Chapter 7
Design of Piled Foundations

7.0 NOTATION

\( a \)  
Deflection due to slenderness of a circular pile

\( a_r \)  
Distance of shear plane from nearest support

\( a_x \)  
Deflection due to slenderness producing additional moment about \( x \)-axis

\( a_y \)  
Deflection due to slenderness producing additional moment about \( y \)-axis

\( A_c \)  
Net area of concrete in a pile cross-section

\( A_p \)  
Cross-sectional area of pile (\( m^2 \))

\( A_s \)  
Surface area of pile in contact with soil

\( A_v \)  
Total area of link bars perpendicular to longitudinal bars

\( A_{sc} \)  
Total area of steel reinforcement in a pile

\( A_{st} \)  
Area of tensile reinforcement in pile cap

\( A_{sv} \)  
Area of steel effective in resisting shear in a pile

\( A_{st} \)  
Area of tensile steel in a pile section resisting moment about \( b \)-axis

\( A_{sy} \)  
Area of tensile steel in a pile section resisting moment about \( h \)-axis

\( b \)  
Width of reinforced concrete section

\( b' \)  
Overall dimension of rectangular pile section

\( b'' \)  
Effective depth of tensile reinforcement in \( b \) direction

\( B \)  
Width or diameter of pile

\( B \)  
Overall width of a group of piles

\( c \)  
Soil cohesion for a stratum (kN/m^2)

\( C_H \)  
Horizontal load-carrying capacity of a single pile

\( C_V \)  
Vertical load-carrying capacity of a single pile

\( d \)  
Effective depth to tensile reinforcement in a concrete section

\( D \)  
Depth of a group of piles below ground

\( D_r \)  
Relative density

\( e_x \)  
Eccentricity of combined unfactored vertical load on pile cap in \( x \)-direction

\( e_y \)  
Eccentricity of combined unfactored vertical load on pile cap in \( y \)-direction

\( e_{hx} \)  
Eccentricity in \( x \)-direction of combined unfactored horizontal load \( H_x \)

\( e_{hy} \)  
Eccentricity in \( y \)-direction of combined unfactored horizontal load \( H_y \)

\( E_t \)  
Stress–strain modulus of pile material (kN/m^2)

\( E_s \)  
Stress–strain modulus of soil (kN/m^2)

\( f_c \)  
Stress in concrete due to prestress alone

\( f_s \)  
Skin resistance at soil/pile interface

\( f_t \)  
Maximum design principal tensile stress in concrete

\( f_y \)  
Characteristic yield strength of steel reinforcement

\( f_{ci} \)  
Cube strength of concrete at transfer of prestress
$f_{cp}$ Average concrete stress in a prestressed concrete section after losses
$f_{cu}$ Characteristic cube strength of concrete at 28 days
$f_{pe}$ Average tensile stress in steel tendons after all losses
$f_{pu}$ Characteristic ultimate strength of steel tendons
$f_{yv}$ Characteristic yield strength of shear reinforcement
$h$ Overall depth of pile cap
$h$ Overall dimension of a rectangular pile
$h$ Overall diameter of a circular pile
$h'$ Effective depth of tensile reinforcement in a rectangular pile in $h$-direction
$H$ Unfactored horizontal load on a single circular pile
$H_x$ Unfactored combined horizontal loads on pile cap in $x$-direction
$H_y$ Unfactored combined horizontal loads on pile cap in $y$-direction
$H_{px}$ Unfactored horizontal load on a single pile in $x$-direction
$H_{py}$ Unfactored horizontal load on a single pile in $y$-direction
$H_{xu}$ Ultimate horizontal load on pile cap in $x$-direction
$H_{yu}$ Ultimate horizontal load on pile cap in $y$-direction
$H_{pux}$ Ultimate horizontal load on a single pile in $x$-direction
$H_{pyu}$ Ultimate horizontal load on a single pile in $y$-direction
$I_1$ Moment of inertia of pile (m$^4$)
$I_z$ Polar moment of inertia of a group of piles about $z$-axis through CG
$I_{xx}$ Moment of inertia of a group of piles about $x-x$ axis through CG of group
$I_{yy}$ Moment of inertia of a group of piles about $y-y$ axis through CG of group
$k_s$ Modulus of subgrade reaction of soil (kN/m$^3$)
$K_s$ Coefficient of friction
$K_t$ Factor used to determine transmission length of prestressing wires or strand
$L_c$ Effective length of pile for calculation of slenderness ratio
$L_o$ Unsupported length of pile
$L_t$ Transmission length of prestressing wires or strands
$L$ Depth of penetration of pile
$L$ Overall length of a group of piles
$L_b$ Average depth of pile in ground
$m$ Modular ratio $E_s/E_c$
$m_v$ Coefficient of volume compressibility (m$^2$/kN)
$M$ Factored bending moment in a circular pile section
$M_o$ Moment to produce zero stress at tension fibre of a prestressed section with 0.8$f_{cp}$ (average uniform prestress)
$M_p$ Unfactored bending moment in a single circular pile
$M_{px}$ Unfactored combined moment on pile cap about $x$-axis
$M_{py}$ Unfactored combined moment on pile cap about $y$-axis
$M'_{px}$ Modified bending moment about $x$-axis to account for biaxial bending
$M'_{py}$ Modified bending moment about $y$-axis to account for biaxial bending
$M'^*_{px}$ Unfactored moment about $x$-axis due to eccentric surcharge on pile cap
$M'^*_{py}$ Unfactored moment about $y$-axis due to eccentric surcharge on pile cap
$M_{px}$ Unfactored bending moment in a single pile about $x$-axis due to $H_{px}$
$M_{py}$ Unfactored bending moment in a single pile about $y$-axis due to $H_{py}$
$M_{xx}$ Unfactored combined moment on pile group about $x$-axis
$M_{yy}$ Unfactored combined moment on pile group about $y$-axis
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{p_{ru}}$</td>
<td>Ultimate bending moment in pile about x-axis</td>
</tr>
<tr>
<td>$M_{p_{uy}}$</td>
<td>Ultimate bending moment in pile about y-axis</td>
</tr>
<tr>
<td>$M_{add,x}$</td>
<td>Additional bending moment in pile about x-axis due to slenderness</td>
</tr>
<tr>
<td>$M_{add,y}$</td>
<td>Additional bending moment in pile about y-axis due to slenderness</td>
</tr>
<tr>
<td>$n$</td>
<td>Slenderness ratio in a prestressed pile</td>
</tr>
<tr>
<td>$N$</td>
<td>Statistical average of SPT number for a soil stratum</td>
</tr>
<tr>
<td>$N$</td>
<td>Combined vertical load on pile cap — unfactored</td>
</tr>
<tr>
<td>$N_q$</td>
<td>Soil bearing capacity coefficient as per Terzaghi</td>
</tr>
<tr>
<td>$N_u$</td>
<td>Ultimate vertical load on a circular pile</td>
</tr>
<tr>
<td>$N_r$</td>
<td>Soil bearing capacity coefficient as per Terzaghi</td>
</tr>
<tr>
<td>$N'_c$</td>
<td>Adjusted bearing capacity factor for cohesion</td>
</tr>
<tr>
<td>$N'_q$</td>
<td>Adjusted bearing capacity factor for $L/B &gt; 1$</td>
</tr>
<tr>
<td>$N_{uz}$</td>
<td>Design ultimate capacity of a concrete section subjected to axial load only</td>
</tr>
<tr>
<td>$N_{bal}$</td>
<td>Design axial load capacity of a balanced section ($= 0.25 f_{cb}bd$)</td>
</tr>
<tr>
<td>$p$</td>
<td>Percentage of tensile reinforcement in a circular pile</td>
</tr>
<tr>
<td>$p_s$</td>
<td>Percentage of tensile reinforcement in a pile section to resist bending about x-axis</td>
</tr>
<tr>
<td>$p_y$</td>
<td>Percentage of tensile reinforcement in a pile section to resist bending about y-axis</td>
</tr>
<tr>
<td>$P$</td>
<td>Total vertical load on a group of piles</td>
</tr>
<tr>
<td>$P_a$</td>
<td>Allowable unfactored vertical load on pile</td>
</tr>
<tr>
<td>$P_u$</td>
<td>Ultimate axial compressive load on pile</td>
</tr>
<tr>
<td>$P_{pu}$</td>
<td>End-bearing resistance of pile</td>
</tr>
<tr>
<td>$P_{si}$</td>
<td>Skin friction resistance of pile</td>
</tr>
<tr>
<td>$q$</td>
<td>Effective vertical stress at pile point</td>
</tr>
<tr>
<td>$q_e$</td>
<td>Statistical average of cone resistance of soil in a stratum (kN/m²)</td>
</tr>
<tr>
<td>$q_u$</td>
<td>Unconfined compressive strength (kN/m²)</td>
</tr>
<tr>
<td>$q_{es}$</td>
<td>Side friction resistance in a cone penetrometer</td>
</tr>
<tr>
<td>$R$</td>
<td>Number of piles in a group</td>
</tr>
<tr>
<td>$R_{H}$</td>
<td>Initial estimate of number of piles based on total horizontal load</td>
</tr>
<tr>
<td>$R_{V}$</td>
<td>Initial estimate of number of piles based on total vertical load</td>
</tr>
<tr>
<td>$s$</td>
<td>Spacing of nodes in pile for finite element analysis</td>
</tr>
<tr>
<td>$S_a$</td>
<td>Spacing of links used as shear reinforcement</td>
</tr>
<tr>
<td>$T$</td>
<td>Unfactored torsion on a group of piles</td>
</tr>
<tr>
<td>$T_a$</td>
<td>Allowable unfactored tension load on pile</td>
</tr>
<tr>
<td>$T_u$</td>
<td>Ultimate axial tensile load on pile</td>
</tr>
<tr>
<td>$U$</td>
<td>Perimeter at punching shear plane in a pile cap</td>
</tr>
<tr>
<td>$v$</td>
<td>Shear stress in concrete in pile cap</td>
</tr>
<tr>
<td>$v_e$</td>
<td>Design concrete shear stress in concrete</td>
</tr>
<tr>
<td>$v_x$</td>
<td>Shear stress in concrete for shear due to bending about x-axis</td>
</tr>
<tr>
<td>$v_y$</td>
<td>Shear stress in concrete for shear due to bending about y-axis</td>
</tr>
<tr>
<td>$v_c$</td>
<td>Modified design shear stress to take into account axial compression</td>
</tr>
<tr>
<td>$v_{ex}$</td>
<td>Design shear stress in concrete for shear due to bending about x-axis</td>
</tr>
<tr>
<td>$v_{ey}$</td>
<td>Design shear stress in concrete for shear due to bending about y-axis</td>
</tr>
<tr>
<td>$V$</td>
<td>Ultimate shear force in a circular pile section</td>
</tr>
<tr>
<td>$V_z$</td>
<td>Shear resistance of a concrete section</td>
</tr>
<tr>
<td>$V_{co}$</td>
<td>Shear resistance of uncracked prestressed section</td>
</tr>
<tr>
<td>$V_{cr}$</td>
<td>Shear resistance of cracked prestressed section</td>
</tr>
</tbody>
</table>
$W$  Weight of pile (kN)
$z$  Depth of lever arm

$\alpha$  Coefficient for calculation of skin resistance of a pile
$\beta$  Factor for computation of effective length of a pile
$\beta'$  Factor for conversion of biaxial bending moment into uniaxial bending
$\gamma$  Unit weight of soil (kN/m$^3$)
$\delta$  Angle of friction between soil and concrete
$\mu$  Poisson’s ratio
$\phi$  Angle of internal friction
$\phi'$  Nominal diameter of tendon in prestressed concrete section

7.1 VERTICAL LOAD – SINGLE PILE CAPACITY

$P_u = P_{pu} + \Sigma P_{si} - W$
$T_u = \Sigma P_{si} + W$

where $P_u =$ ultimate compressive load on pile
$T_u =$ ultimate tensile load on pile
$\Sigma P_{si} =$ skin friction resistance
$P_{pu} =$ end-bearing resistance
$W =$ weight of pile

First method for point resistance

$P_{pu} = A_p (38N) \left( \frac{L_b}{B} \right) \leq 380N \left( A_p \right)$  (see Reference 6, page 602)

where $A_p =$ cross-sectional area of pile (m$^2$)
$N =$ statistical average of the SPT number in a zone of about $8B$
above to $3B$ below the pile point
\[ B = \text{width or diameter of pile} \]
\[ L_\theta = \text{average depth of pile in the ground} \]

**Second method for point resistance**

\[ P_{pu} = A_p q_c \]  
(see Reference 6, page 602)

where  
\[ A_p = \text{cross-sectional area of pile (m}^2) \]
\[ q_c = \text{statistical average of cone point resistance in a zone of about} \]
\[ 8B \text{ above to } 3B \text{ below pile point (kN/m}^2) \]

**Third method for point resistance**

\[ P_{pu} = A_p (N'_c c + \tilde{q} N'_q) \]  
(see Reference 6, page 598)

where  
\[ A_p = \text{cross-sectional area of pile (m}^2) \]
\[ c = \text{cohesion or undrained shear strength } S_u = q_u/2 \text{kN/m}^2 \]
\[ q_u = \text{unconfined compressive strength} \]
\[ \tilde{q} = \text{effective vertical stress at pile point} \]
\[ N'_c = \text{adjusted bearing capacity factor for cohesion (see Fig. 7.2)} \]
\[ N'_q = \text{bearing capacity factor adjusted for } L/b > 1 \text{ dependent on initial angle of shearing resistance } \phi \text{ (see Fig. 7.2). (See Reference 8, page 600.)} \]
\[ L = \text{depth of penetration} \]
\[ B = \text{width or diameter of pile} \]

$L/B$ should be greater than $L_c/B$ as obtained from Fig. 7.2 for the value of $\phi$.

