Chapter 7

Design of Piled Foundations

7.0 NOTATION

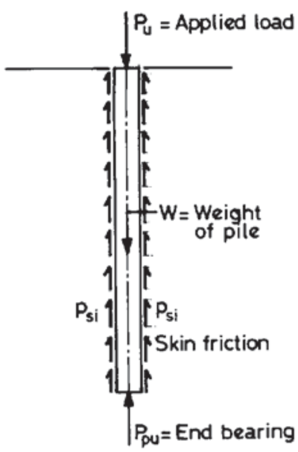
a	Deflection due to slenderness of a circular pile
a_v	Distance of shear plane from nearest support
a_x	Deflection due to slenderness producing additional moment about x-axis
a_y	Deflection due to slenderness producing additional moment about y-axis
A_c	Net area of concrete in a pile cross-section
A_p	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_{sx}	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_y
e_{hy}	Eccentricity in y-direction of combined unfactored horizontal load H_x
E_f	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_{pxu}	Ultimate horizontal load on a single pile in x-direction
H_{pyu}	Ultimate horizontal load on a single pile in y-direction
I_f	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_e	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 $0.8f_{cp}$ (average uniform prestress)
M_p	Unfactored bending moment in a single circular pile
M_x	Unfactored combined moment on pile cap about x-axis
M_y	Unfactored combined moment on pile cap about y-axis
M'_x	Modified bending moment about x-axis to account for biaxial bending
M'_y	Modified bending moment about y-axis to account for biaxial bending
M^*_x	Unfactored moment about x-axis due to eccentric surcharge on pile cap
M^*_y	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_{py}
M_{py}	Unfactored bending moment in a single pile about y-axis due to H_{px}
M_{xx}	Unfactored combined moment on pile group about x-axis
M_{yy}	Unfactored combined moment on pile group about y-axis

M_{pxu}	Ultimate bending moment in pile about x-axis
M_{pyu}	Ultimate bending moment in pile about y-axis
$M_{add\,x}$	Additional bending moment in pile about x-axis due to slenderness
$M_{add\,y}$	Additional bending moment in pile about y-axis due to slenderness
n	Slenderness ratio in a prestressed pile
N	Statistical average of SPT number for a soil stratum
N	Combined vertical load on pile cap − unfactored
N_q	Soil bearing capacity coefficient as per Terzaghi
N_u	Ultimate vertical load on a circular pile
N_γ	Soil bearing capacity coefficient as per Terzaghi
N'_c	Adjusted bearing capacity factor for cohesion
N'_q	Adjusted bearing capacity factor for $L/B > 1$
N_{uz}	Design ultimate capacity of a concrete section subjected to axial load only
N_{bal}	Design axial load capacity of a balanced section $(= 0.25 f_{cu} bd)$
p	Percentage of tensile reinforcement in a circular pile
p_x	Percentage of tensile reinforcement in a pile section to resist bending about x-axis
p_y	Percentage of tensile reinforcement in a pile section to resist bending about y-axis
P	Total vertical load on a group of piles
P_a	Allowable unfactored vertical load on pile
P_u	Ultimate axial compressive load on pile
P_{pu}	End-bearing resistance of pile
P_{si}	Skin friction resistance of pile
$\bar{q}$	Effective vertical stress at pile point
q_c	Statistical average of cone resistance of soil in a stratum (kN/m^2)
q_u	Unconfined compressive strength (kN/m^2)
q_{cs}	Side friction resistance in a cone penetrometer
R	Number of piles in a group
R_{iH}	Initial estimate of number of piles based on total horizontal load
R_{iV}	Initial estimate of number of piles based on total vertical load
s	Spacing of nodes in pile for finite element analysis
S_v	Spacing of links used as shear reinforcement
T	Unfactored torsion on a group of piles
T_a	Allowable unfactored tension load on pile
T_u	Ultimate axial tensile load on pile
U	Perimeter at punching shear plane in a pile cap
v	Shear stress in concrete in pile cap
v_c	Design concrete shear stress in concrete
v_x	Shear stress in concrete for shear due to bending about x-axis
v_y	Shear stress in concrete for shear due to bending about y-axis
v'_c	Modified design shear stress to take into account axial compression
v_{cx}	Design shear stress in concrete for shear due to bending about x-axis
v_{cy}	Design shear stress in concrete for shear due to bending about y-axis
V	Ultimate shear force in a circular pile section
V_c	Shear resistance of a concrete section
V_{co}	Shear resistance of uncracked prestressed section
V_{cr}	Shear resistance of cracked prestressed section

W	Weight of pile (kN)
z	Depth of lever arm
α	Coefficient for calculation of skin resistance of a pile
β	Factor for computation of effective length of a pile
β	Factor for conversion of biaxial bending moment into uniaxial bending
γ	Unit weight of soil (kN/m^3)
δ	Angle of friction between soil and concrete
μ	Poisson's ratio
ϕ	Angle of internal friction
ϕ	Nominal diameter of tendon in prestressed concrete section

7.1 VERTICAL LOAD – SINGLE PILE CAPACITY



SK 7/1 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
ΣP_{si} = skin friction resistance
P_{pu} = end-bearing resistance
W = weight of pile

First method for point resistance

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

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 = width or diameter of pile
L_b = 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 = cross-sectional area of pile (m^2)
q_c = statistical average of cone point resistance in a zone of about $8B$ above to $3B$ below pile point (kN/m^2)

Third method for point resistance

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

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

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

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 = pile perimeter × pile length over which f_s acts (m^2)
f_s = skin resistance (kN/m^2)

First method of skin resistance

$f_s = 2N\,kN/m^2$ for large volume displacement piles
$f_s = N\,kN/m^2$ for small volume displacement piles

where N = statistical average blow count in stratum for SPT.

Second method of skin resistance

$f_s = 0.005 q_c\,kN/m^2$

where q_c = cone penetration resistance (kN/m^2).

Third method of skin resistance

$f_s = q_{cs}\,kN/m^2$ for small volume displacement piles

$f_s = 1.5q_{cs}$ 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/m^2)
$\bar{q}$ = effective vertical stress (kN/m^2)
δ = 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).

Pile type	δ	K_s for low D_r	K_s for high D_r
Steel	20°	0.5	1.0
Concrete	0.75ϕ	1.0	2.0
Wood	0.67ϕ	1.5	4.0

(See Reference 7, page 136.)

Table 7.2 Values of α (Reference 7, page 126).

Soil condition		Values of α				
	D/B	$c = 50$	$c = 100$	$c = 150$	$c = 200$	$c = 250$
Sands or sandy gravel	<10	1.0	1.0	1.0	1.0	1.0
overlying stiff to very	20	1.0	0.9	0.75	0.75	0.75
stiff cohesive soil	>40	0.9	0.65	0.4	0.4	0.4
Soft clays or silts	10	0.35	0.30	0.25	0.2	0.2
overlying stiff to very	>20	0.75	0.70	0.63	0.55	0.5
stiff cohesive soil						
Stiff to very stiff	10	0.9	0.7	0.3	0.2	0.2
cohesive soils without	>40	1.0	0.9	0.3	0.3	0.3
overlying strata						

The units of c are kN/m^2

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

$$P_a = \frac{P_u}{2.5} \qquad T_a = \frac{T_u}{2.5}$$

where P_a = allowable pile load in compression
T_a = allowable pile load in tension

7.2 HORIZONTAL LOAD – SINGLE PILE CAPACITY

Method 1 Cohesive soils

$$k_sB = 1.3\left(\frac{E_sB^4}{E_fI_f}\right)^{\frac{1}{12}}\left(\frac{E_s}{1-\mu^2}\right)$$

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_f = stress–strain modulus of pile material (kN/m^2)
I_f = moment of inertia of pile (m^4)
μ = 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^2)
N = SPT number of blows.
(4) $E_s = 3\ (1-2\mu)/m_v$ where m_v = coefficient of volume compressibility (m^2/kN).

Method 2 Cohesive soils

$k_s = 240q_u$ kN/m^3

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

Cohesionless soils

$k_s = 80\ [C_2\bar{q}N_q + C_1\ (0.5\ \gamma\ BN_\gamma)]$ 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
$\bar{q}$ = effective stress (kN/m^2)
γ = 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:

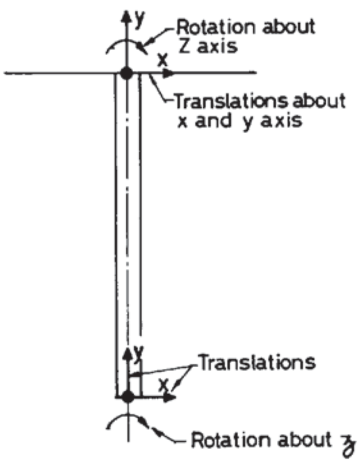
Finite element model of vertical pile

Spring stiffness = SBk_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_γ (Reference 8, page 137).

ϕ (degrees)	N_q	N_γ
0	1.0	0
5	1.6	0.1
10	2.5	0.4
15	3.9	1.2
20	6.4	2.9
25	10.7	6.8
30	18.4	15.1
35	33.3	33.9
40	64.2	79.5
45	134.9	200.8
50	319.0	568.5



SK 7/2 Two-dimensional model of pile in soil (degrees of freedom – top and bottom of pile).

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	Restrained at bottom
Rotation z	Free at bottom

(2) Fixed head pile

Translations x, y	Free at top
Rotation z	Rigid at top
Translations y	Restrained 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

$S \geq 2B$ for end-bearing piles
$S \geq 3B$ for friction piles

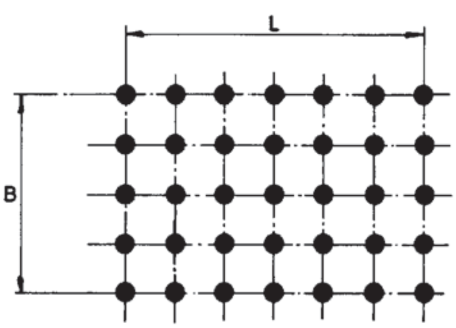
where S = spacing of piles
B = least width or diameter of pile.

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

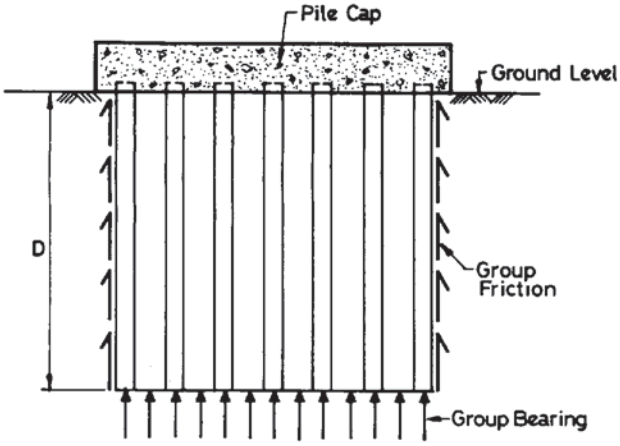
7.3.2 Pile group capacity

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

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



SK 7/3 Group of piles – plan of overall dimensions of group.



SK 7/4 Elevation of group of piles showing group capacity.

where c = average cohesion of clay
= average S_u = average $q_u/2$
α = 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 = $BL\ (N_c'c + \bar{q}N_q')$

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_q' = bearing capacity factor (see Fig. 7.2)
N_c' = bearing capacity factor (see Fig. 7.2)

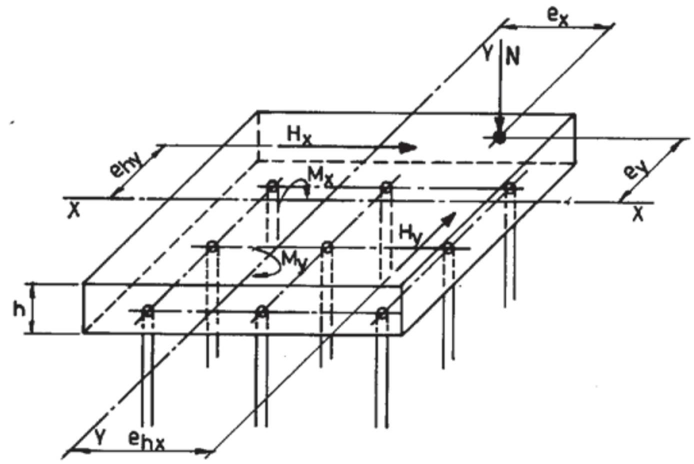
Note: Total vertical load on a group of piles should not exceed the group capacity. Individual pile loads inside the group will be limited by the single pile capacity. Piles carrying horizontal load and spaced at $3B$ or more need not be checked for group effects due to horizontal load.

