609 \text{ mm} \]
\[ d_y = h - c - 1.5 \phi_{bar} = 650 - 35 - (1.5 \times 12) = 597 \text{ mm} \]

Main Reinforcement – Longitudinal Bar

\[ K = \frac{M_{xx}}{f_{ck} b d^2} = \frac{553 \times 10^6}{30 \times 2800 \times 609^2} = 0.018 < K_{bal} = 0.167 \]
\[ \therefore \text{Compression reinforcement is NOT required} \]

\[ z = d \left[ 0.25 - \left( \frac{K}{1.134} \right) \right] = 0.98d > 0.95d \]

\[ A_{s,req} = \frac{M_{xx}}{0.87 f_{yk} z} = \frac{553 \times 10^6}{0.87 \times 500 \times 0.95 \times 609} = 2197 \text{ mm}^2 \]
Minimum & Maximum Area of Reinforcement

\[ A_{s,\text{min}} = 0.26 \left( \frac{f_{ctm}}{f_{yk}} \right) bd = 0.26 \left( \frac{2.90}{500} \right) 0.0013bd \geq 0.0013bd \]

\[ \therefore A_{s,\text{min}} = 0.0013bd = 0.0013 \times 2800 \times 609 = 2217 \text{ mm}^2 \]

\[ A_{s,\text{max}} = 0.04A_c = 0.04bh = 0.04 \times 2800 \times 609 = 72800 \text{ mm}^2 \]

Since \( A_s < A_{s,\text{min}} \), Use \( A_{s,\text{min}} = 2217 \text{ mm}^2 \)

Provide 21H12 (\( A_s = 2375 \text{ mm}^2 \))
Example 2: Pad Footing (Axial Load & Moment)

**Main Reinforcement – Transverse Bar**

\[ K = \frac{M_{yy}}{f_{ck}bd^2} = \frac{435 \times 10^6}{30 \times 3500 \times 597^2} = 0.018 < K_{bal} = 0.167 \]

\[ \therefore \text{Compression reinforcement is } \textbf{NOT} \text{ required} \]

\[ z = d \left[ 0.25 - \left( \frac{K}{1.134} \right) \right] = 0.99d > 0.95d \]

\[ A_{s,req} = \frac{M_{yy}}{0.87f_{yk}z} = \frac{435 \times 10^6}{0.87 \times 500 \times 0.95 \times 597} = 1765\text{mm}^2 \]
Example 2: Pad Footing (Axial Load & Moment)

Minimum & Maximum Area of Reinforcement

\[ A_{s,min} = 0.26 \left( \frac{f_{ctm}}{f_{yk}} \right) bd = 0.26 \left( \frac{2.90}{500} \right) 0.0013bd \geq 0.0013bd \]

\[ \therefore A_{s,min} = 0.0013bd = 0.0013 \times 3500 \times 597 = 3147 \text{ mm}^2 \]

\[ A_{s,max} = 0.04A_c = 0.04bh = 0.04 \times 3500 \times 597 = 91000 \text{ mm}^2 \]

Since \( A_s < A_{s,min} \), Use \( A_{s,min} = 3147 \text{ mm}^2 \)

**Provide 28H12 (A_s = 3167 mm^2)**
Example 2: Pad Footing (Axial Load & Moment)

(i) Vertical Shear

Critical shear at $1.0d$ from face of column:

Average pressure at critical section:

$$= 144 + \left(\frac{2.891}{3.50}\right) \times 18 = 159 \text{ kN/m}^2$$

∴ Design shear force, $V_{Ed} = 159 \times 0.966 \times 2.80 = 431 \text{ kN}$

Note:

Bar extend beyond critical section at $= 966 - 35 = 931 \text{ mm}$

$> (l_{bd} + d) = 36\varnothing + d = (36 \times 12) + 609 = 1041 \text{ mm}$

∴ $A_{sl} = 0 \text{ mm}^2$
(i) Vertical Shear

\[ k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{609}} = 1.57 < 2.0 \]

\[ \rho_l = \frac{A_{sl}}{bd} = 0 \]

\[ \therefore V_{Rd,c} = [0.12k(100\rho_l f_{ck})^{1/3}]bd \]
\[ = [0.12 \times 1.57(100 \times 0 \times 30)^{1/3}]2800 \times 609 = 0 \text{ N} = 0 \text{ kN} \]

\[ \therefore V_{min} = [0.035k^{3/2}\sqrt{f_{ck}}]bd \]
\[ = [0.035 \times 1.57^{3/2}\sqrt{30}]2800 \times 609 = 644949 \text{ N} = 645 \text{ kN} \]

\[ V_{Ed} (430 \text{ kN}) < V_{min} (645 \text{ kN}) \quad \Rightarrow \text{OK} \]
(ii) Punching Shear

Critical shear at $2.0d$ from face of column:

Average $d = \frac{609 + 597}{2} = 603$ mm

$\therefore 2d = 1206$ mm

Control perimeter;

$u = 2(350 + 250) + (2\pi \times 1206) = 8779$ mm

Area within perimeter;

$A = (0.35 \times 0.25) + (2 \times 0.35 \times 1.206) + (2 \times 0.25 \times 1.206) + (\pi \times 1.206^2) = 6.10 \text{ m}^2$

$< (l_{bd} + d) = 36\phi + d = (36 \times 12) + 609 = 1041$ mm

$\therefore \text{Reinforcement NOT contributed to punching resistance}$
(ii) Punching Shear

Average punching shear force at control perimeter:
\[ V_{Ed} = 153 \left[ (2.80 \times 3.50) - 6.10 \right] = 566 \text{ kN} \]

Punching shear stress:
\[ v_{Ed} = \frac{\beta V_{Ed}}{u d} \]

Where
\[ \beta = 1 + k \left( \frac{M_{Ed}}{V_{Ed}} \right) \left( \frac{u_1}{W_1} \right) \]

\[ k = 0.65 \Rightarrow \left[ \frac{c_1}{c_2} = \frac{350}{250} = 1.4 \right] \]

\[ W_1 = 0.5 c_1^2 + c_1 c_2 + 4 c_2 d + 16 d^2 + 2 \pi d c_1 \]
\[ = 0.5(350^2) + (350 \times 250) + (4 \times 250 \times 603) + 16(603^2) + (2 \pi \times 603 \times 350) = 7.9 \times 10^6 \text{ mm}^2 \]

\[ \therefore \beta = 1 + 0.65 \left( \frac{50 \times 10^6}{566 \times 10^3} \right) \left( \frac{8779}{7.9 \times 10^6} \right) = 1.06 \]

Therefore,
\[ v_{Ed} = \frac{1.06 \times 566 \times 10^3}{8779 \times 609} = 0.11 \text{ N/mm}^2 \]
Example 2: Pad Footing (Axial Load & Moment)

(ii) Punching Shear

Punching shear resistance:

\[ k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{609}} = 1.57 < 2.0 \]

\[ v_{Rd,c} = v_{min} = 0.035 k^{3/2} f_{ck}^{1/2} \]
\[ = 0.035 (1.57)^{3/2} (30)^{1/2} \]
\[ = 0.38 \text{ N/mm}^2 > v_{Ed} (0.11 \text{ N/mm}^2) \rightarrow \text{OK} \]
(iii) Maximum Punching Shear at Column Perimeter

Maximum punching shear force:
\[ V_{Ed,\text{max}} = 1500 \text{ kN} \]

Column perimeter,
\[ u_o = 2(350 + 250) = 1200 \text{ mm} \]