**Note:**  
Find point resistance by more than one method if soil test data allow and take the lowest for a conservative estimate.

**Determination of skin resistance**

\[ \Sigma P_{si} = \Sigma A_s f_s \]

where  
\[ A_s = \text{pile perimeter } \times \text{pile length over which } f_s \text{ acts (m}^2) \]
\[ f_s = \text{skin resistance (kN/m}^2) \]

**First method of skin resistance**

\[ f_s = 2N_k \text{kN/m}^2 \quad \text{for large volume displacement piles} \]
\[ f_s = N_k \text{kN/m}^2 \quad \text{for small volume displacement piles} \]

where  
\[ N = \text{statistical average blow count in stratum for SPT}. \]

**Second method of skin resistance**

\[ f_s = 0.005 q_c \text{kN/m}^2 \]

where  
\[ q_c = \text{cone penetration resistance (kN/m}^2). \]

**Third method of skin resistance**

\[ f_s = q_{ck} \text{kN/m}^2 \quad \text{for small volume displacement piles} \]
\[ f_s = 1.5q_{cs} \text{ to } 2.0q_{cs} \] for large volume displacement piles
where \( q_{cs} \) = side friction resistance in cone penetrometer.

**Fourth method of skin resistance**

\[ f_s = \alpha c + 0.5 \bar{q} K_s \tan \delta \] (see Reference 8, page 603)
where \( c \) = average cohesion or \( S_u \) of stratum (kN/m²)
\( \bar{q} \) = effective vertical stress (kN/m²)
\( \delta \) = angle of friction between soil and pile
\( K_s \) = coefficient of friction
\( D_r \) = relative density of sand.

**Table 7.1 Values of \( K_s \) (Reference 8, page 603).**

<table>
<thead>
<tr>
<th>Pile type</th>
<th>( \delta )</th>
<th>( K_s ) for low ( D_r )</th>
<th>( K_s ) for high ( D_r )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>20°</td>
<td>0.5</td>
<td>1.0</td>
</tr>
<tr>
<td>Concrete</td>
<td>0.75( \phi )</td>
<td>1.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Wood</td>
<td>0.67( \phi )</td>
<td>1.5</td>
<td>4.0</td>
</tr>
</tbody>
</table>

(See Reference 7, page 136.)

**Table 7.2 Values of \( \alpha \) (Reference 7, page 126).**

<table>
<thead>
<tr>
<th>Soil condition</th>
<th>( D/B )</th>
<th>( c = 50 )</th>
<th>( c = 100 )</th>
<th>( c = 150 )</th>
<th>( c = 200 )</th>
<th>( c = 250 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sands or sandy gravel</td>
<td>&lt;10</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>overlying stiff to very stiff cohesive soil</td>
<td>&gt;40</td>
<td>0.9</td>
<td>0.65</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
</tr>
<tr>
<td>Soft clays or silts</td>
<td>10</td>
<td>0.35</td>
<td>0.30</td>
<td>0.25</td>
<td>0.2</td>
<td>0.2</td>
</tr>
<tr>
<td>overlying stiff to very stiff cohesive soil</td>
<td>&gt;20</td>
<td>0.75</td>
<td>0.70</td>
<td>0.63</td>
<td>0.55</td>
<td>0.5</td>
</tr>
<tr>
<td>Stiff to very stiff cohesive soils without overlying strata</td>
<td>10</td>
<td>0.9</td>
<td>0.7</td>
<td>0.3</td>
<td>0.2</td>
<td>0.2</td>
</tr>
<tr>
<td>Stiff to very stiff cohesive soils with overlying strata</td>
<td>&gt;40</td>
<td>1.0</td>
<td>0.9</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
</tr>
</tbody>
</table>

The units of \( c \) are kN/m²

**Note:** Find skin resistance by more than one method if soil test data allow and take an average.

\[ P_s = \frac{P_u}{2.5} \quad T_s = \frac{T_u}{2.5} \]

where \( P_s \) = allowable pile load in compression
\( T_s \) = allowable pile load in tension
7.2 HORIZONTAL LOAD – SINGLE PILE CAPACITY

Method 1 Cohesive soils

\[ k_s B = 1.3 \left( \frac{E_s B^4}{E_d I_t} \right)^{1/3} \frac{E_s}{1 - \mu^2} \]

as per Vesic, 1961 (see Reference 6).

where \( k_s \) = modulus of subgrade reaction (kN/m\(^3\))
\( B \) = width or diameter of pile (m)
\( E_s \) = stress–strain modulus of soil (kN/m\(^2\))
\( E_t \) = stress–strain modulus of pile material (kN/m\(^2\))
\( I_t \) = moment of inertia of pile (m\(^4\))
\( \mu \) = Poisson's ratio of soil

\( E_s \) may be obtained by the following methods:

(1) Triaxial tests.
(2) Borehole pressuremeter tests.
(3) \( E_s = 650N(kN/m^3) \)
\( N = SPT \) number of blows.
(4) \( E_s = 3 (1 - 2\mu)/m_s \) where \( m_s \) = coefficient of volume compressibility (m\(^3\)/kN).

Method 2 Cohesive soils

\[ k_s = 240q_u \text{ kN/m}^3 \]

where \( q_u \) = unconfined compression strength (kN/m\(^2\)).

Cohesionless soils

\[ k_s = 80 \left[ C_2 \phi N_q + C_1 (0.5 \gamma BN_q) \right] \text{ kN/m}^3 \]

as per Vesic (see Reference 8, page 631 and page 323, equation 9–8).

where \( C_1 = C_2 = 1.0 \) for square piles
\( C_1 = 1.3 \) to 1.7 for circular piles
\( C_2 = 2.0 \) to 4.4 for circular piles
\( \phi \) = effective stress (kN/m\(^2\))
\( \gamma \) = unit weight of soil
\( B \) = width or diameter of pile

\( N_q \) and \( N\gamma \) may be obtained from the following table (Hansen equations) – see Reference 8, page 137, Table 4–4:

Finite element model of vertical pile

Spring stiffness = \( SB k_s \) kN/m

where \( S \) = node spacing not greater than \( B \)
\( B \) = width or diameter of pile (m)
\( k_s \) = modulus of subgrade reaction (kN/m\(^3\))
Table 7.3 Values of $N_q$ and $N_r$ (Reference 8, page 137).

<table>
<thead>
<tr>
<th>$\phi$ (degrees)</th>
<th>$N_q$</th>
<th>$N_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.0</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>1.6</td>
<td>0.1</td>
</tr>
<tr>
<td>10</td>
<td>2.5</td>
<td>0.4</td>
</tr>
<tr>
<td>15</td>
<td>3.9</td>
<td>1.2</td>
</tr>
<tr>
<td>20</td>
<td>6.4</td>
<td>2.9</td>
</tr>
<tr>
<td>25</td>
<td>10.7</td>
<td>6.8</td>
</tr>
<tr>
<td>30</td>
<td>18.4</td>
<td>15.1</td>
</tr>
<tr>
<td>35</td>
<td>33.3</td>
<td>33.9</td>
</tr>
<tr>
<td>40</td>
<td>64.2</td>
<td>79.5</td>
</tr>
<tr>
<td>45</td>
<td>134.9</td>
<td>200.8</td>
</tr>
<tr>
<td>50</td>
<td>319.0</td>
<td>568.5</td>
</tr>
</tbody>
</table>

Note: For horizontal loads which are not constant and are reversible or repetitive, the top $1.5B$ of pile may be assumed unsupported by soil.

Boundary conditions

(1) Free head pile
- Translations $x$, $y$: Free at top
- Rotation $z$: Free at top
- Translations $y$: Restrainted at bottom
- Rotation $z$: Free at bottom

(2) Fixed head pile
- Translations $x$, $y$: Free at top
- Rotation $z$: Rigid at top
- Translations $y$: Restrainted at bottom
- Rotation $z$: Free at bottom

Material type
For sustained horizontal load due to dead load, water pressure, earth
pressure, etc., use short-term Young's modulus of concrete for bending moment computations but long-term Young's modulus of concrete for pile head deformation.

For short-term horizontal loads due to wind, earthquake, crane surge, etc., use short-term Young's modulus of concrete for bending moment and deflection computations.

**Software**
Use any fully validated software which has a suite for analysis of 2-D plane frame with sprung boundaries.

**Member type**
For rectangular pile use minimum width $B$ in all computations involving $B$. A cracked section moment of inertia may be used for reinforced concrete piles based on Section 2.1.

### 7.3 PILE GROUP EFFECTS

#### 7.3.1 Spacing of piles

\[
\begin{align*}
S &\geq 2B & \text{for end-bearing piles} \\
S &\geq 3B & \text{for friction piles}
\end{align*}
\]

where \( S \) = spacing of piles
\( B \) = least width or diameter of pile.

*Note:* Piles carrying horizontal load should not be spaced at less than $3B$.

#### 7.3.2 Pile group capacity

Ultimate group capacity = group friction capacity + group end-bearing capacity

Ultimate group friction capacity = $2D(B + L)\alpha$

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**SK 7/3 Group of piles – plan of overall dimensions of group.**
where  \( c \) = average cohesion of clay
\( = \) average \( S_u = \) average \( q_u/2 \)
\( \alpha \) = coefficient (from Section 7.1, Table 7.2)
\( D \) = depth of pile group below ground
\( B \) = overall width of group
\( L \) = overall length of group.

Ultimate group end-bearing capacity = \( BL (N'_c + qN'_{u}) \)

where  \( c \) = cohesion or undrained shear strength \( S_u = q_u/2 \) at bottom of pile group
\( q_u \) = unconfined compressive strength
\( \bar{q} \) = effective stress at bottom of pile group
\( N'_c \) = bearing capacity factor (see Fig. 7.2)
\( N'_{u} \) = bearing capacity factor (see Fig. 7.2)

**Note:** Total vertical load on a group of piles should not exceed the group capacity. Individual pile loads inside the group will be limited by the single pile capacity. Piles carrying horizontal load and spaced at 3\( B \) or more need not be checked for group effects due to horizontal load.

\[
\text{Allowable group capacity} = \frac{\text{ultimate group capacity} + \text{ultimate group end-bearing capacity}}{2.5}
\]

### 7.4 ANALYSIS OF PILE LOADS AND PILE CAPS

#### 7.4.1 Rigid pile cap

\( N \) = combined vertical load on pile cap – unfactored
\( M_x = \) combined moment about \( x-x \) – unfactored
\( M_y = \) combined moment about \( y-y \) – unfactored
$H_x = \text{combined horizontal load on pile cap - unfactored in } x-x \text{ direction}$

$H_y = \text{combined horizontal load on pile cap - unfactored in } y-y \text{ direction}$

$e_x = \text{eccentricity of } N \text{ from CG of pile group in } x-x \text{ direction}$

$e_y = \text{eccentricity of } N \text{ from CG of pile group in } y-y \text{ direction}$

$e_{hx} = \text{eccentricity of } H_x \text{ from CG of pile group in } x-x \text{ direction}$

$e_{hy} = \text{eccentricity of } H_y \text{ from CG of pile group in } y-y \text{ direction}$

$h = \text{depth of pile cap}$

**Loads on pile group**

$P = \text{vertical load on pile group}$

$= N + \text{weight of pile cap} + \text{weight of backfill on pile cap} + \text{surcharge}$

$= M_x + Ne_y + H_x h + M_{xx}^*$

$M_{yy} = \text{moment about } y-y \text{ on pile group}$

$= M_y + Ne_x + H_y h + M_{yy}^*$
where $M_r^x$ and $M_r^y$ are moments with respect to CG of pile group due to eccentric surcharge on backfill or pile cap.

$T = \text{torsion on pile group}$

$= H_r e_{hy} + H_r e_{hx}$

$I_{xx} = \Sigma y^2$ about $x-x$ axis passing through CG of pile group

$I_{yy} = \Sigma z^2$ about $y-y$ axis passing through CG of pile group

$I = I_{xx} + I_{yy}$

$R = \text{number of piles in group.}$

Vertical load on a pile = \( \left( \frac{P}{R} \right) \pm \left( \frac{M_{xx} y}{I_{xx}} \right) \pm \left( \frac{M_{yy} x}{I_{yy}} \right) \)
Horizontal load on any pile = resultant of \( \frac{(H^2_x + H^2_y)^{\frac{1}{2}}}{R} \) and \( \frac{T(x^2 + y^2)^{\frac{1}{2}}}{I_z} \)

**Sign convention**
- Vertical loads: downwards positive
- Torsion on pile group: clockwise positive
- Moments on pile group: clockwise positive
- \( +ve \ M_{xy} \) produces compression in piles which have \( +ve \ y \) ordinates.
- \( +ve \ M_{xy} \) produces compression in piles which have \( +ve \ X \) ordinates.
- \( H_x \) is positive in direction of increasing \( x \) in positive direction.
- \( H_y \) is positive in direction of increasing \( y \) in positive direction.
- Eccentricities are \( +ve \) for \( +ve \ X \) and \( +ve \) for \( +ve \ y \).

