Chapter 6

Design of Pad Foundations

6.0 NOTATION

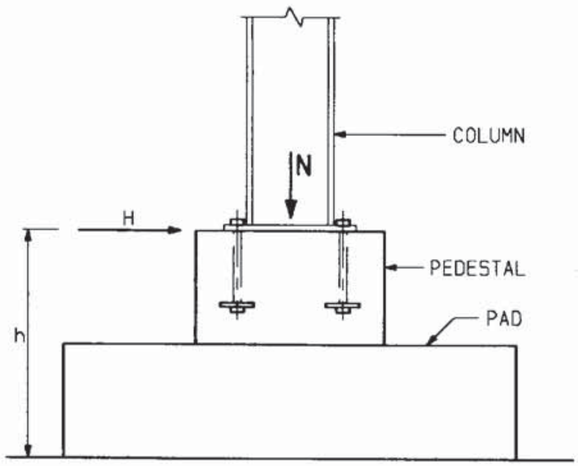
a_{cr}	Point on surface of concrete to nearest face of a bar
A	Length of a side of a rectangular pad foundation
A_s	Area of tensile reinforcement
A_{sx}	Area of tensile reinforcement to resist bending about x-axis
A_{sy}	Area of tensile reinforcement to resist bending about y-axis
b	Width of reinforced concrete section
B	Width of a side of a rectangular pad foundation (least dimension)
c	Soil cohesion (kN/m^2)
c_{min}	Minimum cover to tensile reinforcement
C_x	Size of column in x-direction
C_y	Size of column in y-direction
C_{kd}	Cone resistance by static cone penetration tests (kg/cm^2)
d	Effective depth to tensile reinforcement of a concrete section
d_x	Effective depth to tensile reinforcement resisting moment about x-axis
d_y	Effective depth to tensile reinforcement resisting moment about y-axis
D	Depth of foundation below ground level
e_x	Resultant eccentricity of all column vertical loads in x-direction
e_y	Resultant eccentricity of all column vertical loads in y-direction
e_{xg}	Eccentricity of vertical end reaction from ground beams in the x-direction
e_{yg}	Eccentricity of vertical end reaction from ground beams in the y-direction
E_c	Modulus of elasticity of concrete
E_s	Modulus of elasticity of steel
f_s	Tensile stress in steel reinforcement
f_y	Characteristic yield strength of steel
f_{cu}	Characteristic cube strength of concrete at 28 days
F	Frictional resistance to horizontal movement under pad foundation
h	Overall depth of concrete section/thickness of pad
H	Effective depth of soil under foundation for computation of settlement
H_a	Active pressure on side of a foundation (kN)
H_p	Passive resistance on side of a foundation (kN)
H_u	Ultimate factored horizontal load at underside of a foundation
H_x	Unfactored horizontal shear from column on foundation in the x-direction

H_y	Unfactored horizontal shear from column on foundation in the y-direction
H_{xu}	Factored horizontal shear from column on foundation in x-direction
H_{yu}	Factored horizontal shear from column on foundation in y-direction
K_a	Active pressure coefficient of soil
K_h	Modulus of subgrade reaction for horizontal movement in soil
K_p	Rankine passive pressure coefficient of soil
l_x	Dimension of a rectangular pad footing in x-direction
l_y	Dimension of a rectangular pad footing in y-direction
m	Modular ratio E_s/E_c
m_v	Coefficient of volume compressibility of soil (m^2/MN)
M_x	Unfactored moment from column on foundation about x-axis
M_y	Unfactored moment from column on foundation about y-axis
M_x^*	Unfactored moment on foundation about x-axis due to eccentric surcharge
M_y^*	Unfactored moment on foundation about y-axis due to eccentric surcharge
M_{xg}	Unfactored fixed end moment from ground beams on foundation about x-axis
M_{xu}	Factored moment from column on foundation about x-axis
M_{xx}	Combined unfactored total moment on foundation about x-axis
M_{yg}	Unfactored fixed end moment from ground beams on foundation about y-axis
M_{yu}	Factored moment from column on foundation about y-axis
M_{yy}	Combined unfactored total moment on foundation about y-axis
$M_{x\,u}^*$	Factored moment on foundation about x-axis due to eccentric surcharge
$M_{y\,u}^*$	Factored moment on foundation about y-axis due to eccentric surcharge
M_{xgu}	Factored fixed-end moment from ground beams on foundation about x-axis
M_{xxu}	Combined factored total moment on foundation about x-axis
M_{ygu}	Factored fixed-end moment from ground beams on foundation about y-axis
M_{yyu}	Combined factored total moment on foundation about y-axis
n_h	Coefficient to determine horizontal modulus of subgrade reaction
N	Unfactored vertical load from column on foundation
N_c	Soil bearing capacity coefficient as per Terzaghi
N_q	Soil bearing capacity coefficient as per Terzaghi
N_u	Factored vertical load from column on foundation
N_γ	Soil bearing capacity coefficient as per Terzaghi
p	Total overburden pressure at foundation level
p_o	Effective overburden pressure at foundation level/centre of layer
p_x	Percentage of tensile reinforcement to resist moment about x-axis
p_y	Percentage of tensile reinforcement to resist moment about y-axis
P	Unfactored combined total vertical load on soil under a pad foundation
P_s	Sliding resistance of concrete pad foundation on soil

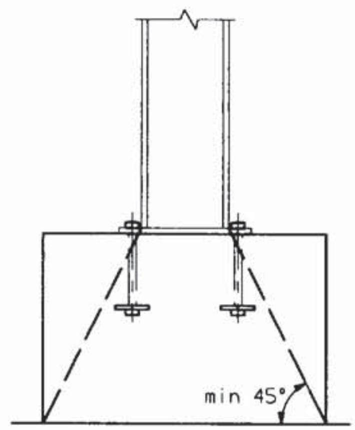
P_u	Factored combined total vertical load on soil under a pad foundation
P_v	Allowable vertical load on soil under a pad foundation
P_{Hx}	Sliding resistance of base in x-direction
P_{Hy}	Sliding resistance of base in y-direction
q_n	Net pressure on soil for settlement computation (MN/m^2)
q_u	Unconfined compressive strength (kN/m^2)
q_{ult}	Ultimate bearing capacity of soil under a pad foundation
r or R	Radius of circular footing
R	Restraint factor for computation of early thermal cracking
s	Shear strength of soil
s_u	Shear strength from unconfined tests ($= q_u/2$)
T_1	Differential temperature in a concrete pour for calculation of early thermal cracking
U_n	Perimeter of column at prescribed multiples of effective depth of pad
U_o	Perimeter of column footprint on pad foundation
v_c	Design concrete shear stress
v_n	Shear stress in concrete at perimeter defined by U_n
V	Shear force across critical section in a pad foundation
V_n	Shear force in a critical perimeter defined by U_n
V_u	Factored end shear of ground beam
w_{max}	Maximum crack width (mm)
x	Depth of neutral axis in a concrete section from compression face
z	Depth of lever arm
Z	Depth of top of pad foundation below ground level
α	Coefficient of thermal expansion of concrete/°C
γ	Unit weight of soil (kN/m^3)
γ_w	Unit weight of water (kN/m^3)
δ	Angle of friction between soil and concrete
Δ	Horizontal movement of foundation
Δ_{max}	Maximum allowable horizontal movement of foundation
ε_h	Calculated strain in concrete at a depth h from compression face
ε_m	Strain corrected for stiffening effect
ε_r	Tensile strain in concrete due to temperature gradient causing early thermal cracking
ε_s	Strain at centre of steel reinforcement
ε_{mh}	Strain at depth h corrected for stiffening effect
ρ_{crit}	Critical percentage of steel required to distribute early thermal cracking
σ_z	Vertical stress at centre of a layer of soil due to net foundation pressure
ϕ	Angle of internal friction

6.1 ANALYSIS FOR BEARING PRESSURE ON SOIL

6.1.1 Isolated single column pad (bearing pressure calculations)



SK 6/1 Typical column foundation in reinforced concrete.



SK 6/2 Typical mass concrete foundation.

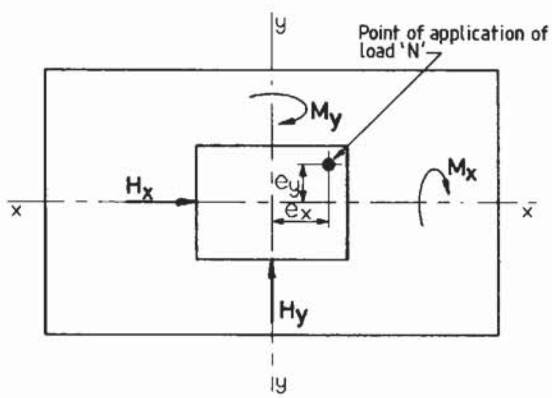
Loads from column

N = combined vertical load unfactored

M_x = combined moment about $x-x$ unfactored
M_y = combined moment about $y-y$ unfactored

H_x = combined horizontal shear in x direction unfactored
H_y = combined horizontal shear in y direction unfactored

e_x = eccentricity in x direction of vertical load N from CG of base
e_y = eccentricity in y direction of vertical load N from CG of base



SK 6/3 Typical loads from column on foundation shown on plan.

Loads at underside of pad on soil

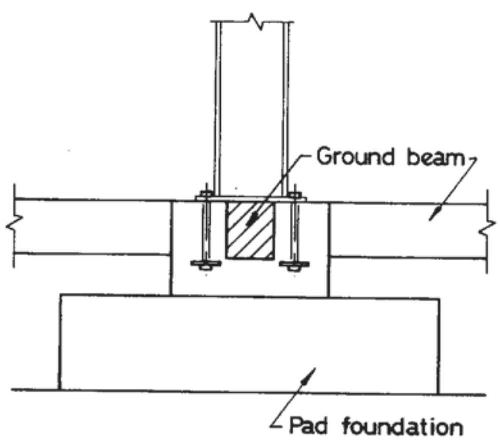
P = vertical load = N + weight of foundation + weight of backfill + surcharge on backfill

M_{xx} = moment about $x-x$ = $M_x + Ne_y + H_y h + M_x^*$
M_{yy} = moment about $y-y$ = $M_y + Ne_x + H_x h + M_y^*$

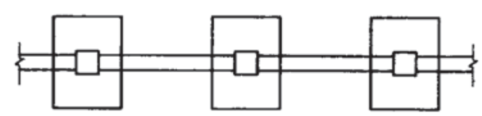
where M_x^* and M_y^* are moments with respect to CG of base due to eccentric surcharge on backfill.

Note: In finding the load on the soil at the underside of the pad footing the directions of the loads, eccentricities and moments must be taken into account. With reversible horizontal loads and moments, all possible combinations should be examined. Eccentric heavy surcharge on part of the backfill on foundation may in certain cases produce higher bearing pressure and should be investigated.

6.1.2 Single column pads connected by ground beams (bearing pressure calculations)



SK 6/4 Typical arrangement of ground beams to column foundation.



SK 6/5 Plan of foundations connected by ground beams.

Assumptions

(1) The pad foundation is assumed rigid and its rotation is very small.
(2) The ground beam may be designed as fixed to the foundation with zero rotation at the ends.
(3) The horizontal loads in any orthogonal direction from all columns with connected foundations will be algebraically added and then divided by the number of columns. The total horizontal load in any direction will be shared equally between connected foundations.
(4) Because of the very high rotational stiffness of the pad foundations relative to the ground beam, it is assumed that the horizontal loads, moments and load eccentricities at the top of the foundation will cause

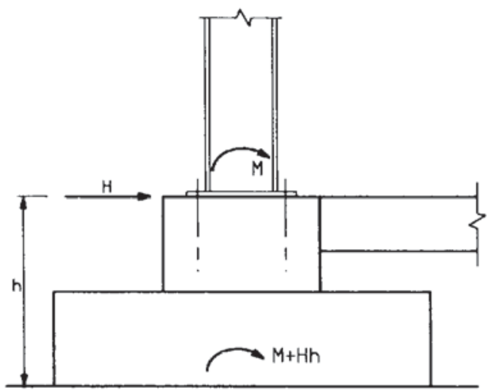
cantilever moment on the soil–pad foundation interface and the ground beam will be unaffected.

(5) The pad foundation will be designed to resist the fixed-end moments from the connected ground beams. The ground beams may also be designed and detailed as pin-jointed to the foundation when there will be no fixed-end moments on the foundation.

(6) The pad foundation should be designed to resist the fixed-end moments from ground beams due to differential settlements, if any, of connected foundations. The ground beams may also be designed and detailed as pin-jointed to the foundation when the fixed-end moments due to differential settlements will be negligible.

Note: To avoid excessive stresses and serious damage, ground beams should preferably be cast on a compressible or rapidly degradable layer of material such that some free vertical movement is allowed to cater for vertical ground movements and differential settlements.

Loads from columns



SK 6/6 Horizontal shear causing additional moment.

N = combined vertical load – unfactored

M_x = combined moment about x–x – unfactored

M_y = combined moment about y–y – unfactored

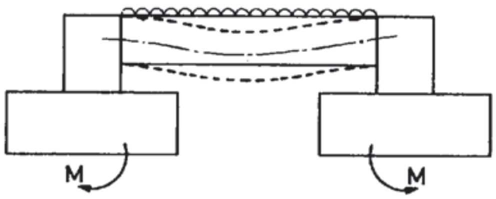
H_x = combined horizontal shear in x direction – unfactored

e_x = eccentricity in x direction of vertical load N from CG of base
e_y = eccentricity in y direction of vertical load N from CG of base

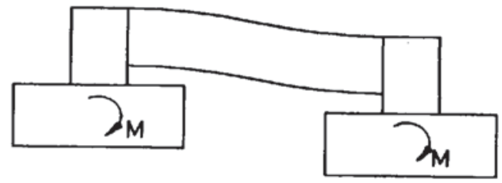
Loads from ground beams

ΣV = combined end shear (vertical) unfactored of all beams

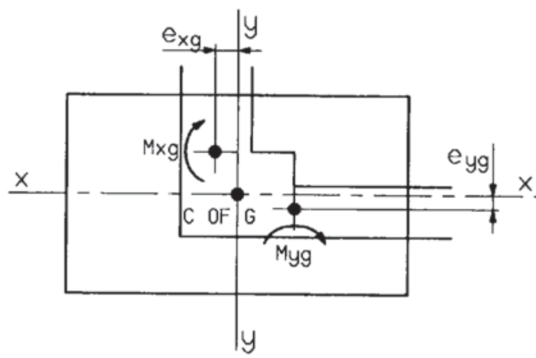
ΣM_{xg} = combined fixed-end moment about x–x unfactored (beams running y–y direction)



SK 6/7 Fixed-end moments from ground beams.



SK 6/8 Fixed-end moments due to differential settlement.



SK 6/9 Bending moment and eccentricity of load from ground beams.

ΣM_{yg} = combined fixed-end moment about $y-y$ unfactored (beams running $x-x$ direction)

e_{xg} = eccentricity of vertical shear V from CG of foundation

e_{yg} = eccentricity of vertical shear V from CG of foundation

Note: M_{xg} and M_{yg} should include the effects of dead load, live load and differential settlements on the ground beam – unfactored.

Loads at underside of pad on soil

P = vertical load $= N + \Sigma V +$ weight of foundation + weight of backfill + surcharge on backfill

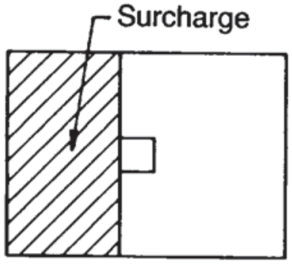
M_{xx} = moment on $x-x = M_x + \Sigma M_{xg} + Ne_y + \Sigma(Ve_{yg}) + H_y h + M_x^*$

M_{yy} = moment on $y-y = M_y + \Sigma M_{yg} + Ne_x + \Sigma(Ve_{xg}) + H_x h + M_y^*$

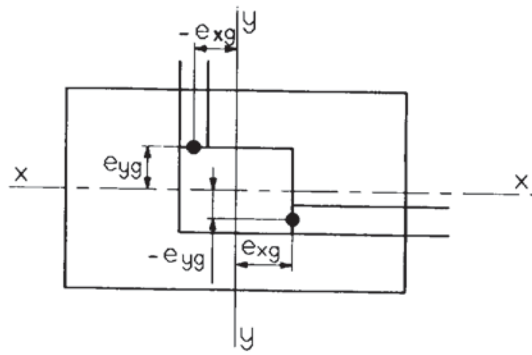
H = horizontal shears = H_x and H_y

where M_x^* and M_y^* are moments with respect to CG of base due to eccentric surcharge on backfill.

Note: In finding the load on the soil at the underside of the pad footing the directions of the loads, eccentricities and moments must be taken into account. With reversible horizontal loads and moments, all possible combinations should be examined. Eccentric heavy surcharge on part of the



SK 6/10 Eccentricity of surcharge on plan of pad foundation.



SK 6/11 Eccentricity of load from simply supported end of ground beam.

backfill on the foundation may in certain cases produce higher bearing pressure on the soil and should be investigated.

6.1.3 Isolated multiple column pad (bearing pressure calculation)

Loads from columns

ΣN = summation of all column vertical loads – unfactored

ΣM_x = algebraic summation of all column moments about x–x – unfactored

ΣM_y = algebraic summation of all column moments about y–y – unfactored

ΣH_x = algebraic summation of all column horizontal shears in x-direction – unfactored

ΣH_y = algebraic summation of all column horizontal shears in y-direction – unfactored

e_x = distance in the x-direction of CG of all column vertical loads from CG of base

e_y = distance in the y-direction of CG of all column vertical loads from CG of base

Loads at underside of pad on soil

P = vertical load = ΣN + weight of foundation + weight of backfill + surcharge on backfill

M_{xx} = moment about x–x = $\Sigma M_x + \Sigma N e_y + \Sigma H_y h + M_x^*$

M_{yy} = moment about y–y = $\Sigma M_y + \Sigma N e_x + \Sigma H_x h + M_y^*$

H = horizontal shears = ΣH_x and ΣH_y

where M_x^* and M_y^* are due to eccentric surcharge.

6.1.4 Multiple column pads connected by ground beams (bearing pressure calculations)

Assumptions See Section 6.1.2.

Loads from columns See Section 6.1.3.

Loads from ground beams See Section 6.1.2.

Loads at underside of pad foundation

P = vertical load = $\Sigma N + \Sigma V$ + weight of foundation + weight of backfill + surcharge on backfill

M_{xx} = moment on $x-x$ = $\Sigma \mathrm{M_x} + \Sigma N e_y + \Sigma M_{xg} + \Sigma(V e_{yg}) + \Sigma H_y h + M_x^*$

M_{yy} = moment on $y-y$ = $\Sigma \mathrm{M_y} + \Sigma N e_x + \Sigma M_{yg} + \Sigma(V e_{xg}) + \Sigma H_x h + M_y^*$

H = horizontal shear = ΣH_x and ΣH_y

where M_x^* and M_y^* are due to eccentric surcharge on backfill.

6.2 ANALYSIS FOR ULTIMATE LOAD

6.2.1 Isolated single column pad

Note: Use load factors and combinations as stated in Section 6.3.

Load from column

N_u = combined vertical load – factored

M_{xu} = combined moment about $x-x$ – factored

M_{yu} = combined moment about $x-x$ – factored

H_{xu} = combined horizontal shear x-direction – factored

H_{yu} = combined horizontal shear y-direction – factored

e_x = eccentricity of N_u in x-direction

e_y = eccentricity of N_u in y-direction

Loads at underside of pad foundation

$P_u = N_u + 1.4$ (weight of foundation + weight of backfill) + 1.6 (surcharge on backfill)

$M_{xxu} = M_{xu} + N_u e_y + H_{yu} h + M_{xu}^*$

$$M_{yyu} = M_{yu} + N_u e_x + H_{xu} h + M^*_{yu}$$

$$H_u = H_{xu} \quad \text{and} \quad H_{yu}$$

where M^*_{xu} and M^*_{yu} are ultimate moments on base due to eccentric surcharge on backfill.