Punching shear stress:
\[ \nu_{Ed} = \frac{\beta V_{Ed}}{u_o d} \]

Where
\[ \beta = 1 + k \left( \frac{M_{Ed}}{V_{Ed}} \right) \left( \frac{u_o}{W_1} \right) \]

\[ k = 0.65 \Rightarrow \left[ \frac{c_1}{c_2} = \frac{350}{250} = 1.4 \right] \]

\[ W_1 = 0.5c_1^2 + c_1c_2 \]
\[ = 0.5(350^2) + (350 \times 250) = 0.15 \times 10^6 \text{ mm}^2 \]
(iii) Maximum Punching Shear at Column Perimeter

\[ \beta = 1 + 0.65 \left( \frac{50 \times 10^6}{1500 \times 10^3} \right) \left( \frac{1200}{0.15 \times 10^6} \right) = 1.17 \]

Therefore, \( v_{Ed} = \frac{1.17 \times 1500 \times 10^3}{1200 \times 603} = 2.44 \text{ N/mm}^2 \)

Maximum shear resistance:

\[ v_{Rd, max} = 0.5 \left[ 0.6 \left( 1 - \frac{f_{ck}}{250} \right) \right] \left( \frac{f_{ck}}{1.5} \right) \]

\[ = 0.5 \left[ 0.6 \left( 1 - \frac{30}{250} \right) \right] \left( \frac{30}{1.5} \right) \]

\[ = 5.28 \text{ N/mm}^2 > v_{Ed} \]

\( \Rightarrow \text{ OK} \)

Soil pressure = 153 kN/m²
Cracking

\( h = 650 \text{ mm} > 200 \text{ mm} \)

Assume steel stress is under \textit{quasi-permanent} loading:
\[
= 0.6 \left( \frac{f_{yk}}{1.15} \right) \left( \frac{A_{s,req}}{A_{s,prov}} \right) = 0.6 \left( \frac{500}{1.15} \right) \left( \frac{2197}{2375} \right) = 241 \text{ N/mm}^2
\]

For design crack width 0.3 mm:
Maximum allowable bar spacing = 150 mm
Actual bar spacing at \( x-x \):
\[
\frac{[2800-2(35)-12]}{20} = 136 \text{ mm} < 150 \text{ mm}
\]
Actual bar spacing at \( y-y \):
\[
\frac{[3500-2(35)-12]}{27} = 126 \text{ mm} < 150 \text{ mm}
\]

\textbf{Cracking OK}
Example 2: Pad Footing (Axial Load & Moment)

Detailing

Plan View

Section View
Example 3

DESIGN OF COMBINED FOOTING
Example 3: Design of Combined Footing

\[ N_A = 1610 \text{ kN (Ultimate)} \]
\[ N_B = 1950 \text{ kN (Ultimate)} \]

- Column size: 300 \( \times \) 300 mm
- Column size: 400 \( \times \) 400 mm
- \( f_{ck} = 35 \text{ N/mm}^2 \)
- \( f_{yk} = 500 \text{ N/mm}^2 \)
- \( \gamma_{soil} = 200 \text{ N/mm}^2 \)
- Unit weight of concrete = 25 kN/m\(^3\)
- Cover = 40 mm
- Assumed \( \phi_{bar} = 12 \text{ mm} \)
Example 3: Design of Combined Footing

Size

Service axial: 
\[ N_A = \frac{1610 \text{ kN}}{1.40} = 1150 \text{ kN} \]
\[ N_B = \frac{1950 \text{ kN}}{1.40} = 1393 \text{ kN} \]

Total service axial, \( N_{\text{total}} \)
\[ = 1150 + 1393 = 2543 \text{ kN} \]

Assumed selfweight 10% of service load, \( W = 254.3 \text{ kN} \)

Area of footing required
\[ = \frac{(N+W)}{\gamma_{\text{soil}}} = \frac{(2543+254.3)}{200} = 14.0 \text{ m}^2 \]

Try footing size, \( B \times H \times h = 2.70 \text{ m} \times 6.00 \text{ m} \times 0.65 \text{ m} \)

Area, \( A = 16.2 \text{ m}^2 \)

Selfweight, \( W = 16.2 \times 0.65 \times 25 = 252.7 \text{ kN} \)
Example 3: Design of Combined Footing

Size (Continued)

Check Service Soil Bearing Capacity = \( \frac{(N+W)}{A} = \frac{(2543+252.7)}{16.2} \)

= 173 kN/m\(^2\) \( \leq \) 200 kN/m\(^2\)  \( \Rightarrow \) OK

Arrange position of footing so that the distribution of soil pressure is uniform:

\[ N_A = 1150 \text{ kN} \]
\[ N_B = 1393 \text{ kN} \]
\[ \Sigma M_A = 0 \]
\[ 1393(3.4) + 2543x = 0 \]
\[ x = 1.86 \text{ m} \]
Example 3: Design of Combined Footing

Analysis

Soil pressure at ultimate load, \( P = \frac{N_{Ed}}{A} = \frac{(1610+1950)}{16.2} = 219.8 \text{ kN/m}^2 \)
Soil pressure per m width, \( w = 219.8 \times 2.7 \text{ m} = 593.5 \text{ kN/m} \)
Example 3: Design of Combined Footing

Shear Force & Bending Moment Diagram

- **SFD (kN)**
- **BMD (kNm)**

The diagram shows the shear force and bending moment diagrams for a combined footing with loads of 1610 kN and 1950 kN. The calculations include:

- Shear force: 593.5 kN/m
- Bending moment: 676 kNm

The diagram also includes the distance x = 1.86 meters, which is relevant for the design calculations.
Example 3: Design of Combined Footing

Main Reinforcement

Longitudinal Reinforcement: Bottom

Effective depth: \( d_x = h - c - 0.5 \phi_{bar} = 650 - 40 - (0.5 \times 12) = 604 \text{ mm} \)

\[
K = \frac{M_{Ed}}{f_{ck} bd^2} = \frac{473 \times 10^6}{35 \times 2700 \times 604^2} = 0.014 < K_{bal} = 0.167
\]

\[\therefore \text{Compression reinforcement is NOT required}\]

\[
z = d \left[ 0.25 - \left( \frac{K}{1.134} \right) \right] = 0.99d > 0.95d
\]

\[
A_{s,req} = \frac{M_{Ed}}{0.87f_{yk}z} = \frac{473 \times 10^6}{0.87 \times 500 \times 0.95 \times 604} = 1894 \text{ mm}^2 < A_{s,min} = 2722 \text{ mm}^2
\]
Example 3: Design of Combined Footing

Main Reinforcement

Longitudinal Reinforcement: Top

\[ K = \frac{M_{Ed}}{f_{ck}bd^2} = \frac{353 \times 10^6}{35 \times 2700 \times 604^2} = 0.010 < K_{bal} = 0.167 \]

\[ z = d \left[ 0.25 - \left( \frac{K}{1.134} \right) \right] = 0.99d > 0.95d \]

\[ A_{s,req} = \frac{M_{Ed}}{0.87f_{yk}z} = \frac{353 \times 10^6}{0.87 \times 500 \times 0.95 \times 604} = 1413 \text{ mm}^2 < A_{s,min} = 2722 \text{ mm}^2 \]
Example 3: Design of Combined Footing

Minimum & Maximum Area of Reinforcement

\[ A_{s,\text{min}} = 0.26 \left( \frac{f_{ctm}}{f_{yk}} \right) bd = 0.26 \left( \frac{3.2}{500} \right) 0.0017bd \geq 0.0013bd \]

\[ \therefore A_{s,\text{min}} = 0.0017bd = 0.0017 \times 2700 \times 604 = 2722 \text{ mm}^2 \]

\[ A_{s,\text{max}} = 0.04A_c = 0.04bh = 0.04 \times 2700 \times 650 = 70200 \text{ mm}^2 \]