**Bending moments in pile cap**

![Diagram](image)

SK 7/9 Critical sections for bending moment in a pile cap.

Take sections \( X - X \) or \( Y - Y \) through pile cap at faces of columns or base plates. Find pile reactions due to combined and load factored basic load cases. Consider all upward and downward loadings across sections \( X - X \) and \( Y - Y \). Find bending moments across section. Find horizontal load on each pile by using the following expressions:

\[
H_{puu} = \frac{H_{pyu}}{R}
\]

\[
H_{pyu} = \frac{H_{yu}}{R}
\]

where \( R \) is number of piles in pile cap. Find bending moments in pile \( M_{puu} \) corresponding to \( H_{puu} \) and \( M_{pyu} \) corresponding to \( H_{pyu} \) assuming an end fixity to pile cap following the method in Section 7.2. \( H_m \) and \( H_{yu} \) are combined factored ultimate horizontal loads.
Algebraically add the bending moments in pile cap due to vertical load and pile fixity moments due to horizontal load to find design bending moments in pile cap.

7.4.2 Flexible pile cap

Large pile caps including piled raft foundations should be modelled as flexible. The modelling will normally be carried out using either a grillage suite of a computer program or a general-purpose finite element program. The piles should be modelled as springs in the vertical direction. The vertical spring stiffness should be obtained from test results on site. A parametric study can be carried out using minimum and maximum stiffness of the pile if there is a large variation.

**Grillage model**

(1) Divide pile cap into an orthogonal grillage network of beams. Ensure that piles are located at crossing of orthogonal beams. Each grillage beam represents a certain width of pile cap.
(2) Use short-term Young's modulus for concrete material properties.
(3) Full section concrete stiffness properties may be used for hypothetical grillage beams (hypothetical width × depth of pile cap).
(4) Piles will be modelled as sprung supports vertically.
(5) Vertical loads on pile cap may be dispersed at 45° up to center depth of pile cap.
(6) Apply at each node with a pile, the moments given by the following formulae:

\[ M_x = \frac{H_f h}{R} \quad \text{about } x\text{-axis} \]
SK 7/11 Plan of raft on piles showing idealised grillage elements – flexible analysis.

SK 7/12 Part section through raft showing details of grillage idealisation.

\[ M_y = \frac{H_y h}{R} \quad \text{about y-axis} \]

(7) Find horizontal load on each pile by using the following expressions:

\[ H_{px} = \frac{H_x}{R} \quad \text{and} \quad H_{py} = \frac{H_y}{R} \]

where \( R \) is total number of piles in group.
(8) Find bending moments in pile, $M_{px}$ corresponding to $H_{px}$ and $M_{py}$ corresponding to $H_{py}$, assuming an end fixity to pile cap following method in Section 7.2. Apply these moments to pile cap grillage model as nodal loads. The pile head to pile cap connection may be assumed as hinged and then $M_{px}$ and $M_{py}$ will be zero.

(9) Find bending moments in pile cap by grillage analysis. Divide bending moments by width of hypothetical strips of pile cap representing grillage beams and obtain $M_x$, $M_y$ and $M_{xy}$ in pile cap per metre width. Apply load factors and combine basic load cases. Modify these combined moments by Wood–Armer method to find design bending moments.\[^{[11,12]}\]

(10) Combine basic load cases at serviceability limit state to find reactions at pile nodes. Compare maximum reaction with pile capacity.

**Finite-element model**

![Finite-element model](sk713.png)

SK 7/13 Typical finite element modelling of a circular raft on piles.

(1) Create a finite element model of pile cap using either 4-noded or 8-noded plate bending elements. The elements may only have three degrees of freedom at each node viz. $z$, $\theta x$ and $\theta y$. The piles will be represented by vertical springs.

Piles will come at nodes in finite element model. Between two piles' nodes there should be a minimum of one plate node without pile.

(2) Use short-term Young's modulus for concrete material properties.

(3) Full section concrete section properties may be used in the analysis.

(4) Vertical loads on pile cap may be dispersed at $45^\circ$ up to central depth of pile cap. These loads may be applied as nodal loads or uniformly distributed loads on plate elements depending on software used.

(5) Apply at each node with a pile, the moments given by the following formulae.

$$M_x = \frac{H_x h}{R} \quad \text{about } x\text{-axis}$$

$$M_y = \frac{H_y h}{R} \quad \text{about } y\text{-axis}$$
(6) Find horizontal load on each pile by using the following expressions:

\[ H_{px} = \frac{H_x}{R} \quad \text{and} \quad H_{py} = \frac{H_y}{R} \]

where \( R \) is total number of piles in group.

(7) Find bending moments in pile, \( M_{px} \) corresponding to \( H_{py} \) and \( M_{py} \) corresponding to \( H_{px} \), assuming an end fixity to pile cap following method in Section 7.2. Apply these moments as nodal loads in finite element model at nodes with piles. These moments will be zero in the case of a hinged connection of pile to pile cap.

(8) Carry out analysis using a validated general-purpose finite element software. Apply load factors to combine basic load cases. Modify the combined \( M_x \), \( M_y \), and \( M_z \) using the Wood–Armer method to find design bending moments.\[11,12\]

(9) Combine basic load cases at serviceability limit state to find reactions at pile nodes. Compare maximum reaction with rated pile capacity.

7.5 LOAD COMBINATIONS

Applied loads on pile cap will be combined using the following principles.

7.5.1 Pile load calculations

\[ L_{C1}: \quad 1.0DL + 1.0IL + 1.0EP + 1.0CLV + 1.0CLH \]
\[ L_{C2}: \quad 1.0DL + 1.0EP + 1.0CLV + 1.0CLH + 1.0WL \quad \text{(or 1.0EL)} \]
\[ L_{C3}: \quad 1.0DL + 1.0IL + 1.0EP + 1.0WL \quad \text{(or 1.0EL)} \]
\[ L_{C4}: \quad 1.0L + 1.0WL \quad \text{(or 1.0EL)} \]

where

\( DL \) = dead load
\( IL \) = imposed load
\( EP \) = earth pressure and water pressure
\( CLV \) = crane vertical loads
\( CLH \) = crane horizontal loads
\( WL \) = wind load
\( EL \) = earthquake load.

7.5.2 Bending moment and shear calculations in pile cap or piles

\[ L_{C5}: \quad 1.4DL + 1.6IL + 1.4EP \]
\[ L_{C6}: \quad 1.2DL + 1.2IL + 1.2EP + 1.2WL \quad \text{(or 1.2EL)} \]
\[ L_{C7}: \quad 1.4DL + 1.4WL \quad \text{(or 1.4EL)} + 1.4EP \]
\[ L_{C8}: \quad 1.0DL + 1.4WL \quad \text{(or 1.4EL)} + 1.4EP \quad \text{(if adverse)} \]
\[ L_{C9}: \quad 1.4DL + 1.4CLV + 1.4CLH + 1.4EP \]
\[ L_{C10}: \quad 1.4DL + 1.6CLV + 1.4EP \]
\[ L_{C11}: \quad 1.4DL + 1.6CLH + 1.4EP \]
\[ L_{C12}: \quad 1.2DL + 1.2CLV + 1.2CLH + 1.2EP + 1.2WL \quad \text{(or 1.2EL)} \]
7.6 STEP-BY-STEP DESIGN PROCEDURE FOR PILED FOUNDATIONS

Step 1 Select type of pile
The type of pile will depend on the following principal factors:

- Environmental issues like noise, vibration.
- Location of structure.
- Type of structure.
- Ground conditions.
- Durability requirements.
- Programme duration.
- Cost.

The commonly available types of piles can be broadly classified as below.

Large-displacement piles (driven)

- Precast concrete.
- Prestressed concrete.
- Steel tube with closed end.
- Steel tube filled with concrete.

Small-displacement piles (driven)

- Precast concrete tube with open end.
- Prestressed concrete tube with open end.
- Steel H-section.
- Screw pile.

Non-displacement piles

- Bored and cast-in-situ concrete pile.
- Steel tube in bored hole filled with concrete.
- Steel or precast section in drilled hole.

Step 2 Determine vertical capacity of single pile
Follow Section 7.1.

Step 3 Determine horizontal capacity of single pile
Follow Section 7.2.

Note: Horizontal capacity of a single pile is limited by maximum deflection of pile cap that structure can accommodate and also by pile structural capacity.

Step 4 Determine approximate number of piles and spacing

\[ R_{IV} = \frac{P}{C_V} \]
\[ R_{iH} = \frac{H}{C_H} \]

\[ R_i = R_{iV} \text{ or } R_{iH}, \text{ whichever is greater} \]

where \( R_i \) = approximate number of piles
\( P = \text{total vertical load on pile cap - unfactored} \)
\( C_V = \text{rated working load capacity of pile - vertical load} \)
\( C_H = \text{rated working load capacity of pile - horizontal load} \)
\( H = \text{total horizontal load on pile cap - unfactored} \)
\[ = (H_x^2 + H_y^2)^{\frac{1}{2}} \]

Spacing of piles should be according to Section 7.3. To minimise the cost of pile cap, the spacing should be kept close to minimum allowed. Larger spacing increases the pile group capacity and pile group moment capacity.

---

SK 7/14 Determination of approximate number of piles.

1. Select a group of piles with approximate number of piles = \( R_i \).
2. Find CG of pile group and locate orthogonal axes \( x-x \) and \( y-y \) through the CG.
3. Find CG of group of piles on left of axis \( y-y \) and right of axis \( y-y \).
4. Find the \( x \)-axis distance between these two CGs and call it \( S_x \).
5. Similarly, find \( S_y \) about \( y \)-axis.
6. Find \( M_x/P = e_x \) and \( M_y/P = e_y \), where \( M_x \) and \( M_y \) are total combined applied moments on pile cap about \( x-x \) and \( y-y \) respectively.
7. Find \( e_x/S_x \) and \( e_y/S_y \).
8. Find \( E_x \) and \( E_y \) from Fig. 7.1.
9. \[ R = \frac{1.1 \cdot R_{iV}}{E_x E_y} \geq R_{iH} \]

where \( R \) = number of piles in group for checking pile load.

**Note:** The factor 1.1 is introduced to cater for additional vertical loads from self-weight of pile cap, surcharge on pile caps, backfilling, etc.

Revise the number of piles in group from \( R_i \) to \( R \).
Step 5 **Determine size of pile cap**
Allow 1.5B from centre of pile to edge of pile cap. Depth of pile cap is governed by the following:

- Shrinking and swelling of clay.
- Frost attacks.
- Holding down bolt assemblies for columns.
- Water table and soluble sulphates.
- Pile anchorage.
- Punching shear capacity of pile cap.

Step 6 **Carry out load combination**
Follow Section 7.5.

Step 7 **Check pile group effects**
Follow Section 7.3.

Step 8 **Carry out analysis of pile cap**
Follow Section 7.4.

Step 9 **Determine cover to reinforcement**
From the soils investigations report, find the concentration of sulphates expressed as SO₃.
Find, from Table 17 of BS 8004: 1986[3], the appropriate type of concrete.

<table>
<thead>
<tr>
<th>Class of exposure</th>
<th>Total SO₃ percentage</th>
<th>Minimum cover on blinding (mm)</th>
<th>Minimum cover elsewhere (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>&lt;0.2</td>
<td>35</td>
<td>75</td>
</tr>
<tr>
<td>2</td>
<td>0.2 to 0.5</td>
<td>40</td>
<td>80</td>
</tr>
<tr>
<td>3</td>
<td>0.5 to 1.0</td>
<td>50</td>
<td>90</td>
</tr>
<tr>
<td>4</td>
<td>1.0 to 2.0</td>
<td>60</td>
<td>100</td>
</tr>
<tr>
<td>5</td>
<td>&gt;2.0</td>
<td>60</td>
<td>100</td>
</tr>
</tbody>
</table>

**Note:** Concrete in ‘class of exposure 5’ needs protective membrane, or coating. The uneven heads of piles normally necessitate a minimum 75 mm cover over blinding for pile caps. The concrete piles will have minimum cover as specified elsewhere.

Step 10 **Calculate area of reinforcement in pile cap**

\[ M = \text{bending moment as found in Step 8 at ultimate limit state} \]

\[ K = \frac{M}{f_{cu}bd^2} \leq 0.156 \]

where \( f_{cu} \) = concrete characteristic cube strength at 28 days
\[ b = \text{width of section over which moment acts} \]
\[ d = \text{effective depth to tension reinforcement} \]

If \( K \) is greater than 0.156, increase depth of pile cap.

\[ A_{st} = \frac{M}{0.87f_y} \]

\[ z = d \left[ 0.5 + \sqrt{\left(0.25 - \frac{K}{0.9}\right)} \right] \leq 0.95d \]

Distribute this area of reinforcement uniformly across the section.