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

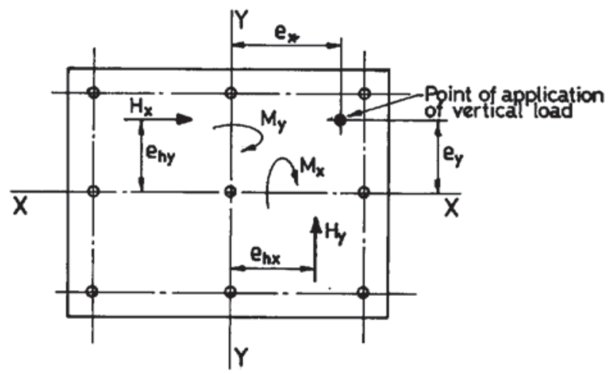
7.4 ANALYSIS OF PILE LOADS AND PILE CAPS

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



SK 7/5 Loads and eccentricity on pile cap.



SK 7/6 Plan view of loads and eccentricity on pile cap.

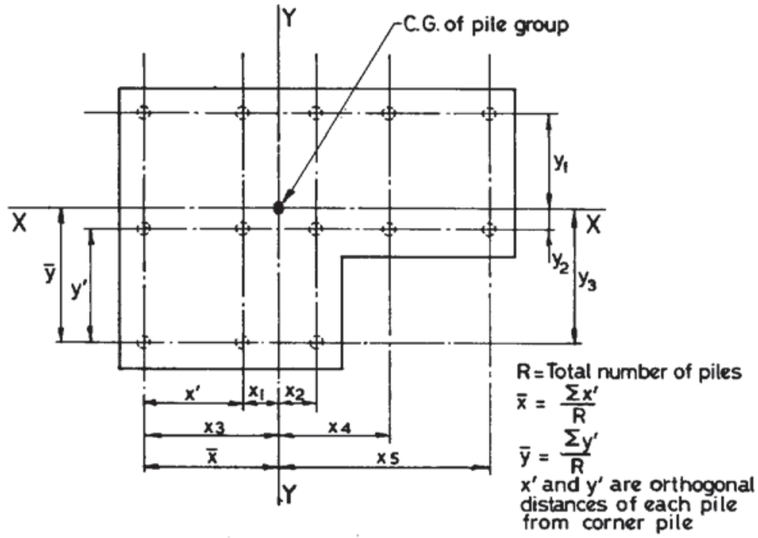
H_x = combined horizontal load on pile cap – unfactored in $x-x$ direction
H_y = combined horizontal load on pile cap – unfactored in $y-y$ direction
e_x = eccentricity of N from CG of pile group in $x-x$ direction
e_y = eccentricity of N from CG of pile group in $y-y$ direction
e_{hx} = eccentricity of H_y from CG of pile group in $x-x$ direction
e_{hy} = eccentricity of H_x from CG of pile group in $y-y$ direction
h = depth of pile cap.

Loads on pile group

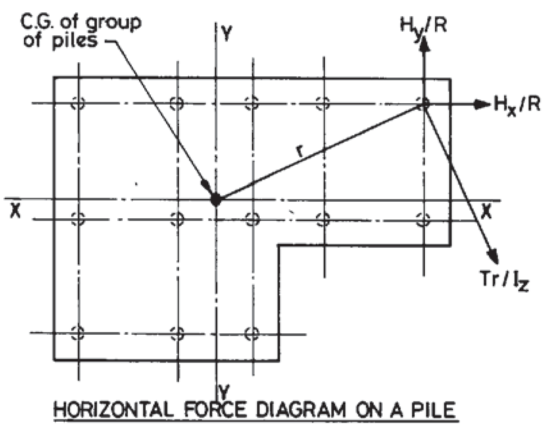
P = vertical load on pile group
= N + weight of pile cap + weight of backfill on pile cap + surcharge on backfill

M_{xx} = moment about $x-x$ on pile group
$= M_x + Ne_y + H_yh + M_x^*$

M_{yy} = moment about $y-y$ on pile group
$= M_y + Ne_x + H_xh + M_y^*$



SK 7/7 Typical pile foundation showing CG of group and co-ordinates of piles.



SK 7/8 Group of piles subject to horizontal loads and torsion.

where M_x^* and M_y^* are moments with respect to CG of pile group due to eccentric surcharge on backfill or pile cap.

T = torsion on pile group
$= H_x e_{hy} + H_y e_{hx}$

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

R = number of piles in group.

$$\text{Vertical load on a pile} = \left(\frac{P}{R}\right) \pm \left(\frac{M_{xx}y}{I_{xx}}\right) \pm \left(\frac{M_{yy}x}{I_{yy}}\right)$$

Horizontal load on any pile = resultant of $\dfrac{(H_x^2 + H_y^2)^{\frac{1}{2}}}{R}$ and $\dfrac{T(x^2 + y^2)^{\frac{1}{2}}}{I_z}$

Sign convention

Vertical loads: downwards positive
Torsion on pile group: clockwise positive

Moments on pile group: clockwise positive

+ve M_{xx} produces compression in piles which have +ve y ordinates.

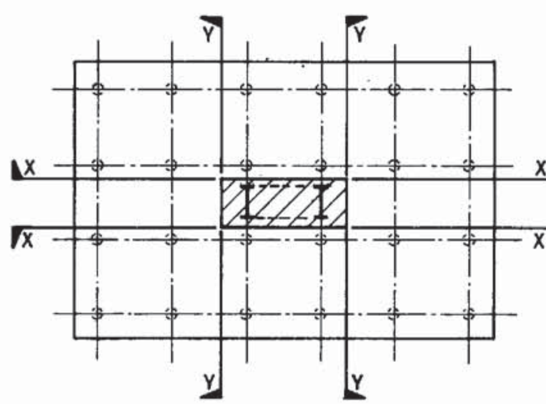
+ve M_{yy} produces compression in piles which have +ve X ordinates.

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



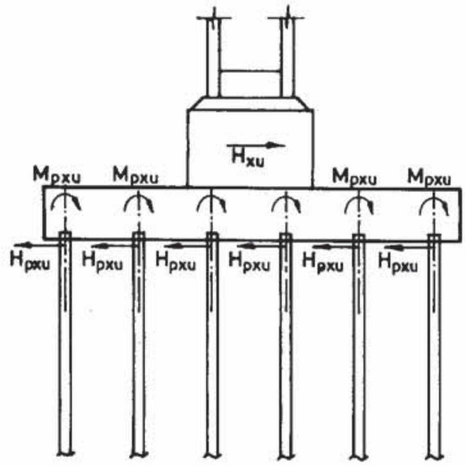
SK 7/9 Critical sections for bending moment in a pile cap.

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:

$$H_{pxu} = \frac{H_{xu}}{R}$$

$$H_{pyu} = \frac{H_{yu}}{R}$$

where R is number of piles in pile cap. Find bending moments in pile M_{pxu} corresponding to H_{pyu} and M_{pyu} corresponding to H_{pxu} assuming an end fixity to pile cap following the method in Section 7.2. H_{xu} and H_{yu} are combined factored ultimate horizontal loads.



SK 7/10 Additional bending moment in pile cap due to pile fixity.

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.

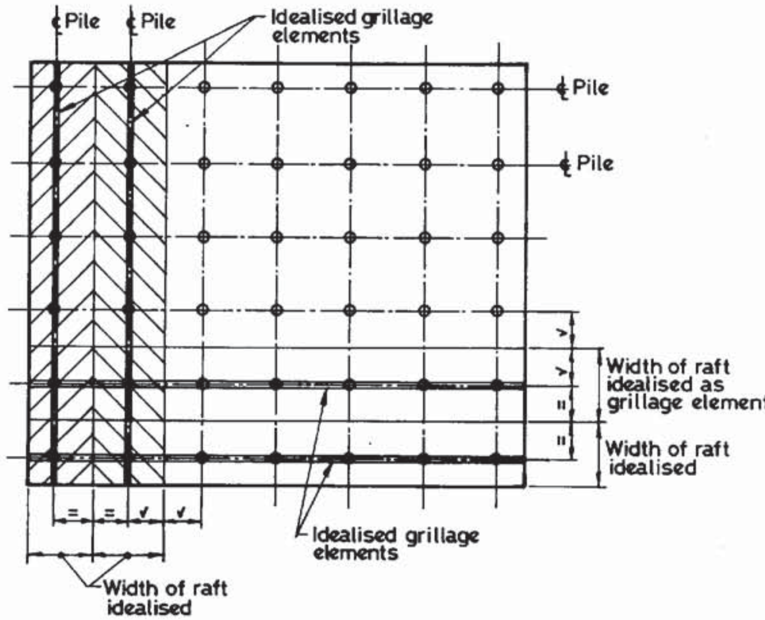
7.4.2 Flexible pile cap

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.

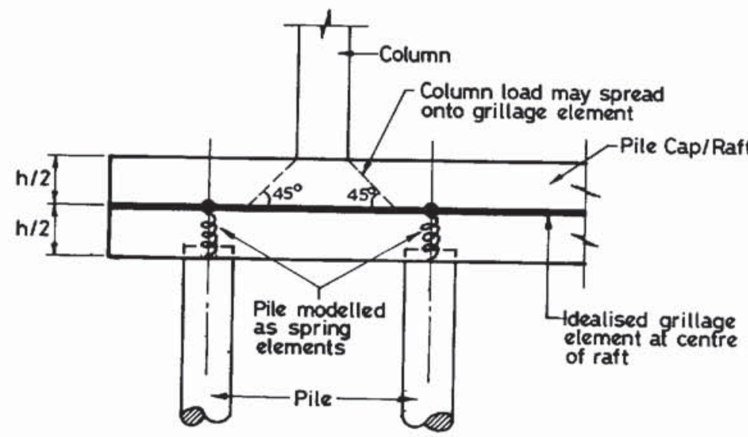
Grillage model

(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 central depth of pile cap.
(6) Apply at each node with a pile, the moments given by the following formulae:

$$M_x = \frac{H_y h}{R} \qquad \textit{about } x\text{-axis}$$



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.

$$M_y = \frac{H_x h}{R} \quad \textit{about } y\text{-axis}$$

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

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

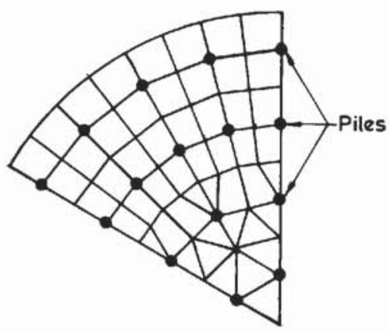
where R is total number of piles in group.

(8) Find bending moments in pile, M_{px} corresponding to H_{py} and M_{py} corresponding to H_{px}, 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



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, θx and θy. 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° 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.