Provide 25H12 at both top and bottom \((A_{s,\text{prov}} = 2828 \text{ mm}^2)\)
Example 3: Design of Combined Footing

Main Reinforcement

Transverse Reinforcement: Bottom

Consider $b = 1000$ mm:

$\therefore$ Soil pressure per 1 m width, $w = 219.8 \times 1.0 \text{ m} = 219.8 \text{ kN/m}$

\[ M_{Ed} = 219.8 \times 1.2 \times \frac{1.2}{2} \]

\[ = 158 \text{ kNm/m} \]
Example 3: Design of Combined Footing

Main Reinforcement

Transverse Reinforcement: Bottom

Effective depth: \( d_y = h - c - 1.5 \phi_{bar} = 650 - 40 - (1.5 \times 12) = 592 \text{ mm} \)

\[
K = \frac{M_E d}{f_{ck} b d^2} = \frac{158 \times 10^6}{35 \times 1000 \times 592^2} = 0.013 \quad < K_{bal} = 0.167
\]

\( z = d \left[ 0.25 - \left( \frac{K}{1.134} \right) \right] = 0.99d > 0.95d
\]

\[
A_{s,req} = \frac{M_E d}{0.87 f_{yk} z} = \frac{158 \times 10^6}{0.87 \times 500 \times 0.95 \times 592} = 647 \text{ mm}^2/\text{m} < A_{s,min} = 988 \text{ mm}^2/\text{m}
\]

\( \therefore \) Compression reinforcement is \textbf{NOT} required
Example 3: Design of Combined Footing

Minimum & Maximum Area of Reinforcement

\[ A_{s,\text{min}} = 0.26 \left( \frac{f_{ctm}}{f_{yk}} \right) bd = 0.26 \left( \frac{3.2}{500} \right) 0.0017bd \geq 0.0013bd \]

\[ \therefore A_{s,\text{min}} = 0.0017bd = 0.0017 \times 1000 \times 592 = 988 \text{ mm}^2/\text{m} \]

\[ A_{s,\text{max}} = 0.04A_c = 0.04bh = 0.04 \times 1000 \times 650 = 26000 \text{ mm}^2/\text{m} \]

Since \( A_s < A_{s,\text{min}} \), then use \( A_{s,\text{min}} = 988 \text{ mm}^2/\text{m} \)

Provide H12-100 (\( A_{s,\text{prov}} = 1131 \text{ mm}^2/\text{m} \))

For secondary bar reinforcement:

Provide H12-100 (\( A_{s,\text{prov}} = 1131 \text{ mm}^2/\text{m} \))
(i) Vertical Shear

Critical shear at $1.0d$ from face of column:

$$V_{Ed} = \left(\frac{1.02}{1.62}\right) \times 964 = 605 \text{ kN}$$

$$k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{604}} = 1.6 < 2.0$$

$$\rho_l = \frac{A_{sl}}{bd} = \frac{2828}{2700 \times 604} = 0.0017 \leq 0.02$$

$$\therefore V_{Rd,c} = \left[0.12k (100 \rho_l f_{ck})^{1/3}\right]bd$$

$$= \left[0.12 \times 1.6 (100 \times 0.0017 \times 35)^{1/3}\right]2700 \times 604 = 562369 \text{ N} = 562.4 \text{ kN}$$

$$\therefore V_{min} = \left[0.035k^{3/2}\sqrt{f_{ck}}\right]bd$$

$$= \left[0.035 \times 1.6^{3/2}\sqrt{35}\right]2700 \times 604 = 6667734 \text{ N} = 667 \text{ kN}$$

$$V_{Ed} (605 \text{ kN}) < V_{min} (667 \text{ kN}) \quad \Rightarrow \quad \text{OK}$$
(ii) Punching Shear

Critical shear at \(2.0d\) from face of column:

Average \(d = \frac{604 + 592}{2} = 598\) mm

\[ \therefore 2d = 1196\) mm

Control perimeter;

\[ u = 2(400 + 400) + (2\pi \times 1196) = 9116\) mm

Area within perimeter;

\[ A = (0.40 \times 0.40) + (2 \times 0.40 \times 1.196) + (2 \times 0.40 \times 1.196) + (\pi \times 1.196^2) = 6.57\) m\(^2\)
(ii) Punching Shear

Average punching shear force at control perimeter:

\[ V_{Ed} = 1950 - (219.8 \times 6.57) = 506 \text{ kN} \]

Punching shear stress:

\[ v_{Ed} = \frac{V_{Ed}}{u d} = \frac{506 \times 10^3}{9116 \times 598} = 0.09 \text{ N/mm}^2 \]

Punching shear resistance:

\[ k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{609}} = 1.57 < 2.0 \]

\[ v_{Rd,c} = v_{min} = 0.035k^{3/2}f_{ck}^{1/2} = 0.035 \times (1.6)^{3/2} \times (35)^{1/2} = 0.41 \text{ N/mm}^2 \]

\[ v_{Ed} (0.09 \text{ N/mm}^2) \rightarrow \text{OK} \]
(iii) Maximum Punching Shear at Column Perimeter

Maximum punching shear force:

\[ V_{Ed,max} = 1950 \text{ kN} \]

Column perimeter, \( u_o = 4 \times 400 = 1600 \text{ mm} \)

Maximum shear resistance:

\[ V_{Rd,max} = 0.5ud \left[ 0.6 \left( 1 - \frac{f_{ck}}{250} \right) \right] \left( \frac{f_{ck}}{1.5} \right) \]
\[ = 0.5 \times 1600 \times 598 \left[ 0.6 \left( 1 - \frac{35}{250} \right) \right] \left( \frac{35}{1.5} \right) \]
\[ = 2880 \text{ kN} > V_{Ed,max} \quad \Rightarrow \text{OK} \]
Cracking

\[ h = 650 \text{ mm} > 200 \text{ mm} \]

Assume steel stress is under \textit{quasi-permanent} loading:

\[ \sigma = 0.6 \left( \frac{f_{yk}}{1.15} \right) \left( \frac{A_{s,req}}{A_{s,prov}} \right) = 0.6 \left( \frac{500}{1.15} \right) \left( \frac{1894}{2828} \right) = 175 \text{ N/mm}^2 \]

For design crack width 0.3 mm:
Maximum allowable bar spacing = 250 mm
Actual bar spacing 1 = \[ \frac{2700 - 2(40) - 12}{24} = 109 \text{ mm} < 250 \text{ mm} \]
Actual bar spacing 2 = \[ \frac{2700 - 2(40) - 12}{24} = 109 \text{ mm} < 250 \text{ mm} \]
Actual bar spacing 3 = 100 mm < 250 mm

\textbf{Cracking OK}
Example 4

DESIGN OF STRAP FOOTING
Example 4: Design of Strap Footing

- $f_{ck} = 30 \text{ N/mm}^2$
- $f_{yk} = 500 \text{ N/mm}^2$
- $\gamma_{soil} = 200 \text{ N/mm}^2$
- Unit weight of concrete = 25 kN/m$^3$
- Cover = 40 mm
- Assumed $\phi_{bar} = 12 \text{ mm}$