**Note:** The effective depth to tension reinforcement will be different in the two orthogonal directions.

**Step II** Check shear stress in pile cap

![Critical section for checking shear stress in pile cap](image)

The critical section for checking shear stress in a pile cap is \( \phi/5 \) into the pile. All piles with centres outside this line should be considered for calculating shear across this section in pile cap. For shear enhancement, \( a_v \) is from face of column to this critical section. No enhancement of shear stress is allowed if \( a_v \) is greater than \( 1.5d \). Where pile spacing is more than \( 3\phi \) then enhancement of shear should be applied only on strips of width \( 3\phi \). The rest of the section will be limited to unenhanced shear stress.

\[ V = \frac{\Sigma P}{Bd} \leq v_e \quad \text{or enhanced} \ v_{et} \text{ if applicable} \]

where \( \Sigma P = \text{sum of all pile reactions at ultimate loading on left of section} \)

\[ B = \text{width of pile cap at critical section} \]
\[ d = \text{average effective depth at critical section} \]

\[ v_{c1} = \frac{v_c (2d)}{a_v} \leq 0.8f_{cu} \text{ or } 5 \text{ N/mm}^2 \]

For rectangular piles the critical section may be considered at face of pile.

The value of \( v_{c1} \) can be found from Figs 11.2 to 11.5 depending on percentage of tensile reinforcement and \( f_{cu} \).

Shear capacity of section should be greater than or equal to applied shear. Ultimate limit state analysis results should be used for checking shear capacity.

**Step 12** Check punching shear stress in pile cap

SK 7/17 Perimeters for punching shear checks.
When the spacing of piles is greater than 3 times the diameter of a pile then the punching shear plane for column should be considered. For rectangular piles the plane can be considered at face of pile. The stress on this punching shear plane should not exceed \( v_c \) depending on the percentage of tensile reinforcement in pile cap.

Check of punching shear stress is also required at perimeter at face of column or pile. This shear stress should not exceed \( 0.8/v_{cu} \) or 5 N/mm\(^2\).

SK 7/18 Further perimeters for punching shear checks in a pile cap.

The punching shear planes for piles will depend on location of pile with respect to edge of pile cap.

Find the perimeter \( U \) at punching shear plane.

\[
\nu = \frac{P}{Ud} \leq v_c
\]

where \( P \) = ultimate vertical column load or ultimate vertical pile reaction

\( v_c \) = design concrete shear stress obtained from Figs 11.2 to 11.5.

Percentage area of tensile reinforcement for computation of design concrete shear stress will be average percentage across punching shear planes.

**Step 13** Check area of reinforcement in pile

Effective length of pile, \( l_e = \beta l_o \)

where \( l_o \) = unsupported length of pile (piles which are not subjected to horizontal load may be assumed fully supported by ground from ground level; piles subjected to horizontal load may be assumed supported by ground at a depth of 1.5b below ground level where \( b \) is width of pile or diameter of pile)
\[ \beta = \begin{cases} 1.2 & \text{for piles with head fixed to pile cap} \\ 1.6 & \text{for piles with head free to rotate.} \end{cases} \]

**Rectangular piles**

![Rectangular pile diagram](image)

**SK 7/19** Typical section through a rectangular pile.

(A) If \( l_e/b \leq 10 \), then treat piles as a short column.

(i) **Pile with no moment**

\[ N = 0.4f_{cu}bh + 0.75A_{sc}f_y \]

Check \( N \geq \) applied direct load on pile.

(ii) **Pile subjected to uniaxial moment**

Find \( e = M/N \) and then \( e/lh \).

Find \( N/lh \) and select appropriate table from Tables 11.8 to 11.17 depending on \( f_{cu} \) and \( k = l/lh \).

From appropriate table find \( p \) which satisfies value of \( N/lh \) for given \( e/lh \).

Find \( A_{sc} = pbh/100 \).

Put \( A_{sc}/2 \) on each face of pile equidistant from axis of moment.

**Note:** The moment \( M \) in pile is due to horizontal load as obtained in Step 3 following Section 7.2.

(iii) **Pile subjected to biaxial moment**

Assuming diameter of reinforcement and finding cover from Step 9, find \( h' \) and \( b' \).

Find \( M_e/h' \) and \( M_e/b' \).

If \( M_e/h' > M_e/b' \), then

\[ M'_e = M_e + \beta M_y \left( \frac{h'}{b'} \right) \]

If \( M_e/b' > M_e/h' \), then

\[ M'_e = M_e + \beta M_x \left( \frac{b'}{h'} \right) \]
Find $N/f_{cu}bh$. The values of $\beta$ are given in the table below.

<table>
<thead>
<tr>
<th>$N/f_{cu}bh$</th>
<th>0</th>
<th>0.1</th>
<th>0.2</th>
<th>0.3</th>
<th>0.4</th>
<th>0.5</th>
<th>(\geq 0.6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\beta$</td>
<td>1.00</td>
<td>0.88</td>
<td>0.77</td>
<td>0.65</td>
<td>0.53</td>
<td>0.42</td>
<td>0.30</td>
</tr>
</tbody>
</table>

Design as uniaxial bending with $N$ and $M_x$ or $M_y$ whichever is more prominent. Find $A_{sc}$ in manner described in (ii) for pile subjected to uniaxial moment.

(B) If $l_e/b > 10$, then treat pile as a slender column.

$$a_x = \frac{1}{2000} \left( \frac{l_e}{h} \right)^2 hK$$

$$a_y = \frac{1}{2000} \left( \frac{l_e}{b} \right)^2 bK$$

Select $A_{sc}$.

$$K = \frac{N_{uz} - N}{N_{uz} - N_{bal}} \leq 1$$

$$N_{uz} = 0.45f_{cu}A_c + 0.87f_yA_{sc}$$

$$N_{bal} = 0.25f_{cu}bh$$

$$A_c = bh - A_{sc}$$

$M_{add,x} = Na_x$

$M_{add,y} = Na_y$

Combine these additional moments with moments obtained from analysis as in Step 3 following Section 7.2. Design pile subjected to biaxial bending as described previously.

Circular piles

![Circular pile diagram](image_url)

**SK 7/20** Typical section through a circular pile. Use minimum six bars.
(A) If $l_1/h \leq 10$, then treat pile as a short column.

(i) Pile with no moment
Assume size of reinforcement and at least six bars.

$$A_c = 0.25\pi h^2 - A_{sc}$$

$$N = 0.4f_{cu}A_c + 0.75A_{sc}f_y$$

Check $N \geq$ applied vertical load on pile.

(ii) Pile with moment
Find $e = Ml/N$ and the $e/R$, where $2R = h$.
Find $N/h^2$ and select appropriate table from Tables 11.18 to 11.27 corresponding to $f_{cu}$ and $k = h_0/h$.
Find $p$ from appropriate table which satisfies $N/h^2$ for given value of $e/R$.
Find $A_{sc} = p\pi R^2/100$.
Use at least six bars.

(B) If $l_1/h > 10$, then treat pile as a slender column.

$$a = \frac{I_e^2}{2000h} \quad \text{(assume } K = 1 \text{ conservatively)}$$

$$M_{add} = Na$$

Combine this additional moment with moment obtained by analysis in Step 3 following Section 7.2. Design pile with moment as described in (ii) above.

**Step 14 Check stresses in prestressed concrete piles**

[Diagram of a typical section of a pretensioned prestressed pile.]

Stresses may be checked at the serviceability limit state only as per BS 8110: Part 1, Section 4.\[1\]

Permissible maximum compressive fibre stress in concrete = $0.4f_{cu}$

Assume pile as Class 3 member with a limiting crack width of 0.1 mm.
Hypothetical flexural tensile stress in concrete = \(4.1 \text{ N/mm}^2\)
for Grade 40
\(= 4.8 \text{ N/mm}^2\)
for Grade 50 and above

Depth factors to modify tensile stress are shown in the following table.

<table>
<thead>
<tr>
<th>Depth (mm)</th>
<th>Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Up to 400</td>
<td>1.0</td>
</tr>
<tr>
<td>500</td>
<td>0.95</td>
</tr>
<tr>
<td>600</td>
<td>0.9</td>
</tr>
</tbody>
</table>

\(N\) = direct service load on pile

\(M_{xx}\) = bending moment as obtained from Step 3 about axis \(x-x\)

\(M_{yy}\) = bending moment as obtained from Step 3 about axis \(y-y\).

Assume the pile section is uncracked.

Find \(A_c\) = area of concrete

\(I_{xx}\) = moment of inertia about \(x-x\) axis

\(I_{yy}\) = moment of inertia about \(y-y\) axis

\(P\) = residual prestress after all losses.

Maximum compressive stress in concrete = \(\left(\frac{P + N}{A_c}\right) + \left(\frac{M_{yy}}{I_{xx}}\right)\)
\[+ \left(\frac{M_{xx}}{I_{yy}}\right)\]

Maximum tensile stress in concrete = \(\left(\frac{P + N}{A_c}\right) - \left(\frac{M_{yy}}{I_{xx}}\right)\)
\[ - \left(\frac{M_{xx}}{I_{yy}}\right)\]

\(m\) = modular ratio

\(f_s\) = strand stress prior to release

\(f_c\) = stress in concrete due to prestress alone.

1. Loss due to elastic shortening = \(\left(\frac{100mf_c}{f_s}\right)\%\)

2. Loss due to relaxation of steel – refer to strand manufacturer’s brochure.

3. Loss due to creep of concrete – follow clause 4.8.5 of BS8110: Part 1.[1]

4. Loss due to shrinkage of concrete – follow clause 4.8.4 of BS8110: Part 1.[1]

Note: Prestressed piles designed as fixed to pile cap must extend into pile cap by
a minimum distance equal to transmission length given by the following equation:

\[ l_t = \frac{K_t \phi}{\sqrt{f_{cu}}} \text{ (mm)} \]

where \( f_{cu} \) = concrete cube strength at 28 days

- \( K_t = 600 \) for plain or indented wire
- \( = 400 \) for crimped wire
- \( = 240 \) for 7-wire standard or super strand
- \( = 360 \) for 7-wire drawn strand

\( \phi \) = nominal diameter of tendon.

**Step 15** Check shear capacity of RC pile

![Diagram](image)

SK 7/22 Typical section through a rectangular pile subject to biaxial bending and shear.

Ultimate limit state shear forces in pile are \( H_{pcu} \) and \( H_{pyu} \). Corresponding bending moments in pile are \( M_{pxu} \) and \( M_{pyu} \). The ultimate coexistent direct load on pile is \( N_u \).

**Rectangular piles**

*No shear check is necessary if:*

- \( M_{pxu}/N_u \leq 0.6h \)
- \( M_{pyu}/N_u \leq 0.6b \)
- \( H_{pxu}/bh' \leq 0.8\sqrt{f_{cu}} \leq 5 \text{ N/mm}^2 \)
- \( H_{pyu}/hb' \leq 0.8\sqrt{f_{cu}} \leq 5 \text{ N/mm}^2 \)

*Shear check is necessary if:*

- \( M_{pxu}/N_u > 0.6h \) and/or \( M_{pyu}/N_u > 0.6b \)

Find \( \nu_x = H_{pyu}/bh' \) and \( \nu_y = H_{pxu}/hb' \)

Find \( p_x = 100A_{sx}/bh' \) and \( p_y = 100A_{sy}/hb' \)

Find \( \nu_{cx} \) and \( \nu_{cy} \) corresponding to \( p_x \) and \( p_y \) from Figs 11.2 to 11.5.
Check \[ \frac{\nu_x}{\nu_{cx}} + \frac{\nu_y}{\nu_{cy}} \leq 1 \]

If this check fails, provide shear reinforcement in the form of links.

Note: \( \nu_{cx} \) and \( \nu_{cy} \) may be enhanced by using the following formulae due to presence of an axial load \( N_u \):

\[ \nu'_{cx} = \nu_{cx} + \frac{0.6 N_u H_{pyu} h}{A_c M_{pyu}} \leq 0.8 f_{cu} \leq 5 \text{ N/mm}^2 \]

\[ \nu'_{cy} = \nu_{cy} + \frac{0.6 N_u H_{pyu} b}{A_c M_{pyu}} \leq 0.8 f_{cu} \leq 5 \text{ N/mm}^2 \]

\( H_{pyu} h/M_{pyu} \) and \( H_{pyu} b/M_{pyu} \) should be less than or equal to 1.0.

Shear reinforcement

\[ A_{sv} = \frac{b S_v (\nu - \nu'_{c})}{0.87 f_{yy}} \]

where \( A_{sv} \) = total area of legs in direction of shear
\( b \) = width of section perpendicular to direction of shear
\( S_v \) = spacing of links
\( f_{yy} \leq 460 \text{ N/mm}^2 \) for links.

Circular piles

\( N_u = \) ultimate vertical load with \( H_{pu} \)
\( H_{pu} = \) combined ultimate horizontal load
\( M_{pu} = \) moment in pile due to \( H_{pu} \)
No shear check is necessary if:

\[ \frac{M_{pu}}{N_u} \leq 0.60h \quad \text{and} \quad \frac{H_{pu}}{0.75A_c} \leq 0.8\sqrt{f_{cu}} \leq 5 \text{ N/mm}^2 \]

where \( A_c = 0.25\pi h^2 \).