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

$$M_y = \frac{H_x h}{R} \quad \text{about } y\text{-axis}$$

(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_{xy} 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

LC_1: $1.0DL + 1.0IL + 1.0EP + 1.0CLV + 1.0CLH$
LC_2: $1.0DL + 1.0EP + 1.0CLV + 1.0CLH + 1.0WL$ (or $1.0EL$)
LC_3: $1.0DL + 1.0IL + 1.0EP + 1.0WL$ (or $1.0EL$)
LC_4: $1.0L + 1.0WL$ (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

LC_5: $1.4DL + 1.6IL + 1.4EP$
LC_6: $1.2DL + 1.2IL + 1.2EP + 1.2WL$ (or $1.2EL$)
LC_7: $1.4DL + 1.4WL$ (or $1.4EL$) $+ 1.4EP$
LC_8: $1.0DL + 1.4WL$ (or $1.4EL$) $+ 1.4EP$ (if adverse)
LC_9: $1.4DL + 1.4CLV + 1.4CLH + 1.4EP$
LC_{10}: $1.4DL + 1.6CLV + 1.4EP$
LC_{11}: $1.4DL + 1.6CLH + 1.4EP$
LC_{12}: $1.2DL + 1.2CLV + 1.2CLH + 1.2EP + 1.2WL$ (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***

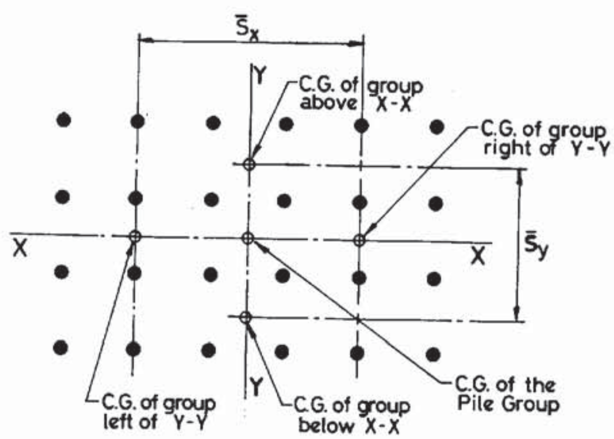
$$R_{\mathrm{iV}} = \frac{P}{C_{\mathrm{V}}}$$

$$R_{iH} = \frac{H}{C_H}$$

$$R_i = R_{iV} \quad \text{or} \quad R_{iH}, \text{ whichever is greater}$$

where R_i = approximate number of piles
P = total vertical load on pile cap – unfactored
C_V = rated working load capacity of pile – vertical load
C_H = rated working load capacity of pile – horizontal load
H = 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 $\bar{S}_x$.
(5) Similarly, find $\bar{S}_y$ about y-axis.
(6) Find $M_x/P = e_y$ and $M_y/P = e_x$, where M_x and M_y are total combined applied moments on pile cap about $x-x$ and $y-y$ respectively.
(7) Find $e_x/\bar{S}_x$ and $e_y/\bar{S}_y$.
(8) Find E_x and E_y from Fig. 7.1.
(9) $R = \dfrac{1.1\, R_{iV}}{E_x E_y} \geq R_{iH}$

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 SO_3.
Find, from Table 17 of BS 8004: 1986[2], the appropriate type of concrete.

Table 7.4 Minimum cover to reinforcement for class of exposure.

Class of exposure	Total SO_3 percentage	Minimum cover on blinding (mm)	Minimum cover elsewhere (mm)
1	<0.2	35	75
2	0.2 to 0.5	40	80
3	0.5 to 1.0	50	90
4	1.0 to 2.0	60	100
5	>2.0	60	100

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 = 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 = width of section over which moment acts
d = effective depth to tension reinforcement.

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

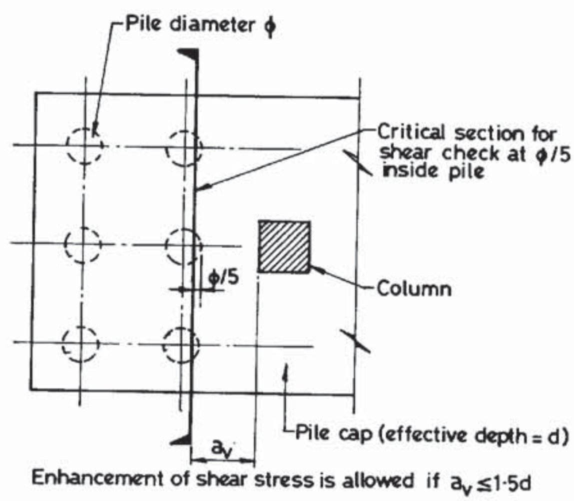
$$A_{st} = \frac{M}{0.87f_y z}$$

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

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 11 Check shear stress in pile cap



SK 7/15 Critical section for checking shear stress in pile cap.

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 3ϕ then enhancement of shear should be applied only on strips of width 3ϕ. The rest of the section will be limited to unenhanced shear stress.

$$V = \frac{\Sigma P}{Bd} \leq v_c \qquad \text{or enhanced } v_{c1} \text{ if applicable}$$

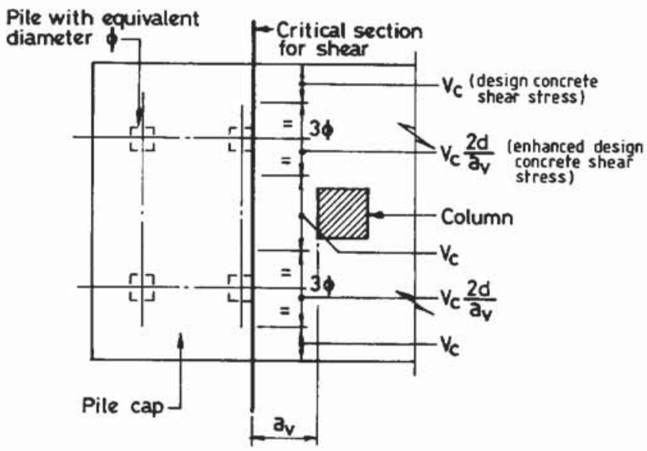
where ΣP = sum of all pile reactions at ultimate loading on left of section

B = width of pile cap at critical section

d = average effective depth at critical section

$$v_{c1} = v_c \left(\frac{2d}{a_v}\right) \leq 0.8\sqrt{f_{cu}} \quad \text{or} \quad 5\,\text{N/mm}^2$$

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

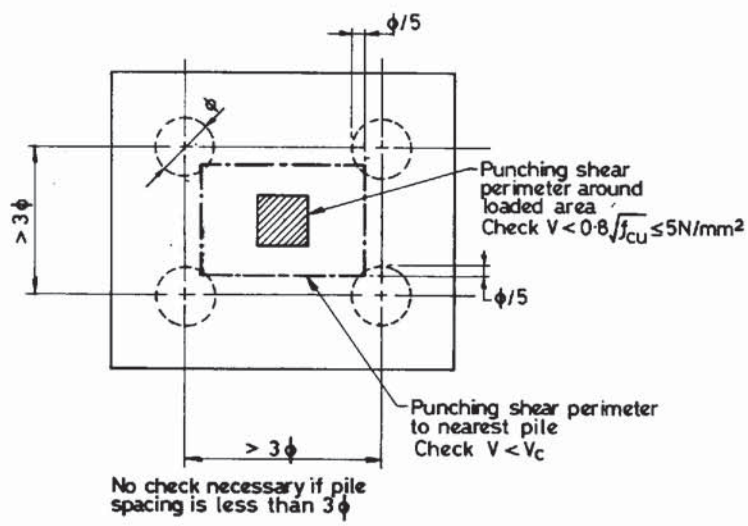


SK 7/16 Diagram showing zones of enhanced shear stress on critical section.

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.

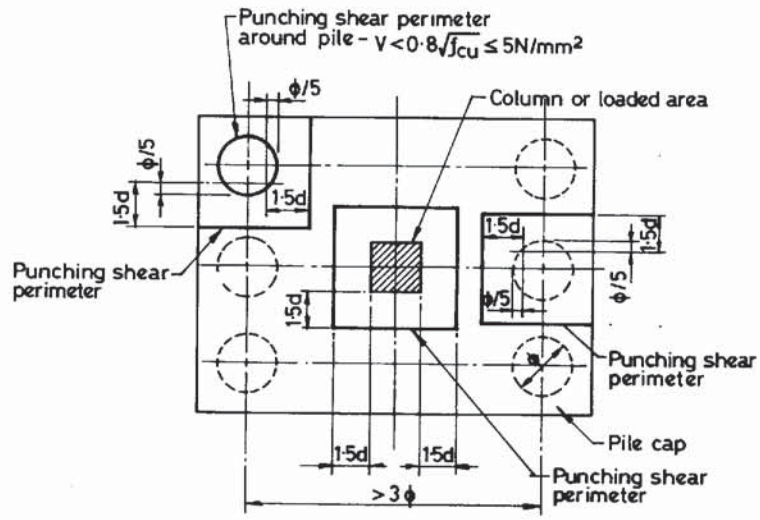
Step 12 Check punching shear stress in pile cap



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 $0.8\sqrt{f_{cu}}$ or $5\,\text{N/mm}^2$.



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.

$$v = \frac{P}{Ud} \leq v_c$$

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.

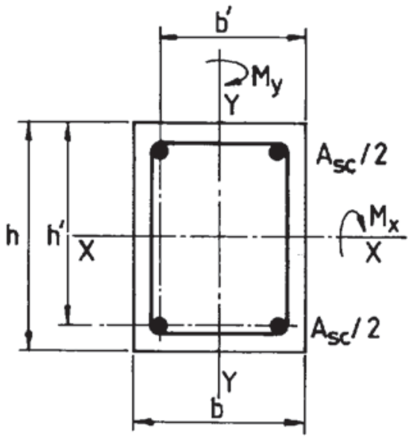
Step 13 Check area of reinforcement in pile

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)

$\beta = 1.2$ for piles with head fixed to pile cap
$= 1.6$ for piles with head free to rotate.

Rectangular piles



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*

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

Check $N \geq$ applied direct load on pile.
(ii) *Pile subjected to uniaxial moment*
Find $e = M/N$ and then e/h.
Find N/bh and select appropriate table from Tables 11.8 to 11.17 depending on f_{cu} and $k = d/h$.
From appropriate table find p which satisfies value of N/bh for given e/h.
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_x/h' and M_y/b'.
If $M_x/h' > M_y/b'$, then

$$M'_x = M_x + \beta M_y \left(\frac{h'}{b'}\right)$$

If $M_y/b' > M_x/h'$, then

$$M'_y = M_y + \beta M_x \left(\frac{b'}{h'}\right)$$

Find $N/f_{cu}bh$.
The values of β are given in the table below.

Table 7.5 Values of β for biaxial bending of pile.

$N/f_{cu}bh$	0	0.1	0.2	0.3	0.4	0.5	≥0.6
β	1.00	0.88	0.77	0.65	0.53	0.42	0.30

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}.

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

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

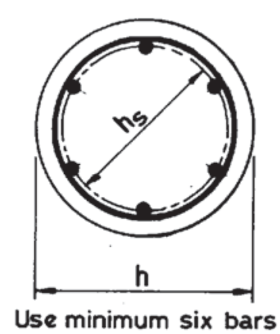
$$N_{bal} = 0.25 f_{cu} bh$$

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

$$M_{add\,x} = Na_x$$
$$M_{add\,y} = Na_y$$

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



SK 7/20 Typical section through a circular pile.

(A) If $l_e/h \leq 10$, then treat pile as a short column.
(i) *Pile with no moment*
Assume size of reinforcement and at least six bars.

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

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

Check $N \geq$ applied vertical load on pile.
(ii) *Pile with moment*
Find $e = M/N$ 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 $k = h_s/h$.
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.

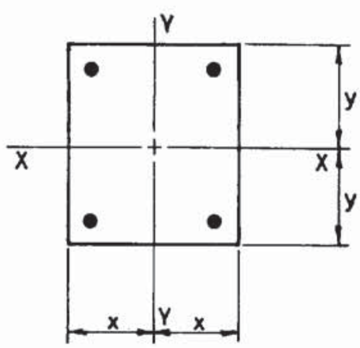
(B) If $l_e/h > 10$, then treat pile as a slender column.

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

$$M_{add} = Na$$

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



SK 7/21 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 $= 0.4 f_{cu}$

Assume pile as Class 3 member with a limiting crack width of 0.1 mm.