$G_k = 800 \text{ kN}, Q_k = 275 \text{ kN}$

$N_A$

$N_B$

$G_k = 1000 \text{ kN}, Q_k = 525 \text{ kN}$

Column size: 300 $\times$ 300 mm

Beam size: 300 $\times$ 900 mm

Column size: 300 $\times$ 300 mm

$H$

2000

6000

700

$S$

$S$

$700$

$R_A$

$R_B$

$800$ kN, $Q = 275$ kN

$1000$ kN, $Q = 525$ kN

$G_k = 800 \text{ kN}, Q_k = 275 \text{ kN}$

$G_k = 1000 \text{ kN}, Q_k = 525 \text{ kN}$

Unit weight of concrete = 25 kN/m$^3$

Cover = 40 mm

Assumed $\phi_{bar} = 12 \text{ mm}$
## Example 4: Design of Strap Footing

### Loading

<table>
<thead>
<tr>
<th>Column</th>
<th>Size (mm × mm)</th>
<th>Load (kN)</th>
<th></th>
<th></th>
<th>Total (1.0G_k + 1.0Q_k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column A</td>
<td>300 × 300</td>
<td>G_k 800</td>
<td>Q_k 275</td>
<td></td>
<td>1075</td>
</tr>
<tr>
<td>Column B</td>
<td>300 × 300</td>
<td>G_k 1000</td>
<td>Q_k 525</td>
<td></td>
<td>1525</td>
</tr>
</tbody>
</table>
Example 4: Design of Strap Footing

Size

Assumed self weight of footing is 10% of service load:
\[ W_A = 108 \text{ kN} \text{ and } W_B = 153 \text{ kN} \]

Beam self weight, \( W_R = 25 \times (0.3 \times 0.9 \times 5.7) = 38 \text{ kN} \)
Example 4: Design of Strap Footing

**Size**

\[ N_A = 1075 \text{ kN} \]
\[ N_B = 1525 \text{ kN} \]

\[ \sum M@B = 0 \]
\[ (R_A - W_A)(6.15 - 1.0) - N_A (6.0) - W_R (3.0) = 0 \]
\[ (R_A - 108)(6.15 - 1.0) - 1075 (6.0) - 38 (3.0) = 0 \]
\[ R_A - 108 = 1275 \]
\[ \therefore R_A = 1382 \text{ kN} \]

\[ \frac{R_A}{2.0H} = 200 \]
\[ \frac{1382}{2.0H} = 200 \]
\[ \therefore H = 3.46 \text{ mm} \]

**Footing A size: 2.00 × 3.50 m**

\[ R_A + R_B = (N_A + N_B + W_A + W_B + W_R) \]
\[ R_A + R_B = (1075 + 1525 + 108 + 153 + 38) \]
\[ R_B = 2898 - R_A \]
\[ R_B = 2898 - 1382 \]
\[ \therefore R_A = 1516 \text{ kN} \]

\[ \frac{R_B}{S^2} = 200 \]
\[ \frac{1516}{S^2} = 200 \]
\[ \therefore S = 2.75 \text{ mm} \]

**Footing B size: 2.75 × 2.75 m**
Example 4: Design of Strap Footing

Footing Size Checks

Total service load = 1075 + 1525 + 123 + 132 + 38 = **2893 kN**

Area of footing = (2.0 × 3.5) + (2.75 × 2.75) = **14.56 m²**

Soil pressure = \[ \frac{2893}{14.56} \] = 199 kN/m² < 200 kN/m² \[ \therefore \text{OK} \]
### Example 4: Design of Strap Footing

#### Footing Self Weight

- **$N_A = 1075\ kN$**
  - 6 m
  - 2 m

- **$N_B = 1525\ kN$**
  - X'

- **$W_A = 123\ kN$**
  - 2 m

- **$W_B = 132\ kN$**
  - 2.75 m

- **$W_R = 38\ kN$**

**Distance from resultant force $R$ to centroid of column B:**

\[
(1075 \times 6.0) + (123 \times 5.15) + (38 \times 3.0) = 2893X'
\]

$X' = 2.48\ m$

**Distance from centroid of footings to centroid of column B:**

\[
(2.0 \times 3.5)(5.15) = [(2.0 \times 3.5) + (2.75 \times 2.75)]X'
\]

$X' = 2.48\ m$
Example 4: Design of Strap Footing

Analysis

Design Load:

\[ N_A = (1.35 \times 800) + (1.50 \times 275) = 1493 \text{ kN} \]
\[ N_B = (1.35 \times 1000) + (1.50 \times 525) = 2138 \text{ kN} \]
\[ W_A = 1.35 \times 123 = 165 \text{ kN} \]
\[ W_B = 1.35 \times 132 = 179 \text{ kN} \]
\[ W_R = 1.35 \times 38 = 52 \text{ kN} \]

Total Design Load, \( P_{\text{total}} \) = 4026 kN

Ultimate soil pressure = \( \frac{P_{\text{total}}}{\text{Area of Footing}} = \frac{4026}{14.56} = 276 \text{ kN/m}^2 \)
Example 4: Design of Strap Footing

Shear Force & Bending Moment Diagram

Shear Force Diagram (SFD):
- 1493 kN
- 2138 kN

Bending Moment Diagram (BMD):
- \( \frac{52}{5.7} = 9.1 \text{ kN/m} \)
- \( 276.5 - \left( \frac{165}{2.0 \times 3.5} \right) = 253 \times 3.5 = 885 \text{ kN/m} \)
- \( 276.5 - \left( \frac{179}{2.75 \times 2.75} \right) = 253 \times 2.75 = 695 \text{ kN/m} \)

\[ 276.5 - \left( \frac{165}{2.0 \times 3.5} \right) = 253 \times 3.5 = 885 \text{ kN/m} \]
\[ 276.5 - \left( \frac{179}{2.75 \times 2.75} \right) = 253 \times 2.75 = 695 \text{ kN/m} \]
**DESIGN OF FOOTING A**

**Main Reinforcement**

Soil pressure, \( w = 253 \times 2.0 \text{ m} = 506 \text{ kN/m} \)

\[
M_{Ed} = 506 \times 1.6 \times \frac{1.6}{2} = 647 \text{ kNm}
\]

Effective depth: \( d = h - c - 0.5 \phi_{bar} = 700 - 40 - (0.5 \times 16) = 652 \text{ mm} \)
Example 4: Design of Strap Footing

\[ K = \frac{M_{Ed}}{f_{ck}bd^2} = \frac{647 \times 10^6}{30 \times 2000 \times 652^2} = 0.025 < K_{bal} = 0.167 \]

\[ \therefore \text{Compression reinforcement is NOT required} \]

\[ z = d \left[ 0.25 - \left( \frac{K}{1.134} \right) \right] = 0.98d > 0.95d \]

\[ A_{s,req} = \frac{M_{Ed}}{0.87f_{yk}z} = \frac{647 \times 10^6}{0.87 \times 500 \times 0.95 \times 652} = 2402 \text{ mm}^2/\text{m} \geq A_{s,min} \]

**Minimum & Maximum Area of Reinforcement**

\[ A_{s,min} = 0.26 \left( \frac{f_{ctm}}{f_{yk}} \right) bd = 0.26 \left( \frac{2.90}{500} \right) 0.0015bd \geq 0.0013bd \]

\[ \therefore A_{s,min} = 0.0015bd = 0.0015 \times 2000 \times 652 = 1964 \text{ mm}^2 \text{ or } 982 \text{ mm}^2/\text{m} \]

\[ A_{s,max} = 0.04A_c = 0.04bh = 0.04 \times 2000 \times 700 = 56000 \text{ mm}^2 \]