Shear check is necessary if:

\[ \frac{M_{pu}}{N_u} > 0.60h \]

Shear stress, \( \nu = \frac{H_{pu}}{0.75A_c} \)

\( p = 100A_s/1.5A_c \) assuming 50% of bars effectively in tension

where \( A_s \) = total area of steel in pile.
Find $v_c$ corresponding to $p$ from Figs 11.2 to 11.5. 

The shear stress $v_c$ may be enhanced by using the following formula due to presence of an axial load $N_u$:

$$v_c' = v_c + \frac{0.6N_uH_{pu}h}{A_M p} \leq 0.8\sqrt{f_{cu}} \leq 5\text{ N/mm}^2$$

$H_{pu}h/M_{pu}$ should be less than or equal to 1.0.

If $v > v_c'$, then use shear reinforcement.

$$V_s = 0.87f_{yw}A_v\left(\frac{z}{S}\right) \quad V_c = 0.75v_c'A_c$$

where $A_v = \text{total area of link bars perpendicular to longitudinal bars, i.e.}$

the two legs of hoop reinforcement

$f_{yw} = \text{characteristic yield strength of link reinforcement}$

$S = \text{spacing of links}$.

Find $z/R$ from appropriate table from Tables 11.18 to 11.27 corresponding to $f_{es}$, $h_u/h$, $p$, $N/R^2$ and $e/R$.

Check $H_{pu} \leq V_s + V_c$

The total shear resistance for inclined links =

$$V_s = \left[0.87f_{yw}A_{sv} (\cos \alpha + \sin \alpha \cot \beta) \left(\frac{z}{S}\right)\right]$$

where $A_{sv} = \text{total area of link bars i.e. the two legs of hoop reinforcement}$.

$\beta$ may be taken as 45° when $\alpha$ is angle of inclination of link.

**Step 16** Check shear capacity of prestressed pile

![Diagram](image)

**SK 7/27 Typical section and elevation of a prestressed concrete pile.**

$$V_{co} = 0.67bh(f_i^2 + 0.8f_{cp}f_i)^{\frac{1}{3}}$$

$$V_{cr} = \left(1 - \frac{0.55f_{pc}}{f_{pu}}\right) v_c bd + \frac{M_v V}{M} \geq 0.1bdv/\sqrt{f_{cu}}$$

$$V_c = V_{co} \text{ or } V_{cr} \text{ as the case may be (kN) \ - design ultimate shear resistance}$$
\[ V_{co} = \text{shear resistance of section uncracked (kN)} \]
\[ V_{cr} = \text{shear resistance of section cracked (kN)} \]
\[ f_t = \text{maximum design principal stress at the centroidal axis} = 0.24\sqrt{f_{cu}} \]
\[ f_{cp} = \text{design compressive stress at centroidal axis of concrete section due to prestress alone} \]
\[ f_{pe} = \text{design effective prestress in tendons after all losses} \leq 0.6f_{pu} \]
\[ f_{pu} = \text{characteristic ultimate strength of tendons} \]
\[ v_{c} = \text{design concrete shear strength from Figs 11.2 to 11.5 where percentage of steel reinforcement should include tendons plus any ordinary untensioned longitudinal steel reinforcement in tensile zone of section} \]
\[ d = \text{effective depth to centroid of reinforcing steel in tension zone where reinforcing steel should include tendons and any untensioned reinforcement} \]
\[ f_{cu} = \text{characteristic cube concrete strength at 28 days} \]
\[ M_o = \text{moment to produce zero stress at tension fibre with } 0.8f_{cp} \text{ on section.} \]

![Diagram](image)

SK 7/28 Stress diagram for a symmetrical rectangular prestressed pile due to \( M_o \).

If \( H_{pu} < 0.5V_c \), no shear reinforcement is required.
If \( H_{pu} \geq 0.5V_c \), then provide shear reinforcement as follows.

**Shear reinforcement**
If horizontal shear on pile, \( H_{pu} \), is less than or equal to \( (V_c + 0.4bd) \) then,

\[ \frac{A_{sw}}{S_w} = \frac{0.4b}{0.87f_{yw}} \]

If horizontal shear on pile, \( H_{pu} \), is more than \( (V_c + 0.4bd) \) then,
\[ \frac{A_{sv}}{S_v} = \frac{H_{pa} - V_c}{0.87f_{yv}d} \]

**Note:** For biaxial bending and shear, check requirement for shear reinforcement for each direction of bending separately, but allow for contribution of concrete shear resistance \( V_c \) in one direction of loading only for calculation of shear reinforcement. (See Step 7 of Section 4.3.1.)

**Step 17** Check minimum reinforcement in RC pile
For rectangular and circular piles, \( 100A_{sc}/A_c \geq 0.4 \).

**Step 18** Check minimum prestress in prestressed pile

Find slenderness ratio of pile = \( n = \frac{l}{b} \)

where \( b \) = minimum width of pile
\( l \) = total length of prestressed pile at commencement of driving.

Minimum prestress after losses = 60 \( \text{psi} \)
or
\( = 0.4n \text{N/mm}^2 \)

If diesel hammer is used,
minimum prestress in concrete = 5 \( \text{N/mm}^2 \)

**Step 19** Maximum reinforcement in pile
\( 100A_{sc}/A_c \leq 6 \)

**Step 20** Containment of reinforcement in pile
Minimum dia. of links = 0.25 \( \times \) largest bar \( \geq 6 \text{ mm} \)
Maximum spacing of links = 12 \( \times \) smallest dia. of bar

**Step 21** Links in prestressed piles
At top and bottom 3\( B \) length of pile, provide 0.6\% of volume of pile in volume of link.

**Step 22** Minimum tension reinforcement in pile cap
\( A_s \geq 0.0013bh \) in both directions

**Step 23** Curtailment of bars in pile cap
A minimum anchorage of 12 times diameter of bar should be provided at ends by bending bar up vertically. Additionally check that full tension anchorage bond length is provided from critical section for bending in a pile cap where design for flexure and requirement for flexural steel in tension is determined. In finding anchorage bond length beyond that section, actual area of steel provided may be taken into account.

**Step 24** Spacing of bars in pile cap
Clear spacing of bars should not exceed 3\( d \) or 750 mm.
**Percentage of reinforcement, 100A_s/\bar{b}d (%)**  

<table>
<thead>
<tr>
<th>Percentage of reinforcement, 100A_s/\bar{b}d (%)</th>
<th>Maximum clear spacing of bars in pile cap (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 or over</td>
<td>160</td>
</tr>
<tr>
<td>0.75</td>
<td>210</td>
</tr>
<tr>
<td>0.5</td>
<td>320</td>
</tr>
<tr>
<td>0.3</td>
<td>530</td>
</tr>
<tr>
<td>Less than 0.3</td>
<td>3d or 750</td>
</tr>
</tbody>
</table>

*Note:* This will deem to satisfy a crack width limitation of 0.3 mm.

**Step 25** *Early thermal cracking*
See Chapter 3.

**Step 26** *Assessment of crack width in flexure*
See Chapter 3.

**Step 27** *Connections*
See Chapter 10 for connection of pile to pile cap and column to pile cap.

### 7.7 WORKED EXAMPLE

**Example 7.1** *Pile cap for an internal column of a building*

Size of column = 800 mm \(\times\) 800 mm  
Spacing of column = 8 m \(\times\) 8 m on plan
### Unfactored column loads

<table>
<thead>
<tr>
<th></th>
<th>Dead</th>
<th>Imposed</th>
<th>Wind</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical load, $N$ (kN)</td>
<td>1610</td>
<td>1480</td>
<td>-</td>
</tr>
<tr>
<td>Horizontal shear, $H_x$ (kN)</td>
<td>28</td>
<td>18</td>
<td>156</td>
</tr>
<tr>
<td>Horizontal shear, $H_y$ (kN)</td>
<td>-</td>
<td>-</td>
<td>112</td>
</tr>
<tr>
<td>Moment, $M_x$ (kNm)</td>
<td>-</td>
<td>-</td>
<td>448</td>
</tr>
<tr>
<td>Moment, $M_y$ (kNm)</td>
<td>112</td>
<td>72</td>
<td>624</td>
</tr>
</tbody>
</table>

### Geotechnical information (see SK 7/30)

**Stratum 1**
Average thickness of layer = 1.5 m
Classification: very loose yellow brown to brownish grey sandy silt.

Average $N = 3$ (SPT)
$\begin{align*}
  c &= 11.3 \text{kN/m}^2 \\
  \phi &= 4^\circ \\
  \gamma &= 26 \text{kN/m}^3
\end{align*}$

**Stratum 2**
Average thickness of layer = 9 m
Classification: soft to medium bluish-grey clayey silt.

Average $N = 5$ (SPT)
$\begin{align*}
  c &= 20.2 \text{kN/m}^2 \\
  \phi &= 5^\circ \\
  \gamma &= 24 \text{kN/m}^3 \\
  \gamma_{sat} &= 27 \text{kN/m}^3
\end{align*}$

**Stratum 3**
Average thickness of layer = 2 m
Classification: stiff to very stiff bluish-grey silty clay.

Average $N = 14$ (SPT)
$\begin{align*}
  c &= 60 \text{kN/m}^2 \\
  \phi &= 6^\circ \\
  \gamma_{sat} &= 26 \text{kN/m}^3
\end{align*}$

**Stratum 4**
Average thickness of layer = 7 m
Classification: dense to very dense mottled brown sandy silt.

Average $N = 24$ (SPT)
$\begin{align*}
  c &= 13.8 \text{kN/m}^2 \\
  \phi &= 31^\circ \\
  \gamma_{sat} &= 27 \text{kN/m}^3
\end{align*}$
SK 7/30 Average ground condition soil strata.
Stratum 5
Average thickness of layer = 15 m
Classification: very stiff to hard silty clay.
Average \( N = 31 \) (SPT)
\( c = 71.5 \text{ kN/m}^2 \)
\( \phi = 8^\circ \)
\( \gamma_{sat} = 28 \text{ kN/m}^3 \)
Water table at 3.0 m below ground level.

**Step 1 Select type of pile**
Considering all the factors as described in Step 1 of Section 7.6 it is decided to use a non-displacement pile. Choose 600 mm diameter bored and cast-in-situ concrete pile.

**Step 2 Determine vertical capacity of pile**
Follow Section 7.1.
\[ P_u = P_{pu} + \Sigma P_{si} - W \]

*First method of point resistance*
\[ P_{pu} = A_p(38N)(\frac{L_b}{B}) \]
Assume pile to go into Stratum 5 and stop at 8.0 m within Stratum 5.
\( L_b = \) average length of pile = \((1.5 + 9 + 2 + 7 + 8) \) m = \(27.5 \) m
\( A_p = \) cross-sectional area of pile = \(\pi \times \frac{0.6^2}{4} = 0.283 \) m\(^2\)
\( B = 0.60 \) m
\( N = \) statistical average of SPT in a zone of about \(8B\) above to \(3B\) below pile point = \(31\)
\[ P_{pu} = 0.283 \times 38 \times 31 \times \frac{27.5}{0.6} = 15280 \text{ kN} \]
\[ \leq 380N(A_p) = 380 \times 31.0 \times 0.283 = 3334 \text{ kN} \]

*Second method of point resistance*
\[ P_{pu} = A_p(N'_c + qN'_u) \]
\( A_p = 0.283 \text{ m}^2 \)
\( c = 71.5 \text{ kN/m}^2 \)
\( \gamma_w = 10 \text{ kN/m}^3 \)
\( q = \) effective vertical stress at pile point
\[ = 1.5 \times 26 + 1.5 \times 24 + 7.5 \times 27 + 2 \times 26 + 7 \times 27 + 8 \times 27 \]
\[ - (27.5 - 3) \times 10 \]
\[ = 489.5 \text{ kN/m}^2 \]
SK 7/31 The pile penetrating different strata.

\[ L = 27.5 \text{ m} \quad B = 0.60 \text{ m} \]
\[ L/B = 46 \quad \phi = 8^\circ \]

From Fig. 7.2,
\[ N'_q = 3 \quad N'_c = 15 \quad \text{and} \quad L_q/B = 3.5 \]
\[ \frac{L}{B} > \frac{L_c}{B} \]

\[ P_{pu} = 0.283 \left( (15 \times 71.5) + (3 \times 489.5) \right) = 719 \text{ kN} \]
SK 7/32 Condition at bottom of pile.

**Determination of skin resistance**

\[ \Sigma P_{si} = \Sigma A_s f_s \]

Used non-displacement pile of 600 mm diameter.