6.2.2 Single column pads connected by ground beams

Note: Use load factors and combinations as stated in Section 6.3. For assumptions, see Section 6.1.2.

Load from columns See Section 6.2.1.

Load from ground beams

ΣV_u = combined factored end shear of all beams

ΣM_{xgu} = combined factored end moment about $x-x$ (beams running $y-y$)

ΣM_{ygu} = combined factored end moment about $y-y$ (beams running $x-x$)

e_{xg} = eccentricity of V_u from CG of base in x-direction (beams $x-x$)

e_{yg} = eccentricity of V_u from CG of base in y-direction (beams $y-y$)

Note: M_{xgu} and M_{ygu} should include the effects of dead load, live load and differential settlements on the ground beam.

Loads at underside of pads foundation on soil

$P_u = N_u + \Sigma V_u + 1.4$ (weight of foundation + weight of backfill) + 1.6 (surcharge on backfill)

$$M_{xxu} = M_{xu} + \Sigma M_{xgu} + N_u e_y + \Sigma(V_u e_{yg}) + H_{yu} h + M^*_{xu}$$

$$M_{yyu} = M_{yu} + \Sigma M_{xgu} + N_u e_x + \Sigma(V_u e_{xg}) + H_{xu} h + M^*_{yu}$$

$$H_u = H_{xu} \quad \text{and} \quad H_{yu}$$

where M^*_{xu} and M^*_{yu} are ultimate moments on base due to eccentric surcharge on backfill.

6.2.3 Multiple column pads

Note: For multiple column pad foundations with or without ground beams, use the same philosophy as in Sections 6.2.1 and 6.2.2 but with the following loads from all columns on the base summed up, algebraically.

ΣN_u = summation of all ultimate vertical loads from columns

ΣM_{xu} = summation of all ultimate moments about $x-x$ axis

ΣM_{yu} = summation of all ultimate moments about $y-y$ axis

ΣH_{xu} = summation of all horizontal shears in x-direction

ΣH_{yu} = summation of all horizontal shears in y-direction

e_x = resultant eccentricity of all column vertical loads in x-direction

e_y = resultant eccentricity of all column vertical loads in y-direction

6.3 LOAD COMBINATIONS

Loads from the columns will be combined using the following principles.

6.3.1 Bearing pressure calculations

LC_1: $1.0DL + 1.0IL + 1.0EP + 1.0CLV + 1.0CLH$
No increase in allowable bearing capacity.
LC_2: $1.0DL + 1.0EP + 1.0CLV + 1.0CLH + 1.0WL$ (or $1.0EL$)
25% increase in allowable bearing capacity.
LC_3: $1.0DL + 1.0IL + 1.0EP + 1.0WL$ (or $1.0EL$)
25% increase in allowable bearing capacity.
LC_4: $1.0DL + 1.0WL$ (or $1.0EL$)
25% increase in allowable bearing capacity.

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

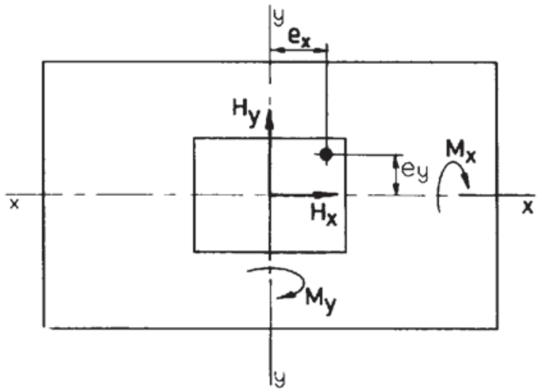
6.3.2 Bending moment and shear calculations

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}: $1.2DL + 1.2CLV + 1.2CLH + 1.2EP + 1.2\,WL$ (or $1.2EL$)

6.3.3 Settlement computation

LC_{13}: $1.0DL + 0.5IL$
(vertical direct loads only)

6.4 SIGN CONVENTION



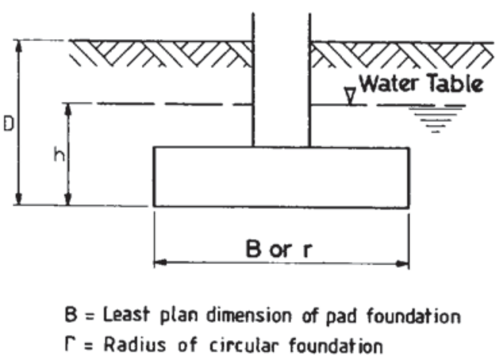
SK 6/12 Sign convention showing positive loads and eccentricity.

To avoid mistakes the following sign convention should be followed:

Vertical loads: downwards positive
Moments on base: clockwise positive
Horizontal shears: left to right positive: +ve x-direction
bottom to top positive: +ve y-direction
Eccentricities: +ve for +ve x and +ve for +ve y

6.5 ESSENTIALS OF SOIL MECHANICS

6.5.1 Ultimate bearing capacity



SK 6/13 Typical parameters for the calculation of bearing capacity.

From soil investigation and laboratory tests the following parameters should be available:

c = soil cohesion (kPa)

ϕ = angle of internal friction (degrees or radians)

γ = unit weight of soil (kN/m^3)

p = total overburden pressure at foundation level

$p_o = p - \gamma_w h$

h = height of water above foundation level

γ_w = unit weight of water (kN/m^3)

q_{ult} = ultimate bearing capacity as per Terzaghi (conservative approach)

Bearing capacity calculations for cohesionless and (c-ϕ) soils.

For continuous foundation

$$q_{ult} = cN_c + p_o(N_q - 1) + 0.5\gamma BN_\gamma + p$$

For square foundation

$$q_{ult} = 1.3cN_c + p_o(N_q - 1) + 0.4\gamma BN_\gamma + p$$

For circular foundation

$$q_{ult} = 1.3cN_c + p_o(N_q - 1) + 0.3\gamma BN_\gamma + p$$

$$a = e^{(0.75\pi - \phi/2)\tan\phi}$$

$$N_q = \frac{a^2}{2\cos^2(45^\circ + \phi/2)}$$

$$N_c = (N_q - 1)\cot\phi$$

$$N_\gamma = 0.5\tan\phi\left(\frac{K_{py}}{\cos^2\phi} - 1\right)$$

Table 6.1 Values of K_{py} (as per Reference 6).

ϕ	0	5	10	15	20	25	30	35	40	45	50
K_{py}	10.8	12.2	14.7	18.6	25.0	35.0	52.0	82.0	141.0	298.0	800.0

N_q, N_c and N_γ may be obtained from Fig. 6.1.
Allowable bearing capacity = $q_{ult}/3$

Bearing capacity calculations for cohesive soils

$$q_{ult} = cN_c + p$$

N_c may be obtained from Table 6.2.
Allowable bearing capacity = $q_{ult}/3$

Table 6.2 Values of N_c for cohesive soils (as per Reference 6).

Types of footing	D/B or $D/2r$	Values of N_c
Circular or square footing	0	6.2
	0.5	7.3
	1.0	8.2
	1.5	9.1
	2.0	9.3
	2.5	9.3
	3.0	9.3
	3.5	9.3
	4.0	9.3
Strip footing	0	5.2
	0.5	6.2
	1.0	7.1
	1.5	7.7
	2.0	8.1
	2.5	8.2
	3.0	8.2
	3.5	8.2
	4.0	8.2

D = depth below ground to underside of pad foundation
B = least plan dimension of pad foundation
r = radius of circular pad foundation

Note: There are many different ways of calculating ultimate bearing capacity which take into account depth, water tables, load inclinations, various shapes of foundations, soil layers, etc. It is normal practice to have the allowable bearing capacities for various sizes of foundations determined by specialists carrying out the site investigation.
In the absence of laboratory tests for finding c, γ and ϕ, values from Tables 6.3 and 6.4 may be used to determine allowable bearing capacity if the description of the soil is known.

Table 6.3 Typical values of angle of internal friction, ϕ.

Soil type	Angle of internal friction, ϕ (degrees)
Medium gravel	40–55
Sandy gravel	45–50
Loose dry sand	28–34
Loose saturated sand	28–34
Dense dry sand	35–46
Dense saturated sand	33–44
Loose silty sand	20–22
Dense silty sand	25–30
Saturated clay	0

Table 6.4 Typical values of cohesive strength, c.

Soil type	Cohesive strength, c (kN/m^2)
Hard boulder clays	>300
Hard fissured clays	>300
Deep London and gault clays	>300
Hard weathered shales	>300
Hard weathered mudstones	>300
Very stiff boulder clay	150–300
Very stiff blue London clay	150–300
Very stiff weathered Keuper Marl	150–300
Stiff boulder clay	75–150
Stiff blue London clay	75–150
Stiff weathered Keuper Marl	75–150
Firm normally consolidated clay	40–75
Upper weathered 'brown' London clay	40–75
Soft normally consolidated clay	20–40

Note: Presumed allowable bearing capacities for various types of soil and grades of chalk and Keuper Marl may be obtained from Tables 2 and 3 of BS 8004: 1986.[2]

6.5.2 Settlement of foundation

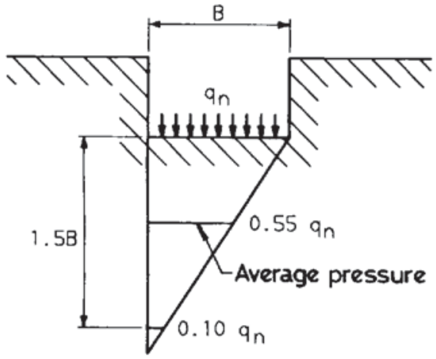
Method 1 Quick approximate method

Soil parameter: Coefficient of volume compressibility (in m^2/MN) = m_v

Values of m_v should be available after soil investigation tests. Approximate values of m_v for clays may be obtained from Table 6.5.

Table 6.5 Typical values of coefficient of volume compressibility, m_v.

Soil type	Coefficient of volume compressibility, m_v (m^2/MN)
Heavily overconsolidated boulder clays	<0.05
Stiff weathered rocks	<0.05
Hard London clays	<0.05
Boulder clays	0.05–0.1
Very stiff London clays	0.05–0.1
Upper blue London clays	0.10–0.3
Weathered boulder clay	0.10–0.3
Weathered Keuper Marl	0.10–0.3
Normally consolidated alluvial clays	0.30–1.5
Estuarine clays	0.30–1.5
Organic alluvial clays and peats	>1.5



SK 6/14 Pressure distribution for settlement computation.

Consolidation settlement = $m_v\ \sigma_z\ H$

where m_v = average m_v of all layers up to a depth of $1.5B$
B = width of foundation (least dimension)
σ_z = $0.55q_n$ (average pressure in centre of layers)
q_n = net pressure on the soil (MN/m^2)
H = $1.5B$ (metres)

Note: Immediate settlement is ignored in these calculations.

Method 2 Settlement from static cone penetration tests

Soil parameter: Cone resistance (in kg/cm^2) = C_{kd}

Constant of compressibility = $C = \dfrac{147C_{kd}}{p_o}$

where p_o = effective pressure at the centre of layer = $p - \gamma_w h$ (in kN/m^2)
p = total overburden pressure at the centre of layer (kN/m^2)
h = height of water to the centre of layer (metres)
γ_w = unit weight of water (kN/m^3)

Settlement of layer = $S = \dfrac{H}{C}\log_e\left(\dfrac{p + \sigma_z}{p_o}\right)$ metres

where σ_z = vertical stress at centre of layer (kN/m^2) as a result of net foundation pressure (q_n)
H = thickness of the layer of soil (metres)

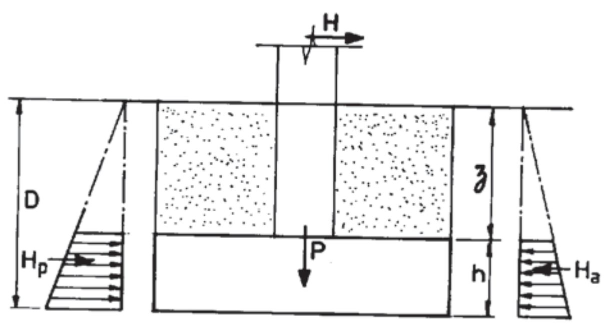
σ_z may be obtained from Fig. 6.2 (see Reference 5).

Total estimated settlement = summation of settlement of each layer

Note: The cone penetrometer curve should be broken down into separate layers, each having approximately the same value of cone resistance within the layer. Even if the cone penetrometer curve does not indicate any layering of soil, the settlement should be computed in layers because the value of σ_z falls off rapidly with depth.

6.5.3 Sliding resistance

6.5.3.1 Sliding of concrete on soil



SK 6/15 Sliding resistance of pad foundation.

Sliding resistance of concrete foundation on soil, $P_s = F + H_p - H_a$

where F = frictional resistance under base

H_p = passive resistance due to horizontal movement of foundation

H_a = active pressure due to horizontal movement of foundation

$F = P \tan \delta$

Δ = horizontal movement of foundation into soil (metres)

Δ_{max} = maximum allowable horizontal movement on the basis of soil shear strength (metres)

For cohesionless soil $\Delta_{max} = \left(\dfrac{K_p}{K_h}\right) \gamma$

For cohesive soil $\Delta_{max} = \dfrac{\gamma D + 2c}{K_h}$

$K_h = n_h / B$ for cohesionless soil

$= k_{si}/1.5B$ for cohesive soil

(See Table 6.7 for typical values of n_h and k_{si}.)

h = thickness of concrete pad foundation

δ = friction angle between concrete and soil (see Table 6.6).

ϕ = angle of internal friction of backfill material (see Table 6.3)

$K_p = \tan^2 (45° + \phi/2)$

γ = unit weight of backfill material (kN/m^3)
B = width of foundation over which horizontal soil pressure is active (metres)

$H_p = 0.5\Delta BhK_h\,(Z + D)$ for cohesionless soil
$= K_h\Delta Bh$ for cohesive soil

K_a = active pressure coefficient of the backfill material
$= \tan^2(45° - \phi/2)$ for cohesionless soil
= negligible for cohesive soils

$H_a = 0.5K_a\,(Z + D)\,Bh\gamma$ for cohesionless soil
P = total vertical load on the soil including the weight of foundation and backfill
D = depth of soil to underside of pad foundation
Z = depth of soil to top of pad foundation

Note: In practice it is very difficult to decide how much horizontal movement may be allowed without causing excessive stresses in other parts of the structure. It is good practice to provide total sliding resistance by frictional resistance only. The Rankine passive pressure coefficient, K_p, should not be used in these computations because a large movement is necessary to generate the full passive resistance. A factor of safety of 1.5 should be allowed against sliding.

Check: $P_s \geq 1.5H$

6.5.3.2 *Horizontal bearing capacity of soil*

Allowable horizontal bearing capacity of soil, $P_H = qA\tan\left(\frac{\phi}{1.5}\right) + \frac{2}{3}cA$

where c = cohesion of foundation soil (kN/m^2)
A = area of foundation
q = average unit pressure under foundation (kN/m^2)
ϕ = angle of internal friction of foundation soil

Check: $P_H \geq F$

6.6 BEARING PRESSURE CALCULATIONS

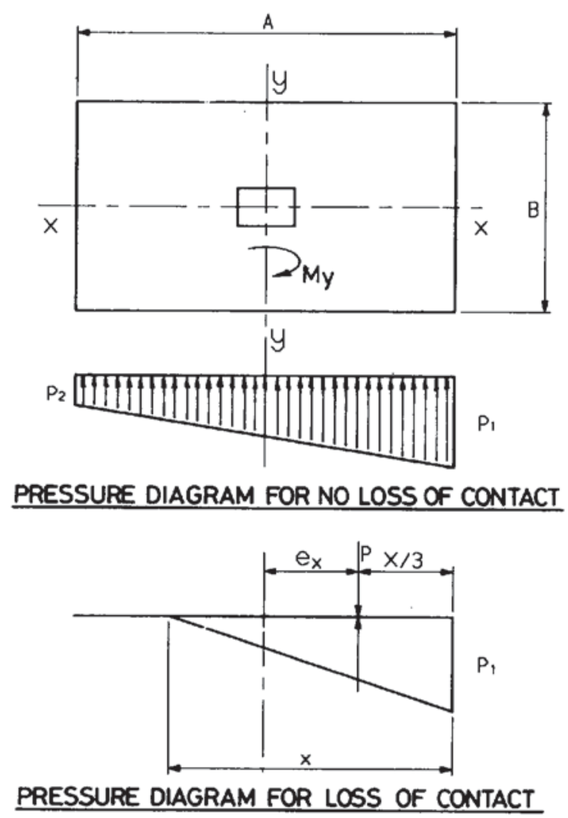
6.6.1 Rectangular pad – uniaxial bending – no loss of contact

$$e_x = \frac{M_{yy}}{P} \qquad e_y = 0$$

$$p_1 = \left(\frac{P}{AB}\right) + \left(\frac{6M_{yy}}{A^2B}\right) \qquad p_2 = \left(\frac{P}{AB}\right) - \left(\frac{6M_{yy}}{A^2B}\right)$$

Table 6.6 Typical values of friction angle between concrete and soil, δ.

Concrete on the following soil types	Friction angle, δ (degrees)
Clean sound rock	35
Clean gravel, gravel–sand mixtures	29–31
Coarse sand	29–31
Clean fine-to-medium sand	24–29
Silty medium-to-coarse sand	24–29
Clayey gravel	24–29
Clean fine sand	19–24
Silty-to-clayey fine-to-medium sand	19–24
Fine sandy silt	17–19
Very stiff and hard residual clay	22–26
Medium stiff and stiff clay	17–19
Bituminous or water-proofing membrane	0–5



SK 6/16 Pressure diagrams for uniaxial bending and direct load on base.

Table 6.7 Typical coefficients of horizontal modulus of subgrade reaction.

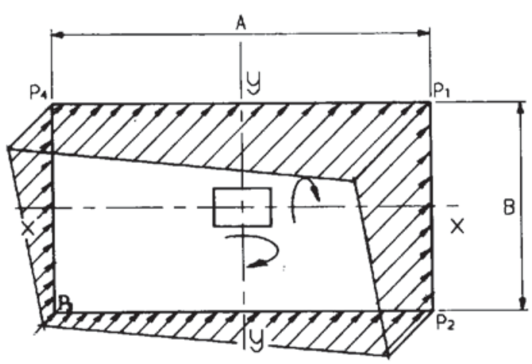
	Values of n_h (cohesionless) (MN/m^3)		
	Loose	Medium	Dense
Dry or moist sand	2.2	6.6	17.6
Submerged sand	1.26	4.4	10.7
	Values of k_{si} (cohesive) (MN/m^3)		
Types of clay	Stiff	Very stiff	Hard
	7.2	14.4	28.8

6.6.2 Rectangular pad – uniaxial bending – loss of contact

$$p_1 = \frac{2P}{(1.5A - 3e_x)B}$$

$$x = 1.5A - 3e_x$$

6.6.3 Rectangular pad – biaxial bending – no loss of contact



SK 6/17 Typical pressure diagram for biaxial bending and no loss of contact.

$$p_1 = \left(\frac{P}{AB}\right) + \left(\frac{6M_{yy}}{A^2B}\right) + \left(\frac{6M_{xx}}{AB^2}\right)$$

$$p_2 = \left(\frac{P}{AB}\right) + \left(\frac{6M_{yy}}{A^2B}\right) - \left(\frac{6M_{xx}}{AB^2}\right)$$

$$p_3 = \left(\frac{P}{AB}\right) - \left(\frac{6M_{yy}}{A^2B}\right) - \left(\frac{6M_{xx}}{AB^2}\right)$$

$$p_4 = \left(\frac{P}{AB}\right) - \left(\frac{6M_{yy}}{A^2B}\right) + \left(\frac{6M_{xx}}{AB^2}\right)$$

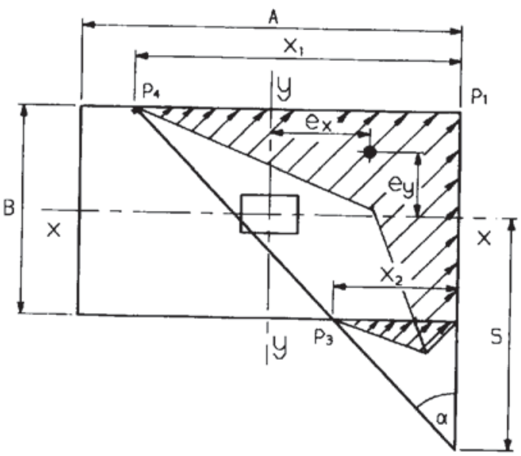
6.6.4 Rectangular pad – biaxial bending – loss of contact

Partial contact of soil/foundation (see Fig. 6.3).
The resultant of soil pressure diagram under base has co-ordinates e_x and e_y.