Hypothetical flexural tensile stress in concrete = $4.1\,N/mm^2$ for Grade 40
= $4.8\,N/mm^2$ for Grade 50 and above

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

Depth (mm)	Factor
Up to 400	1.0
500	0.95
600	0.9

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.

$$\text{Maximum compressive stress in concrete} = \left(\frac{P+N}{A_c}\right) + \left(\frac{M_x y}{I_{xx}}\right) + \left(\frac{M_y x}{I_{yy}}\right)$$

$$\text{Maximum tensile stress in concrete} = \left(\frac{P+N}{A_c}\right) - \left(\frac{M_x y}{I_{xx}}\right) - \left(\frac{M_y x}{I_{yy}}\right)$$

m = modular ratio
f_s = strand stress prior to release
f_c = stress in concrete due to prestress alone.

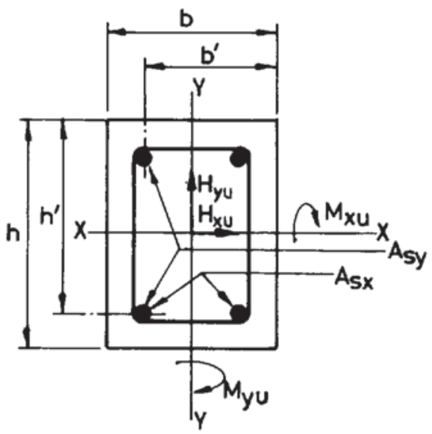
(1) Loss due to elastic shortening = $\left(\frac{100mf_c}{f_s}\right)\%$
(2) Loss due to relaxation of steel – refer to strand manufacturer's brochure.
(3) Loss due to creep of concrete – follow clause 4.8.5 of BS 8110: Part 1.[1]
(4) Loss due to shrinkage of concrete – follow clause 4.8.4 of BS 8110: 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
ϕ = nominal diameter of tendon.

Step 15 Check shear capacity of RC pile



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

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

Rectangular piles

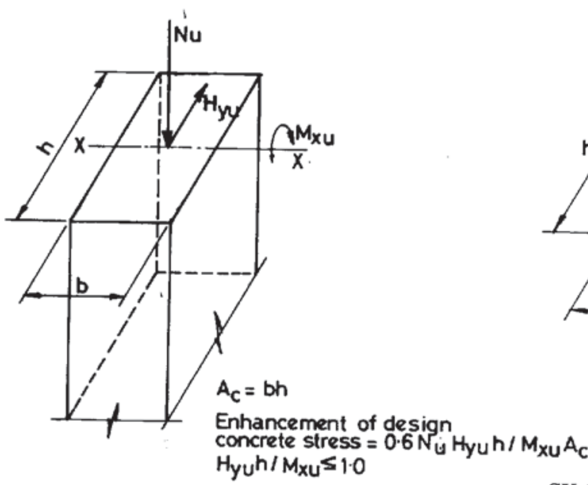
No shear check is necessary if:
$M_{pxu}/N_u \leq 0.6h$
and $M_{pyu}/N_u \leq 0.6b$
and $H_{pyu}/bh' \leq 0.8\sqrt{f_{cu}} \leq 5\,\text{N/mm}^2$
and $H_{pxu}/hb' \leq 0.8\sqrt{f_{cu}} \leq 5\,\text{N/mm}^2$

Shear check is necessary if:
$M_{pxu}/N_u > 0.6h$ and/or $M_{pyu}/N_u > 0.6b$
Find $v_x = H_{pyu}/bh'$ and $v_y = H_{pxu}/hb'$
Find $p_x = 100A_{sx}/bh'$ and $p_y = 100A_{sy}/hb'$

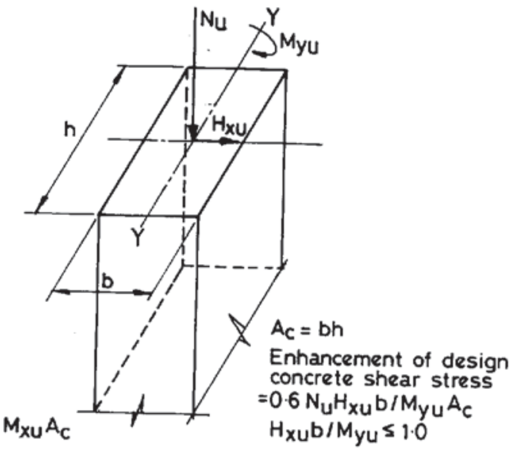
Find v_{cx} and v_{cy} corresponding to p_x and p_y from Figs 11.2 to 11.5.

$$\text{Check} \quad \frac{v_x}{v_{cx}} + \frac{v_y}{v_{cy}} \leqslant 1$$

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



SK 7/23 Shear stress enhancement due to presence of axial load.



SK 7/24 Shear stress enhancement due to presence of axial load.

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

$$v'_{cx} = v'_{cx} + \frac{0.6 N_u H_{pyu} h}{A_c M_{pxu}} \leq 0.8\sqrt{f_{cu}} \leq 5\,\text{N/mm}^2$$

$$v'_{cy} = v'_{cy} + \frac{0.6 N_u H_{pxu} b}{A_c M_{pyu}} \leq 0.8\sqrt{f_{cu}} \leq 5\,\text{N/mm}^2$$

$H_{pyu}h/M_{pxu}$ and $H_{pxu}b/M_{pyu}$ should be less than or equal to 1.0.

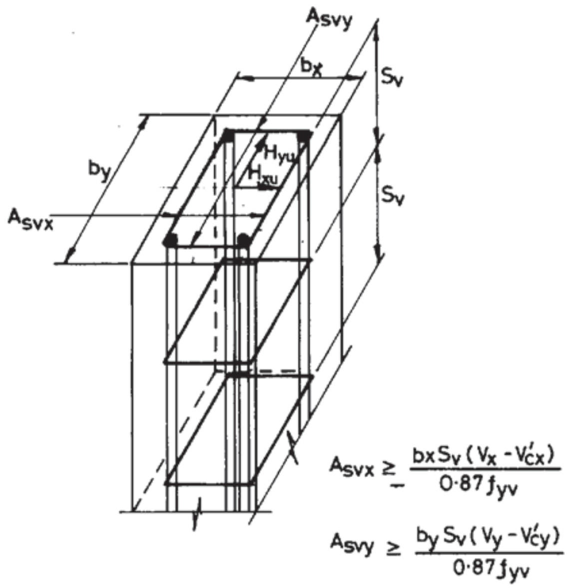
Shear reinforcement

$$A_{sv} = \frac{b S_v (v - v'_c)}{0.87 f_{yv}}$$

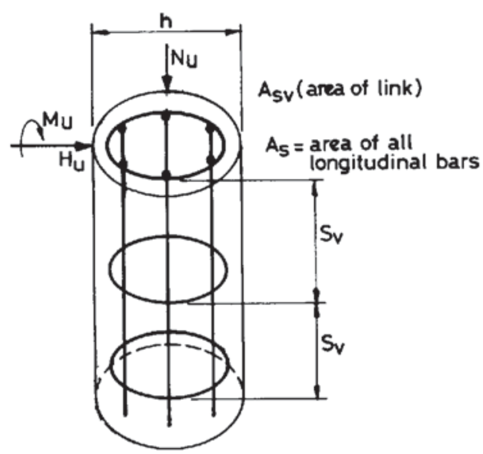
where A_{sv} = total area of legs in direction of shear
b = width of section perpendicular to direction of shear
S_v = spacing of links
$f_{yv} \leq 460\,\text{N/mm}^2$ for links.

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}



SK 7/25 Shear reinforcement in a rectangular pile.



SK 7/26 Shear reinforcement in a circular pile.

No shear check is necessary if:

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

where $A_c = 0.25\pi h^2$.

Shear check is necessary if:

$M_{pu}/N_u > 0.60h$

Shear stress, $v = H_{pu}/0.75A_c$

$p = 100A_s/1.5A_c$ 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:

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

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

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

$$V_s = 0.87f_{yv}A_v\left(\frac{z}{s}\right) \qquad V_c = 0.75v_c'A_c$$

where A_v = total area of link bars perpendicular to longitudinal bars, i.e. the two legs of hoop reinforcement
f_{yv} = characteristic yield strength of link reinforcement
S = spacing of links.

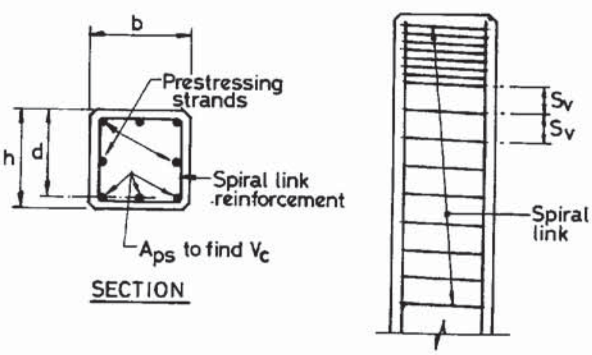
Find z/R from appropriate table from Tables 11.18 to 11.27 corresponding to f_{cu}, h_s/h, p, N/R^2 and e/R.

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

The total shear resistance for inclined links =
$V_s = [0.87f_yA_{sv}\ (\cos\alpha + \sin\alpha\cot\beta)\ (z/S)]$

where A_{sv} = total area of link bars i.e. the two legs of hoop reinforcement.
β may be taken as 45° when α is angle of inclination of link.

Step 16 Check shear capacity of prestressed pile



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

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

$$V_{cr} = \left(1 - \frac{0.55f_{pe}}{f_{pu}}\right) v_cbd + \frac{M_oV}{M} \geq 0.1bd\sqrt{f_{cu}}$$

$V_c = V_{co}$ or V_{cr} as the case may be (kN) – design ultimate shear resistance

V_{co} = shear resistance of section uncracked (kN)

V_{cr} = shear resistance of section cracked (kN)

f_t = maximum design principal stress at the centroidal axis $= 0.24\sqrt{f_{cu}}$

f_{cp} = design compressive stress at centroidal axis of concrete section due to prestress alone

f_{pe} = design effective prestress in tendons after all losses $\leq 0.6 f_{pu}$

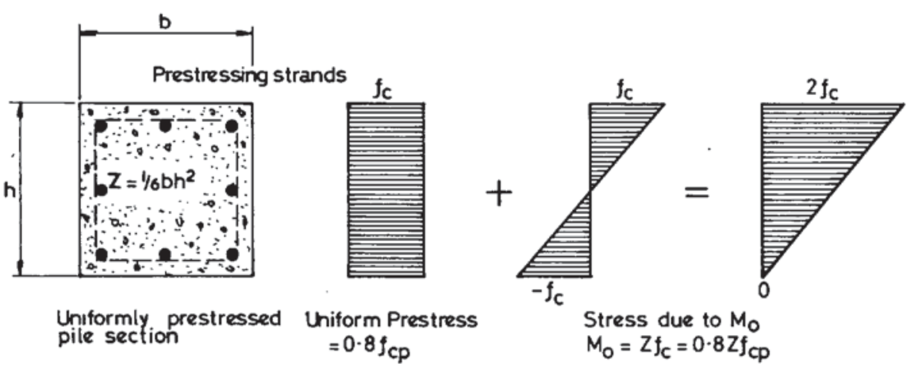
f_{pu} = characteristic ultimate strength of tendons

v_c = 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 = effective depth to centroid of reinforcing steel in tension zone where reinforcing steel should include tendons and any untensioned reinforcement

f_{cu} = characteristic cube concrete strength at 28 days

M_o = moment to produce zero stress at tension fibre with $0.8 f_{cp}$ on section.