**First method of skin resistance**

\[ f_s = \frac{N}{A_s^2} \text{ kN/m}^2 \]

**Stratum 1**

\[ A_s = \text{perimeter} \times \text{depth of stratum} \]
\[ = \pi \times 0.60 \times 1.5 \]
\[ = 2.83 \text{ m}^2 \]
\[ f_s = 3 \text{ kN/m}^2 \]
\[ P_{s1} = 3 \times 2.83 = 8.5 \text{ kN} \]

**Stratum 2**

\[ A_s = \pi \times 0.60 \times 9 = 17 \text{ m}^2 \]
\[ f_s = 5 \text{ kN/m}^2 \]
\[ P_{s2} = 5 \times 17 = 85 \text{ kN} \]

**Stratum 3**

\[ A_s = \pi \times 0.60 \times 2 = 3.8 \text{ m}^2 \]
\[ f_s = 14 \text{ kN/m}^2 \]
\[ P_{s3} = 14 \times 3.8 = 53.2 \text{ kN} \]
Stratum 4

\[ A_{s4} = \pi \times 0.60 \times 7 = 13.2 \text{ m}^2 \]

\[ f_{s4} = 24 \text{ kN/m}^2 \]

\[ P_{si4} = 13.2 \times 24 = 316.8 \text{ kN} \]

Stratum 5

\[ A_{s5} = \pi \times 0.60 \times 8 = 15.1 \text{ m}^2 \]

\[ f_{s5} = 31 \text{ kN/m}^2 \]

\[ P_{si5} = 15.1 \times 31 \times 468.1 \text{ kN} \]

\[ \Sigma P_{si} = 931.6 \text{ kN} \]

Fourth method of skin resistance

\[ f_s = \alpha c + 0.5qK_s \tan \delta \]

Ignore the second term because \( \delta \) is very small.

Stratum 1

\[ \alpha = 0.75 \quad c = 11.3 \text{ kN/m}^2 \]

\[ P_{si1} = A_{s1} \times f_{s1} \]

\[ P_{si1} = 0.75 \times 11.3 \times 2.83 = 24 \text{ kN} \]

\[ A_{s1} = 2.83 \text{ m}^2 \]

Stratum 2

\[ \alpha = 0.75 \quad c = 20.2 \text{ kN/m}^2 \]

\[ P_{si2} = 0.75 \times 20.2 \times 17 = 257.2 \text{ kN} \]

\[ A_{s2} = 17 \text{ m}^2 \]

Stratum 3

\[ \alpha = 0.75 \quad c = 60 \text{ kN/m}^2 \]

\[ P_{si3} = 0.75 \times 60 \times 3.8 = 171 \text{ kN} \]

\[ A_{s3} = 3.8 \text{ m}^2 \]

Stratum 4

\[ A_{s4} = 13.2 \text{ m}^2 \]

\[ \alpha = 2.0 \quad \text{say with high } D_t \]

\[ c = 13.8 \text{ kN/m}^2 \quad \phi = 31^\circ \]

\[ K_s = 2.0 \quad \text{from chart} \]

\[ \delta = 0.75\phi = 23.25^\circ \]

\[ \tan \delta = 0.43 \]
\( \bar{q} = \text{effective vertical stress at middle of layer} \)
\[ = 1.5 \times 26 + 1.5 \times 24 + 7.5 \times 27 + 2 \times 26 + 3.5 \times 27 - (16 - 3) \times 10 \]
\[ = 294 \text{kN/m}^2 \]

\[ f_s = ac + 0.5 \bar{q} K_s \tan \delta \]

\[ P_{ud} = 13.2 \left[ 2 \times 13.8 \right] + (0.5 \times 294 \times 2 \times 0.43) ] = 2033 \text{kN} \]

The fourth method of skin resistance is giving much higher values than the first method and may be ignored from the point of view of conservatism.

\[ P_u = P_{pu} + P_{sm} \]
\[ = 719 + 932 \]
\[ = 1651 \text{kN} \]

Allowable working load on pile \( \frac{1651}{2.5} = 660 \text{kN} \)

Designed pile is 600 mm diameter bored and cast in-situ concrete pile with an average length of 27.5 m to carry a working load of 660 kN. This is a conservative theoretical estimate of single pile vertical load capacity and must be verified by actual pile tests on site.

**Step 3** Determine horizontal capacity of single pile

See Section 7.2.

Assume cohesive soil.

**Method 1**

\( E_s = 650N \) where \( N = \text{SPT No.} \)

\( E_s \) of Stratum 1 = 650 \times 3 = 1950 \text{kN/m}^2

\( E_s \) of Stratum 2 = 650 \times 5 = 3250 \text{kN/m}^2

\( E_s \) of Stratum 3 = 650 \times 14 = 9100 \text{kN/m}^2

\( E_s \) of Stratum 4 = 650 \times 24 = 15600 \text{kN/m}^2

\( E_s \) of Stratum 5 = 650 \times 31 = 20150 \text{kN/m}^2

\( k_s B = 1.3 \left( \frac{E_s B^4}{E_l I_l} \right)^{\frac{1}{4}} \left( \frac{E_s}{1 - \mu^2} \right) \)

\( E_l = 28 \times 10^6 \text{kN/m}^2 \) for pile concrete

\( I_l = \left( \frac{\pi}{64} \right) D^4 = \left( \frac{\pi}{64} \right) \times 0.60^4 = 6.36 \times 10^{-3} \text{ m}^4 \)

\( k_{s1} B = 1672 \text{kN/m}^2 \quad k_{s1} = 2787 \text{kN/m}^3 \)

\( k_{s2} B = 2909 \text{kN/m}^2 \quad k_{s2} = 4848 \text{kN/m}^3 \)

\( k_{s3} B = 8875 \text{kN/m}^2 \quad k_{s3} = 14792 \text{kN/m}^3 \)

\( k_{s4} B = 15914 \text{kN/m}^2 \quad k_{s4} = 26523 \text{kN/m}^3 \)

\( k_{s5} B = 20999 \text{kN/m}^2 \quad k_{s5} = 34998 \text{kN/m}^3 \)
Method 2

\[ k_3 = 240q_u \text{kN/m}^2 \]
\[ = 480 \text{kN/m}^2 \]

\[ k_{31} = 480 \times 11.3 = 5424 \text{kN/m}^3 \]
\[ k_{32} = 480 \times 20.2 = 9696 \text{kN/m}^3 \]
\[ k_{33} = 480 \times 60 = 28800 \text{kN/m}^3 \]
\[ k_{34} = 480 \times 13.8 = 6624 \text{kN/m}^3 \]
\[ k_{35} = 480 \times 71.5 = 34320 \text{kN/m}^3 \]

The values given by Method 1 are smaller or softer which will produce larger deflection and bending moments in pile.

For the sake of conservatism use values given by Method 1.

\[ S = \text{node spacing for finite element analysis} = 0.60 \text{m} \]
\[ B = 0.60 \text{m} \]

spring stiffness = \[ SBk_3 \text{kN/m} \]

---

SK 7/33 Finite element model of pile.
Ignore top $1.5B$ of pile for lateral support from soil.
The whole length of pile need not be modelled.

**Stratum 1**
Spring stiffness = $0.60 \times 0.60 \times 2787$

$= 1003 \text{ kN/m}$

**Stratum 2**
Spring stiffness = $0.60 \times 0.60 \times 4848$

$= 1745 \text{ kN/m}$

**Stratum 3**
Spring stiffness = $0.60 \times 0.60 \times 14792$

$= 5325 \text{ kN/m}$

**Stratum 4**
Spring stiffness = $0.60 \times 0.6 \times 26523 = 9548 \text{ kN/m}$

Assume full fixity of pile with pile cap.
Apply unit load at top of pile and find pile stiffness and bending moment and shear in pile using a two-dimensional computer program.

$A = 0.283 \text{ m}^2 \quad I = 6.36 \times 10^{-3} \text{ m}^4$

**Results of computer run**
Maximum moment = $2.48 \text{ kNm}/\text{kN}$
Pile top deflection = $0.12 \text{ mm}/\text{kN}$

Single pile horizontal stiffness = \( \frac{1000}{0.12} = 8333 \text{ kN/m} \)

**Step 4 Determine approximate number of piles and spacing**
Maximum vertical load on pile cap = $1610 + 1480 = 3090 \text{ kN} = P$

\[ R_{IV} = \frac{P}{C_v} = \frac{3090}{660} = 4.7 \]

Assume maximum allowable horizontal displacement of pile cap is $10 \text{ mm}$.
Maximum horizontal load = $28 + 18 + 156 = 202 \text{ kN} = H$

Maximum horizontal load on pile to limit deflection to $10 \text{ mm}$
$= 8333 \times 0.010$
$= 83 \text{ kN} \quad \text{per pile}$

\[ R_{IH} = \frac{H}{C_H} = \frac{202}{83} = 2.4 \]

\[ R_i = \text{greater of } R_{IV} \text{ and } R_{IH} = 4.7 \]

$1.1R_i = 4.7 \times 1.1 = 5.17$

Use $6 \text{ no. piles.}$
Step 5 *Determine size of pile cap*

- $B =$ diameter of pile = 0.6 m
- $1.5B = 1.5 \times 0.6 = 0.9$ m

Allow 0.9 m from centre of pile to edge of pile cap. Assume 0.9 m depth of pile cap.
SK 7/36 Layout of piles under pile cap.

Spacing of piles $\geq 3B \geq 3 \times 0.6 = 1.8 \text{ m}$
Size of pile cap assumed is $5.4 \text{ m} \times 3.6 \text{ m} \times 0.9 \text{ m}$.

**Step 6** Carry out load combination

*Estimation of load on pile*

$LC_1 = 1.0DL + 1.0IL$

$N = 1610 + 1480 = 3090 \text{ kN}$

$H_x = 28 + 18 = 46 \text{ kN}$

$H_y = 0 \text{ kN}$

$M_x = 0 \text{ kNm}$

$M_y = 112 + 72 = 184 \text{ kNm}$

$LC_3 = 1.0DL + 1.0IL + 1.0WL$

$N = 3090 \text{ kN}$

*Wind in x−x direction*

$H_x = 46 + 156 = 202 \text{ kN}$

$H_y = 0 \text{ kN}$

$M_x = 0 \text{ kNm}$

$M_y = 184 + 624 = 808 \text{ kNm}$

*Wind in y−y direction*

$H_x = 46 \text{ kN}$

$H_y = 112 \text{ kN}$

$M_x = 448 \text{ kNm}$

$M_y = 184 \text{ kNm}$

$LC_4 = 1.0DL + 1.0WL$. 
$N = 1610 \text{kN}$

*Wind in $x-x$ direction*

$H_x = 28 + 156 = 184 \text{kN}$

$H_y = 0 \text{kN}$

$M_x = 0 \text{kNm}$

$M_y = 112 + 624 = 736 \text{kNm}$

*Wind in $y-y$ direction*

$H_x = 28 \text{kN}$

$H_y = 112 \text{kN}$

$M_x = 448 \text{kNm}$

$M_y = 112 \text{kNm}$

*Estimation of loads on piles for bending moment and shear calculations in pile cap*

$LC_x = 1.4DL + 1.6IL$

$N = 1.4 \times 1610 + 1480 \times 1.6 = 4622 \text{kN}$

$H_x = 1.4 \times 28 + 1.6 \times 18 = 68 \text{kN}$

$H_y = 0 \text{kN}$

$M_x = 0 \text{kNm}$

$M_y = 1.4 \times 112 + 1.6 \times 72 = 272 \text{kNm}$

$LC_y = 1.2DL + 1.2IL + 1.2WL$

$N = 1.2 \times 1610 + 1.2 \times 1480 = 3708 \text{kN}$

*Wind in $x-x$ direction*

$H_x = 1.2 \times (28 + 18 + 156) = 242.4 \text{kN}$

$H_y = 0 \text{kN}$

$M_x = 0 \text{kNm}$

$M_y = 1.2 \times (112 + 72 + 624) = 969.6 \text{kNm}$

*Wind in $y-y$ direction*

$H_x = 1.2 \times (28 + 18) = 55.2 \text{kN}$

$H_y = 1.2 \times 112 = 134.4 \text{kN}$

$M_x = 1.2 \times 448 = 537.6 \text{kNm}$

$M_y = 1.2 \times (112 + 72) = 220.8 \text{kNm}$

$LC_o = 1.4DL + 1.4WL$

$N = 1.4 \times 1610 = 2254 \text{kN}$

*Wind in $x-x$ direction*

$H_x = 1.4 (28 + 156) = 257.6 \text{kN}$

$H_y = 0 \text{kN}$
SK 7/35 Layout of piles under pile cap.

Spacing of piles \( \geq 3B \geq 3 \times 0.6 = 1.8 \text{ m} \)

Size of pile cap assumed is \( 5.4 \text{ m} \times 3.6 \text{ m} \times 0.9 \text{ m} \).