6.6.4.1 ***Resultant in Zone 1 of base*** (see Fig. 6.3)
Factor of safety for overturning is less than 1.5. Redesign size of base.

6.6.4.2 ***Resultant in Zone 2 of base*** (see Fig. 6.3)
No loss of contact of base. Calculate pressures as in Section 6.6.3.

6.6.4.3 ***Resultant in Zone 3 of base*** (see Fig. 6.3)



SK 6/18 Biaxial bending – loss of contact. Zero pressure on line p_3–p_4.

$$e_x = \frac{M_{yy}}{P} \qquad e_y = \frac{M_{xx}}{P}$$

$$S = \frac{B}{12}\left[\frac{B}{e_y} + \left(\frac{B^2}{e_y^2} - 12\right)^{\frac{1}{2}}\right]$$

$$\tan\alpha = \frac{3(A - 2e_x)}{2(S + e_y)}$$

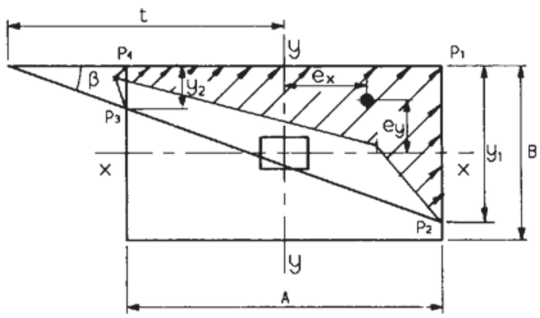
$$p_1 = \left(\frac{12P}{B\tan\alpha}\right)\left(\frac{B + 2S}{B^2 + 12S^2}\right)$$

$$p_2 = \left[\frac{S - \dfrac{B}{2}}{S + \dfrac{B}{2}}\right] p_1$$

$$p_3 = p_4 = 0$$

$$x_1 = \left(S + \frac{B}{2}\right) \tan\alpha \qquad x_2 = \left(S - \frac{B}{2}\right) \tan\alpha$$

6.6.4.4 Resultant in Zone 4 of base (see Fig. 6.3)



SK 6/19 Biaxial bending – loss of contact. Zero pressure on line p_2–p_3.

$$t = \frac{A}{12}\left[\frac{A}{e_x} + \left(\frac{A^2}{e_x^2} - 12\right)^{\frac{1}{2}}\right]$$

$$\tan\beta = \frac{3(B - 2e_y)}{2(t + e_x)}$$

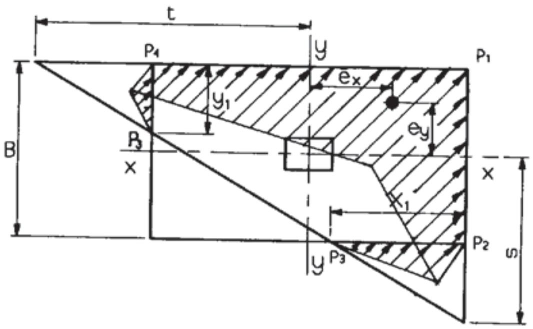
$$p_1 = \left(\frac{12P}{A\tan\beta}\right)\left(\frac{A + 2t}{A^2 + 12t^2}\right)$$

$$p_4 = \left[\frac{t - \dfrac{A}{2}}{t + \dfrac{A}{2}}\right] p_1$$

$$p_2 = p_3 = 0$$

$$y_1 = \left(t + \frac{A}{2}\right) \tan\beta \qquad y_2 = \left(t - \frac{A}{2}\right) \tan\beta$$

6.6.4.5 Resultant in Zone 5 of base (see Fig. 6.3)



SK 6/20 Biaxial bending – loss of contact. Zero pressure on line p_3–p_3.

$$k = \frac{e_x}{A} + \frac{e_y}{B}$$

$$p_1 = \left(\frac{P}{AB}\right)k[12 - 3.9(6k - 1)(1 - 2k)(2.3 - 2k)]$$

S and t are as in Sections 6.6.4.3 and 6.6.4.4.

$$p_2 = \left[\frac{S - \dfrac{B}{2}}{S + \dfrac{B}{2}}\right]p_1$$

$$p_3 = 0$$

$$p_4 = \left[\frac{t - \dfrac{A}{2}}{t + \dfrac{A}{2}}\right]p_1$$

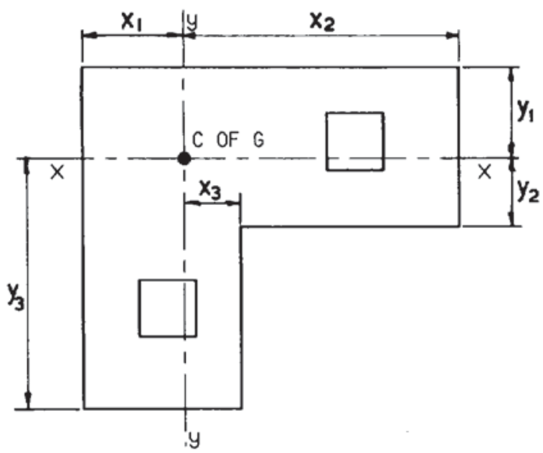
$$x_1 = \left(S - \frac{B}{2}\right)\left[\frac{t + \dfrac{A}{2}}{S + \dfrac{B}{2}}\right] \qquad y_1 = \left(t - \frac{A}{2}\right)\left[\frac{S + \dfrac{B}{2}}{t + \dfrac{A}{2}}\right]$$

Note: To find maximum pressure at a corner the design chart in Fig. 6.4 may be used. At the initial design stage when the size of the foundation is being determined, this design chart becomes very useful.

Find e_x/A and e_y/B.
Read from Fig. 6.4 the value of K.

$$\text{Maximum pressure} = \frac{PK}{AB}$$

6.6.5 Multiple column – biaxial bending – no loss of contact



SK 6/21 Typical unsymmetrical pad foundation and co-ordinates of corners to find bearing pressure.

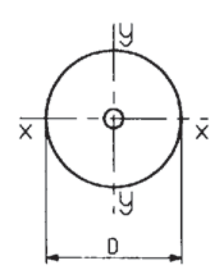
(1) Find area of foundation, A.
(2) Find centre of gravity of plan area of foundation.
(3) Find second moments of area about two orthogonal axes going through CG of area of foundation (I_{xx} and I_{yy}).
(4) Find maximum ordinates X and Y of corner points of foundation with respect to CG of area of foundation.
(5) Find eccentricities of all vertical loads from CG of area of foundation.
(6) Find total vertical load through CG of foundation and total moments about orthogonal axes passing through CG of area of foundation (P, M_{xx} and M_{yy}).
(7) Find maximum and minimum pressures at various points on foundation using the equation below.

$$P_n = \frac{P}{A} + \frac{M_{xx}y_n}{I_{xx}} + \frac{M_{yy}x_n}{I_{yy}}$$

Note: This method is valid only when there is no loss of contact between the foundation and the soil. Use a consistent sign convention as in Section 6.4.

6.6.6 Circular pad – biaxial bending – no loss of contact

$$A = \frac{\pi D^2}{4} = 0.7854D^2$$



SK 6/22 Circular pad foundation.

$$Z = \frac{\pi D^3}{32} = 0.0982D^3$$

$$M = (M_{xx}^2 + M_{yy}^2)^{\frac{1}{2}}$$

$$e = \frac{M}{P} \leq \frac{D}{8}$$

$$p_{\max} = \frac{P}{A} + \frac{M}{Z}$$

Design forces in pad foundation

Maximum shear force across diameter $D = V = 1.285pR^2 + 1.571qR^2$

Maximum bending moment across diameter $D = M_1 = 0.595pR^3 + 0.667qR^3$

where R = radius of circular pad = $D/2$
q = minimum pressure = $(P/A) - (M/Z)$
$p + q$ = maximum pressure = $(P/A) + (M/Z)$
$p = 2M/Z$.

6.7 Step-by-step design procedure for pads

Step 1 ***Select type and depth of foundation***

The types of foundations are as follows:

(A) Reinforced concrete pad with single column.
(B) Reinforced concrete pad with multiple column.
(C) Reinforced concrete pad with single column and ground beams.
(D) Reinforced concrete pad with multiple column and ground beams.
(E) Mass concrete pad with single column.

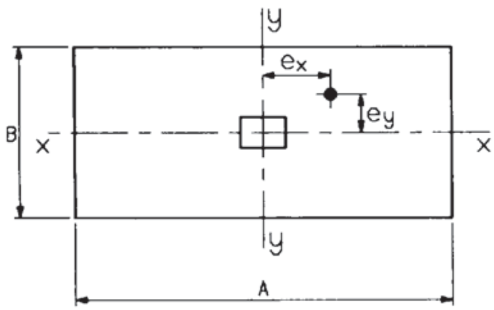
Note: Type E may be used for light single-storey structures only.

The depth of the foundation is governed by the following:

- Shrinking and swelling of clay.
- Frost attacks.
- Holding-down bolt arrangement of columns.
- Suitable bearing stratum.
- Water table and soluble sulphates.
- Width of foundation which is normally kept more than depths for shallow foundations.

Step 2 ***Select approximate size***

From the ground investigation report and from Tables 1–3 of BS 8004: 1986,[2] find the presumed allowable bearing capacity.
Find total maximum unfactored vertical load from column.
Find maximum unfactored moments M_x and M_y from column.



SK 6/23 Equivalent eccentricity of direct load on pad foundation.

Find eccentricities $e_x = \dfrac{M_{yy}}{P}$ and $e_y = \dfrac{M_{xx}}{P}$

Assume $A \geq 6e_x$ and $B \geq 6e_y$

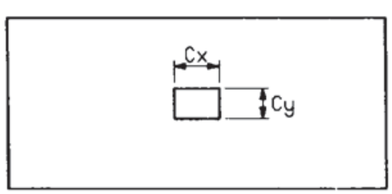
For biaxial bending,

find e_x/A and e_y/B, and from Fig. 6.4 find the value of K.

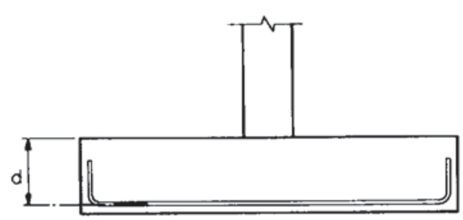
Maximum approximate pressure $= \dfrac{PK}{AB}$

Check whether the maximum pressure is lower than presumed allowable bearing capacity.

Note: At this stage some of the loads from the self-weight of the foundation, ground beams, backfill, eccentricities of surcharges, etc. have not been included and hence a margin has to be left in the bearing pressure to account for these. Moreover, the actual bearing pressure computations and settlement computations may further enhance the size of the foundation.



SK 6/24 Dimensions of column or pedestal on pad foundation.



SK 6/25 Effective depth of pad.

Determine minimum thickness of pad:
Find $v_{max} = 0.8\sqrt{f_{cu}}$ or $5\,N/mm^2$ whichever is lesser
Find column perimeter $= U_o = 2(C_x + C_y)$.
Find total ultimate vertical load from column $= N_u$
Find $d \geq N_u/(v_{max}\ U_o)$
also $d \geq 0.5[(C_1^2 + 4C_2)^{0.5} - C_1]$ whichever gives larger d.

$C_1 = U_o/6$
$C_2 = N_u/12v_c$ (assume v_c, which is dependent on percentage of tensile reinforcement)

Choose overall depth of pad allowing for cover.

Note: Consideration need not be given to anchorage length of column bars in pad foundation if all column bars are in compression.

Step 3 Calculate bearing capacity of soil
Follow Section 6.5.

Step 4 Calculate column load combinations
Follow Section 6.3.

Step 5 Calculate approximate settlement
Follow Method 1 of Section 6.5.2.

Note: The approximate settlement computation will help to determine the level of differential settlement for which the building should be designed.

Step 6 Carry out analysis for bearing pressure
Follow Section 6.1.

Step 7 Calculate bearing pressures under foundation
Follow Section 6.6.

Step 8 Calculate sliding resistance of foundation
Follow Section 6.5.3.

Step 9 Check combined sliding and bearing

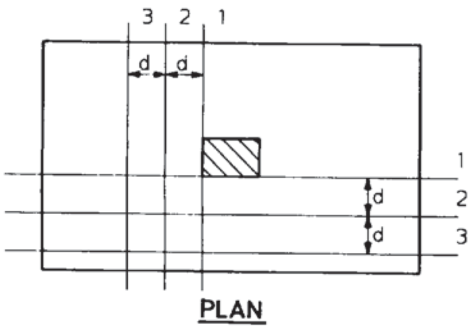
Check: $$\left(\frac{P}{P_v}\right) + \left(\frac{H_x}{P_{Hx}}\right) + \left(\frac{H_y}{P_{Hy}}\right) \leq 1$$

where P = total vertical load unfactored
P_v = area of base × allowable bearing capacity
H_x, H_y = total horizontal load unfactored in x- and y-directions
P_{Hx}, P_{Hy} = sliding resistance of base (Step 8) in x- and y-directions (see Section 6.5.3.)

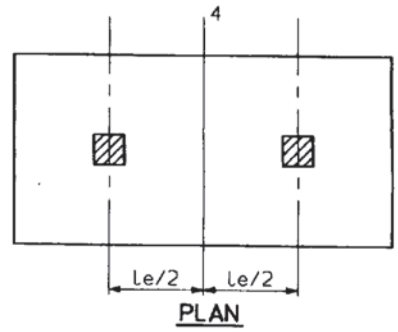
Step 10 Carry out analysis of bearing pressure for bending moment and shear
Follow Section 6.2.

Step 11 Calculate bearing pressure for bending moment and shear
Follow Section 6.6.

Step 12 Calculate bearing moments and shears in pad
Critical sections for bending moments and shears:



SK 6/26 Critical section for checking bending moments and shears in pad foundation.



SK 6/27 Critical section for bending moment.

- Sections 1, 4 – design bending moment.
- Section 1 – shear.
- Section 2 – shear.
- Section 3 – shear.

Note: When calculating bending moments and shears the downward loads on the pad will be considered with the upwards loads.
In a complicated unsymmetrical combined column foundation the bending moment and shear force diagram based on the pressure distribution should be drawn and critical sections determined from these diagrams.

Step 13 ***Determine cover to reinforcement***
From the soil investigations report, find the concentration of sulphates expressed as SO_3.
From Table 17 of BS 8004: 1986,[2] find the appropriate type of concrete.

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

Note: Concrete in 'class of exposure 5' needs protective coating.

Step 14 ***Calculate area of tension reinforcement and distribution***

M = bending moment due to ultimate loads in pad

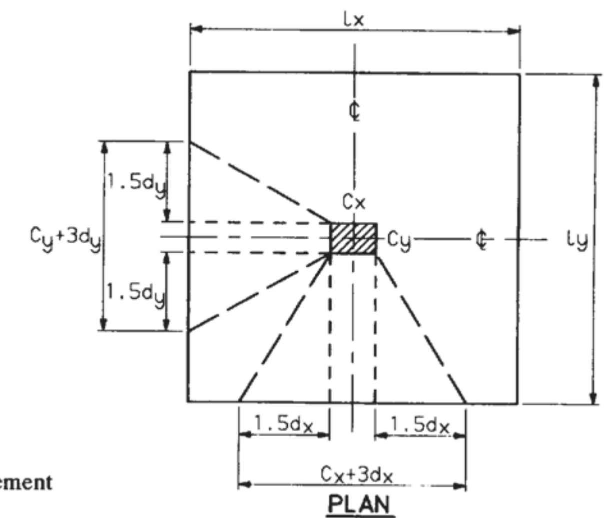
Find effective depth, d.

b = total width of section

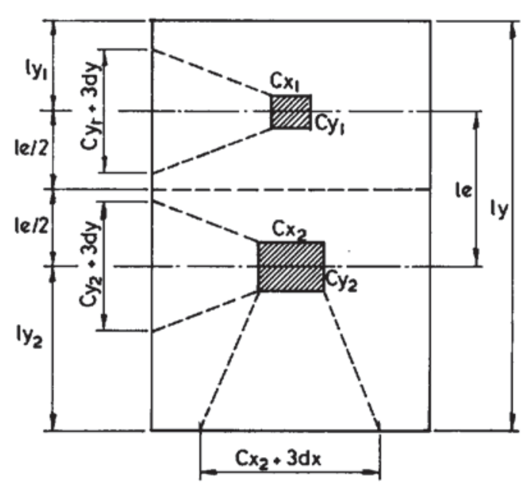
$$K = \frac{M}{f_{cu}bd^2} \leq 0.156$$

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

$$A_s = \frac{M}{0.87f_y z}$$



SK 6/28 Rules for distribution of reinforcement in pad foundation.



SK 6/29 Detailing rules for pad foundation with two columns.

Note: Increase depth of pad foundation if K is greater than 0.156.

Distribution of tension reinforcement

Case 1
If $l_x > 1.5\ (C_x + 3d_x)$, distribute two-thirds of total reinforcement in y-direction in the zone $(C_x + 3d_x)$.

Case 2
If $l_y > 1.5\ (C_y + 3d_y)$, distribute two-thirds of total reinforcement in x-direction in the zone $(C_y + 3d_y)$.

Case 3
If $l_x > 1.5\ (C_{x1} + 3d_x)$ or $1.5(C_{x2} + 3d_x)$, whichever is the lesser, distribute two-thirds of total reinforcement (top) in y-direction in the zone $(C_{x1} + 3d_x)$ or $(C_{x2} + 3d_x)$, whichever is the lesser.

Case 4
For bottom reinforcement in the combined foundation, follow the rules in Case 1 and Case 2, assuming individual foundations to the centre of column spacing $(l_c/2)$.

Note: d_x relates to effective depth for resistance against moment M_x which is about the orthogonal axis x. Similarly, d_y relates to effective depth for resistance against moment M_y which is about the orthogonal axis y, or, in other words, the reinforcement in the x-direction is to resist moment about the y axis and the effective depth is d_y.

Step 15 ***Check shear stress***
See Step 12.

Shear at Section 1

Check shear stress:

$$v_1 = \frac{V_1}{bd} \leq 0.8\sqrt{f_{cu}} \quad \text{or} \quad 5\,\text{N/mm}^2$$

where V_1 = total shear at Section 1
b = total width of Section 1
d = effective depth of Section 1.

Shear at Section 2

Check shear stress:

$$v_2 = \frac{V_2}{bd} \leq 2v_c$$

The value of v_c is obtained from Figs 11.2 to 11.5, depending on $100A_s/bd$ where A_s is the total area of tensile reinforcement in the section.

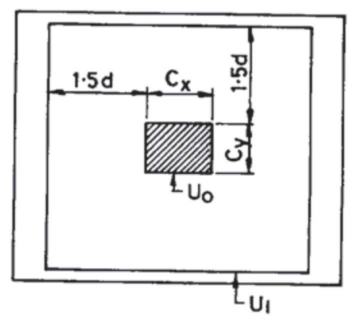
Shear at Section 3

Check shear stress:

$$v_3 = \frac{V_3}{bd} \leq v_c$$

Note: Change the thickness of the pad if the shear stress at any section exceeds the allowable limit. It is not cost-effective to provide shear reinforcement in the pad foundation.