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,

$$\frac{A_{sv}}{S_v} = \frac{0.4b}{0.87 f_{yv}}$$

If horizontal shear on pile, H_{pu}, is more than $(V_c + 0.4bd)$ then,

$$\frac{A_{sv}}{S_v} = \frac{H_{pu} - V_c}{0.87 f_{yv} d}$$

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 $= n = \dfrac{l}{b}$

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

Minimum prestress after losses = 60n psi
or = $0.4n$ N/mm^2

If diesel hammer is used,

minimum prestress in concrete = 5 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 ≥ 6 mm

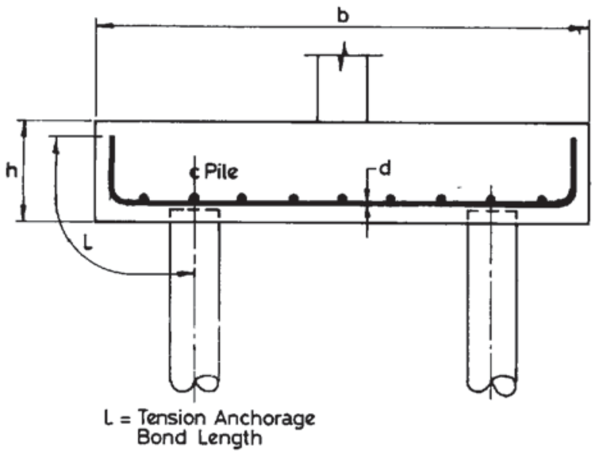
Maximum spacing of links = $12 \times$ smallest dia. of bar

Step 21 ***Links in prestressed piles***
At top and bottom $3B$ 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 $3d$ or 750 mm.



SK 7/29 Typical section through a pile cap.

Percentage of reinforcement, $100A_s/bd$ (%)	Maximum clear spacing of bars in pile cap (mm)
1 or over	160
0.75	210
0.5	320
0.3	530
Less than 0.3	$3d$ or 750

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 × 800 mm
Spacing of column = 8 m × 8 m on plan

Unfactored column loads

	Dead	Imposed	Wind
Vertical load, N (kN)	1610	1480	—
Horizontal shear, H_x (kN)	28	18	156
Horizontal shear, H_y (kN)	—	—	112
Moment, M_x (kNm)	—	—	448
Moment, M_y (kNm)	112	72	624

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)
$c = 11.3\,\text{kN/m}^2$
$\phi = 4°$
$\gamma = 26\,\text{kN/m}^3$

Stratum 2
Average thickness of layer = 9 m

Classification: soft to medium bluish-grey clayey silt.

Average N = 5 (SPT)
$c = 20.2\,\text{kN/m}^2$
$\phi = 5°$
$\gamma = 24\,\text{kN/m}^3$
$\gamma_{sat} = 27\,\text{kN/m}^3$

Stratum 3
Average thickness of layer = 2 m

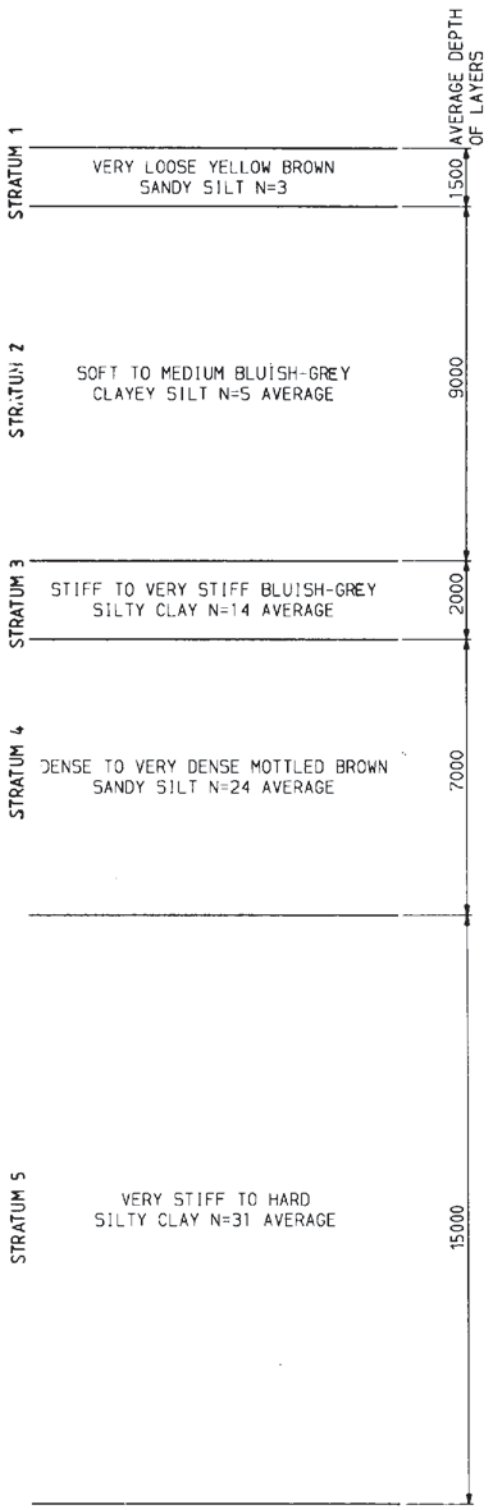
Classification: stiff to very stiff bluish-grey silty clay.

Average N = 14 (SPT)
$c = 60\,\text{kN/m}^2$
$\phi = 6°$
$\gamma_{sat} = 26\,\text{kN/m}^3$

Stratum 4
Average thickness of layer = 7 m

Classification: dense to very dense mottled brown sandy silt.

Average N = 24 (SPT)
$c = 13.8\,\text{kN/m}^2$
$\phi = 31°$
$\gamma_{sat} = 27\,\text{kN/m}^3$



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)

$c = 71.5\,\text{KN/m}^2$

$\phi = 8°$

$\gamma_{sat} = 28\,\text{kN/m}^3$

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

$$P_{pu} = A_p(38N)\left(\frac{L_b}{B}\right)$$

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 $= \pi \times \dfrac{0.6^2}{4} = 0.283\,\text{m}^2$

$B = 0.60\,\text{m}$

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

$$P_{pu} = A_p(N'_c c + \bar{q}N'_q)$$

$A_p = 0.283\,\text{m}^2$

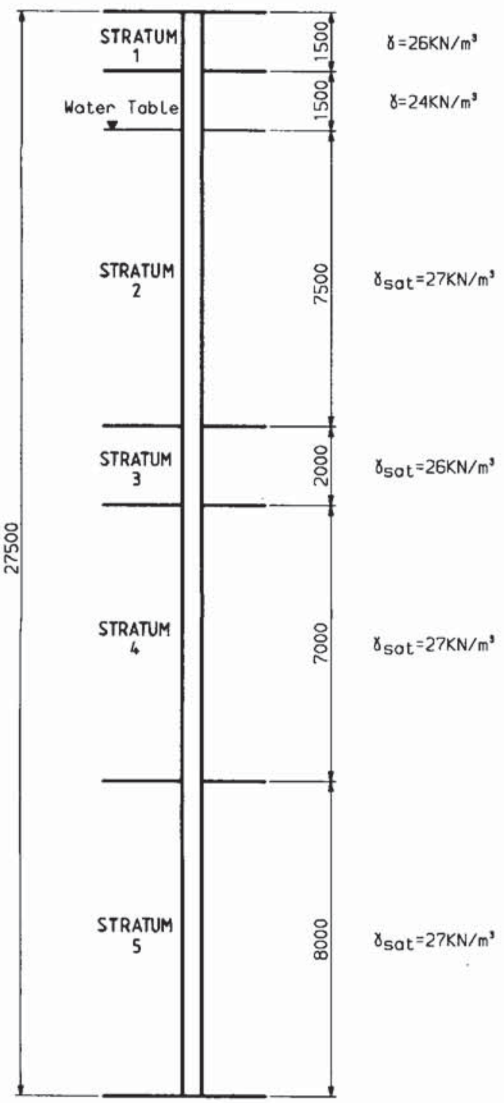
$c = 71.5\,\text{kN/m}^2$

$\gamma_w = 10\,\text{kN/m}^3$

$\bar{q}$ = 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.

$L = 27.5\,\text{m} \qquad B = 0.60\,\text{m}$

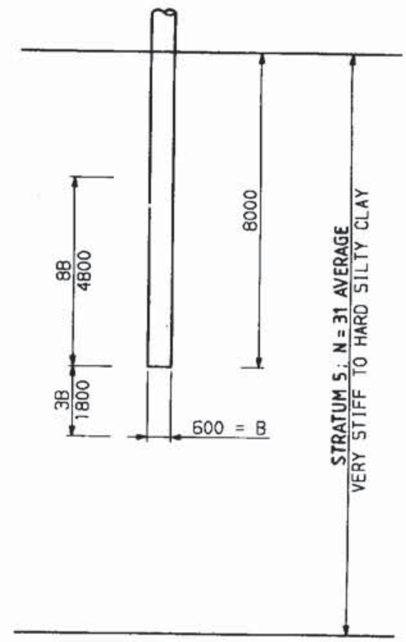
$L/B = 46 \qquad \phi = 8°$

From Fig. 7.2,

$N'_q = 3 \qquad N'_c = 15 \quad \text{and} \quad L_c/B = 3.5$

$$\frac{L}{B} >> \frac{L_c}{B}$$

$P_{pu} = 0.283\,[(15 \times 71.5) + (3 \times 489.5)] = 719\,\text{kN}$



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

$f_s = N \text{kN/m}^2$

Stratum 1

$$A_{s1} = \text{perimeter} \times \text{depth of stratum}$$
$$= \pi \times 0.60 \times 1.5$$
$$= 2.83 \text{m}^2$$

$$f_{s1} = 3 \text{kN/m}^2$$

$$P_{si1} = 3 \times 2.83 = 8.5 \text{kN}$$

Stratum 2

$$A_{s2} = \pi \times 0.60 \times 9 = 17 \text{m}^2$$

$$f_{s2} = 5 \text{kN/m}^2$$

$$P_{si2} = 5 \times 17 = 85 \text{kN}$$

Stratum 3

$$A_{s3} = \pi \times 0.60 \times 2 = 3.8 \text{m}^2$$

$$f_{s3} = 14 \text{kN/m}^2$$

$$P_{si3} = 14 \times 3.8 = 53.2 \text{kN}$$

Stratum 4

$A_{s4} = \pi \times 0.60 \times 7 = 13.2\,m^2$

$f_{s4} = 24\,kN/m^2$

$P_{si4} = 13.2 \times 24 = 316.8\,kN$

Stratum 5

$A_{s5} = \pi \times 0.60 \times 8 = 15.1\,m^2$

$f_{s5} = 31\,kN/m^2$

$P_{si5} = 15.1 \times 31 \times 468.1\,kN$

$\Sigma P_{si} = 931.6\,kN$

Fourth method of skin resistance

$f_s = \alpha c + 0.5\bar{q}K_s \tan\delta$

Ignore the second term because δ is very small.

Stratum 1

$\alpha = 0.75 \qquad c = 11.3\,kN/m^2$

$P_{si1} = A_{s1} \times f_{s1}$
$P_{si1} = 0.75 \times 11.3 \times 2.83 = 24\,kN$

$A_{s1} = 2.83\,m^2$

Stratum 2

$\alpha = 0.75 \qquad c = 20.2\,kN/m^2$

$P_{si2} = 0.75 \times 20.2 \times 17 = 257.2\,kN$

$A_{s2} = 17\,m^2$

Stratum 3

$\alpha = 0.75 \qquad c = 60\,kN/m^2$

$P_{si3} = 0.75 \times 60 \times 3.8 = 171\,kN$

$A_{s3} = 3.8\,m^2$

Stratum 4

$A_{s4} = 13.2\,m^2$

$\alpha = 2.0$ say with high D_r

$c = 13.8\,kN/m^2 \qquad \phi = 31°$

$K_s = 2.0$ from chart

$\delta = 0.75\phi = 23.25°$

$\tan\delta = 0.43$

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

$f_s = \alpha c + 0.5\bar{q}\ K_s \tan\delta$

$P_{si4} = 13.2\ [2 \times 13.8) + (0.5 \times 294 \times 2 \times 0.43)] = 2033\,\text{kN}$

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.

$$P_u = P_{pu} + P_{su} = 719 + 932 = 1651\,\text{kN}$$

$$\text{Allowable working load on pile} = \frac{1651}{2.5} = 660\,\text{kN}$$

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 = SPT No.