**Step 6 Carry out load combination**

*Estimation of load on pile*

\[ LC_1 = 1.0DL + 1.0IL \]

\[ N = 1610 + 1480 = 3090 \text{kN} \]

\[ H_x = 28 + 18 = 46 \text{kN} \]

\[ H_y = 0 \text{kN} \]

\[ M_x = 0 \text{kNm} \]

\[ M_y = 112 + 72 = 184 \text{kNm} \]

\[ LC_3 = 1.0DL + 1.0IL + 1.0WL \]

\[ N = 3090 \text{kN} \]

*Wind in x-x direction*

\[ H_x = 46 + 156 = 202 \text{kN} \]

\[ H_y = 0 \text{kN} \]

\[ M_x = 0 \text{kNm} \]

\[ M_y = 184 + 624 = 808 \text{kNm} \]

*Wind in y-y direction*

\[ H_x = 46 \text{kN} \]

\[ H_y = 112 \text{kN} \]

\[ M_x = 448 \text{kNm} \]

\[ M_y = 184 \text{kNm} \]

\[ LC_4 = 1.0DL + 1.0WL \]
\[ c = 71.5 \text{kN/m}^2 \quad \text{at bottom of group} \]
\[ \bar{q} = \text{effective stress at bottom of group} = 489.5 \text{kN/m}^2 \quad \text{(see Step 2)} \]
\[ \frac{N_q}{N_e} = \begin{cases} 3 \\ 15 \end{cases} \text{ for } \phi = 8^\circ \]

Group end-bearing capacity = \[1.8 \times 3.6 \times (15 \times 71.5 + 489.5 \times 3) = 16465 \text{kN}\]

Ultimate group capacity = \[7996 + 16465 = 24461 \text{kN}\]

Allowable group capacity = \[\frac{24461}{2.5} = 9784 \text{kN}\]

Allowable group capacity based on single pile capacity = \[6 \times 660 = 3960 \text{kN}\]

Design basis is single pile capacity.

**Step 8 Carry out analysis of pile cap**

Assume that pile cap is rigid. Assume 500 mm backfill on top of pile cap. Assume a surcharge of 5 kN/m² on backfill with no eccentricity.

*It is always advisable to use the table as presented.*

\[ W = \text{weight of pile cap} \]
\[ \quad + \text{weight of backfill on pile cap} \]
\[ \quad + \text{weight of surcharge on backfill} \]
\[ = 5.4 \text{m} \times 3.6 \text{m} \times 0.9 \text{m} \times 24 \text{kN/m}^3 \]
\[ + 5.4 \times 3.6 \times 0.5 \times 20 \text{kN/m}^3 \]
\[ + 5.4 \times 3.6 \times 5 \text{kN/m}^3 \]
\[ = 712 \text{kN} \]

Maximum service load on pile without wind = 665 kN

Maximum service load on pile with wind = 771 kN

---

**SK 7/37 Calculations of pile group stiffness.**
<table>
<thead>
<tr>
<th>Load case</th>
<th>N</th>
<th>$M_x$</th>
<th>$M_y$</th>
<th>$M_z$</th>
<th>$P$ or $P_a$</th>
<th>$M_{xy}$</th>
<th>$M_{yz}$</th>
<th>$M_{xz}$</th>
<th>$T$</th>
</tr>
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<tbody>
<tr>
<td>LC1</td>
<td>30900</td>
<td>0</td>
<td>848</td>
<td>184</td>
<td>0</td>
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<td>LC2</td>
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<td>848</td>
<td>184</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
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<tr>
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<td>0</td>
<td>848</td>
<td>184</td>
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<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
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<tr>
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<td>736</td>
<td>112</td>
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<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
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<td>LC5</td>
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<td>68</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
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<tr>
<td>LC6</td>
<td>3708</td>
<td>0</td>
<td>537.6</td>
<td>226</td>
<td>0</td>
<td>0</td>
<td>0</td>
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<td>0</td>
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<tr>
<td>LC7</td>
<td>2254</td>
<td>0</td>
<td>156.8</td>
<td>39.2</td>
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<td>0</td>
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<td>0</td>
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<tr>
<td>LC8</td>
<td>2254</td>
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<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Analysis of loads on pile cap.

$M_{xx} = M_x + N \cdot d_x + H_{xy} + M_{xy}$

$M_{yy} = M_y + N \cdot d_y + H_{xz} + M_{xz}$

$M_{zz} = M_z + N \cdot d_z + H_{zy} + M_{yz}$

$P = N + W$

$P_a = N + 1.4W$ (or 1.2W)
**Loads on pile.**

<table>
<thead>
<tr>
<th>Load case</th>
<th>( P ) or ( P_u )</th>
<th>( H_x )</th>
<th>( H_y )</th>
<th>( M_{xx} )</th>
<th>( M_{yy} )</th>
<th>( T )</th>
<th>( Q_{\text{max}} )</th>
<th>( Q_{\text{min}} )</th>
<th>( H ) or ( H_{pu} )</th>
<th>( M_p ) or ( M_{pu} )</th>
<th>( \delta ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( LC_1 )</td>
<td>3802</td>
<td>46</td>
<td>—</td>
<td>225.4</td>
<td>—</td>
<td>665</td>
<td>602</td>
<td>7.67</td>
<td>19.0</td>
<td>0.9</td>
<td></td>
</tr>
<tr>
<td>( LC_3 )</td>
<td>3802</td>
<td>202</td>
<td>—</td>
<td>989.8</td>
<td>—</td>
<td>771</td>
<td>496</td>
<td>33.67</td>
<td>83.5</td>
<td>4.0</td>
<td></td>
</tr>
<tr>
<td>( LC_4 )</td>
<td>3802</td>
<td>46</td>
<td>112</td>
<td>548.8</td>
<td>225.4</td>
<td>767</td>
<td>501</td>
<td>20.18</td>
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<td>2.4</td>
<td></td>
</tr>
<tr>
<td>( LC_5 )</td>
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<td>46</td>
<td>112</td>
<td>548.8</td>
<td>225.4</td>
<td>767</td>
<td>501</td>
<td>20.18</td>
<td>50.0</td>
<td>2.4</td>
<td></td>
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<tr>
<td>( LC_6 )</td>
<td>2322</td>
<td>184</td>
<td>—</td>
<td>901.6</td>
<td>—</td>
<td>512</td>
<td>262</td>
<td>30.67</td>
<td>76.1</td>
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<td></td>
</tr>
<tr>
<td>( LC_7 )</td>
<td>2322</td>
<td>28</td>
<td>112</td>
<td>548.8</td>
<td>137.2</td>
<td>508</td>
<td>266</td>
<td>19.24</td>
<td>47.7</td>
<td>2.3</td>
<td></td>
</tr>
<tr>
<td>( LC_8 )</td>
<td>2322</td>
<td>28</td>
<td>112</td>
<td>548.8</td>
<td>137.2</td>
<td>508</td>
<td>266</td>
<td>19.24</td>
<td>47.7</td>
<td>2.3</td>
<td></td>
</tr>
<tr>
<td>( LC_9 )</td>
<td>2322</td>
<td>28</td>
<td>112</td>
<td>548.8</td>
<td>137.2</td>
<td>508</td>
<td>266</td>
<td>19.24</td>
<td>47.7</td>
<td>2.3</td>
<td></td>
</tr>
<tr>
<td>( LC_{10} )</td>
<td>2322</td>
<td>28</td>
<td>112</td>
<td>548.8</td>
<td>137.2</td>
<td>508</td>
<td>266</td>
<td>19.24</td>
<td>47.7</td>
<td>2.3</td>
<td></td>
</tr>
<tr>
<td>( LC_{11} )</td>
<td>2322</td>
<td>28</td>
<td>112</td>
<td>548.8</td>
<td>137.2</td>
<td>508</td>
<td>266</td>
<td>19.24</td>
<td>47.7</td>
<td>2.3</td>
<td></td>
</tr>
</tbody>
</table>

\[ I_{xx} = \Sigma y^2 = 4.86 \text{m}^2 \quad I_{yy} = \Sigma x^2 = 12.96 \text{m}^2 \quad I_{zz} = I_{xx} + I_{yy} = 17.82 \text{m}^2 \]

\[ Q_{\text{max}} = \frac{P}{R} + \frac{M_{xx,y}}{I_{xx}} + \frac{M_{yy,x}}{I_{yy}} \quad Q_{\text{min}} = \frac{P}{R} - \frac{M_{xx,y}}{I_{xx}} - \frac{M_{yy,x}}{I_{yy}} \]

\[ H = \frac{\sqrt{(H_1^2 + H_2^2)}}{R} \quad R = \text{no. of piles} = 6 \]

\( M_p = \text{bending moment in pile} = 2.48H \) (see Step 3) \quad x = 1.8 \text{m} \quad y = 0.9 \text{m} \]

\( \delta = \text{horizontal displacement at top of pile} = 0.12H \text{mm} \) (see Step 3)
SK 7/38 General arrangement of pile cap and piles.

Allowable service load on pile without wind = 660 kN  OK
Allowable service load on pile with wind = 660 \times 1.25 = 825 kN  OK

*Bending moment and shear force in pile cap*

SK 7/39 Critical sections for calculation of bending moment in pile cap.
Sections 1–1 and 2–2 are taken at the face of column.
Assume column size = 800 mm × 800 mm

Dead load of pile cap + surcharge + backfill = 0.9 × 24 + 0.5 × 20 + 5 = 36.6 kN/m²

Applying load factors for different load cases:
1.4 × 36.6 = 51.2 kN/m²
1.2 × 36.6 = 43.9 kN/m²

\[ M'_{11} = \frac{3.6 \times 51.2 \times 2.3^2}{2} = 487.5 \text{ kNm} \]
\[ \text{or } \frac{3.6 \times 43.9 \times 2.3^2}{2} = 418.0 \text{ kNm} \]

\[ M'_{12} = \frac{5.4 \times 51.2 \times 1.4^2}{2} = 271.0 \text{ kNm} \]
\[ \text{or } \frac{5.4 \times 43.9 \times 1.4^2}{2} = 232.3 \text{ kNm} \]

**Step 9**  Determine cover to reinforcement
From soil test reports, the total SO₃ is 0.75%. This means it is Class 3 exposure (see table in Step 9 of Section 7.6).

Minimum cover on blinding concrete = 50 mm
Minimum cover elsewhere = 90 mm
Assume 90 mm cover for pile cap everywhere.

**Step 10**  Calculate area of reinforcement in pile cap
\[ M = \text{bending moment in pile cap } \text{as found in Step 8.} \]
\[ M_{11} = 2264.9 \text{ kNm} \text{ from table in Step 8.} \]
Bending moments and shear in pile cap.

<table>
<thead>
<tr>
<th>Load case</th>
<th>$Q_1$</th>
<th>$Q_2$</th>
<th>$Q_3$</th>
<th>$Q_4$</th>
<th>$M'_{11}$</th>
<th>$M'_{22}$</th>
<th>$M'_{11}$</th>
<th>$M_{11}$</th>
<th>$M_{22}$</th>
<th>$V'_{33}$</th>
<th>$V_{44}$</th>
<th>$V'_{33}$</th>
<th>$V_{44}$</th>
<th>$V'_{33}$</th>
<th>$V_{44}$</th>
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<tbody>
<tr>
<td>$LC_1$</td>
<td>890</td>
<td>937</td>
<td>983</td>
<td>983</td>
<td>-487.5</td>
<td>-271.0</td>
<td>2752.4</td>
<td>1405</td>
<td>2264.9</td>
<td>1134.0</td>
<td>-199.1</td>
<td>-298.6</td>
<td>1966</td>
<td>2810</td>
<td>1766.9</td>
</tr>
<tr>
<td>$LC_6$</td>
<td>595</td>
<td>760</td>
<td>925</td>
<td>925</td>
<td>-418.0</td>
<td>-232.3</td>
<td>2590.0</td>
<td>1140</td>
<td>2172.0</td>
<td>907.7</td>
<td>-170.7</td>
<td>-256.0</td>
<td>1850</td>
<td>2280</td>
<td>1679.3</td>
</tr>
<tr>
<td>$LC_7$</td>
<td>844</td>
<td>882</td>
<td>920</td>
<td>676</td>
<td>-418.0</td>
<td>-232.3</td>
<td>2234.4</td>
<td>1323</td>
<td>1816.4</td>
<td>1090.7</td>
<td>-170.7</td>
<td>-256.0</td>
<td>1596</td>
<td>2646</td>
<td>1425.3</td>
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<tr>
<td>$LC_7$</td>
<td>367</td>
<td>542</td>
<td>717</td>
<td>717</td>
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<td>-271.0</td>
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<td>813</td>
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<td>542.0</td>
<td>-199.1</td>
<td>-298.6</td>
<td>1434</td>
<td>1626</td>
<td>1234.9</td>
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<tr>
<td>$LC_7$</td>
<td>657</td>
<td>684</td>
<td>711</td>
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<td>-487.5</td>
<td>-271.0</td>
<td>1593.2</td>
<td>1026</td>
<td>1105.7</td>
<td>755.0</td>
<td>-199.1</td>
<td>-298.6</td>
<td>1138</td>
<td>2052</td>
<td>938.9</td>
</tr>
</tbody>
</table>

$Q_1$, $Q_2$, $Q_3$ and $Q_4$ are pile reactions

$M'_{11} = 1.4 \ (Q_3 + Q_4)$, $M'_{22} = 0.5 \ (Q_1 + Q_2 + Q_3)$, $M_{11} = M'_{11} + M'_{22}$

$V'_{33} = Q_3 + Q_4$, $V_{44} = Q_1 + Q_2 + Q_3$, $V'_{33} = V_{33} + V'_{33}$

$M_{11}$, $M_{22}$, $V_{33}$ and $V_{44}$ are bending moments and shears in pile cap due to dead load of pile cap + surcharge

$M'_{11}$, $M'_{22}$, $V'_{33}$ and $V'_{44}$ are bending moments and shears in pile cap due to pile reaction

$M_{11}$, $M_{22}$, $V_{33}$ and $V_{44}$ are combined bending moments and shears in pile cap

$\phi = 600 \text{ mm}$, $\phi/s = 120 \text{ mm}$, $\phi$ = diameter of pile
For this load case, pile fixity moment = 19.0 kNm per pile.