Step 16 Check punching shear



SK 6/30 Punching shear perimeters in pad foundation.

N_u = maximum ultimate vertical load from column
Find perimeters U_o, U_1 and U_2.

$$d = 0.5(d_x + d_y)$$

$$U_o = 2(C_x + C_y)$$
$$U_1 = (U_o + 12d)$$

Check: $$v_o = \frac{N_u}{U_o d} \leq 0.8\sqrt{f_{cu}} \quad \text{or} \quad 5\,\text{N/mm}^2$$

$$v_1 = \frac{N_u - p_1 A_1}{U_1 d} \leq v_c$$

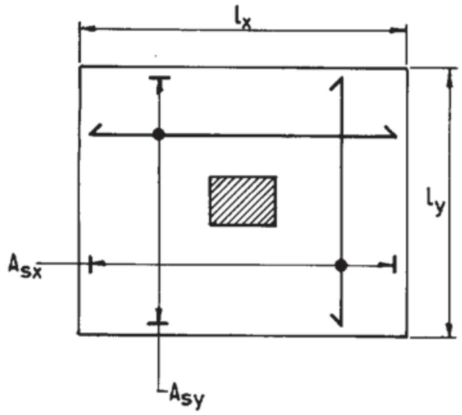
$$A_1 = (C_x + 3.0d)(C_y + 3.0d)$$

where p_1 = average upwards pressure over area A_1 enclosed by perimeter U_1.

The value of v_c is obtained from Figs 11.2 to 11.5 corresponding to p_x or p_y, whichever is the lesser.

$$p_x = \frac{100A_{sx}}{l_x d_x}$$

$$p_y = \frac{100A_{sy}}{l_y d_y}$$



SK 6/31 Typical tensile reinforcement plan for pad foundation.

Change the thickness of the pad if the punching shear stress exceeds v_c, otherwise shear reinforcement will be required as per Step 7 of Section 3.3.

Note: Apply the same principle individually to each column for a combined foundation.

Step 17 ***Check minimum reinforcement for flexure***

Minimum tensile reinforcement $= 0.0013bh$ in both directions ($f_y = 460\,\text{N/mm}^2$)

Note: Provide this minimum reinforcement also at the top of the foundation where top reinforcement is required for flexure.

Step 18 ***Check spacing of reinforcement***

Clear spacing of bars should not exceed $3d$ or 750 mm.

Percentage of reinforcement $100A_s/bd$ in pad (%)	Maximum clear spacing of bars for $f_y = 460\,\text{N/mm}^2$ (mm)
1 or over	160
0.75	210
0.5	320
0.3	530
less than 0.3	$3d$ or 750

Note: The above rules for spacing of bars in tension will in most cases ensure adequate control of crack widths to 0.3 mm where the cover does not exceed 50 mm.

Step 19 ***Check early thermal cracking***

Determine pour configuration:

(1) Massive pour cast on blinding concrete: $R = 0.1$ to 0.2
(2) Massive pour cast on existing mass concrete: $R = 0.3$ to 0.4 at base
$R = 0.1$ to 0.2 at top

where R = restraint factor.

Determine the value of temperature, T_1, for OPC concrete cast on ground from the table.

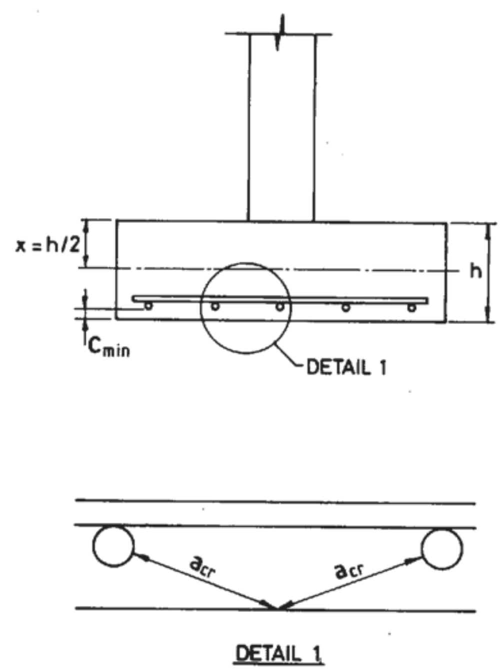
Section thickness (mm)	T_1(°C)
300	17
500	28
700	28
1000	28

Calculate $\varepsilon_r = 0.8T_1\alpha R$

where $\alpha = 12 \times 10^{-6}$/°C, or values from Table 2.3 may be used

$$W_{\max} = \frac{3a_{cr}\,\varepsilon_r}{1 + 2\left(\dfrac{a_{cr} - C_{\min}}{h - x}\right)}$$

Note: The design crack width is 0.3 mm. If this is exceeded, closer spacings of bars may be used.



SK 6/32 Early thermal crack width calculation.

Step 20 ***Check minimum reinforcement to distribute thermal cracking***

$\rho_{crit} = 0.0035$ for $f_y = 460\,\text{N/mm}^2$

Up to 300 mm thickness of pad foundation

$A_{st} = 0.00175bh$ near top surface in each direction

From 300 mm to 500 mm thickness of pad foundation

$A_{st} = 0.001\,75bh\,\text{mm}^2$ near top surface in each direction

$A_{sb} = 0.35b\,\text{mm}^2$ near bottom surface in each direction

Over 500 mm thickness of pad foundation

$A_{st} = 0.875b\,\text{mm}^2$ near top surface in each direction

$A_{sb} = 0.35b\,\text{mm}^2$ near bottom surface in each direction

where b = width of pad perpendicular to the direction of reinforcement (mm)
h = overall thickness of pad (mm).

Step 21 ***Check crack width due to flexure***

Serviceability limit state
Loading conditions LC_1 to LC_4 in Section 6.3.
Find bending moment M across a critical section, as in Step 12.

$$m = \frac{E_s}{E_c} = 15 \qquad \text{for long-term loading}$$

$$x = \left[\left(\frac{mA_s}{b}\right)^2 + \frac{2mA_sd}{b}\right]^{\frac{1}{2}} - \frac{mA_s}{b}$$

$$z = d - \frac{x}{3}$$

$$f_s = \frac{M}{A_sz}$$

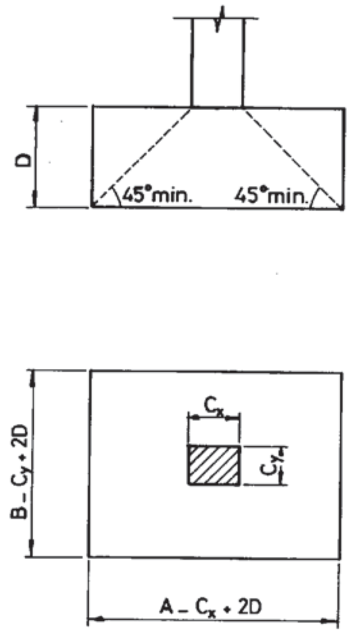
$$\varepsilon_s = \frac{f_s}{E_s} \qquad \varepsilon_h = \left(\frac{h-x}{d-x}\right)\varepsilon_s$$

$$\varepsilon_{mh} = \varepsilon_h - \frac{b(h-x)^2}{3E_sA_s(d-x)}$$

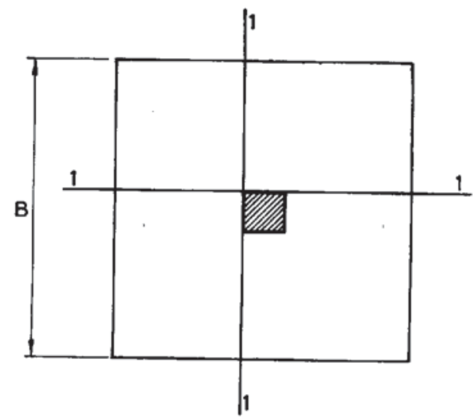
$$W_{max} = \frac{3a_{cr}\,C_{min}}{1 + \dfrac{2(a_{cr} - C_{min})}{h-x}}$$

Note: In extremely severe exposure conditions it is prudent to check crack widths and provide adequate reinforcement to limit it to an allowable value.

Step 22 Design mass concrete foundation



SK 6/33 Typical plan and elevation of mass concrete pad foundation.



SK 6/34 Critical sections in mass concrete foundation.

The size and depth will be determined based on a 45° dispersion of load from column through mass concrete on to soil.
D = depth of foundation.
At critical section 1, find the bending moment and shear from the soil pressure diagram for unfactored loads: serviceability limit state.

M = bending moment across critical section
V = shear across critical section

Check: f_{tb} = stress in concrete in bending tension

$$= \frac{6M}{BD^2} \text{ or } \frac{6M}{AD^2} \leq 0.37\sqrt{f_{cu}}$$

v_d = shear stress in concrete

$$= \frac{V}{BD} \text{ or } \frac{V}{AD} \leq 0.037 f_{cu}$$

$$\text{Punching shear stress} = \frac{V}{2(C_x + C_y)D} \leq 0.037 f_{cu}$$

$$\text{Bearing stress under column base plate} = \frac{V}{C_x C_y} \leq 0.275 f_{cu}$$

Step 23 ***Calculate settlement***
Follow Method 2 of Section 6.5.2.
Use load combination LC_{13} of Section 6.3.

Note: The settlement calculations should be carried out to give a better understanding of the global effects on the structure. It may be necessary to alter the sizes of some of the pad foundations in a structure in order to even out the differential settlements. It is also important in certain cases to feed back these settlements in the analysis of the structure.

6.8 WORKED EXAMPLES

Example 6.1 ***RC pad with single column***
Internal column.

Column size $= 400\,\text{mm} \times 400\,\text{mm}$
Column spacing $= 6\,\text{m} \times 6\,\text{m}$ on plan

The unfactored column loads are given in the following table:

	Dead	Imposed	Wind
Vertical load N (kN)	610	480	–
Horizontal shear H_x (kN)	–	–	42
Horizontal shear H_y (kN)	–	–	38
Moment M_x (kNm)	–	–	95
Moment M_y (kNm)	–	–	105

Suitable bearing stratum at 1000 mm below ground level. Medium dense silty sand.

Step 1 ***Select type and depth of foundation***

Type: Reinforced concrete pad with single RC column

Depth: 1000 mm below finished ground level
1150 mm below finished floor level.

Depth selected from considerations of:

- Frost attack.
- Swelling of soil.
- Suitable bearing stratum.

Step 2 ***Select approximate size***

Presumed allowable bearing capacity from BS 8004: 1986[2] $= 150\,\text{kN/m}^2$

Maximum Vertical load $V = 610 + 480 = 1090\,\text{kN}$

Maximum eccentricity $= e_x = M_y/V = 105/1090 = 0.1\,\text{m}$

$6e_x = 0.6\,\text{m} \leq A$

$\dfrac{V}{150} = 7.3\,\text{m}^2$

Assume a $3\,\text{m} \times 3\,\text{m}$ foundation pad with area of $9\,\text{m}^2$: $A = 3\,\text{m}$ and $B = 3\,\text{m}$.

Determine minimum thickness of pad
Assume grade of concrete = C30

$v_{max} = 0.8\sqrt{f_{cu}}$ or $5\,\text{N/mm}^2$ (whichever is lesser) $= 4.38\,\text{N/mm}^2$

$U_o = 2(C_x + C_y) = 2(400 + 400) = 1600\,\text{mm}$

Total factored load from column

$N_u = 1.4 \times 610 + 1.6 \times 480 = 1622\,\text{kN}$

$$d \geq \frac{N_u}{v_{max} U_o} = \frac{1622 \times 10^3}{4.38 \times 1600} = 231\,\text{mm}$$

$$\text{or} \quad d \geq \frac{1}{2}[(C_1^2 + 4C_2)^{\frac{1}{2}} - C_1] = 430\,\text{mm}$$

$$\text{where} \quad C_1 = \frac{U_o}{6} = \frac{1600}{6} = 267\,\text{mm}$$

$$C_2 = \frac{N_u}{12 v_c} = \frac{1622 \times 10^3}{12 \times 0.45} = 300\,370\,\text{mm}^2$$

Assumed $v_c = 0.45\,\text{N/mm}^2$ which corresponds to about 0.3% tension reinforcement for $f_{cu} = 30\,\text{N/mm}^2$. Choose overall depth of pad equal to 500 mm allowing for adequate cover.

Step 3 ***Calculate bearing capacity of soil***
(See Section 6.5.1.)

Note: This step may not be necessary if the allowable bearing capacity for the selected size of foundation is already available from the soils investigation report. However, for completeness of a foundation design problem, this step is included.

From field and laboratory tests the following soil parameters of the bearing stratum are known:

ground water table = 2.0 m below ground level

$h = 0$, i.e. height of water table above foundation level is zero.

γ = unit weight of soil = $18\,\text{kN/m}^3$

p = total overburden pressure at the foundation level.
$= 18 \times 1$ (below ground)
$= 18\,\text{kN/m}^2$

$$p_o = p - \gamma_w h = 18\,\text{kN/m}^2$$

γ_w = unit weight of water

c = soil cohesion = $10\,\text{kN/m}^2$

ϕ = angle of internal friction = $22° = 0.384$ radian

$$\begin{aligned} a &= e^{(0.75\pi - \phi/2)\tan\phi} \\ &= e^{(0.75\pi - 0.192)\tan 22°} \\ &= 2.3974 \end{aligned}$$

$$N_q = \frac{a^2}{2\cos^2\left(45° + \dfrac{\phi}{2}\right)} = 9.19$$

$$N_c = (N_q - 1)\cot\phi = 20.3$$

$K_{py} = 29.0$ from Table 6.1 in Section 6.5.1

$$N_\gamma = \frac{\tan\phi}{2}\left(\frac{K_{py}}{\cos^2\phi} - 1\right) = 6.6$$

$$\begin{aligned} q_{ult} &= 1.3cN_c + p_o(N_q - 1) + 0.4\gamma B N_\gamma + p \\ &= 1.3 \times 10 \times 20.3 + 18 \times (9.19 - 1) + 0.4 \times 18 \times 3 \times 6.6 + 18 \\ &= 572\,\text{kN/m}^2 \end{aligned}$$

$$\text{Allowable bearing capacity} = \frac{q_{ult}}{3} = \frac{572}{3} = 190\,\text{kN/m}^2$$

Step 4 ***Calculate column load combinations***
(See Section 6.3.)

Bearing pressure calculations

$LC_1 = 1.0DL + 1.0IL$
$LC_3 = 1.0DL + 1.0IL + 1.0WL$

LC_1: Combined vertical column load, $N = 610 + 480 = 1090\,\text{kN}$
$H_x = 0 \quad H_y = 0 \quad M_x = 0 \quad M_y = 0$

LC_3: N = vertical load = $1090\,\text{kN}$
$H_x = 42\,\text{kN} \quad M_y = 105\,\text{kNm} \quad H_y = 0 \quad M_x = 0$

Note: By inspection, wind in one direction only may be checked for a square foundation.

Bending moment and shear calculations

$LC_5 = 1.4DL + 1.6IL$
$LC_6 = 1.2DL + 1.2IL + 1.2WL$
$LC_7 = 1.4DL + 1.4WL$

LC_5: $N_u = 1.4 \times 610 + 1.6 \times 480 = 1622\,\text{kN}$
$H_{xu} = 0 \quad H_{yu} = 0 \quad M_{xu} = 0 \quad M_{yu} = 0$

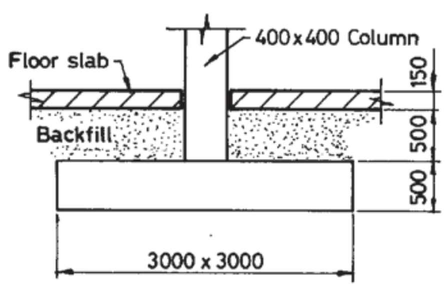
LC_6: $N_u = 1.2 \times 610 + 1.2 \times 480 = 1308\,\text{kN}$
$H_{xu} = 1.2 \times 42 = 50.4\,\text{kN} \quad H_{yu} = 0$
$M_{xu} = 0 \quad M_{yu} = 1.2 \times 105 = 126\,\text{kNm}$

LC_7: $N_u = 1.4 \times 610 = 854\,\text{kN}$
$H_{xu} = 1.4 \times 42 = 58.8\,\text{kN} \quad H_{yu} = 0$
$M_{yu} = 1.4 \times 105 = 147\,\text{kNm}$

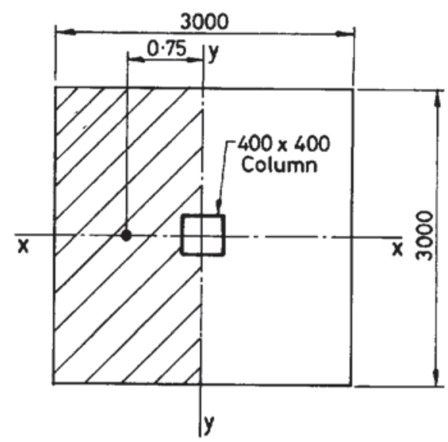
Step 5 ***Calculate approximate settlement***

This step may be ignored since the foundations are not connected by ground beams and the differential settlements will have little effect on the design of this foundation.

Step 6 ***Carry out analysis for bearing pressure***



SK 6/35 Section through pad foundation.



SK 6/36 Eccentric surcharge on pad foundation on plan.

Weight of foundation $= 9\,\text{m}^2 \times 0.50\,\text{m} \times 24\,\text{kN/m}^3$
$= 108\,\text{kN}$

Weight of backfill + ground slab $= 9\,\text{m}^2 \times 0.50\,\text{m} \times 18\,\text{kN/m}^3 + 9\,\text{m}^2 \times 0.15\,\text{m} \times 24\,\text{kN/m}^3$
$= 113.4\,\text{kN}$

Surcharge on ground slab $= 5\,\text{kN/m}^2$

Weight of surcharge on half foundation $= 4.5\,\text{m}^2 \times 5\,\text{kN/m}^2 = 22.5\,\text{kN}$

Eccentricity of surcharge $= 0.75\,\text{m}$

Weight of surcharge on full foundation $= 45\,\text{kN}$

LC_1: p = total vertical load
$= 1090 + 108 + 113.4 + 45$
$= 1356.4\,\text{kN}$

$H_x = 0 \quad H_y = 0 \quad M_x = 0 \quad M_y = 0$

LC_3: $P = 1090 + 108 + 113.4 + 22.5$
$= 1333.9\,\text{kN}$
$H_x = 42\,\text{kN} \quad M_y = 105\,\text{kNm} \quad H_y = 0$
$M_{xx} = 0 \quad M_{yy} = 105 + 42 \times 0.45 + 22.5 \times 0.75 = 140.8\,\text{kNm}$

Step 7 ***Calculate bearing pressure under foundation***
(See Section 6.6.)

$$LC_1:\ p = \frac{1356.4}{9} = 150.7\,\text{kN/m}^2 < 190\,\text{kN/m}^2$$

$$LC_2:\ e_x = \frac{M_{yy}}{P}$$
$$= 0.104\,\text{m} < A/6 = 0.50\,\text{m}$$

$$p_1 = \frac{P}{AB} + \frac{6M_{yy}}{A^2B}$$
$$= \frac{1333.9}{9} + \frac{6 \times 140.8}{9 \times 3}$$
$$= 179.5\,\text{kN/m}^2 < 190 \times 1.25 = 237.5\,\text{kN/m}^2$$

Note: 25% overstress on allowable bearing capacity may be allowed for combinations including wind.

Bearing pressures are within allowable limits.

Step 8 ***Calculate sliding resistance of foundation***
Ignore passive resistance because horizontal movement of the foundation should be avoided.
(See Section 6.5.3.)
Assume $\delta = 17°$ from Table 6.6.