E_s of Stratum 1 = 650×3 = $1950\,\text{kN/m}^2$
E_s of Stratum 2 = 650×5 = $3250\,\text{kN/m}^2$
E_s of Stratum 3 = 650×14 = $9100\,\text{kN/m}^2$
E_s of Stratum 4 = 650×24 = $15\,600\,\text{kN/m}^2$
E_s of Stratum 5 = 650×31 = $20\,150\,\text{kN/m}^2$

$$k_sB = 1.3\left(\frac{EsB^4}{E_fI_f}\right)^{\frac{1}{12}}\left(\frac{E_s}{1-\mu^2}\right)$$

$E_f = 28 \times 10^6\,\text{kN/m}^2$ for pile concrete

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

$k_{s1}B = 1672\,\text{kN/m}^2$ $k_{s1} = 2787\,\text{kN/m}^3$
$k_{s2}B = 2909\,\text{kN/m}^2$ $k_{s2} = 4848\,\text{kN/m}^3$
$k_{s3}B = 8875\,\text{kN/m}^2$ $k_{s3} = 14\,792\,\text{kN/m}^3$
$k_{s4}B = 15\,914\,\text{kN/m}^2$ $k_{s4} = 26\,523\,\text{kN/m}^3$
$k_{s5}B = 20\,999\,\text{kN/m}^2$ $k_{s5} = 34\,998\,\text{kN/m}^3$

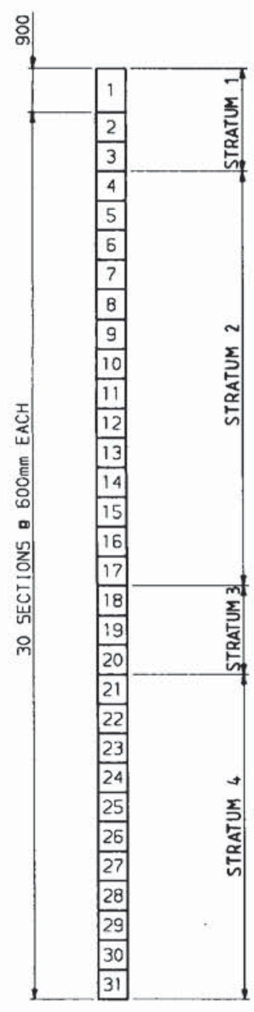
Method 2

$$k_s = 240q_u \text{ kN/m}^2$$
$$= 480c \text{ kN/m}^2$$

$$k_{s1} = 480 \times 11.3 = 5424 \text{ kN/m}^3$$
$$k_{s2} = 480 \times 20.2 = 9696 \text{ kN/m}^3$$
$$k_{s3} = 480 \times 60 = 28\,800 \text{ kN/m}^3$$
$$k_{s4} = 480 \times 13.8 = 6624 \text{ kN/m}^3$$
$$k_{s5} = 480 \times 71.5 = 34\,320 \text{ kN/m}^3$$

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 = node spacing for finite element analysis = 0.60 m
B = 0.60 m
spring stiffness = SBk_s 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 $= 0.60 \times 0.60 \times 14\,792$
$= 5325\,\text{kN/m}$

Stratum 4
Spring stiffness $= 0.60 \times 0.6 \times 26\,523 = 9548\,\text{kN/m}$

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.

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

Results of computer run

Maximum moment $= 2.48\,\text{kNm/kN}$

Pile top deflection $= 0.12\,\text{mm/kN}$

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

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 mm.

Maximum horizontal load $= 28 + 18 + 156 = 202\,\text{kN} = H$

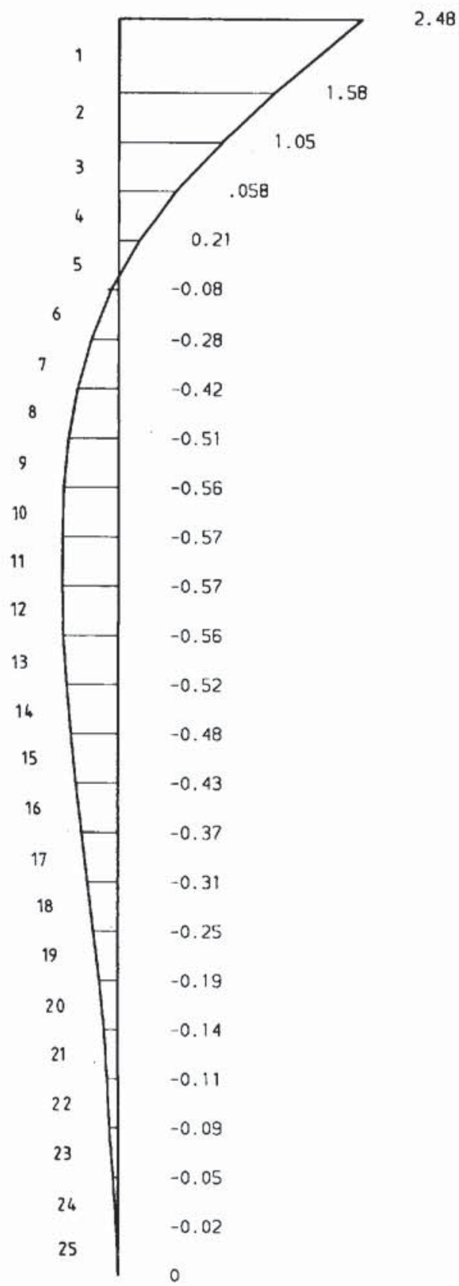
Maximum horizontal load on pile to limit deflection to 10 mm
$= 8333 \times 0.010$
$= 83\,\text{kN}$ per pile

$$R_{iH} = \frac{H}{C_H} = \frac{202}{83} = 2.4$$

R_i = greater of R_{iV} and $R_{iH} = 4.7$

$1.1R_i = 4.7 \times 1.1 = 5.17$

Use 6 no. piles.



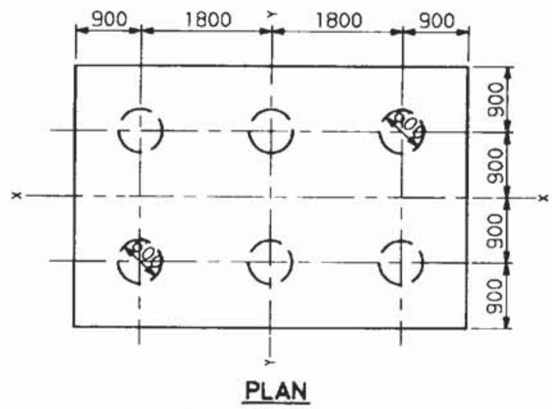
SK 7/34 Bending moment (kNm) due to 1 kN horizontal load at top of pile.

Step 5 Determine size of pile cap

B = diameter of pile = 0.6 m

$1.5B = 1.5 \times 0.6 = 0.9\,\text{m}$

Allow 0.9 m from centre of pile to edge of pile cap.
Assume 0.9 m depth of pile cap.



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

$H_x = 46\,\text{kN}$
$H_y = 112\,\text{kN}$

$M_x = 448\,\text{kNm}$
$M_y = 184\,\text{kNm}$

$LC_4 = 1.0DL + 1.0WL$

$N = 1610\,\text{kN}$

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

$LC_5 = 1.4DL + 1.6IL$

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

$H_x = 1.4 \times 28 + 1.6 \times 18 = 68\,\text{kN}$
$H_y = 0\,\text{kN}$

$M_x = 0\,\text{kNm}$
$M_y = 1.4 \times 112 + 1.6 \times 72 = 272\,\text{kNm}$

$LC_6 = 1.2DL + 1.2IL + 1.2WL$

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

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

$H_x = 1.2 \times (28 + 18) = 55.2\,\text{kN}$
$H_y = 1.2 \times 112 = 134.4\,\text{kN}$

$M_x = 1.2 \times 448 = 537.6\,\text{kNm}$
$M_y = 1.2 \times (112 + 72) = 220.8\,\text{kNm}$

$LC_7 = 1.4DL + 1.4WL$

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

Wind in x–x direction

$H_x = 1.4\,(28 + 156) = 257.6\,\text{kN}$
$H_y = 0\,\text{kN}$

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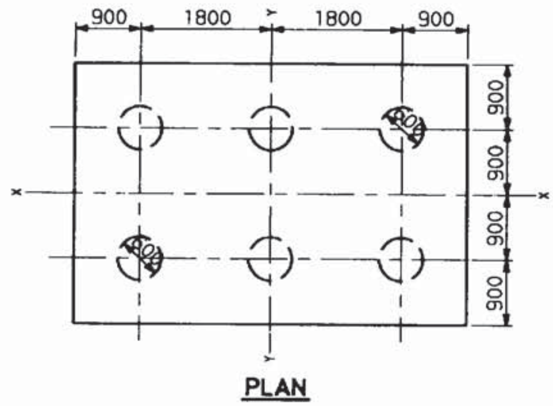


SK 7/35 Layout of piles under pile cap.

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

$H_x = 46\,\text{kN}$
$H_y = 112\,\text{kN}$

$M_x = 448\,\text{kNm}$
$M_y = 184\,\text{kNm}$

$LC_4 = 1.0DL + 1.0WL$

$c = 71.5\,kN/m^2$ at bottom of group

$\bar{q}$ = effective stress at bottom of group = $489.5\,kN/m^2$ (see Step 2)

$$\left.\begin{aligned} N'_q &= 3 \\ N'_c &= 15 \end{aligned}\right\} \text{for } \phi = 8°$$

Group end-bearing capacity = $1.8 \times 3.6 \times (15 \times 71.5 + 489.5 \times 3)$
= 16465 kN

Ultimate group capacity = 7996 + 16465 = 24461 kN

Allowable group capacity = $\dfrac{24461}{2.5}$ = 9784 kN

Allowable group capacity based on single pile capacity = 6×660 = 3960 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 $5\,kN/m^2$ on backfill with no eccentricity.
It is always advisable to use the table as presented.

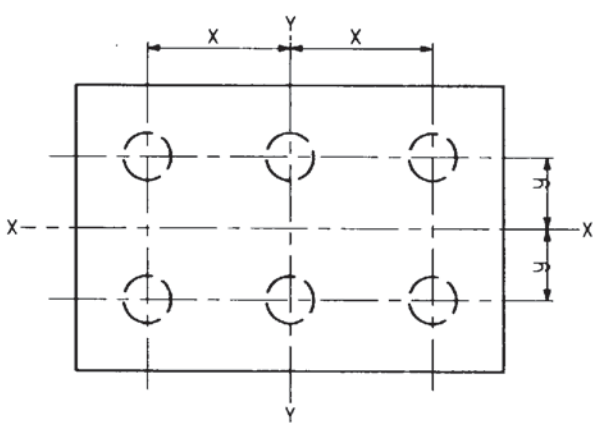
W = weight of pile cap
+ weight of backfill on pile cap
+ weight of surcharge on backfill

$= 5.4\,m \times 3.6\,m \times 0.9\,m \times 24\,kN/m^3$
$+ 5.4 \times 3.6 \times 0.5\,m \times 20\,kN/m^3$
$+ 5.4 \times 3.6 \times 5\,kN/m^2$

= 712 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.

Analysis of loads on pile cap.

Load case	N	M_x	M_y	H_x	H_y	e_x	e_y	e_{hx}	e_{hy}	h	P or P_u	M_x^*	M_y^*	M_{xx}	M_{yy}	T
LC_1	3090	0	184	46	0	0	0	0	0	0.9	3802	0	0	0	225.4	0
LC_3	3090	0	808	202	0	0	0	0	0	0.9	3802	0	0	0	989.8	0
LC_3	3090	448	184	46	112	0	0	0	0	0.9	3802	0	0	548.8	225.4	0
LC_4	1610	0	736	184	0	0	0	0	0	0.9	2322	0	0	0	901.6	0
LC_4	1610	448	112	28	112	0	0	0	0	0.9	2322	0	0	548.8	137.2	0
LC_5	4622	0	272	68	0	0	0	0	0	0.9	5619	0	0	0	333.2	0
LC_6	3708	0	969.6	242.4	0	0	0	0	0	0.9	4562	0	0	0	1187.6	0
LC_6	3708	537.6	220.8	55.2	134.4	0	0	0	0	0.9	4562	0	0	658.6	270.5	0
LC_7	2254	0	1030.4	257.6	0	0	0	0	0	0.9	3251	0	0	0	1262.2	0
LC_7	2254	627.2	156.8	39.2	156.8	0	0	0	0	0.9	3251	0	0	768.3	192.1	0

$M_{xx} = M_x + Ne_y + H_y h + M_x^*$ $\quad M_{yy} = M_y + Ne_x + H_x h + M_y^*$

$T = H_x e_{hy} + H_y e_{hx}$ $\quad P = N + W$ $\quad P_u = N + 1.4W$ (or $1.2W$)

Loads on pile.