Pile fixity moment on pile cap is opposite in sign to moment $M_{11}$ and may be ignored.
Assume 20 mm diameter reinforcement.

$d_x = 900 - 90 (\text{cover}) - 10 (\text{half bar dia.}) = 800 \text{ mm} \quad b = 3.6 \text{ m}$

$f_{cu} = 30 \text{ N/mm}^2 \quad \text{for concrete in pile cap}$

$K = \frac{M_{11}}{f_{cu}bd^2} = \frac{2264.9 \times 10^6}{30 \times 3600 \times 800^2} = 0.033$

$z = d \left[0.5 + \sqrt{\left(0.25 - \frac{K}{0.9}\right)}\right]$

$= 0.96d \approx 0.95d = 760 \text{ mm}$

$A_{st} = \frac{M_{11}}{0.87f_{sz}} = \frac{2264.9 \times 10^6}{0.87 \times 460 \times 760} = 7447 \text{ mm}^2$

Assume $f_y = 460 \text{ N/mm}^2$ for HT reinforcement

Area of 20 mm dia. bar = 314 mm$^2$ \quad $24 \times 314 = 7536 \text{ mm}^2$

Use 24 no. 20 mm diameter bars equally spaced (approximate spacing 150 mm) in the $x-x$ direction.

$M_{22} = 1134 \text{ kNm} \quad \text{from table in Step 8.}$

Ignore the effect of pile fixity moments.
Assume 12 mm diameter reinforcement.

$d_y = 900 - 90(\text{cover} - 20(\text{bar dia.}) - 6(\text{half bar}) = 784 \text{ mm}$
Design of Piled Foundations

\[ K = \frac{M_{22}}{f_{cd}bd^2} = \frac{1134 \times 10^6}{30 \times 5400 \times 784^2} = 0.011 \]

\[ z = 0.95d \text{ by inspection} \]
\[ = 0.95 \times 784 = 745 \text{ mm} \]

\[ A_u = \frac{M_{22}}{0.87f_{sz}} = \frac{1134 \times 10^6}{0.87 \times 460 \times 745} = 3803 \text{ mm}^2 \]

Area of 12 mm dia. bar = 113 mm² × 34 = 3842 mm²

Use 34 no. 12 mm diameter bars equally spaced (approximate spacing 155 mm) in the y-y direction.

(See also Step 27 for minimum reinforcement.)

All bars are high tensile reinforcement to be placed at bottom of pile cap. There is no requirement for bars on top of pile cap.

Step 11 Check shear stress in pile cap

\[ V_{33} = \text{shear on critical section 3-3} \]
\[ = 1766.9 \text{ kN} \text{ (see table in Step 8)} \]

\[ a_v = 2700 - 400 \text{ (half column)} - 1080 = 1220 \text{ mm} \]
1.5\(d_c = 1.5 \times 800 = 1200\) mm

\(a_v > 1.5d_c\) hence no enhancement of shear stress is allowed

\[
\nu = \frac{V}{bd} = \frac{1766.9 \times 10^3}{3600 \times 800} = 0.61 \text{ N/mm}^2
\]

\[
p = \frac{100A_v}{bd} = \frac{100 \times 7536}{3600 \times 800} = 0.26\%
\]

\[
\nu_c = 0.425 \text{ N/mm}^2 < 0.61 \text{ N/mm}^2 \quad \text{from Fig. 11.3}
\]

The cheapest alternative is to bring the outer piles in towards the centre of pile cap by 20 mm in the \(x-x\) direction only. This has very little effect on pile reactions.

\[
a_v = 1200 \text{ mm} \quad 1.5d_c = 1200 \text{ mm}
\]

\[
\frac{2d}{a_v} = \frac{2 \times 800}{1200} = 1.333
\]

Increase grade of concrete from \(f_{cu} = 30 \text{ N/mm}^2\) to \(f_{cu} = 40 \text{ N/mm}^2\) in pile cap.

\[
\nu_{ct} = 0.47 \text{ N/mm}^2 \quad \text{from Figs 11.2 to 11.5}
\]

\[
\nu_{ct} = \nu_{ct}\left(\frac{2d}{a_v}\right) = 0.47 \times 1.333
\]

\[
= 0.63 \text{ N/mm}^2 > 0.61 \text{ N/mm}^2 \quad \text{OK}
\]

\[
V_{44} = \text{shear on critical section 4-4} = 2511.4 \text{ kN} \quad \text{(see table in Step 8)}.
\]

\[
a_v = 1800 - 1200 + 120 - 400 \text{ (half column)} = 320 \text{ mm}
\]

\[
1.5d_v = 1.5 \times 784 = 1176 \text{ mm} > a_v
\]
$$2d_v = \frac{2 \times 784}{320} = 4.9$$

$$p = \frac{100A_{sc}}{bd} = \frac{100 \times 3482}{5400 \times 784} = 0.08\%$$

(See Step 22 for minimum percentage of reinforcement.)

$$\nu_{ct} = 0.40 \text{N/mm}^2 \text{ for } f_{ct} = 40 \text{N/mm}^2$$

$$\nu_{c2} = 0.40 \times 4.9 = 1.96 \text{N/mm}^2$$

$$\nu_c = \frac{V}{bd} = \frac{2511 \times 10^3}{5400 \times 784}$$

$$= 0.59 \text{N/mm}^2 < 1.96 \text{N/mm}^2 \text{ OK}$$

**Step 12 Check punching shear stress in pile cap**

![Diagram of a pile cap with critical planes highlighted]

**SK 7/44 Critical planes for punching shear of piles in pile cap.**

\[ U_1 = \text{perimeter of column} = 2(800 + 800) = 3200 \text{mm} \]

Since pile spacing is not greater than 3 times diameter of pile, then punching shear stress at critical perimeter for column need not be checked.

\[ U_2 = \text{perimeter on punching shear critical plane for pile load} = 2300 + 2256 = 4556 \text{mm} \]

Ultimate maximum column load, \( N = 4622 \text{ kN} \) from table in Step 8.

Ultimate maximum pile load, \( Q = 983 \text{ kN} \)
Column punching shear stress \( \frac{N}{U_d} = \frac{4622 \times 10^3}{3200 \times 0.5 \times (800 + 784)} \)
\[= 1.82 \text{N/mm}^2 < 0.8\sqrt{f_{cu}} \text{ or } 5 \text{N/mm}^2 \text{ OK} \]

Punching shear stress at perimeter of pile \[\frac{983 \times 10^3}{\pi \times 600 \times 800} \]
\[= 0.65 \text{N/mm}^2 < 0.8\sqrt{f_{cu}} \text{ OK} \]

Pile punching shear stress \[\frac{Q}{U_d} = \frac{983 \times 10^3}{4556 \times 0.5 (800 + 784)} \]
\[= 0.27 \text{N/mm}^2 \]

Minimum \(v_c\) for Grade 40N/mm\(^2\) concrete = 0.40N/mm\(^2\) OK

**Step 13 Check area of reinforcement in pile**

Unsupported length of pile, \(l_o\), is assumed negligible.
Assume \(l_o/h < 10\).
The pile is treated as a short column. From tables in Step 8,
\[Q_{\text{max}} = 983 \text{kN} \quad \text{with } M = 28.1 \text{kNm} \]
\[Q_{\text{min}} = 367 \text{kN} \quad \text{with } M = 106.5 \text{kNm} \]
Max. shear, \(V_{\text{max}} = 42.93 \text{kN} \)
Assume minimum cover is 75 mm.

![Diagram of pile reinforcement](image)

Allowing for links and bar diameter, assume \(h_s = 420 \text{mm}\).
\[h_s = \frac{420}{600} = 0.70 = k \]
\[f_{cu} = 30 \text{N/mm}^2 \quad e = \frac{M}{N} = \frac{28.1}{983} = 0.029 \text{m} \]
\[\frac{e}{R} = \frac{0.029}{0.3} = 0.095 \]
\[\frac{Q_{\text{max}}}{h^2} = \frac{983 \times 10^3}{600 \times 600} = 2.73 \text{N/mm}^2 \]
From Table 11.19, it is observed that minimum reinforcement may be used.

Use minimum reinforcement.

For the second load case,

\[
\frac{Q_{\text{min}}}{h^2} = \frac{367 \times 10^3}{600 \times 600} = 1 \text{ N/mm}^2
\]

\[
e = \frac{R}{1}
\]

Again use minimum reinforcement.

**Step 14** Check stresses in prestressed concrete piles
Not required.

**Step 15** Check shear capacity of RC pile
No shear check is necessary if \(M_{pu}/N_u \leq 0.60h\).

\[
\frac{M_{pu}}{N_u} = \frac{106.5 \times 10^6}{367 \times 10^3} = 290 \text{ mm}
\]

\[0.60h = 0.60 \times 600 = 360 \text{ mm}\]

No shear check is necessary.

\[
\frac{H_{pu}}{0.75A_c} = \frac{42.93 \times 10^3}{0.75 \times \pi \times 600^2/4}
\]

\[= 0.20 \text{ N/mm}^2 < 0.8\sqrt{f_{cu}} \quad \text{OK}\]

**Step 16** Check shear capacity of prestressed pile
Not required.

**Step 17** Check minimum reinforcement in RC pile

\[
\frac{100A_{sc}}{A_c} \geq 0.4
\]

\[A_{sc} = \frac{A_c \times 0.4}{100} = \frac{\pi \times 300^2 \times 0.4}{100} = 1131 \text{ mm}^2\]

Use 6 no. 16 mm dia. HT bars (1206 mm²).

**Step 18** Check minimum prestress in prestressed pile
Not required.

**Step 19** Maximum reinforcement in pile
Not required.
Step 20 Containment of reinforcement in pile
Minimum dia. of links = 0.25 x bar dia. = 4 mm ≥ 6 mm
Maximum spacing of links = 12 x smallest dia. of bar = 12 x 16 = 192 mm
Use 6 mm dia. links at 175 mm centres.

Step 21 Links in prestressed piles
Not required.

Step 22 Minimum tension reinforcement in pile cap
$A_r \geq 0.0013bh$ in both directions

Minimum reinforcement in the $x$-$x$ direction = $0.0013 \times 3600 \times 900 = 4212 \, \text{mm}^2$

Provided 7536 mm$^2$ (see Step 10).

Minimum reinforcement in the $y$-$y$ direction = $0.0013 \times 5400 \times 900 = 6318 \, \text{mm}^2$

Area of 16 mm dia. bar = $201 \, \text{mm}^2$  $32 \times 201 = 6432 \, \text{mm}^2$

Area required = 3842 mm$^2$ from Step 10

Use 32 no. 16 mm dia. bars equally spaced (approximate spacing 170 mm) in the $y$-$y$ direction.

![SK 7/46 Pile cap reinforcement revised to suit minimum reinforcement.]

Step 23 Curtailment of bars in pile cap
Minimum anchorage at ends of bars is 12 x dia. of bar.
$12 \times 20 = 240 \, \text{mm}$
$12 \times 16 = 192 \, \text{mm}$

Provide a minimum 250 mm bent up length of pile bottom reinforcement.

*Check full anchorage bond length of the main tension bars.*
\( f_{cu} = 40 \text{ N/mm}^2 \)

Reinforcement used is Type 2 deformed bars.
From Table 3.29 of BS8110: Part 1: 1985,[1]

tension anchorage length = \( 32\phi = 32 \times 20 = 640 \text{ mm} \)

More than 640 mm length of bar is available beyond section 1–1 in Step 8.

**Step 24  Spacing of bars in pile cap**

Maximum percentage of reinforcement = \( p = \frac{100A_s}{bd} \)

\[
= \frac{100 \times 7536}{3600 \times 800} = 0.26\%
\]

Maximum allowed clear spacing for \( p \) less 0.3% is 3\( d \) or 750 mm, whichever is less.
Spacing of bars adopted is 150 mm.

**Step 25  Early thermal cracking**

If it is felt necessary to limit early thermal cracking of concrete in pile cap then minimum reinforcement on sides and top of pile cap should be provided based on method of calculation shown in Chapter 2.

**Step 26  Assessment of crack width in flexure**

Normally the calculations in Step 24 will deem to satisfy the crack width limitations of BS8110: Part 1: 1985.[1]

If calculations are necessary to prove the limitations of crack width due to flexure in pile cap then methods shown in Chapter 3 should be followed.

**Step 27  Connection of pile to pile cap**

From Step 17, 16 mm HT Type 2 deformed bars are used.
From Table 3.29 of BS8110,

full anchorage bond length = \( 32\phi; 32 \times 16 = 512 \text{ mm} \)

The bars from the pile will project 600 mm into pile cap. (See general recommendations for design of connections in Chapter 10.)
Fig. 7.1 Determination of pile efficiency.

Fig. 7.2 Bearing capacity factors for deep foundations.