$P = 610 + 108 + 113.4$ (dead load only) $= 831.4\,\text{kN}$

$P_s = P \tan\delta = 831.4 \times \tan 17° = 254\,\text{kN} > 1.5H = 1.5 \times 42 = 63\,\text{kN}$

$P_H = qA \tan\phi + cA$
$= 831.4 \times \tan 22° + 10 \times 9$
$= 426\,\text{kN} > P_s = 254\,\text{kN}$

Step 9 ***Check combined sliding and bearing***
$P = 1356.4\,\text{kN}$

$P_v = 190\,\text{kN/m}^2 \times 9\,\text{m}^2 = 1710\,\text{kN}$

$H_x = 42\,\text{kN}$

$P_{Hx} = 426\,\text{kN}$

$$\frac{P}{P_v} + \frac{H_x}{P_{Hx}} = \frac{1356.4}{1710} + \frac{42}{426}$$
$$= 0.89 < 1 \quad \text{OK}$$

Step 10 Carry out analysis of bearing pressure for bending moment and shear

LC_5: $N_u = 1622\,\text{kN}$

$P_u = N_u + 1.4\,(\text{foundation} + \text{backfill}) + 1.6\,(\text{surcharge on backfill})$
$= 1622 + 1.4 \times (108 + 113.4) + 1.6 \times 45$
$= 2004\,\text{kN}$

$H_{xu} = 0 \qquad H_{yu} = 0 \qquad M_{xxu} = 0 \qquad M_{yyu} = 0$

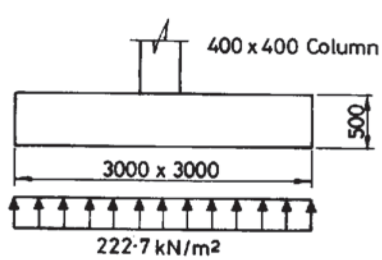
LC_6: $P_u = N_u + 1.2\,(\text{foundation} + \text{backfill} + \text{surcharge})$
$= 1308 + 1.2\,(108 + 113.4 + 22.5)$
$= 1601\,\text{kN}$

$M_{xxu} = 0$
$M_{yyu} = M_{yu} + H_{xu}h + M^*_{yu}$
$= 126 + 50.4 \times 0.45 + 1.2 \times 22.5 \times 0.75$
$= 168.9\,\text{kNm}$

LC_7: $P_u = N_u + 1.4\,(\text{foundation} + \text{backfill})$
$= 854 + 1.4\,(108 + 113.4)$
$= 1164\,\text{kN}$

$M_{xxu} = 0$
$M_{yyu} = M_{yu} + H_{xu}h$
$= 147 + 58.8 \times 0.45$
$= 173.5\,\text{kNm}$

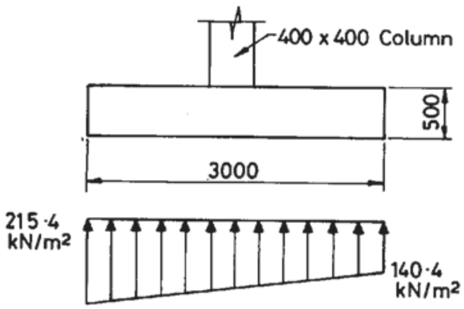
Step 11 Calculate bearing pressure for bending moment and shear



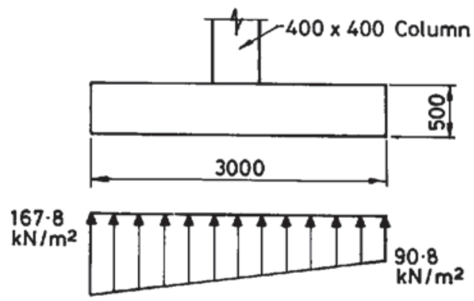
SK 6/37 Uniform bearing pressure for load combination LC_5.

$$LC_5: \quad p = \frac{P_u}{AB} = \frac{2004}{9} = 222.7\,\text{kN/m}^2$$

$$LC_6: \quad p_1 = \frac{P_u}{AB} + \frac{6M_{yyu}}{A^2B}$$
$$= \frac{1601}{9} + \frac{6 \times 168.9}{27}$$
$$= 177.9 + 37.5 = 215.4\,\text{kN/m}^2$$



SK 6/38 Bearing pressure for load combination LC_6.



SK 6/39 Bearing pressure for load combination LC_7.

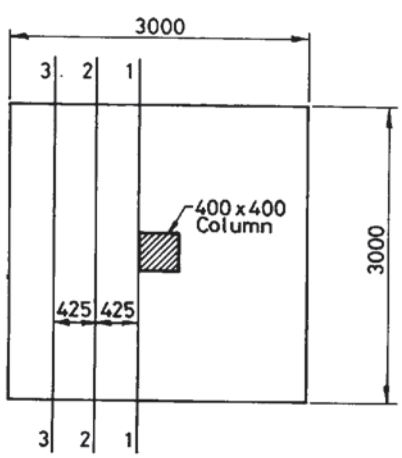
$$p_2 = \frac{P_u}{AB} - \frac{6M_{yyu}}{A^2B}$$
$$= 140.4\,\text{kN/m}^2$$

$$LC_7: \quad p_1 = \frac{P_u}{AB} + \frac{6M_{yyu}}{A^2B}$$
$$= \frac{1164}{9} + \frac{6 \times 173.5}{27}$$
$$= 129.3 + 38.5 = 167.8\,\text{kN/m}^2$$
$$p_2 = 129.3 - 38.5 = 90.8\,\text{kN/m}^2$$

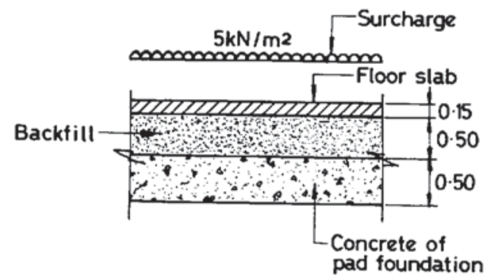
Step 12 Calculate bending moments and shears in pad

LC_5: Downward load on pad $= p_d$
$p_d =$ self-weight of pad + backfill + surcharge

Upward load on pad $= p_u$
$p_u =$ pressure of ground on pad
(see Step 10).



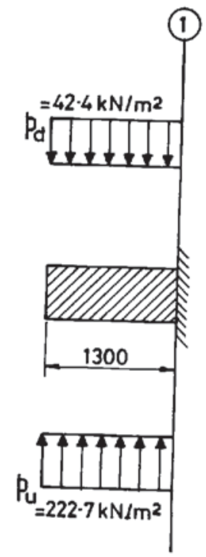
SK 6/40 Critical sections for bending moment and shears on plan of pad foundation.



SK 6/41 Calculation of p_d for downward load on overhang.

$$p_d = (0.50 \times 24 + 0.50 \times 18 + 0.15 \times 24) \times 1.4 + (5\,\text{kN/m}^2) \times 1.6$$
$$= 42.4\,\text{kN/m}^2$$

$$p_u = 222.7\,\text{kN/m}^2 \quad \text{constant}$$



SK 6/42 Typical loading on pad foundation overhang at section 1–1.

Cantilever overhang at section 1–1 = 1500 − 200 = 1300 mm = l

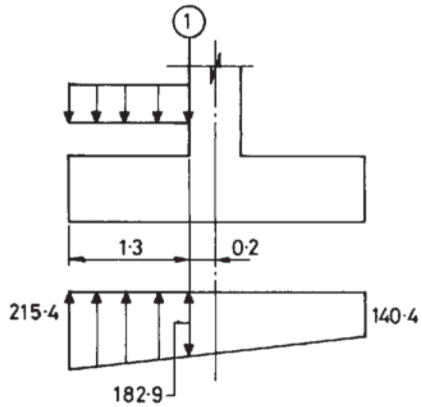
$$\text{Bending moment at section 1} = M_1 = \frac{(p_u - p_d)Bl^2}{2}$$
$$= \frac{180.3 \times 3 \times 1.3^2}{2}$$
$$= 457.1\,\text{kNm}$$

$$\text{Shear at section 1} = V_1 = (p_u - p_d)Bl$$
$$= 180.3 \times 3 \times 1.3$$
$$= 703.2\,\text{kN}$$

Assume $d = 425\,\text{mm}$

Shear at section 2 = $V_2 = (p_u - p_d)B(l - 0.425) = 473.3\,\text{kN}$

Shear at section 3 = $V_3 = (p_u - p_d)B(l - 2 \times 0.425) = 243.4\,\text{kN}$



SK 6/43 Loading on overhang of pad foundation due to LC_6.

LC_6: Pressure at section 1–1 $= 177.9 + \dfrac{37.5 \times 0.2}{1.5} = 182.9\,\text{kN/m}^2$

$$p_d = 1.2(0.50 \times 24 + 0.50 \times 18 + 0.15 \times 24 + 5) = 35.5\,\text{kN/m}^2$$

$$\text{Bending moment} = M_1 = (182.9 - 35.5) \times 3 \times \frac{1.3^2}{2}$$

$$+ (215.4 - 182.9) \times 0.5 \times 1.3 \times 3 \times \frac{2}{3} \times 1.3$$

$$= 428.6\,\text{kNm}$$

The shears at sections 1, 2 and 3 need not be checked. By inspection they will be less critical than LC_5.
LC_7 need not be checked. By inspection it will not be critical.

Step 13 ***Determine cover to reinforcement***
From SI report, total $SO_3 = 0.5\%$
Class of exposure = 3
(See write-up of Step 13 in Section 6.7.)
75 mm blinding concrete will be used.
Minimum cover on blinding concrete = 50 mm
Assume 16 mm diameter HT Type 2 deformed bars.
Effective depth of top layer (symmetrical reinforcement in both directions), $d = 500 - 50 - 16 - 8 = 426\,\text{mm}$

Step 14 ***Calculate area of tensile reinforcement***
Maximum bending moment on section 1–1 = 457.1 kNm (LC_5)

$$K = \frac{M}{f_{cu}bd^2} = \frac{457.1 \times 10^6}{30 \times 3000 \times 426^2} = 0.028$$

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

$$= d\left[0.5 + \sqrt{\left(0.25 - \frac{0.028}{0.9}\right)}\right] = 0.95 \times 426 = 405\,\text{mm}$$

$$A_{st} = \frac{M}{0.87 f_y z} = \frac{457.1 \times 10^6}{0.87 \times 460 \times 405}$$
$$= 2820\,\text{mm}^2$$

Use 15 no. 16 dia. Type 2 HT bars in each direction ($3015\,\text{mm}^2$).

Distribution of tension reinforcement
(See write-up of Step 14 in Section 6.7.)

$C_x = C_y = 400\,\text{mm}$

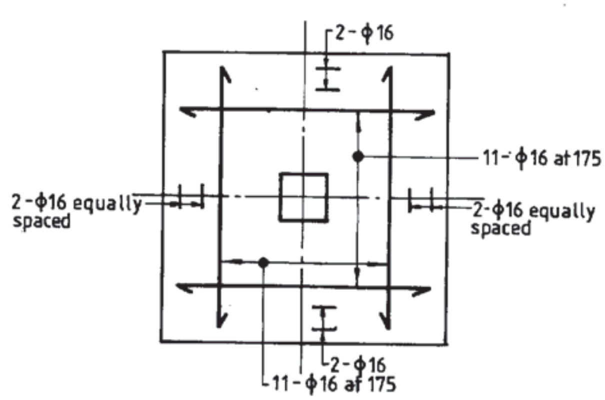
$d_x = 500 - 50 - 8 = 442\,\text{mm}$
$d_y = 500 - 50 - 16 - 8 = 426\,\text{mm}$

$1.5(C_y + 3d_y) = 2517\,\text{mm} < l_y = 3000\,\text{mm}$
$1.5(C_x + 3d_x) = 2589\,\text{mm} < l_x = 3000\,\text{mm}$

$$\frac{2}{3} A_{st} = \frac{2}{3} \times 2820 = 1880\,\text{mm}^2$$

Reinforcement over central $C_y + 3d_y$(1678 mm) and $C_x + 3d_x$(1726 mm)

$$= \frac{1880}{1.678} = 1120\,\text{mm}^2/\text{m}$$



SK 6/44 Distribution of reinforcement in pad foundation.

Use 11 no. 16 mm dia. bars at 175 mm centres ($1149\,\text{mm}^2/\text{m}$) over the central zone in each direction.
Use 2 no. 16 mm dia. bars on each side outside the central zone.
Total number of 16 mm bars used = 15($3015\,\text{mm}^2$)
All bars are HT Type 2.

Step 15 ***Check shear stress***

(See Step 12 − LC_5.)

$$\text{Check}\quad v_1 = \frac{V_1}{bd} \le 0.8\sqrt{f_{cu}} \quad \text{or} \quad 5\,\text{N/mm}^2$$

$$= \frac{703.2 \times 10^3}{3000 \times 426} = 0.55\,\text{N/mm}^2 < 0.8\sqrt{f_{cu}}$$

$$\text{Check}\quad v_2 = \frac{V_2}{bd} \le 2v_c$$

$$= \frac{473.3 \times 10^3}{3000 \times 426} = 0.37\,\text{N/mm}^2$$

A_{st} = 15 no. 16 mm dia. bars = 3015 mm^2

Use larger d (442 mm) for calculation of p.

$$p = \frac{100A_{st}}{bd}$$

$$= \frac{100 \times 3015}{3000 \times 442} = 0.23\%$$

From Fig. 11.3 for $f_{cu} = 30\,\text{N/mm}^2$,

$v_c = 0.42\,\text{N/mm}^2 > v_2$

No more shear checks are necessary.

Step 16 ***Check punching shear***

(See write-up of Step 14 in Section 6.7.)

$d_x = 442\,\text{mm} \qquad d_y = 426\,\text{mm}$

$d = 0.5\,(442 + 426) = 434\,\text{mm}$

$U_o = 2\,(C_x + C_y) = 2(400 + 400) = 1600\,\text{mm}$

$U_1 = (U_o + 12d) = 1600 + 12 \times 434 = 6808\,\text{mm}$

$$v_o = \frac{N_u}{U_{od}} \le 0.8\sqrt{f_{cu}} \quad \text{or} \quad 5\,\text{N/mm}^2$$

(See Step 4 − LC_5.)

$N_u = 1622\,\text{kN}$

$$v_o = \frac{1622 \times 10^3}{1600 \times 434} = 2.34\,\text{N/mm}^2 < 0.8\sqrt{f_{cu}}$$

$p_1 = p_u - p_d = 180.3\,\text{kN/m}^2 \qquad$ (see Step 12)

$$\begin{aligned} A_1 &= (C_x + 3.0d_x)(C_y + 3.0d_y) \\ &= (400 + 3.0 \times 426)(400 + 3.0 \times 442) \times 10^{-6} \\ &= 2.90\,\text{m}^2 \end{aligned}$$

$$v_1 = \frac{N_u - p_1 A_1}{U_1 d}$$

$$= \frac{(1622 - 180.3 \times 2.9) \times 10^3}{6808 \times 434}$$

$$= 0.37\,\text{N/mm}^2 \quad \text{OK}$$

$$v_c = 0.42\,\text{N/mm}^2 \qquad \text{(from Step 15)}$$

Step 17 ***Check minimum reinforcement for flexure***

Minimum tensile reinforcement $= 0.0013bh = 0.0013 \times 3000 \times 500$
$= 1950\,\text{mm}^2 < 3015\,\text{mm}^2$ provided

No top tension in pad foundation.

Step 18 ***Check spacing of reinforcement***

Percentage reinforcement $p = 0.23\%$ (see Step 15)

Maximum spacing = 750 mm not exceeded

Step 19 ***Check early thermal cracking***

(See write-up of Step 19 in Section 6.7.)

$R = 0.15$, say

$T_1 = 28°\text{C}$

$\alpha = 12 \times 10^{-6}/°\text{C}$

$$\varepsilon_r = 0.8T_1\alpha R$$
$$= 0.8 \times 28 \times 12 \times 10^{-6} \times 0.15$$
$$= 4.032 \times 10^{-5}$$

$x = h/2 = 250\,\text{mm}$ (assumed)

$a_{cr} = \sqrt{(74^2 + 137.5^2)} - 8 = 148\,\text{mm}$

$$W_{max} = \frac{3a_{cr}\varepsilon_r}{1 + \left(\dfrac{2(a_{cr} - C_{min})}{h - x}\right)}$$

$$= \frac{3 \times 148 \times 4.032 \times 10^{-5}}{1 + \left(\dfrac{2(148 - 66)}{500 - 250}\right)}$$

$$= 0.01\,\text{mm} < 0.3\,\text{mm}$$

Step 20 ***Check minimum reinforcement to distribute thermal cracking***

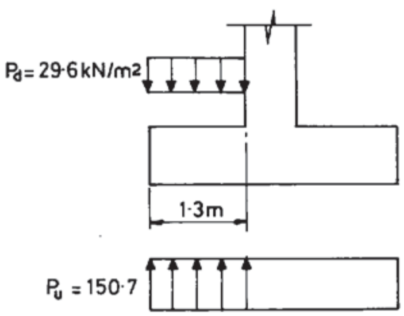
(See write-up of Step 20 in Section 6.7.)

Top reinforcement $= 0.00175bh$
$= 0.00175 \times 3000 \times 500$
$= 2625\,\text{mm}^2$ over 3000 mm

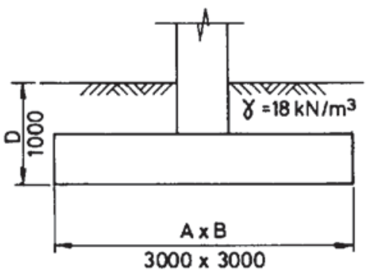
Bottom reinforcement $= 0.35b$
$= 0.35 \times 3000$
$= 1050\,\text{mm}^2$

Note: If thermal cracking has to be avoided, then a top mesh of 16 dia. bars at 200 c/c should be provided. This may seem unnecessary in the present circumstances since the pad will be fully buried in the ground.

Step 21 Check crack width due to flexure



SK 6/45 Pressure diagram for serviceability limit state.



SK 6/46 Calculation of net foundation pressure at serviceability limit state.

Serviceability limit state

Loading condition LC_1. (See Step 7.)

$$p_u = 150.7\,\text{kN/m}^2$$
$$p_d = 0.5 \times 24 + 0.5 \times 18 + 0.15 \times 24 + 5 = 29.6\,\text{kN/m}^2$$

$$M = \frac{(p_u - p_d)Bl^2}{2}$$
$$= \frac{(150.7 - 29.6) \times 3 \times 1.3^2}{2} = 307\,\text{kNm}$$

$m = 15 \qquad A_s = 3015\,\text{mm}^2 \qquad b = 3000\,\text{mm} \qquad d = 426\,\text{mm}$

$$x = \left[\left(\frac{mA_s}{b}\right)^2 + \frac{2mA_sd}{b}\right]^{\frac{1}{2}} - \frac{mA_s}{b}$$
$$= \left[\left(\frac{15 \times 3015}{3000}\right)^2 + \frac{2 \times 15 \times 3015 \times 426}{3000}\right]^{\frac{1}{2}} - \frac{15 \times 3015}{3000}$$
$$= 99\,\text{mm}$$

$$z = d - \frac{x}{3} = 426 - \frac{99}{3} = 393\,\text{mm}$$

$$f_s = \frac{M}{zA_s} = \frac{307 \times 10^6}{393 \times 3015} = 259\,\text{N/mm}^2$$

$$\varepsilon_s = \frac{f_s}{E_s} = \frac{259}{200 \times 10^3} = 1.295 \times 10^{-3}$$

$$\varepsilon_h = \left(\frac{h - x}{d - x}\right)\varepsilon_s = \left(\frac{500 - 99}{426 - 99}\right) \times 1.295 \times 10^{-3} = 1.588 \times 10^{-3}$$

$$\varepsilon_{mh} = \varepsilon_h - \frac{b(h - x)^2}{3E_sA_s(d - x)}$$

$$= 1.588 \times 10^{-3} - \frac{3000 \times (500 - 99)^2}{3 \times 200 \times 10^3 \times 3015 \times (426 - 99)} = 0.773 \times 10^{-3}$$

$c_{min} = 50 + 16 = 66\,\text{mm}$ for second layer

$a_{cr} = 148\,\text{mm}$ (see Step 19)

$$W_{max} = \frac{3a_{cr}\,\varepsilon_{mh}}{1 + \dfrac{2(a_{cr} - c_{min})}{h - x}}$$

$$= \frac{3 \times 148 \times 0.773 \times 10^{-3}}{1 + \dfrac{2(148 - 66)}{500 - 99}}$$

$$= 0.24\,\text{mm} < 0.3\,\text{mm} \quad \text{OK}$$

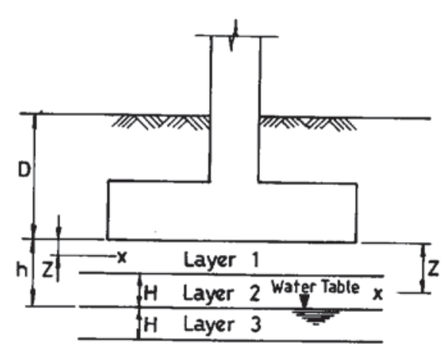
Note: The crack width should be checked if the foundation level is below the water table and the total SO_3 is higher than 1%.