**Main Reinforcement:** Provide 14H16 \( (A_{s,prov} = 2815 \text{ mm}^2) \)

**Secondary Reinforcement:** H16-200 \( (A_{s,prov} = 1005 \text{ mm}^2/\text{m}) \)
Example 4: Design of Strap Footing

(i) Vertical Shear

Critical shear at $1.0d$ from face of column:

\[ \therefore \text{Design shear force, } V_{Ed} = 253 \times 0.948 \times 2.0 = 479.4 \text{ kN} \]

**Note:**
Bar extend beyond critical section at $= 948 - 40 = 908$ mm

\[ > (l_{bd} + d) = 36\varnothing + d = (36 \times 16) + 652 = 1228 \text{ mm} \]

\[ \therefore A_{sl} = 0 \text{ mm}^2 \]
(i) Vertical Shear

\[ k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{652}} = 1.55 < 2.0 \]

\[ \rho_l = \frac{A_{sl}}{bd} = 0 \]

\[ V_{rd,c} = [0.12k(100\rho_l f_{ck})^{1/3}]bd \]
\[ = [0.12 \times 1.55(100 \times 0 \times 30)^{1/3}]3500 \times 652 = 0 \text{ N} = 0 \text{ kN} \]

\[ V_{min} = [0.035k^{3/2}\sqrt{f_{ck}}]bd \]
\[ = [0.035 \times 1.55^{3/2}\sqrt{30}]3500 \times 652 = 484194 \text{ N} = 484.2 \text{ kN} \]

\[ V_{Ed} (479.4 \text{ kN}) < V_{min} (484.2 \text{ kN}) \]

\[ \Rightarrow \text{OK} \]
Example 4: Design of Strap Footing

Cracking

\[ h = 700 \text{ mm} > 200 \text{ mm} \]

Assume steel stress is under *quasi-permanent* loading:

\[ = 0.59 \left( \frac{f_{yk}}{1.15} \right) \left( \frac{A_{s,req}}{A_{s,prov}} \right) = 0.59 \left( \frac{500}{1.15} \right) \left( \frac{2402}{2815} \right) = 219 \text{ N/mm}^2 \]

For design crack width 0.3 mm:

Maximum allowable bar spacing = 200 mm

Actual bar spacing = \[ \frac{[2000-2(40)-16]}{13} = 146 \text{ mm} < 200 \text{ mm} \]

**Cracking OK**
Example 4: Design of Strap Footing

DESIGN OF FOOTING B

Main Reinforcement

Soil pressure, \( w = 253 \times 2.75 \text{ m} = 695.75 \text{ kN/m} \)

\[
M_{Ed} = 695.75 \times 1.225 \times \frac{1.225}{2} = 522 \text{ kNm}
\]

Effective depth: \( d = h - c - 1.5 \phi_{bar} = 700 - 40 - (1.5 \times 16) = 636 \text{ mm} \)
Example 4: Design of Strap Footing

\[ K = \frac{M_{Ed}}{f_{ck}bd^2} = \frac{522 \times 10^6}{30 \times 2750 \times 636^2} = 0.016 < K_{bal} = 0.167 \]

∴ Compression reinforcement is **NOT** required

\[ z = d \left[ 0.25 - \left( \frac{K}{1.134} \right) \right] = 0.99d > 0.95d \]

\[ A_{s,req} = \frac{M_{Ed}}{0.87f_{yk}z} = \frac{522 \times 10^6}{0.87 \times 500 \times 0.95 \times 636} = 1985 \text{ mm}^2/\text{m} < A_{s,min} = 2623 \text{ mm}^2 \]

**Minimum & Maximum Area of Reinforcement**

\[ A_{s,min} = 0.26 \left( \frac{f_{ctm}}{f_{yk}} \right) bd = 0.26 \left( \frac{2.90}{500} \right) 0.0015bd \geq 0.0013bd \]

∴ \[ A_{s,min} = 0.0015bd = 0.0015 \times 2750 \times 636 = 2623 \text{ mm}^2 \]

\[ A_{s,max} = 0.04A_c = 0.04bh = 0.04 \times 2750 \times 700 = 77000 \text{ mm}^2 \]

**Main Reinforcement: Provide 15H16** (\( A_{s,prov} = 3016 \text{ mm}^2 \))
(i) Vertical Shear

Critical shear at 1.0d from face of column:

\[ V_{Ed} = 253 \times 0.589 \times 2.75 = 409.5 \text{ kN} \]

**Note:**
Bar extend beyond critical section at = 589 – 40 = 549 mm
\[ > (l_{bd} + d) = 36\Phi + d = (36 \times 16) + 636 = 1212 \text{ mm} \]
\[ \therefore A_{sl} = 0 \text{ mm}^2 \]
Example 4: Design of Strap Footing

(i) Vertical Shear

\[ k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{636}} = 1.56 < 2.0 \]

\[ \rho_l = \frac{A_{sl}}{bd} = 0 \]

\[ : V_{Rd,c} = [0.12k(100\rho_l f_{ck})^{1/3}]bd \]
\[ = [0.12 \times 1.56(100 \times 0 \times 30)^{1/3}]2750 \times 636 = 0 \text{ N} = 0 \text{ kN} \]

\[ : V_{min} = [0.035k^{3/2}\sqrt{f_{ck}}]bd \]
\[ = [0.035 \times 1.56^{3/2}\sqrt{30}]2750 \times 636 = 653774 \text{ N} = 653.8 \text{ kN} \]

\[ V_{Ed} (409.5 \text{ kN}) < V_{min} (653.8 \text{ kN}) \quad \Rightarrow \quad \text{OK} \]
Example 4: Design of Strap Footing

**Cracking**

\[ h = 700 \text{ mm} > 200 \text{ mm} \]

Assume steel stress is under *quasi-permanent* loading:

\[
0.54 \left( \frac{f_{yk}}{1.15} \right) \left( \frac{A_{s,req}}{A_{s,prov}} \right) = 0.54 \left( \frac{500}{1.15} \right) \left( \frac{2623}{3016} \right) = 204 \text{ N/mm}^2
\]

For design crack width 0.3 mm:
Maximum allowable bar spacing = 200 mm
Actual bar spacing = \[
\frac{[2750-2(40)-16]}{14} = 189 \text{ mm} < 200 \text{ mm}
\]

**Cracking OK**
DESIGN OF TIE BEAM

Design similar to beam design. Refer to RC1.
DESIGN OF PILE FOUNDATIONS
Design of Pile Foundation

• To be used when the soil conditions are poor and uneconomical, or not possible to provide adequate spread foundations
• The piles must extend down to firm soil
• Load carried by either end bearing, friction or combination of both
Design of Pile Foundation

Load from Structure

Pile Cap

Lower Density

Medium Density

High Density
Selection of Piles Type

- Depends on **loading, type of structure, soil strata & site conditions**
- Main types:
  a) Precast reinforced or pre-stressed concrete piles
  b) Cast in-situ reinforced concrete piles
  c) Timber piles
  d) Steel piles
  e) Bakau piles
Determination of Pile Capacity

• Safe load:
  a) Determined from test loading of a pile or using a pile formula
  b) Ultimate load divided by safety factor between 2 & 3
  c) Depends on size & depth, & whether the pile is of the end bearing or friction type

• Pile formula – gives resistance from the energy of the driving force & the final set or penetration of the pile per blow
Determination of Piles Number & Spacing

- Pile load < Single pile capacity
- Piles are usually arranged symmetrically with respect to the column axis
Determination of Piles Number & Spacing

*Foundation Subjected to Axial Load Only*

Load applied at centroid of the group of piles:

\[ F_a = \frac{(N + W)}{n} \]