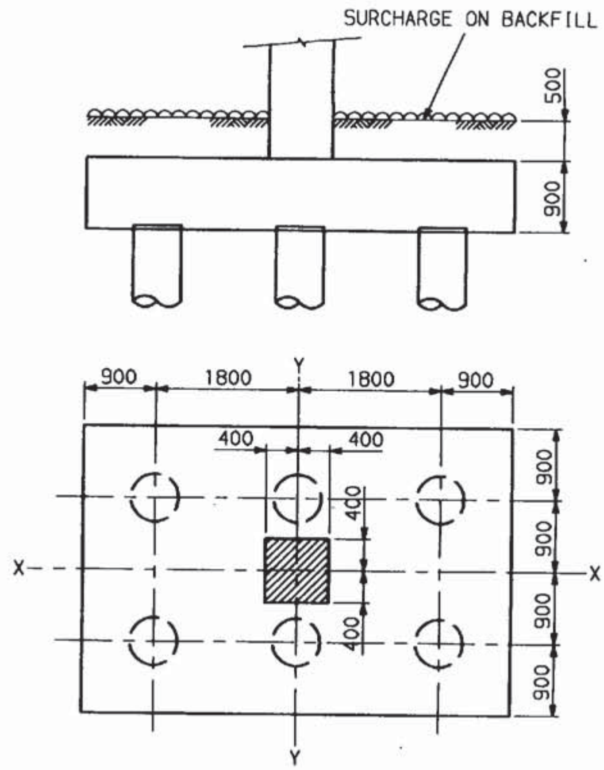
Load case	P or P_u	H_x	H_y	M_{xx}	M_{yy}	T	Q_{max}	Q_{min}	H or H_{pu}	M_p or M_{pu}	δ (mm)
LC_1	3802	46	–	–	225.4	–	665	602	7.67	19.0	0.9
LC_3	3802	202	–	–	989.8	–	771	496	33.67	83.5	4.0
LC_3	3802	46	112	548.8	225.4	–	767	501	20.18	50.0	2.4
LC_4	2322	184	–	–	901.6	–	512	262	30.67	76.1	3.7
LC_4	2322	28	112	548.8	137.2	–	508	266	19.24	47.7	2.3
LC_5	5619	68	–	–	333.2	–	983	890	11.33	28.1	1.4
LC_6	4562	242.4	–	–	1187.6	–	925	595	40.40	100.2	4.8
LC_6	4562	55.2	134.4	658.6	270.5	–	920	601	24.21	60.0	2.9
LC_7	3251	257.6	–	–	1262.2	–	717	367	42.93	106.5	5.2
LC_7	3251	39.2	156.8	768.3	192.1	–	711	373	26.94	66.8	3.2

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

$$Q_{max} = \frac{P}{R} + \frac{M_{xx}y}{I_{xx}} + \frac{M_{yy}x}{I_{yy}} \qquad Q_{min} = \frac{P}{R} - \frac{M_{xx}y}{I_{xx}} - \frac{M_{yy}x}{I_{yy}}$$

$$H = \frac{\sqrt{(H_x^2 + H_y^2)}}{R} \qquad R = \text{no. of piles} = 6$$

M_p = bending moment in pile = $2.48H$ (see Step 3) $\quad x = 1.8\,\text{m} \quad y = 0.9\,\text{m}$
δ = horizontal displacement at top of pile = $0.12H$ mm (see Step 3)

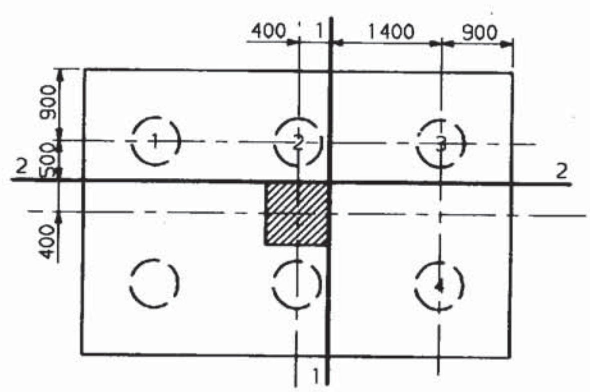


SK 7/38 General arrangement of pile cap and piles.

Allowable service load on pile without wind = 660 kN OK

Allowable service load on pile with wind = 660 × 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:

$1.4 \times 36.6 = 51.2\,\text{kN/m}^2$
$1.2 \times 36.6 = 43.9\,\text{kN/m}^2$

M'_{11} = bending moment due to dead load of pile cap etc. on section 1−1

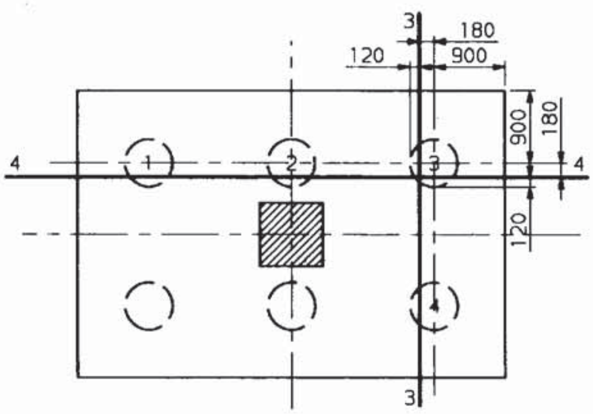
$$= \frac{3.6 \times 51.2 \times 2.3^2}{2} = 487.5\,\text{kNm}$$

$$\text{or} \quad = \frac{3.6 \times 43.9 \times 2.3^2}{2} = 418.0\,\text{kNm}$$

M'_{22} = Bending moment due to dead load of pile cap etc. on section 2−2

$$= \frac{5.4 \times 51.2 \times 1.4^2}{2} = 271.0\,\text{kNm}$$

$$\text{or} \quad = \frac{5.4 \times 43.9 \times 1.4^2}{2} = 232.3\,\text{kNm}$$



SK 7/40 Critical sections for shear.

Step 9 ***Determine cover to reinforcement***

From soil test reports, the total SO_3 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***

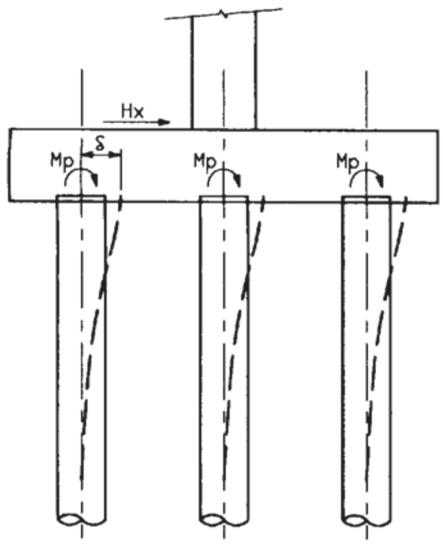
M = bending moment in pile cap   as found in Step 8.

M_{11} = 2264.9 kNm   from table in Step 8.

Bending moments and shear in pile cap.

Load case	Q_1	Q_2	Q_3	Q_4	M'_{11}	M'_{22}	M''_{11}	M''_{22}	M_{11}	M_{22}	V'_{33}	V'_{44}	V''_{33}	V''_{44}	V_{33}	V_{44}
LC_5	890	937	983	983	−487.5	−271.0	2752.4	1405	2264.9	1134.0	−199.1	−298.6	1966	2810	1766.9	2511.4
LC_6	595	760	925	925	−418.0	−232.3	2590.0	1140	2172.0	907.7	−170.7	−256.0	1850	2280	1679.3	2024.0
LC_6	844	882	920	676	−418.0	−232.3	2234.4	1323	1816.4	1090.7	−170.7	−256.0	1596	2646	1425.3	2390.0
LC_7	367	542	717	717	−487.5	−271.0	2007.6	813	1520.1	542.0	−199.1	−298.6	1434	1626	1234.9	1327.4
LC_7	657	684	711	427	−487.5	−271.0	1593.2	1026	1105.7	755.0	−199.1	−298.6	1138	2052	938.9	1753.4

Q_1, Q_2, Q_3 and Q_4 are pile reactions
$M''_{11} = 1.4\ (Q_3 + Q_4)$ $\quad M''_{22} = 0.5\ (Q_1 + Q_2 + Q_3)$ $\quad M_{11} = M'_{11} + M''_{11}$
$V''_{33} = Q_3 + Q_4$ $\quad V''_{44} = Q_1 + Q_2 + Q_3$ $\quad V_{33} = V'_{33} + V''_{33}$
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
$\phi = 600\,\text{mm}$ $\quad \phi/5 = 120\,\text{mm}$ $\quad \phi$ = diameter of pile



SK 7/41 Moments in pile and pile cap due to pile fixity.

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.

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

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

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

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

$$= 0.96d \leq 0.95d = 760\,\text{mm}$$

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

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

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

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

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

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

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

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

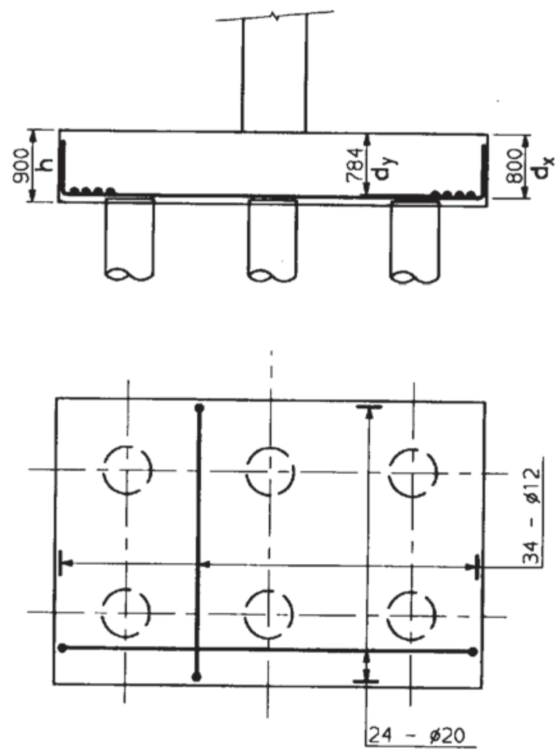
$z = 0.95d$ by inspection
$= 0.95 \times 784 = 745\,\text{mm}$

$$A_{st} = \frac{M_{22}}{0.87 f_y z} = \frac{1134 \times 10^6}{0.87 \times 460 \times 745} = 3803\,\text{mm}^2$$

Area of 12 mm dia. bar = $113\,\text{mm}^2$ $34 \times 113 = 3842\,\text{mm}^2$

Use 34 no. 12 mm diameter bars equally spaced (approximate spacing 155 mm) in the y–y direction.
(See also Step 22 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.



SK 7/42 Pile cap reinforcement.