Step 22 ***Design mass concrete foundation***
Not required.

Step 23 ***Calculate settlement***
Load combination LC_{13}: (See Section 6.3.)

$LC_{13} = 1.0DL + 0.5IL$ (vertical loads only)

$P = 610 + 0.5 \times 480 + 108 + 113.4 + 22.5 = 1093.9\,\text{kN}$



SK 6/47 Calculation of settlement.

$$\text{Gross foundation pressure} = \frac{P}{AB} = 121.5\,\text{kN/m}^2$$

$$\text{Weight of soil removed} = ABD\gamma = 3 \times 3 \times 1 \times 18 = 162\,\text{kN}$$

$$q_n = \text{net foundation pressure} = 121.5 - \frac{162}{9} = 103.5\,\text{kN/m}^2$$

$$\frac{A}{B} = 1 \qquad A = B = 3\,\text{m}$$

Complete the settlement computation table up to $Z = 2.5B$.

Ground water table at 2.0 m below ground level

Total settlement = 24.53 mm

C_{kd} = cone resistance (kg/cm^2)
$C = 147\ C_{kd}/p_o$
σ_z = vertical stress at centre of layer (kN/m^2)
Z = depth to centre of layer
h = height of ground water above centre of layer
$p_o = p - \gamma_w h$

σ_z is obtained from Fig. 6.2.

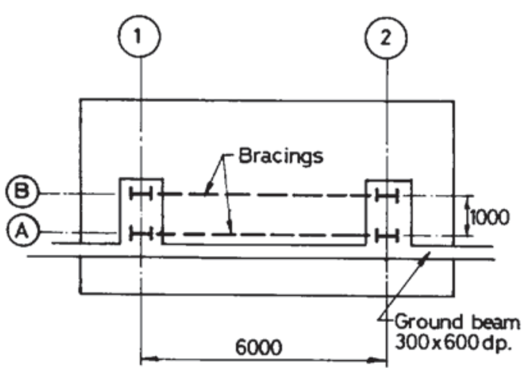
Note: Check whether this long-term predicted settlement is going to cause any problem elsewhere in the structural system.

Step 24 Design connection of pad to column (see Chapter 10)

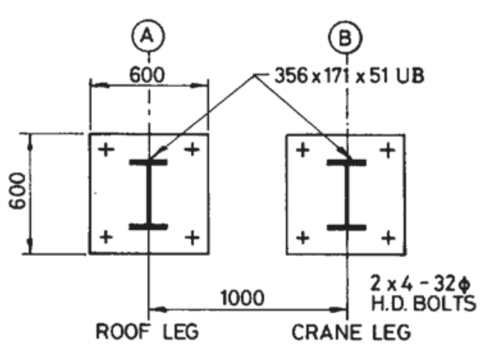
Example 6.2 RC pad with multiple columns

Foundation for the braced columns.

Unfactored loads from the columns.



SK 6/48 Combined pad foundation for a braced bay.



SK 6/49 Typical arrangement of steel columns at braced bay.

Settlement computation table for Example 6.1

H	Z	Z/B	σ_z/q_n	σ_z	C_{kd}	h	p_o	C	S (mm)
1.0	0.5	0.167	0.8	82.8	35	0	27	190	7.38
1.0	1.5	0.5	0.52	53.82	35	0.5	40	129	6.61
1.0	2.5	0.833	0.35	36.22	35	1.5	48	107	5.25
1.0	3.5	1.167	0.22	22.77	65	2.5	56	170	2.00
1.0	4.5	1.5	0.16	16.56	65	3.5	64	149	1.54
1.0	5.5	1.8333	0.10	10.35	90	4.5	72	184	0.73
1.0	6.5	2.167	0.08	8.28	90	5.5	80	165	0.60
1.0	7.5	2.5	0.075	7.76	120	6.5	88	200	0.42

Loading table for Example 6.2

Columns	DL		IL		CLV		CLH		WL_1		WL_2	
	V	H	V	H	V	H	V	H	V	H	V	H
A1	+80	–	+40	–	–	–	∓200	±12	∓105	±9	∓50	±25
A2	+80	–	+40	–	–	–	–	–	∓105	±9	±50	±25
B1	+50	–	+20	–	+900	–	±200	±12	±105	±9	∓50	±25
B2	+50	–	+20	–	–	–	–	–	±105	±9	±50	±25

WL_1 = transverse wind WL_2 = longitudinal wind

600 mm × 300 mm wide ground beam to carry 230 mm brickwork 3 m high.
Suitable bearing stratum at 1200 mm below ground level.
Finished floor level is 150 mm above finished ground level.
Stiff to very stiff clay layer.

Step 1 ***Select type and depth of foundation***

Type: Reinforced concrete pad foundation – combined 2 sets of columns.
Length of 32 mm diameter HD bolts anchorage assembly = 400 mm.
The bottom of grout under base plate will be 500 mm below finished floor level.
It is easier for construction if the HD bolt is in the pedestal.

Height of pedestal = 1000 mm say
Thickness of pad = 650 mm assumed

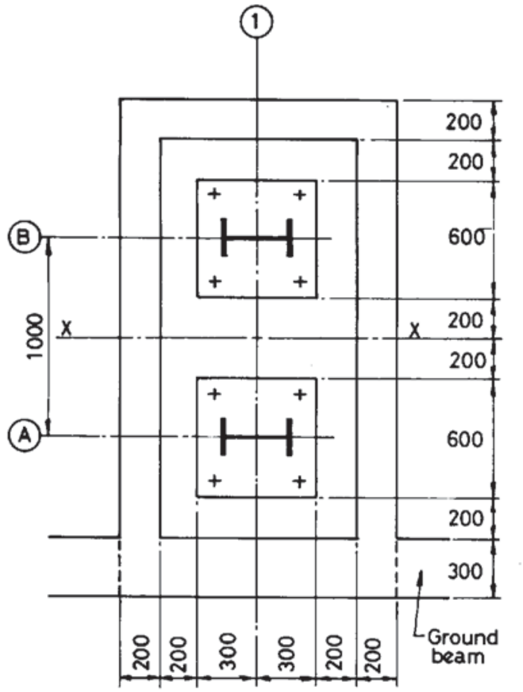
Underside of base at 1650 mm below floor level which is 1500 mm below finished ground level.

Step 2 ***Select approximate size***

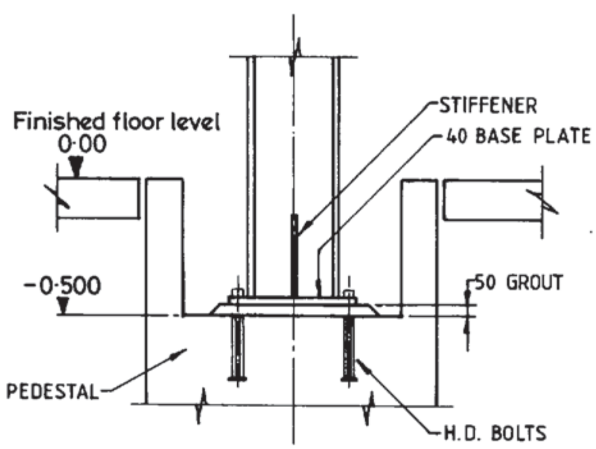
Size of pedestal = 1400 mm × 2500 mm

Presumed allowable bearing capacity from BS 8004: 1986[2] = 200 kN/m^2

Assume a projection of 1750 mm around the pedestals. The trial size



SK 6/50 Typical detail of column base and pedestal on plan.



SK 6/51 Typical section through pedestal column connection.

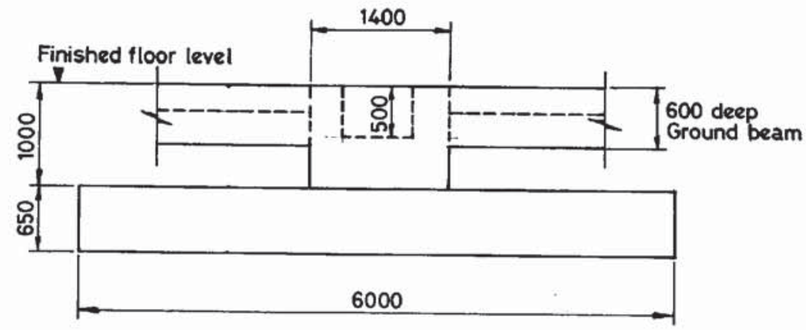
becomes 11000 × 6000. This is based on experience and may need some revision after all the calculations are carried out.
Determine minimum thickness of pad.

Ultimate vertical load on pad through one pedestal =

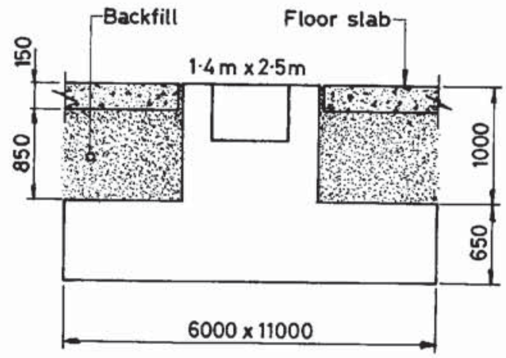
$N_u = 1.4 \times (80 + 80 + 50 + 50) + 1.6 \times (900)$
$= 1850\,\text{kN}$ including weight of pedestal

$U_o = 2(C_x + C_y) = 2 \times (1400 + 2500) = 7800\,\text{mm}$

$C_1 = U_o/6 = 1300\,\text{mm}$
$C_2 = N_u/v_c = 1850/0.4 = 4265\,\text{mm}^2$



SK 6/52 Elevation of pad foundation.



SK 6/53 Section through pad foundation.

$v_c = 0.4\,\text{N/mm}^2$ assumed for $f_{cu} = 30\,\text{N/mm}^2$

$d = 0.5[(C_1^2 + 4C_2)^{0.5} - C_1] = 3.5\,\text{mm}$

Punching shear will not be critical.

Step 3 ***Calculate bearing capacity of soil***

Note: This step may not be necessary if the soils investigation report includes the allowable bearing capacity calculations for different sizes of foundation.

From field and laboratory tests the following soil parameters of the bearing stratum are known.

γ = unit weight of soil = $19\,\text{kN/m}^3$

Ground water table = 2.0 m below ground level

$h = 0$ (ground water table below the level of foundation)

Purely cohesive bearing stratum.

$q_{ult} = cN_c + p$

p = overburden pressure = $\gamma D = 19 \times 1.5\,\text{m} = 28.5\,\text{kN/m}^2$

c = cohesive strength = $75\,\text{kN/m}^2$

$$D = 1.5\,\text{m} \qquad B = 6.0\,\text{m} \qquad \frac{D}{B} = 0.25$$

$N_c = 6.7$ from Table 6.2 in Section 6.5.1

$$q_{\text{ult}} = 75 \times 6.7 + 28.5 = 531\,\text{kN/m}^2$$

$$\text{Allowable bearing capacity} = \frac{q_{\text{ult}}}{3} = 177\,\text{kN/m}^2$$

Step 4 Calculate column load combinations

$$LC_1 = 1.0DL + 1.0IL + 1.0EP + 1.0CLV + 1.0CLH$$

Load case no.	Combination no.	Column no.	Vertical load, V (kN)	Ecc., e_x (m)	Ecc., e_y (m)	Ve_x (kNm)	Ve_y (kNm)	H_x (kN)	H_y (kN)
LC_1	1	A1	−80	−3.0	−0.5	+240	+40	—	+12
	$DL + IL + CLV$	A2	120	3.0	−0.5	+360	−60	—	—
	$+ CLH$	B1	1170	−3.0	+0.5	−3510	+585	—	+12
		B2	70	3.0	+0.5	+210	+35	—	—
	Totals		1280			−2700	+600	—	+24
LC_1	2	A1	320	−3.0	−0.5	−960	−160	—	−12
	$DL + IL + CLV$	A2	120	3.0	−0.5	+360	−60	—	—
	$- CLH$	B1	770	−3.0	0.5	−2310	+385	—	−12
		B2	70	3.0	0.5	+210	+35	—	—
	Totals		1280			−2700	+200	—	−24

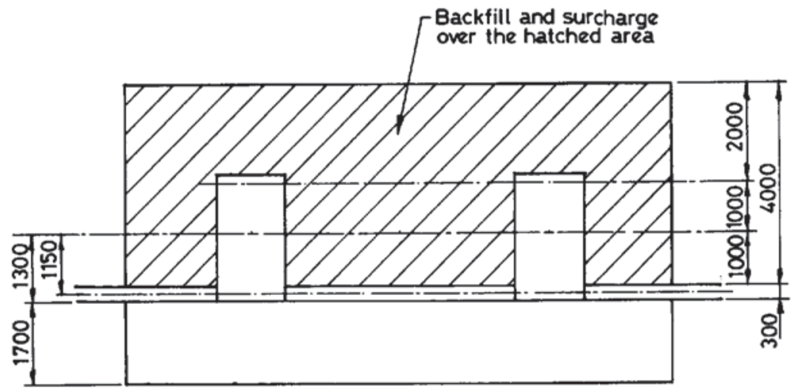
$LC_2 = 1.0DL + 1.0EP + 1.0CLV + 1.0CLH + 1.0WL_1$ (or $\pm\ 1.0WL_2$)
(total 8 possible combinations)

Load case no.	Combination no.	Column no.	Vertical load, V (kN)	Ecc., e_x (m)	Ecc., e_y (m)	Ve_x (kNm)	Ve_y (kNm)	H_x (kN)	H_y (kN)
LC_2	1	A1	−225	−3.0	−0.5	+675	+113	—	+21
	$DL + CLV$	A2	−25	3.0	−0.5	−75	+13	—	+9
	$+ CLH + WL_1$	B1	1255	−3.0	0.5	−3765	+628	—	+21
		B2	155	3.0	0.5	+465	+78	—	+9
	Totals		1160			−2700	+832	—	60
LC_2	2	A1	−70	−3.0	−0.5	+210	+35	−25	+12
	$DL + CLV$	A2	+30	3.0	−0.5	+90	−15	−25	
	$+ CLH - WL_2$	B1	1200	−3.0	0.5	−3600	+600	−25	+12
		B2	0	3.0	0.5	—	—	−25	
	Totals		1160			−3300	+620	−100	+24

Weight of backfill = $(6 \times 11 - 1.4 \times 2.5) \times 0.85\,\text{m} \times 18\,\text{kN/m}^3$
= 956 kN

Weight of ground slab (150 mm) = $(4 \times 11) \times 0.15 \times 25\,\text{kN/m}^3$
= 165 kN

Eccentricity, $e_y = +1.0\,\text{m}$



SK 6/55 Plan of foundation showing areas of superimposed loads.

Surcharge on ground slab @ $25\,\text{kN/m}^2 = 4 \times 11 \times 25 = 1100\,\text{kN}$
Eccentricity, $e_y = 1.0\,\text{m}$

Weight of ground beams + brickwork = $(12 - 2.8)\,\text{m} \times 0.3\,\text{m} \times 0.6\,\text{m} \times 25\,\text{kN/m}^3$ (ground beams) $+3\,\text{m} \times 0.23\,\text{m} \times 22\,\text{kN/m}^3$ (brickwork)
= 224 kN

Eccentricity, $e_y = -1.15\,\text{m}$

Differential settlement = 10 mm assumed (see Step 5)

$$M_y = \frac{6\delta EI}{l^2} = 126\,\text{kNm} \quad \text{for each beam}$$

$E = 14 \times 10^6\,\text{kN/m}^2$ long-term Young's modulus

Beam size = 300 mm × 600 mm

$$\text{Beam end reactions} = \frac{12\delta EI}{l^3} = \frac{2M_y}{l}$$
$$= \frac{252}{6} = 42\,\text{kN}$$

Moment on the foundation = $2 \times (-126) + (-42 \times 6) = -504\,\text{kNm}$

Note: There are many possible alternatives of the differential settlement. The worst, from the point of view of bearing pressure considering other loadings, is found by inspection.

Note: Most of the other combinations can be ignored by inspection. Also by inspection it is clear that load cases LC_3 and LC_4 will not produce more onerous design.

Step 5 ***Calculate approximate settlement***

(See Method 1 in Section 6.5.2.)

Soil parameter required from soil investigation report $= m_v$ m²/MN
Assume this is not available.
From Table 6.5 of Section 6.5.2, assume

$m_v = 0.15\,\mathrm{m^2/MN}$

Consolidation settlement $= m_v \sigma_z H$

q_n = average pressure $= 50\,\mathrm{kN/m^2}$ assumed

$\sigma_z = 0.55 q_n = 27.5\,\mathrm{kN/m^2}$

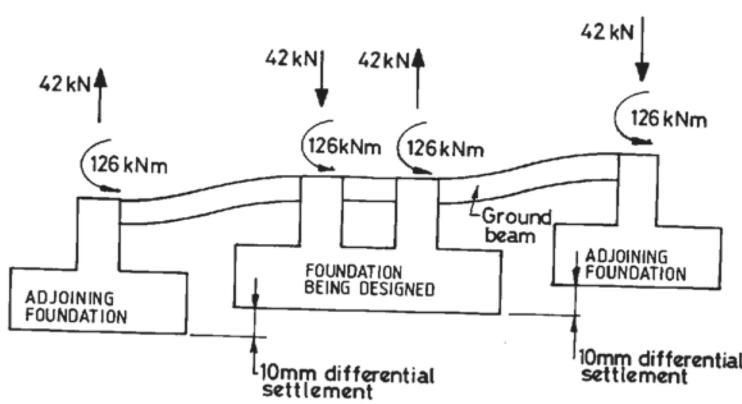
B = width of foundation = 6 m

$H = 1.5B = 9.0\,\mathrm{m}$

$$\text{Settlement} = m_v \sigma_z H = \frac{0.15}{1000} \times 27.5 \times 9.0 \times 1000 = 37\,\mathrm{mm}$$

Assume quarter of this predicted settlement as differential settlement. This is because the predicted maximum settlement is 40 mm, say, the minimum settlement is half (i.e. 20 mm) and the average settlement is 30 mm, so the average differential settlement is 10 mm (40 − 30).

Differential settlement = 10 mm



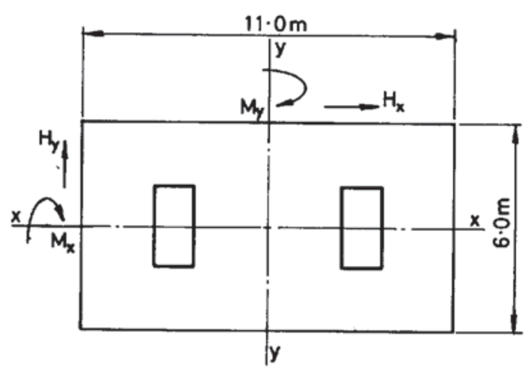
SK 6/54 Loads and moments due to differential settlement of pad foundation.