- \( N \) = Axial load from column
- \( W \) = Self weight of pile cap
- \( n \) = Number of piles
**Determination of Piles Number & Spacing**

*Foundation Subjected to Axial Load & Moment*

- Pile cap assumed to rotate about the centroid of the pile group
- Piles load resisting moment vary uniformly from zero at the centroidal axis to a maximum for the piles farthest away

\[
F_{ai} = \frac{(N + W)}{n} \pm \frac{Mx_i}{I_y}
\]

- \( F_{ai} \) = Load per pile \( i \)
- \( M \) = Moment
- \( x_i \) = Distance from pile \( i \) to centroid of pile cap
- \( I_y \) = Moment of inertial of pile group = \( (x_1^2 + x_2^2 + \ldots + x_n^2) \)
Design of Pile Cap
Design of Pile Cap

column

pile cap

pile
Size & Thickness

- Depends on **number** of piles used, **arrangement of piles** & **shape** of piles.
- Thickness of the cap should be sufficient to provide **adequate bond length** for bars projecting from the piles as well for the dowel bars of the column.
Main Reinforcement

- **Using bending theory or truss theory**
- **Truss Theory:** Force from the supported column is assumed to be transmitted by a triangular truss action – concrete providing the compressive members of the truss & steel reinforcement providing tensile force
### Truss Theory – Axial Force

<table>
<thead>
<tr>
<th>Number of Piles</th>
<th>Dimensions of Pile Cap</th>
<th>Neglecting of Column</th>
<th>Tensile Force to be Resisted by Reinforcement Taking Size of Column into Consideration</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td></td>
<td></td>
<td>$\frac{NL}{4d}$</td>
</tr>
<tr>
<td><img src="image" alt="Diagram 2 Piles" /></td>
<td><img src="image" alt="Diagram 3 Piles" /></td>
<td></td>
<td>$\frac{NL}{9d}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
**Truss Theory – Axial Force**

<table>
<thead>
<tr>
<th>$N$</th>
<th>$l$</th>
<th>Formula</th>
</tr>
</thead>
</table>
| 4   |     | \[
\frac{N}{24ld} (3l^2 - a^2) \] |
| 5   |     | \[
\frac{N}{30ld} (3l^2 - a^2) \] |

**Notation**  \( h_p \) diameter of pile; \( a, b \) dimensions of column; \( \alpha \) spacing factor of piles (normally between 2 and 3 depending on ground conditions)

**Figure 9.7** Tensile force in pile cap

(Source: “Reinforced concrete designers handbooks”, Reynold$^{[11]}$)
Design of Pile Cap

Pile Cap Size
Design of Pile Cap

Pile Cap Size

- $(k + 1) \times \text{pile dia} + 300 \text{ mm}$
- $k \times \text{pile dia}$
- $\frac{(k + 1)}{2} \times \text{pile dia} + 150 \text{ mm}$
- $\frac{(k + 1)}{3} \times \text{pile dia} + 300 \text{ mm}$
- $2(k + \frac{1}{3}) \times \text{pile dia} + \frac{600}{\sqrt{3}} \text{ mm}$
- $\frac{3k}{2} \times \text{pile dia}$
- $\frac{k}{2} \times \text{pile dia}$
- $(k + 1) \times \text{pile dia} + 300 \text{ mm}$
- $(2k + 1) \times \text{pile dia} + 300 \text{ mm}$
Design for Shear

- Critical section taken to be $20\% \times \text{pile diameter (or } \phi_p/5 \text{)}$ inside the face of the column
- Shear enhancement may be considered such that the shear resistance of the concrete may be increased to $V_{Rd,c} \times \frac{2d}{a_v}$ where $a_v = \text{distance from the face of column to the critical section}$
- For spacing of piles $\leq 3 \times \text{pile diameter}$, this enhancement may be applied across the whole critical section
Design for Shear

• **For spacing > 3 \times pile diameter**, then the pile cap should be checked for punching shear on the perimeter
• Check shear force at column face < 0.5v_1f_{cd}ud = 0.5v_1 \left( \frac{f_{ck}}{1.5} \right) ud where u is the perimeter of the column and strength reduction factor, v_1 = 0.6 \left( 1 - \frac{f_{ck}}{250} \right)
**Design of Pile Cap**

**Design for Shear**

- Vertical shear @ 1.0d from column face
- Punching shear @ 2.0d from column face
Detailing of Reinforcement

- Main tension reinforcement should continue pass each pile and bent-up vertically to provide full anchorage length beyond the centre line of each pile
- Normal to provide fully lapped horizontal link of size $\geq 12$ mm @ spacing $\leq 250$ mm
Design of Pile Cap

Detailing of Reinforcement

- Bars projecting into cap from piles
- Starter bars for columns
- Links for starter bars
- Vertical links between bars extending from piles
- Horizontal links around up-standing end of main bars (≥ 12 mm @ 250 mm, say)
- Main bars design to resist tensile forces

Starter bars for columns

Links for starter bars

Vertical links between bars extending from piles

Horizonal links around up-standing end of main bars (≥ 12 mm @ 250 mm, say)
Example 5

DESIGN OF PILE CAP – TRUSS THEORY
Example 5: Design of Pile Cap – Truss Theory

- Axial Load, $N$:
  - $G_k = 3500 \text{ kN}$ & $Q_k = 2500 \text{ kN}$
- $f_{ck} = 30 \text{ N/mm}^2$
- $f_{yk} = 500 \text{ N/mm}^2$
- $\gamma_{soil} = 200 \text{ N/mm}^2$
- Unit weight of concrete = $25 \text{ kN/m}^3$
- Column size = $500 \text{ mm} \times 500 \text{ mm}$
- Cover = 75 mm
- Assumed $\phi_{bar} = 25 \text{ mm}$
- Pile:
  - Pre-stressed spun pile: 500 mm dia.
  - Working load = 1800 kN
Example 5: Design of Pile Cap – Truss Theory

**Size of Pile Cap**

Service load = 3500 + 2500 = 6000 kN
Assumed self weight of pile cap, say \( W = 200 \) kN
Pile capacity, \( F_a = 1800 \) kN

No. of pile required,
\[
n = \frac{(N+W)}{F_a} = \frac{(6000 + 200)}{1800} = 3.4 \quad \therefore \text{Use } n = 4
\]

Pile spacing, \( l = k \phi_p = 3 \phi_p = 3 \times 500 = 1500 \) mm

Width, \( B = (k + 1) \phi_p + 300 = (3 + 1) \times 500 + 300 = 2300 \) mm

Length, \( H = (k + 1) \phi_p + 300 = (3 + 1) \times 500 + 300 = 2300 \) mm

Depth, \( h = 2 \phi_p + 100 = 2(500) + 100 = 1100 \) mm
Try size: \( B \times H \times h = 2.3 \times 2.3 \times 1.1 \) m
Self weight = \( 25(2.3 \times 2.3 \times 1.1) = 145 \text{ kN} < 200 \text{ kN} \) OK

\( \phi_p/5 = 100 \)

\( a_v = 750 - 250 - 250 + 500/5 = 350 \)

Critical shear section
Example 5: Design of Pile Cap – Truss Theory

**Main Reinforcement**

Effective depth, \( d = h - c - 1.5\phi_{\text{bar}} = 1100 - 75 - 1.5(25) = 988 \text{ mm} \)

Ultimate load, \( N = 1.35G_k + 1.5Q_k = 1.35(3500) + 1.5(2500) = 8475 \text{ kN} \)