Step 11 Check shear stress in pile cap

V_{33} = shear on critical section 3–3
= 1766.9 kN   (see table in Step 8)

$a_v = 2700 - 400 \text{ (half column)} - 1080 = 1220\,\text{mm}$

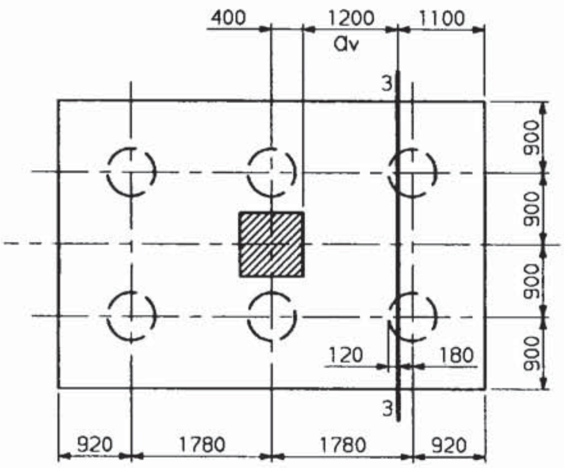
$1.5d_x = 1.5 \times 800 = 1200\,\text{mm}$

$a_v > 1.5d_x$ hence no enhancement of shear stress is allowed

$$v = \frac{V}{bd} = \frac{1766.9 \times 10^3}{3600 \times 800} = 0.61\,\text{N/mm}^2$$

$$p = \frac{100A_s}{bd} = \frac{100 \times 7536}{3600 \times 800} = 0.26\%$$

$v_c = 0.425\,\text{N/mm}^2 < 0.61\,\text{N/mm}^2$ from Fig. 11.3



SK 7/43 Critical shear plane in pile cap.

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.

$a_v = 1200\,\text{mm}$ $1.5d_x = 1200\,\text{mm}$

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

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

$v_{c1} = 0.47\,\text{N/mm}^2$ from Figs 11.2 to 11.5

$$v_{c2} = v_{c1}\left(\frac{2d}{a_v}\right) = 0.47 \times 1.333$$

$$= 0.63\,\text{N/mm}^2 > 0.61\,\text{N/mm}^2 \quad \text{OK}$$

V_{44} = shear on critical section 4–4
= 2511.4 kN (see table in Step 8).

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

$1.5d_y = 1.5 \times 784 = 1176\,\text{mm} > a_v$

$$\frac{2d_y}{a_v} = \frac{2 \times 784}{320} = 4.9$$

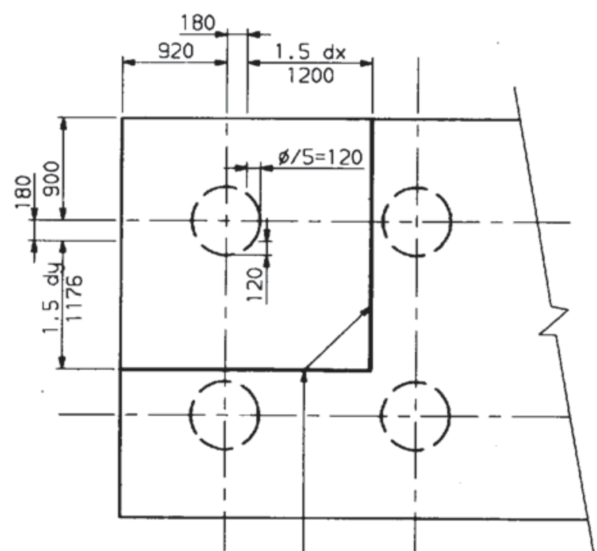
$$p = \frac{100A_{sc}}{bd} = \frac{100 \times 3482}{5400 \times 784} = 0.08\%$$

(See Step 22 for minimum percentage of reinforcement.)

$v_{c1} = 0.40\,\text{N/mm}^2$ for $f_{cu} = 40\,\text{N/mm}^2$
$v_{c2} = 0.40 \times 4.9 = 1.96\,\text{N/mm}^2$

$$v_c = \frac{V}{bd} = \frac{2511 \times 10^3}{5400 \times 784}$$
$$= 0.59\,\text{N/mm}^2 < 1.96\,\text{N/mm}^2 \quad \text{OK}$$

Step 12 Check punching shear stress in pile cap



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

U_1 = perimeter of column = 2 (800 + 800) = 3200 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 = perimeter on punching shear critical plane for pile load
= 2300 + 2256 = 4556 mm

Ultimate maximum column load, N = 4622 kN   from table in Step 8.

Ultimate maximum pile load, Q = 983 kN

$$\text{Column punching shear stress} = \frac{N}{U_1 d} = \frac{4622 \times 10^3}{3200 \times 0.5 \times (800 + 784)}$$

$$= 1.82\,\text{N/mm}^2 < 0.8\sqrt{f_{cu}} \text{ or } 5\,\text{N/mm}^2 \quad \text{OK}$$

$$\text{Punching shear stress at perimeter of pile} = \frac{983 \times 10^3}{\pi \times 600 \times 800}$$

$$= 0.65\,\text{N/mm}^2 < 0.8\sqrt{f_{cu}} \quad \text{OK}$$

$$\text{Pile punching shear stress} = \frac{Q}{U_2 d} = \frac{983 \times 10^3}{4556 \times 0.5\,(800 + 784)}$$

$$= 0.27\,\text{N/mm}^2$$

Minimum v_c for Grade $40\,\text{N/mm}^2$ concrete = $0.40\,\text{N/mm}^2$ OK

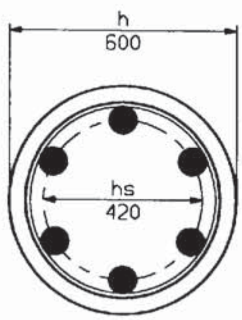
Step 13 Check area of reinforcement in pile

Unsupported length of pile, l_o, is assumed negligible.
Assume $l_c/h < 10$.
The pile is treated as a short column. From tables in Step 8,

$Q_{max} = 983\,\text{kN}$ with $M = 28.1\,\text{kNm}$
$Q_{min} = 367\,\text{kN}$ with $M = 106.5\,\text{kNm}$

Max. shear, $V_{max} = 42.93\,\text{kN}$

Assume minimum cover is 75 mm.



SK 7/45 Pile reinforcement.

Allowing for links and bar diameter, assume $h_s = 420\,\text{mm}$.

$$\frac{h_s}{h} = \frac{420}{600} = 0.70 = k$$

$$f_{cu} = 30\,\text{N/mm}^2 \qquad e = \frac{M}{N} = \frac{28.1}{983} = 0.029\,\text{m}$$

$$\frac{e}{R} = \frac{0.029}{0.3} = 0.095$$

$$\frac{Q_{max}}{h^2} = \frac{983 \times 10^3}{600 \times 600} = 2.73\,\text{N/mm}^2$$

From Table 11.19, it is observed that minimum reinforcement may be used.

Use minimum reinforcement.

For the second load case,

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

$$\frac{e}{R} = 1$$

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 mm^2).

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 × bar dia. = 4 mm ≥ 6 mm

Maximum spacing of links = 12 × smallest dia. of bar = 12 × 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

$A_s \geq 0.0013bh$ in both directions

Minimum reinforcement in the $x-x$ direction = 0.0013 × 3600 × 900 = 4212 mm^2

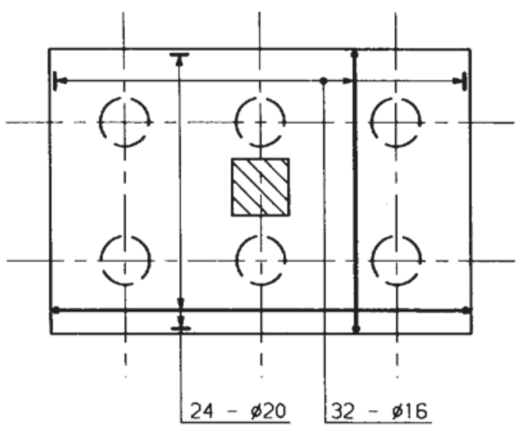
Provided 7536 mm^2 (see Step 10).

Minimum reinforcement in the $y-y$ direction = 0.0013 × 5400 × 900 = 6318 mm^2

Area of 16 mm dia. bar = 201 mm^2 32 × 201 = 6432 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 × dia. of bar.

12 × 20 = 240 mm
12 × 16 = 192 mm

Provide a minimum 250 mm bent up length of pile bottom reinforcement. *Check full anchorage bond length of the main tension bars.*

$f_{cu} = 40\,N/mm^2$

Reinforcement used is Type 2 deformed bars.
From Table 3.29 of BS 8110: Part 1: 1985,[1]

tension anchorage length = $32\phi = 32 \times 20 = 640\,mm$

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

Step 24 Spacing of bars in pile cap

$$\text{Maximum percentage of reinforcement} = p = \frac{100A_s}{bd}$$

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

Maximum allowed clear spacing for p less 0.3% is $3d$ 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 BS 8110: 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 BS 8110,

full anchorage bond length = 32ϕ; $32 \times 16 = 512\,mm$

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

7.8 FIGURES FOR CHAPTER 7

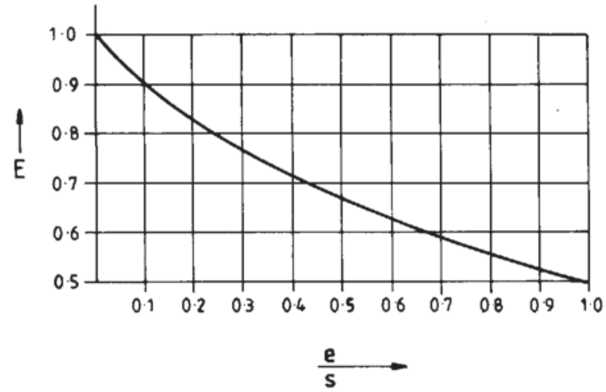


Fig. 7.1 Determination of pile efficiency.

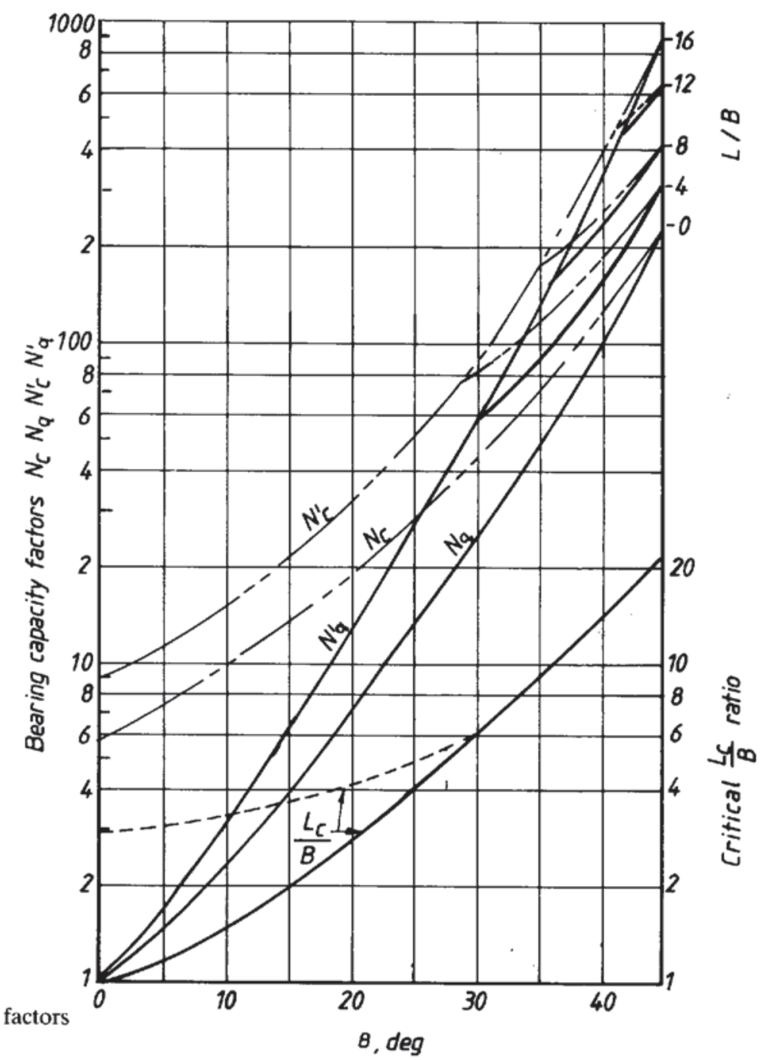


Fig. 7.2 Bearing capacity factors for deep foundations.