Step 6 ***Carry out analysis for bearing pressure***

Self-weight of foundation $= 2 \times 1.4 \times 2.5 \times 1.0 \times 25\,\mathrm{kN/m^3}$ (pedestal) +
$6 \times 11 \times 0.65 \times 25\,\mathrm{kN/m^3}$ (base)
$= 1248\,\mathrm{kN}$

Load case	Load type	Vertical, V(kN)	M_x (kNm)	M_y (kNm)	H_x (kN)	H_y (kN)
	Column vertical	1280	+600	−2700	–	–
	Column horizontal	–	+28	–	–	+24
	Foundation self-weight	1248	–	–	–	–
LC_1	Backfill	956	–	–	–	–
Combination 1	Ground slab	165	+165	–	–	–
	Surcharge on slab	1100	+1100	–	–	–
	Ground beam	224	−258	–	–	–
	Differential settlement	–	–	−504	–	–
LC_1	Totals	4973	+1635	−3204	–	+24

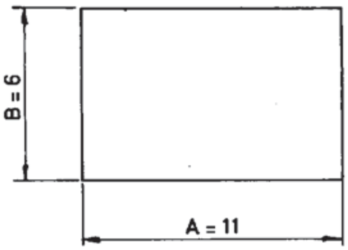
Load case	Load type	Vertical, V(kN)	M_x (kNm)	M_y (kNm)	H_x (kN)	H_y (kN)
	Column vertical	1160	+620	−3300	–	–
	Column horizontal	–	+28	−115	−100	+24
	Foundation self-weight	1248	–	–	–	–
LC_2	Backfill	956	–	–	–	–
Combination 2	Ground slab	165	+165	–	–	–
	Surcharge on slab	1100	+1100	–	–	–
	Ground beam	224	−258	–	–	–
	Differential settlement	–	–	−504	–	–
LC_2	Totals	4853	+1655	−3919	−100	+24



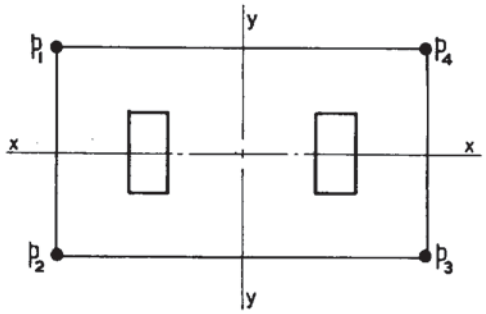
SK 6/56 Sign convention of positive forces on foundation.

Note: The foundations are connected by ground beams in the x-direction. The horizontal force H_x may be distributed equally among all connected foundations. For the sake of conservatism this has not been done.

Step 7 Calculate bearing pressures



SK 6/57 Footprint of foundation.



SK 6/58 Bearing pressure locations.

$$LC_1:\quad p = \left(\frac{p}{AB}\right) \pm \left(\frac{6M_x}{AB^2}\right) \pm \left(\frac{6M_y}{A^2B}\right)$$

$$p_1 = \left(\frac{4973}{6 \times 11}\right) + \left(\frac{6 \times 1635}{11 \times 6^2}\right) + \left(\frac{6 \times 3204}{11^2 \times 6}\right)$$

$$= 75 + 25 + 26$$

$$= 126\,\text{kN/m}^2 < 177\,\text{kN/m}^2 \qquad \text{(see Step 3)}$$

Similarly,

$$p_2 = 75 - 25 + 26 = 76\,\text{kN/m}^2$$
$$p_3 = 75 - 25 - 26 = 24\,\text{kN/m}^2$$
$$p_4 = 75 + 25 - 26 = 74\,\text{kN/m}^2$$

$$LC_2:\quad p_1 = 74 + 25 + 32 = 131\,\text{kN/m}^2$$
$$p_2 = 81\,\text{kN/m}^2$$
$$p_3 = 17\,\text{kN/m}^2$$
$$p_4 = 67\,\text{kN/m}^2$$

All bearing pressures are within allowable limit.

Step 8 Calculate sliding resistance of foundation

Check sliding between concrete and soil: ignore passive resistance.
Assume very aggressive soil and membrane tanking is used.

Assume $\delta = 5°$ $\quad \tan\delta = 0.09 \quad H_x = 100\,\text{kN} \quad H_y = 24\,\text{kN}$

Load case LC_2

Frictional resistance $= P\tan\delta = 4853 \times 0.09 = 437\,\text{kN} > H_x$ and H_y

$$\text{Factor of safety against sliding} = \frac{437}{\sqrt{(H_x^2 + H_y^2)}} = \frac{437}{\sqrt{(100^2 + 24^2)}}$$

$$= 4.2 \quad \text{OK}$$

Check horizontal bearing capacity of soil:
For cohesive soil,
$P_{Hu} = cA = 75 \times 6 \times 11 = 4950\,\text{kN}$

$$P_{Hx} = P_{Hy} = \frac{P_{Hu}}{1.5} = 3300\,\text{kN}$$

Step 9 Check combined sliding and bearing

Load case LC_2

$P = 4853\,\text{kN} \qquad H_x = 100\,\text{kN} \qquad H_y = 24\,\text{kN}$

P_v = allowable bearing capacity × area = 177 × 66 = 11 682 kN

$$\frac{P}{P_v} + \frac{H_x}{P_{Hx}} + \frac{H_y}{P_{Hy}} = \frac{4853}{11\,682} + \frac{100}{3300} + \frac{24}{3300} = 0.45 < 1 \quad \text{okay}$$

Step 10 Carry out analysis of bearing pressure for bending moment and shear

Ultimate column load combinations

Load case no.	Combination no.	Column no.	Vertical load, V(kN)	Ecc., e_x (m)	Ecc., e_y (m)	Ve_x (kNm)	Ve_y (kNm)	H_x (kN)	H_y (kN)
LC_9		A1	−168	−3.0	−0.5	+504	+84	–	+16.8
	1.4DL	A2	+112	3.0	−0.5	+336	−56	–	–
	+1.4CLV	B1	+1610	−3.0	+0.5	−4830	+805	–	+16.8
	+1.4CLH	B2	+70	3.0	+0.5	+210	+35	–	–
	Totals		+1624			−3780	+868	–	+33.6

Ultimate loads on foundation

Load case	Combination	Load type	Vertical, V (kN)	M_x (kNm)	M_y (kNm)	H_x (kN)	H_y (kN)
		Column vertical	1624	+868	−3780	–	–
		Column horizontal	–	+38.6	–	–	33.6
	1.4DL	Foundation self-weight	1747				
	+1.4CLV	Backfill	1338				
LC_9	+1.4CLH	Ground slab	231	+231			
		Ground beam	314	−361			
		Differential settlement			−706		
		Totals	5254	+776.6	−4486	–	33.6

Note: It is most important that a complicated loading system on a multiple column foundation should be investigated in a structured manner using tables as shown. Otherwise mistakes will creep in.

Only one load case has been analysed to show the method. All other load cases should be similarly investigated.

Step 11 Calculate bearing pressure for bending moment and shear

Load case LC_9

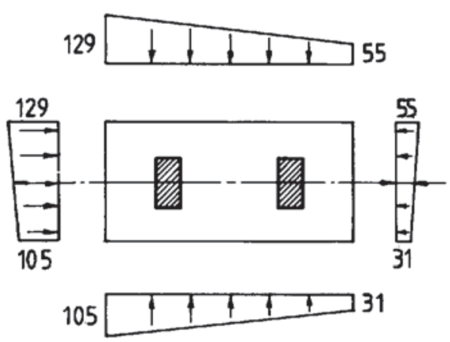
$$p_1 = \left(\frac{P}{AB}\right) + \left(\frac{6M_x}{AB^2}\right) + \left(\frac{6M_y}{A^2B}\right)$$

$$= \left(\frac{5254}{66}\right) + \left(\frac{6 \times 776.6}{11 \times 6^2}\right) + \left(\frac{6 \times 4486}{11^2 \times 6}\right)$$

$$= 80 + 12 + 37 = 129\,\text{kN/m}^2$$

$$p_2 = 80 - 12 + 37 = 105\,\text{kN/m}^2$$
$$p_3 = 80 - 12 - 37 = 31\,\text{kN/m}^2$$
$$p_4 = 80 + 12 - 37 = 55\,\text{kN/m}^2$$

Note: Pressures for load case LC_9 only have been calculated to show the method. In an actual design, other load cases should also be investigated.



SK 6/59 Bearing pressure diagrams for load case LC_9.

Step 12 Calculate bending moments and shears in pad

Load case LC_9

Find pressures at critical sections 1, 2, 3, 4, 5, 6, 7 as shown. They are p_1, p_2, p_3, p_4, etc.

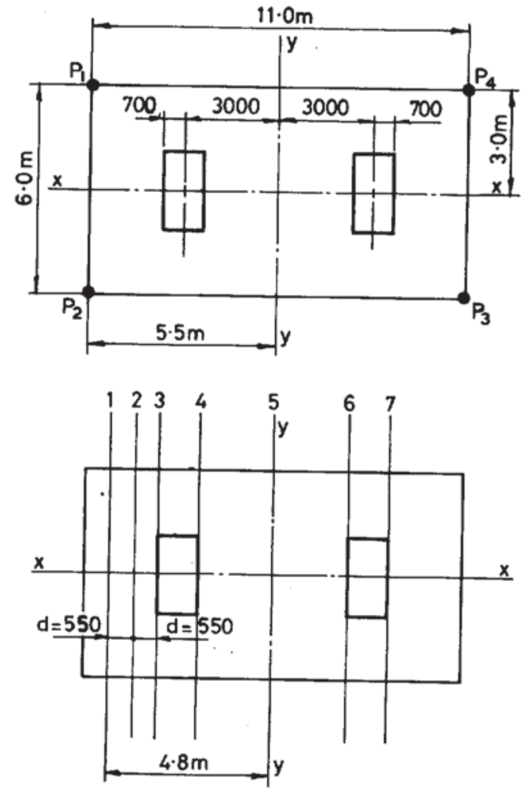
Average pressures at section 1 = p_1
Assume d = 550 mm
Distance from $y-y$ axis = 4.8 m = x_1

$$p_{1,\text{av}} = \left(\frac{P}{AB}\right) + \left(\frac{12M_yx_1}{A^3B}\right)$$

$$= 80 + \frac{12 \times 4486 \times 4.8}{11^3 \times 6}$$

$$= 112\,\text{kN/m}^2$$

Similarly,



SK 6/60 Bearing pressure locations on plan and critical sections for bending moment and shear.

$p_{2,\mathrm{av}} = 109\,\mathrm{kN/m^2}$
$p_{3,\mathrm{av}} = 105\,\mathrm{kN/m^2}$
$p_{4,\mathrm{av}} = 96\,\mathrm{kN/m^2}$
$p_{5,\mathrm{av}} = 80\,\mathrm{kN/m^2}$
$p_{6,\mathrm{av}} = 64\,\mathrm{kN/m^2}$
$p_{7,\mathrm{av}} = 55\,\mathrm{kN/m^2}$

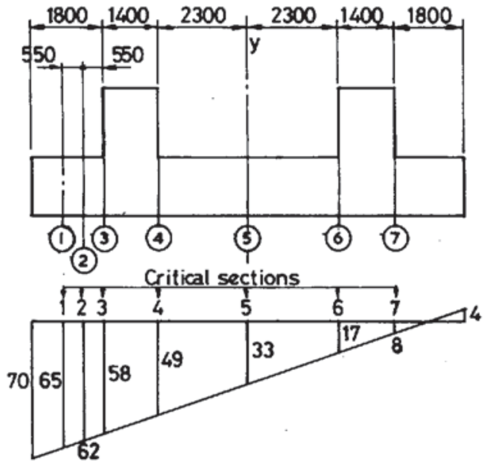
Average pressures at edges are $117\,\mathrm{kN/m^2}$ and $43\,\mathrm{kN/m^2}$.

p_d = downward load on pad (ultimate)
= self-weight of foundation + backfill
$= (0.65\,\mathrm{m} \times 25\,\mathrm{kN/m^3} + 0.85\,\mathrm{m} \times 18\,\mathrm{kN/m^3}) \times 1.4$
$= 44\,\mathrm{kN/m^2}$ uniform excluding ground slab

Equivalent weight of ground slab acting on half the foundation width
$= 0.5 \times 0.15\,\mathrm{m} \times 25\,\mathrm{kN/m^3} \times 1.4$
$= 3\,\mathrm{kN/m^2}$

Total downward load $= 47\,\mathrm{kN/m^2}$

Draw net pressure diagram.



SK 6/61 Net bearing pressure diagram to find bending moments and shears – load case LC_9.

$$\text{Bending moment at section 3} = 6 \times \left[\frac{58 \times 1.8^2}{2} + \frac{(70 - 58)1.8^2}{3}\right]$$
$$= 642\,\text{kNm}$$

LC_9: Column loads on $A_1 + B_1 = (1610 - 168) = 1442\,\text{kN}$ (factored)

Weight of pedestal $= 1.4 \times 1.4\,\text{m} \times 2.5\,\text{m} \times 1.0\,\text{m} \times 25\,\text{kN/m}^3 =$ $123\,\text{kN}$ (factored)

$$\text{Bending moment at section 5} = 6\,\text{m} \times \left[\frac{33 \times 5.5^2}{2} + \frac{(70 - 33) \times 5.5^2}{3}\right]$$
$$- (1442 + 123) \times 3\,\text{m}$$
$$= 538\,\text{kNm} \quad \text{(no top tension)}$$

$$\text{Bending moment at section 7} = 6\,\text{m} \times \left(\frac{8 \times 1.8^2}{2} - \frac{12 \times 1.8^2}{3}\right)$$
$$= 0\,\text{kNm}$$

$$\text{Shear at section 1} = 6\,\text{m} \times \left[(70 + 65) \times \frac{0.7}{2}\right]$$
$$= 284\,\text{kN}$$

$$\text{Shear at section 2} = 6\,\text{m} \times \left[(62 + 70) \times \frac{1.25}{2}\right]$$
$$= 495\,\text{kN}$$

$$\text{Shear at section 3} = 6\,\text{m} \times \left[(70 + 58) \times \frac{1.8}{2}\right]$$
$$= 691\,\text{kN}$$

$$\text{Shear at section 4} = 6\,\text{m} \times \left[(70 + 49) \times \frac{3.2}{2}\right] - (1442 + 123)$$
$$= -423\,\text{kN}$$

Note: It is useful to draw the bending moment diagram for the load case. Similarly bending moments and shears should be calculated for all load cases and all critical sections parallel to the $x-x$ and $y-y$ axes following the recommendations in Step 12 of Section 6.7.

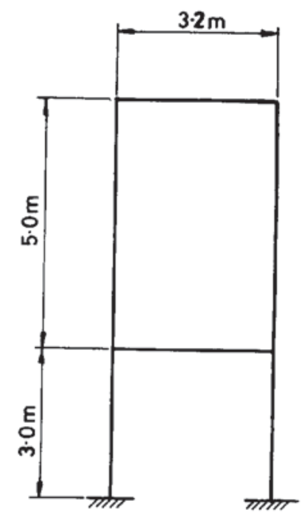
Step 13 ***Determine cover to reinforcement***
Follow Step 13 of Section 6.7.

Step 14 ***Calculate area of tension reinforcement and distribution***
Follow Step 14 of Section 6.7.

Step 15 ***to Step 23***
Similar to Example 6.1.

Note: The numerous other checks required in Step 15 to Step 23 in this example are not shown for brevity. They have already been shown in Example 6.1.

Example 6.3 ***Mass concrete pad – side bearing in cohesive soils***
Foundation for roadside signpost.



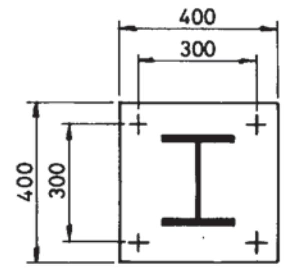
SK 6/62 Roadside signpost.

Vertical load = 18 kN
Horizontal wind shear = 4.5 kN
Bending moment due to wind = 25 kNm

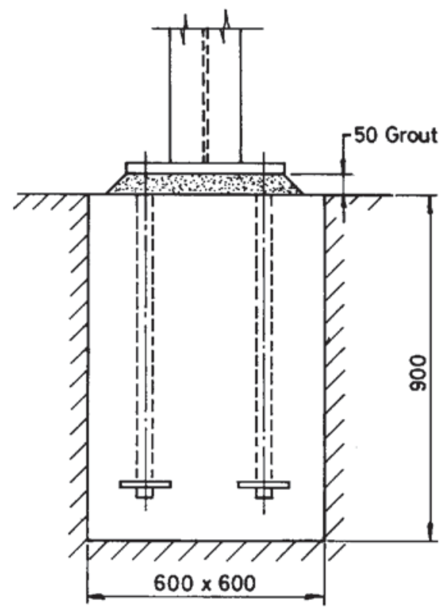
Size of columns = 203 × 203 × 46 kg/m UC
Size of base plate = 400 × 400

Size of foundation bolts = 4 no. M24
Bolt spacing = 300 mm
Soil condition is stiff to very stiff clay.

Step 1 ***Select type and depth of foundation***



SK 6/63 Details of column base plate.



SK 6/64 First trial of a side bearing mass concrete foundation.

Step 2 ***Select approximate size***

Assume 100 mm projection on all sides of base plate.
The foundation size selected is $600 \times 600 \times 900$ deep.

Note: This foundation will be designed as side bearing for the horizontal load and the applied moment. The vertical load will be carried by the direct bearing on the soil. It is very important that this foundation is cast against the soil.

Step 3 ***Calculate bearing capacity of soil***

γ = unit weight of soil = $19\,\text{kN/m}^3$

p = total overburden pressure at foundation level
$= \gamma D = 19 \times 0.9 = 17.1\,\text{kN/m}^2$

h = height of water above foundation level = 0 m

$p_o = p = 17.1\,\text{kN/m}^2$

c = minimum undrained soil cohesion = $150\,\text{kN/m}^2$

$q_{\text{ult}} = c\,N_c + p$

$$\frac{D}{B} = \frac{0.9}{0.6} = 1.5$$

$N_c = 9.1$ from Table 6.2 in Section 6.5.1

$q_{ult} = 9.1 \times 150 + 17.1 = 1382.1\,\text{kN/m}^2$

Allowable vertical bearing capacity $= \dfrac{q_{ult}}{3} = 460\,\text{kN/m}^2$

Maximum horizontal bearing capacity is the ultimate passive resistance of soil given by the following equation:

$$\begin{aligned} q_h &= p_o \tan^2(45° + \phi/2) + 2c\tan(45° + \phi/2) \\ &= 2c \quad \text{at ground level for } \phi = 0° \text{ and } p_o = 0 \\ &= 300\,\text{kN/m}^2 \end{aligned}$$

Step 4 ***Calculate column load combination***
(See Section 6.3.)
For bearing pressure calculations:

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

$N = 18\,\text{kN} \quad H_x = 4.5\,\text{kN} \quad M_x = 25\,\text{kNm} \quad M_y = 0$

For bending moment and shear calculations:

$LC_7 = 1.4DL + 1.4WL$

$N = 25.2\,\text{kN} \quad H_x = 6.3\,\text{kN} \quad M_x = 35\,\text{kNm}$

Step 5 ***Calculate approximate settlement***
This step can be ignored.

Step 6 ***Carry out analysis for bearing pressure***

Self-weight of foundation $= 0.6\,\text{m} \times 0.6\,\text{m} \times 0.9\,\text{m} \times 25\,\text{kN/m}^3 = 8.1\,\text{kN}$

$P = 18 + 8.1 = 26.1\,\text{kN} \quad H_x = 4.5\,\text{kN} \quad M_x = 25\,\text{kNm}$

Step 7 ***Calculate bearing pressures***
Soil is stiff to very stiff clay.
Determine horizontal modulus of subgrade reaction.