From truss analogy:

Tension force, \( T = \frac{Nl}{8d} = \frac{8475 \times 1.5}{8 \times 0.988} = 1609 \text{ kN} \)

Area of reinforcement, \( A_s = \frac{T}{0.87f_{\text{yk}}} = \frac{1609 \times 10^3}{0.87 \times 500} = 3699 \text{ mm}^2 \)

For the whole width of pile cap, \( A_s = 2 \times 3699 = 7399 \text{ mm}^2 \)
Main Reinforcement (continued)

Minimum & Maximum Area of Reinforcement

\[ A_{s,min} = 0.26 \left( \frac{f_{ctm}}{f_{yk}} \right) bd = 0.26 \left( \frac{2.90}{500} \right) 0.0015bd \geq 0.0013bd \]
\[ \therefore A_{s,min} = 0.0015bd = 0.0015 \times 2300 \times 988 = 3421 \text{ mm}^2 \]

\[ A_{s,max} = 0.04A_c = 0.04bh = 0.04 \times 2300 \times 1100 = 101200 \text{ mm}^2 \]

Provide 16H25 (\(A_{s,prov} = 7855 \text{ mm}^2\))
Example 5: Design of Pile Cap – Truss Theory

Shear

(i) Vertical Shear

Critical shear at $\phi_p/5$ section inside pile:

Load per pile = $\frac{8475}{4} = 2119$ kN

2 pile outside critical section $\therefore$ Shear force, $V_{Ed} = 2 \times 2119 = 4238$ kN

Pile spacing $< 3\phi_p$ $\therefore$ Consider shear enhancement on the whole width of the section

$\therefore$ Reduced shear force, $V_{Ed} = 4238 \times \left(\frac{a_v}{2d}\right) = 4238 \times \left(\frac{350}{2 \times 988}\right) = 751$ kN
Example 5: Design of Pile Cap – Truss Theory

Shear

Note:
Bar extend beyond critical section at = 475 + 988 – 75 = 1388 mm
> \( (l_{bd} + d) = 36\phi + d = (36 \times 25) + 988 = 1888 \text{ mm} \) \( \therefore A_{sl} = 0 \text{ mm}^2 \)

\[ a_v = 750 - 250 - 250 + \frac{500}{5} = 350 \]
Example 5: Design of Pile Cap – Truss Theory

**Shear**

\[ k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{988}} = 1.45 < 2.0 \]

\[ \therefore V_{Rd,c} = V_{min} = \left[0.035k^{3/2}\sqrt{f_{ck}}\right]bd \]
\[ = \left[0.035 \times 1.45^{3/2}\sqrt{30}\right]2300 \times 988 \times 10^{-3} = 760 \text{ kN} \]

\[ V_{Ed} (751 \text{ kN}) < V_{min} (760 \text{ kN}) \]

\[ \Rightarrow \text{OK} \]
Example 5: Design of Pile Cap – Truss Theory

Shear

(ii) Punching Shear at Perimeter 2.0d from Column Face

Since pile spacing < 3\(\phi_p\)

\[\therefore \text{No punching shear check in necessary}\]
Example 5: Design of Pile Cap – Truss Theory

Shear

(iii) **Maximum Punching Shear at Column Perimeter**

\[
V_{Rd,\text{max}} = 0.5 \left[ 0.6 \left( 1 - \frac{f_{ck}}{250} \right) \right] \left( \frac{f_{ck}}{1.5} \right) ud \\
= 0.5 \left[ 0.6 \left( 1 - \frac{30}{250} \right) \right] \left( \frac{30}{1.5} \right) (4 \times 500) \times 988 = 10428 \text{ kN} > V_{Ed,\text{max}} = 8475 \text{ kN}
\]

**OK**
Cracking

\[ h = 1100 \text{ mm} > 200 \text{ mm} \]

Assume steel stress is under *quasi-permanent* loading:

\[ = 0.50 \left( \frac{f_{yk}}{1.15} \right) \left( \frac{A_{s,req}}{A_{s,prov}} \right) = 0.50 \left( \frac{500}{1.15} \right) \left( \frac{7399}{7855} \right) = 205 \text{ N/mm}^2 \]

For design crack width 0.3 mm:
Maximum allowable bar spacing = 200 mm
Actual bar spacing = \[ \frac{[2300-2(75)-25]}{15} = 142 \text{ mm} < 200 \text{ mm} \]

**Cracking OK**
Example 5: Design of Pile Cap – Truss Theory

Detailing

- 16H25 @ 140 mm
- 1500 mm spacing
- 500 mm spun pile
- 5H16 Binders
- h = 1100 mm
Example 5

DESIGN OF PILE CAP – BEAM THEORY
Example 5: Design of Pile Cap – Beam Theory

- Axial Load, \( N_{\text{ultimate}} = 4200 \) kN
- Moment, \( M_{\text{ultimate}} = 75 \) kNm
- \( f_{ck} = 30 \) N/mm\(^2\)
- \( f_{yk} = 500 \) N/mm\(^2\)
- \( \gamma_{soil} = 200 \) N/mm\(^2\)
- Unit weight of concrete = 25 kN/m\(^3\)
- Column size = 400 mm \( \times \) 400 mm
- Cover = 75 mm
- Assumed \( \phi_{bar} = 16 \) mm
- Pile:
  - Precast RC pile: 300 mm \( \times \) 300 mm
  - Working load = 600 kN
- Safety factor = 1.40
Size of Pile Cap

Service load = \( \frac{4200}{1.40} = 3000 \) kN

Assumed self weight of pile cap, say \( W = 200 \) kN

Pile capacity, \( F_a = 600 \) kN

No. of pile required, \( n = \frac{(N+W)}{F_a} = \frac{(3000 + 200)}{600} = 5.3 \)  

::: Use \( n = 6 \)
Example 5: Design of Pile Cap – Beam Theory

Size of Pile Cap

Service load = \( \frac{4200}{1.40} = 3000 \) kN

Assumed self weight of pile cap, say \( W = 200 \) kN

Pile capacity, \( F_a = 600 \) kN

No. of pile required, \( n = \frac{(N+W)}{F_a} = \frac{(3000 + 200)}{600} = 5.3 \) ∴ Use \( n = 6 \)
Example 5: Design of Pile Cap – Beam Theory

Size of Pile Cap

Service load = \( \frac{4200}{1.40} \) = 3000 kN

Assumed self weight of pile cap, say \( W = 200 \) kN

Pile capacity, \( F_a = 600 \) kN

No. of pile required, \( n = \frac{(N+W)}{F_a} = \frac{(3000 + 200)}{600} = 5.3 \). Use \( n = 6 \)

Pile spacing, \( l = k\phi_p = 3\phi_p = 3 \times 300 = 900 \) mm

Width, \( B = (k + 1)\phi_p + 300 = (3 + 1) \times 300 + 300 = 1500 \) mm

Length, \( H = (2k + 1)\phi_p + 300 = [2(3) + 1]300 + 300 = 2400 \) mm

Depth, \( h = 2\phi_p + 500 = 2(300) + 500 = 1100 \) mm
Example 5: Design of Pile Cap – Beam Theory

**Size of Pile Cap (continued)**

Try size: \( B \times H \times h = 1.5 \times 2.4 \times 1.1 \) m

Self weight = \( 25(1.5 \times 2.4 \times 1.1) = 99 \text{ kN} < 200 \text{ kN} \) OK

\[ \phi_p/5 = 60 \]

\[ \sigma_v = 1200 - 200 - 450 + 300/5 = 610 \]

\[ I = 4(0.9)^2 = 3.24 \text{ m}^2 \]
Example 5: Design of Pile Cap – Beam Theory

**Analysis**

Service moment, \( M_{service} = \frac{75}{1.40} = 53.6 \text{ kNm} \)

Maximum service load per pile, \( F = \frac{(N+W)}{n} + \frac{Mx}{I} \)

\[
\begin{align*}
F &= \frac{(3000+99)}{6} + \frac{53.6 \times 0.90}{3.24} = 531 \text{ kN} < 600 \text{ kN} \\
\end{align*}
\]

Ultimate load per pile:

\[
\begin{align*}
F_{xx} &= \frac{N_{ult}}{n} + \frac{M_{ult} x}{I} = \frac{4200}{6} + \frac{75 \times 0.90}{3.24} = 721 \text{ kN} \\
F_{yy} &= \frac{N_{ult}}{n} = \frac{4200}{6} = 700 \text{ kN} \\
\end{align*}
\]

Maximum moment at column face:

\[
\begin{align*}
M_{xx} &= 2[721 \times (0.90 - 0.20)] = 1009 \text{ kNm} \\
M_{yy} &= 3[700 \times (0.45 - 0.20)] = 525 \text{ kNm} \\
\end{align*}
\]
**Main Reinforcement**

Effective depth:
\[ d_x = h - c - 0.5\phi_{bar} = 1100 - 75 - 0.5(16) = 1017 \text{ mm} \]
\[ d_y = h - c - 1.5\phi_{bar} = 1100 - 75 - 1.5(16) = 1001 \text{ mm} \]

**Longitudinal Bar**

\[ K = \frac{M_{Ed}}{f_{ck}bd^2} = \frac{1009 \times 10^6}{30 \times 1500 \times 1017^2} = 0.022 < K_{bal} = 0.167 \]

\[ z = d \left[ 0.25 - \left( \frac{K}{1.134} \right) \right] = 0.98d > 0.95d \]

\[ A_s,req = \frac{M_{Ed}}{0.87f_{yk}z} = \frac{1009 \times 10^6}{0.87 \times 500 \times 0.95 \times 1017} = 2401 \text{ mm}^2 \]
Example 5: Design of Pile Cap – Beam Theory

Minimum & Maximum Area of Reinforcement

\[ A_{s,\text{min}} = 0.26 \left( \frac{f_{ctm}}{f_{yk}} \right) bd = 0.26 \left( \frac{2.90}{500} \right) 0.0015bd \geq 0.0013bd \]

\[ \therefore A_{s,\text{min}} = 0.0015bd = 0.0015 \times 1500 \times 1017 = 2298 \text{ mm}^2 \]

\[ A_{s,\text{max}} = 0.04A_c = 0.04bh = 0.04 \times 1500 \times 1100 = 66000 \text{ mm}^2 \]

Provide 12H16 (\( A_{s,\text{prov}} = 2413 \text{ mm}^2 \))
Example 5: Design of Pile Cap – Beam Theory

Transverse Bar

\[ K = \frac{M_{Ed}}{f_{ck}bd^2} = \frac{525 \times 10^6}{30 \times 2400 \times 1001^2} = 0.007 < K_{bal} = 0.167 \]

∴ Compression reinforcement is **NOT** required

\[ z = d \left[ 0.25 - \left( \frac{K}{1.134} \right) \right] = 0.9d > 0.95d \]

\[ A_{s,req} = \frac{M_{Ed}}{0.87f_{yk}z} = \frac{525 \times 10^6}{0.87 \times 500 \times 0.95 \times 1001} = **1269 \text{ mm}^2** \]
Example 5: Design of Pile Cap – Beam Theory

Minimum & Maximum Area of Reinforcement

\[ A_{s,min} = 0.26 \left( \frac{f_{ctm}}{f_{yk}} \right) bd = 0.26 \left( \frac{2.90}{500} \right) 0.0015bd \geq 0.0013bd \]

\[ \therefore A_{s,min} = 0.0015bd = 0.0015 \times 2400 \times 1001 = 3618 \text{ mm}^2 \]

\[ A_{s,max} = 0.04A_c = 0.04bh = 0.04 \times 2400 \times 1100 = 105600 \text{ mm}^2 \]

Since \( A_{s,req} = 1269 \text{ mm}^2 < A_{s,min} = 3618 \text{ mm}^2 \)

Provide 18H16 (\( A_{s,prov} = 3620 \text{ mm}^2 \))
Shear

(i) Vertical Shear

Critical shear at $\phi_p/5$ section inside pile:

2 pile outside critical section $\therefore$ Shear force, $V_{Ed} = 2 \times 721 = \textbf{1442 kN}$

Pile spacing $< 3\phi_p$ $\therefore$ Consider shear enhancement on the whole width of the section

$\therefore$ Reduced shear force, $V_{Ed} = 1442 \times \left(\frac{a_v}{2d}\right) = 1442 \times \left(\frac{610}{2 \times 1017}\right) = \textbf{432 kN}$
Example 5: Design of Pile Cap – Beam Theory

Shear

**Note:**
Bar extend beyond critical section at \( = 315 + 1001 = 1316 \text{ mm} \)
\( > (l_{bd} + d) = 36\phi + d = (36 \times 16) + 1001 = 1577 \text{ mm} \) \( \therefore A_{sl} = 0 \text{ mm}^2 \)
Shear

\[ k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{1001}} = 1.45 < 2.0 \]

\[ V_{Rd,c} = V_{min} = [0.035k^{3/2}\sqrt{f_{ck}}]bd \]
\[ = [0.035 \times 1.45^{3/2}\sqrt{30}]1500 \times 1001 \times 10^{-3} = 501 \text{ kN} \]

\[ V_{Ed}(432 \text{ kN}) < V_{min}(501 \text{ kN}) \quad \Rightarrow \text{OK} \]
Shear

(ii) Punching Shear at Perimeter 2.0d from Column Face

Since pile spacing $< 3\phi_p$

:: No punching shear check is necessary
Example 5: Design of Pile Cap – Beam Theory

Shear

(iii) Maximum Punching Shear at Column Perimeter

\[ V_{Rd,\text{max}} = 0.5 \left[ 0.6 \left( 1 - \frac{f_{ck}}{250} \right) \right] \left( \frac{f_{ck}}{1.5} \right) u_d \]

\[ = 0.5 \left[ 0.6 \left( 1 - \frac{30}{250} \right) \right] \left( \frac{30}{1.5} \right) (4 \times 400) \times 1017 = 8456 \text{ kN} > V_{Ed,\text{max}} = 4200 \text{ kN} \]

**OK**
Cracking

\[ h = 1100 \text{ mm} > 200 \text{ mm} \]

Assume steel stress is under quasi-permanent loading:

\[ = 0.55 \left( \frac{f_{yk}}{1.15} \right) \left( \frac{A_{s,req}}{A_{s,prov}} \right) = 0.55 \left( \frac{500}{1.15} \right) \left( \frac{2401}{2413} \right) = 238 \text{ N/mm}^2 \]

For design crack width 0.3 mm:

Maximum allowable bar spacing = 200 mm

Actual bar spacing 1 = \[ \frac{[2300-2(75)-16]}{17} = 131 \text{ mm} < 200 \text{ mm} \]

Actual bar spacing 2 = \[ \frac{[1500-2(75)-16]}{11} = 121 \text{ mm} < 200 \text{ mm} \]

Cracking OK
Example 5: Design of Pile Cap – Beam Theory

Detailing

18H16

12H16

4H12 Binders

h = 1100