Assume $k_{si} = 14\,\text{MN/m}^3$ (see Table 6.7 in Section 6.5.1)

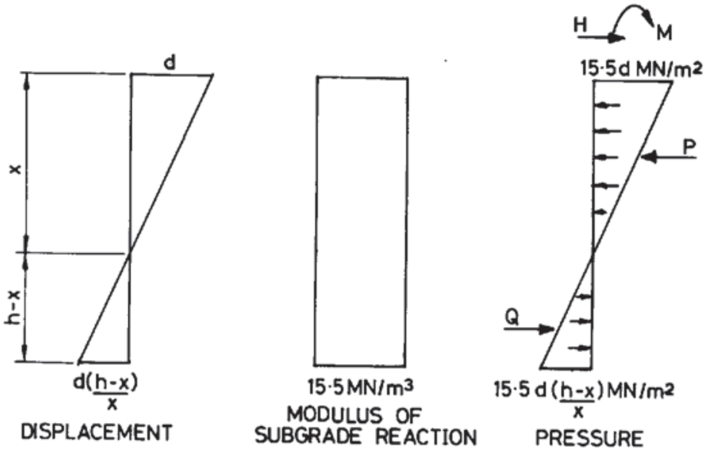
B = width of foundation = 0.6 m

$$K_h = \frac{k_{si}}{1.5B} = \frac{14}{1.5 \times 0.6} = 15.5\,\text{MN/m}^3$$

Note: Horizontal load and vertical load are treated separately to find the bearing pressures. It is assumed that the vertical load will be carried uniformly on the base; size 600 mm × 600 mm. The horizontal load and moment will be carried by side bearing in the manner shown.

Assumptions:

(1) Foundation block is fully rigid.



SK 6/65 Displacement and pressure diagrams of side-bearing foundation in cohesive soil.

(2) Displacement d at top of foundation when subjected to bending moment M_x and horizontal shear H_x.
(3) Conservatively, that there is no net horizontal movement of whole foundation block. Moment and horizontal force are resisted by rotation only. The pressure diagram will be as shown if x is depth from ground level of point of rotation.
(4) Neglect friction on sides of foundation block.
(5) Neglect contribution from non-uniform pressure distribution on bottom surface of foundation.

$$P = 0.5K_hBxd = 4.65dx\,\text{MN}$$

$$Q = 0.5K_hB(h - x)^2 \frac{d}{x} = 4.65(h - x)^2 \frac{d}{x}\text{MN}$$

$h = 0.9\,\text{m}$

H = applied horizontal load = 4.5 kN = 0.0045 MN

Considering horizontal load equilibrium:

$P = H + Q$ or $P - Q = H$

or $$4.65d\left[x - \frac{(0.9 - x)^2}{x}\right] = 4.5 \times 10^{-3}$$

Taking moment about the foundation level:

$$P\left(h - \frac{x}{3}\right) - \frac{Q(h - x)}{3} - M - Hh = 0$$

or $$4.65dx\left(h - \frac{x}{3}\right) - \frac{4.65d(h - x)^3}{3x} = 25 \times 10^{-3} + 4.5 \times 10^{-3} \times 0.9$$

or $4.65d\left[x\left(0.9 - \frac{x}{3}\right) - \frac{(0.9 - x)^3}{3x}\right] = 29.05 \times 10^{-3}$

Solving for the two unknowns d and x using a computer-assisted equation solver.

Displacement $= d = 22\,\text{mm}$
Point of rotation, $x = 460\,\text{mm}$

Maximum allowable shear stress in mass concrete $= 0.037f_{cu} = 0.925\,\text{N/mm}^2$

$$P = 4.65dx = 47.3\,\text{kN}$$

$$Q = 4.65(h - x)^2\left(\frac{d}{x}\right) = 42.8\,\text{kN}$$

Maximum horizontal pressure on the soil $= 15.5d\,\text{MN/m}^2$

$$= 15.5 \times 1000 \times \frac{22}{1000}\,\text{kN/m}^2$$

$$= 341\,\text{kN/m}^2$$

This pressure is higher than the unconfined compressive strength of soil, which is $300\,\text{kN/m}^2$. To prevent local heave of soil, revise size of foundation to 900 mm × 900 mm × 1300 mm.

$$K_h = \frac{k_{si}}{1.5B} = \frac{14}{1.5 \times 0.9} = 10.4\,\text{MN/m}^3$$

$$P = 0.5K_hBxd$$

$$Q = 0.5K_hB(h - x)^2\,\frac{d}{x}$$

$$P - Q = H$$

$h = 1.3\,\text{m}$ $\quad M = 25\,\text{kNm}$ $\quad H = 4.5\,\text{kN}$ $\quad B = 0.9\,\text{m}$

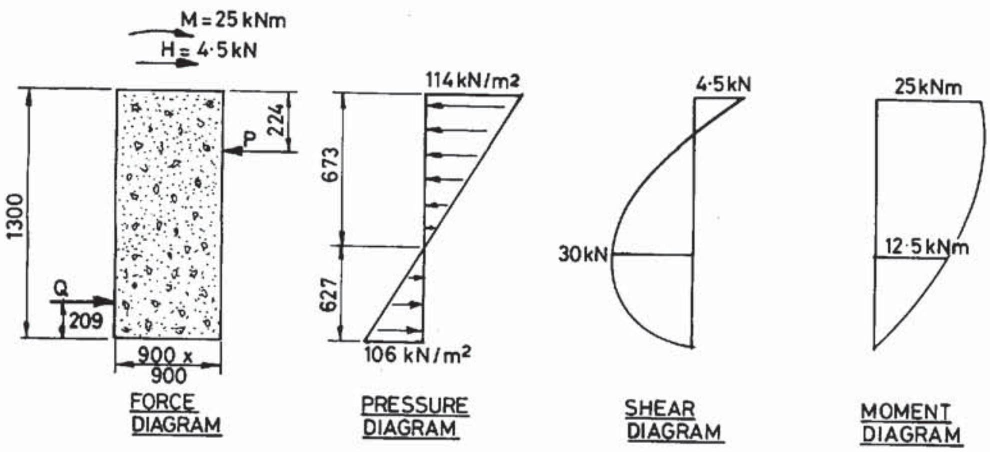
By solving the above equations,

$P = 34.5\,\text{kN}$
$d = 11.0\,\text{mm}$
$x = 673\,\text{mm}$
$Q = 30.0\,\text{kN}$
$p = 114\,\text{kN/m}^2$ maximum pressure $< 300\,\text{kN/m}^2$ ultimate pressure

Factor of safety on ultimate $= \frac{300}{114} = 2.63$

Note: The displacement of 11 mm at ground level will mean 130 mm displacement at top of an 8000 mm high structure. This should be checked for clearances or other obstructions.

Revised weight of foundation = 26.3 kN



SK 6/66 Diagrams at serviceability limit state.

$$\text{Vertical Bearing Pressure} = \frac{P}{A} = \frac{44.3}{0.9 \times 0.9} = 54.7\,\text{kN/m}^2 < 460\,\text{kN/m}^2$$

Step 8 ***Calculate sliding resistance***
Not required.

Step 9 ***Check combined sliding and bearing***
Not required.

Step 10 ***Carry out analysis of bearing pressure for bending moment and shear***
Not required.

Step 11 ***to Step 21***
Not required.

Step 22 ***Design mass concrete foundation***
Use C25 concrete $f_{cu} = 25\,\text{N/mm}^2$

Maximum allowable tension in bending $= 0.37\sqrt{f_{cu}} = 0.37 \times (25)^{\frac{1}{2}} =$ $1.85\,\text{N/mm}^2$

Maximum bending moment = 25 kNm approximately

Maximum shear = 30.0 kN

Section modulus, $Z = \left(\frac{1}{6}\right) \times 900^3 = 1.215 \times 10^8\,\text{mm}^3$

$$\text{Bending tensile stress in mass concrete} = \frac{M}{Z} = \frac{25 \times 10^6}{1.215 \times 10^8} = 0.20\,\text{N/mm}^2$$

Allowable bending tensile stress $= 1.85\,\text{N/mm}^2$ OK

$$\text{Shear stress} = \frac{30 \times 10^3}{900 \times 900} = 0.037\,\text{N/mm}^2 < 0.925\,\text{N/mm}^2 \quad \text{OK}$$

Example 6.4 ***Mass concrete pad – side bearing in cohesionless soils***
Same loading and example as in Example 6.3. Soil is dense sandy gravel with $\phi = 35°$.

Step 1 ***Select type and depth of foundation***
Similar to Step 1 in Example 6.3.

Step 2 ***Select approximate size***
Choose $1000 \times 1000 \times 1500$ deep foundation.

Note: In cohesionless soil the size of the foundation will be larger than in cohesive soil because the allowable horizontal bearing capacity at higher levels is lower. It will be very difficult to cast this foundation against the soil because the sides of the excavation may not stay vertical. It will be necessary to have well compacted granular backfill using mechanical compactors.

Step 3 ***Calculate bearing capacity of soil***

$B = 1\,\text{m}$

γ = unit weight of soil = $18\,\text{kN/m}^3$

p = total overburden pressure at foundation level = $18 \times 1.5\,\text{m} = 27\,\text{kN/m}^2$

$p_o = p - \gamma_w h = 27\,\text{kN/m}^2 \quad \text{as } h = 0$

$c = 0$

ϕ = angle of internal friction = $35°$

$a = e^{(0.75\pi - \phi/2)\tan\phi} = 4.20$

$$N_q = \frac{a^2}{2\cos^2\left(45° + \frac{\phi}{2}\right)} = 41.4$$

$K_{py} = 82.0 \quad$ from Table 6.1 in Section 6.5.1

$$N_\gamma = 0.5\tan\phi\left(\frac{K_{py}}{\cos^2\phi} - 1\right) = 42.4$$

$$\begin{aligned} q_{ult} &= 1.3cN_c + p_o(N_q - 1) + 0.4\gamma BN_\gamma + p \\ &= 27\,(41.4 - 1) + 0.4 \times 18 \times 1 \times 42.4 + 27 \\ &= 1423\,\text{kN/m}^2 \end{aligned}$$

$$\text{Allowable vertical bearing capacity} = \frac{q_{ult}}{3} = 474\,\text{kN/m}^2$$

Maximum horizontal bearing capacity is ultimate passive resistance of soil given by the following equation:

$$q_h = p_o \tan^2\left(45° + \frac{\phi}{2}\right) + 2c \tan\left(45° + \frac{\phi}{2}\right) = \gamma h \tan^2\left(45° + \frac{\phi}{2}\right)$$
$$= 66h\,\mathrm{kN/m^2}$$

Step 4 ***Calculate column load combination***
See Step 4 of Example 6.3.

Step 5 ***Calculate approximate settlement***
Can be ignored.

Step 6 ***Carry out analysis for bearing pressure***
Self-weight of foundation $= 1 \times 1 \times 1.5 \times 25\,\mathrm{kN/m^3} = 37.5\,\mathrm{kN}$

$$P = 37.5 + 18 = 55.5\,\mathrm{kN}$$

$$H_x = 4.5\,\mathrm{kN} \qquad M_x = 25\,\mathrm{kNm}$$

Step 7 ***Calculate bearing pressures***
Assumptions:

(1) Foundation is rigid.
(2) Foundation carries part of moment and total horizontal shear by side bearing up to ultimate horizontal bearing pressure of $66Z\ \mathrm{kN/m^2}$ (see Step 3).
(3) Residual moment is carried by bottom surface of foundation as a variable pressure on surface. Factor of safety against overturning will be 1.5 or more.
(4) Rotational deformation of foundation is proportional to distance from point of rotation.
(5) There is no net horizontal movement.
(6) Point of rotation is assumed at depth x from ground level.

Modulus of subgrade reaction $= K_h = n_h \dfrac{Z}{B}$

$B = 1.0\,\mathrm{m}$
$n_h = 6.6\,\mathrm{MN/m^3}$ assumed

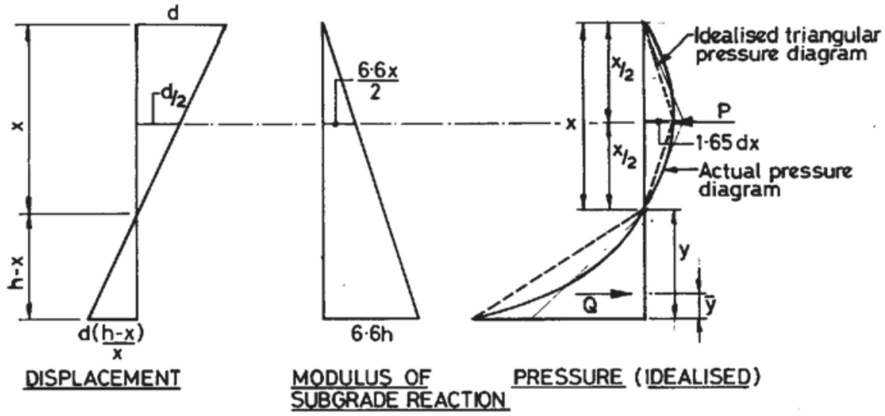
(See Table 6.7 in Section 6.5.3.)

$K_h = 6.6Z\,\mathrm{MN/m^3}$

Z = depth from ground level (metres)

$P = 1.65dx^2 \dfrac{B}{2}$ for idealised triangular pressure distribution

$Q = 6.6hdB \dfrac{(h - x)^2}{2x}$ for idealised triangular pressure distribution



SK 6/67 Displacement and pressure diagrams of side-bearing foundation in cohesionless soil.

$P = Q + H$

$M + Hh + Q\left(h - \frac{x}{3}\right) - P\left(h - \frac{x}{2}\right) = 0$ for idealised triangular distribution

$H = 4.5\,\text{kN}$

Maximum pressure $= p = 1.65dx$ from pressure diagram

$$\text{also } p = \frac{66x}{2000}\ \text{MN/m}^2 \quad \text{(from Step 3)}$$

Equating the two gives

$$1.65dx = \frac{66x}{2000}$$

$d = 20\,\text{mm}$

Ultimate horizontal passive pressure on side of foundation is reached at a depth $x/2$ when horizontal deformation at top reaches 20 mm. It is assumed that moment-carrying capacity of foundation through side bearing will have a limiting value when deformation reaches 20 mm at top.

Find M when
$h = 1.5\,\text{m}$
$d = 20 \times 10^{-3}\,\text{m}$
$H = 4.5 \times 10^{-3}\,\text{MN}$
$B = 1.0\,\text{m}$

or $P = 0.0165x^2$ $\quad Q = 0.099(1.5 - x)^2/x$ $\quad P = Q + H$

and $M = P(1.5 - 0.5x) - \dfrac{Q(1.5 - x)}{3} - Hh$ (MNm)

Solving the equations:

$P = 19.4 \times 10^{-3}\,\text{MN}$
$x = 1.092\,\text{m}$
$Q = 14.9 \times 10^{-3}\,\text{MN}$
$M = 9.7\,\text{kNm}$ (maximum allowed by side bearing)

Residual moment $= 25 - 9.7 = 15.3\,\text{kNm}$

By rigorous analysis:
d' = displacement at bottom
d = displacement at top
X = depth of neutral axis
Y = bottom of foundation to neutral axis
$\bar{Y}$ = point of application of Q from bottom of foundation

$$P = \int_0^X \frac{(X - x)}{X}\, dB6.6x\, \mathrm{d}x = 6.6\, Bd\left(\frac{X^2}{6}\right) = 1.1\, BdX^2$$

$$Q = 6.6d'B\int_0^Y \frac{y(X + y)}{Y}\,\mathrm{d}y = 6.6d'B\left[\left(\frac{XY}{2}\right) + \frac{Y^2}{3}\right]$$
$$= 1.1d'B(3XY + 2Y^2)$$

$$X + Y = 1.5 \qquad \frac{X}{d} = \frac{Y}{d'}$$

$$M + 4.5 \times 1.5 + Q\bar{Y} - P\left(h - \frac{X}{2}\right) = 0$$

$$\bar{Y} = Y - \frac{\int_0^Y y^2(X + y)\,\mathrm{d}y}{\int_0^Y y(X + y)\mathrm{d}y}$$
$$= Y\left[1 - \frac{(X/3 + Y/4)}{(X/2 + Y/3)}\right]$$

Solving the above equations:

$X = 1.0317\,\text{m}$ $\quad Y = 0.4683\,\text{m}$

$d' = 8.9\,\text{mm}$ $\quad$ pressure $= 88.1\,\text{kN/m}^2$
$d = 19.7\,\text{mm}$

$Q = 18.57\,\text{kN}$ $\quad P = 23.07\,\text{kN}$

$M = 13.25\,\text{kNm}$
$\bar{Y} = 0.314Y$

This gives a higher value of M and hence is less conservative.

Vertical load on foundation $= P = 55.5\,\text{kN}$ (see Step 6)

$$e = \frac{M}{P} = \frac{15.3}{55.5} = 0.276\,\text{m} > \frac{A}{6} = 0.167\,\text{m}$$

(See Section 6.6.2.)

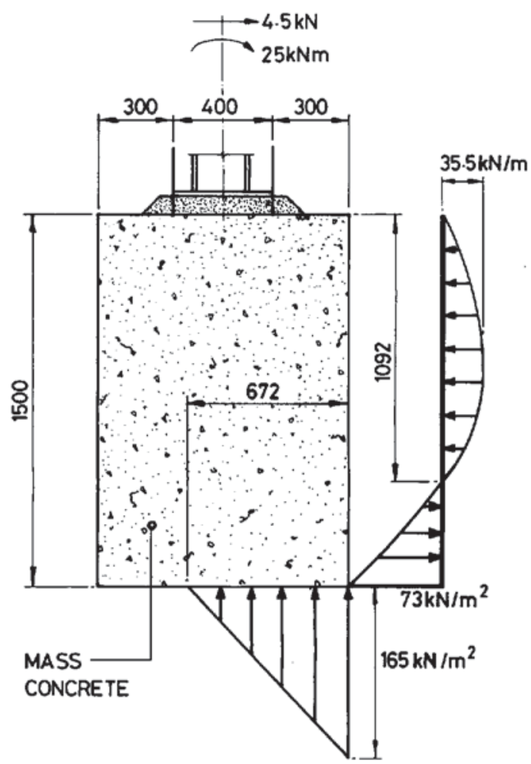
$$P_1 = \frac{2P}{(1.5A - 3e)B}$$
$$= \frac{2 \times 55.5}{(1.5 \times 1 - 3 \times 0.276) \times 1}$$
$$= 165\,\text{kN/m}^2 < 474\,\text{kN/m}^2 \quad \text{(see Step 3)} \quad \text{OK}$$

$x = 1.5A - 3e = 0.672\,\text{m}$

$$\text{Restraining moment} = 55.5 \times \frac{A}{2} = 55.5 \times 0.5$$
$$= 27.75\,\text{kNm}$$

Overturning moment = 15.3 kNm

$$\text{Factor of safety against overturning} = \frac{27.75}{15.3} = 1.8 > 1.5 \quad \text{OK}$$



SK 6/68 Soil pressure diagram on mass concrete side-bearing foundation in cohesionless soil.

Note: This is a very conservative estimate of factor of safety against overturning because in practice the value of moment resisted by side bearing will not be restricted to 9.7 kNm but will increase till the pressure diagram becomes rectangular and not triangular as assumed in the analysis.

Maximum pressure at $x/2$ from ground level $= 1.65dx\,\mathrm{MN/m^2}$
$= 1.65 \times 20 \times 10^{-3} \times 1.092 \times 10^3\,\mathrm{kN/m^2}$
$= 36\,\mathrm{kN/m^2}$

Maximum passive pressure that can be generated at $x/2 = 66 \times 1.092/2 = 36\,\mathrm{kN/m^2}$

Step 8 to Step 21
Not required.

Step 22 Design mass concrete foundation
Use C25 mass concrete $f_{cu} = 25\,\mathrm{N/mm^2}$

Bending about vertical plane:
follow same principle as in Example 6.3.

Bending about horizontal plane:

Overhang = 200 mm

Maximum shear assuming uniform pressure of $165\,\mathrm{kN/m^2}$ less weight of foundation $= 1.5 \times 25 = 37.5\,\mathrm{kN/m^2}$

Net pressure upwards $= 165 - 37.5 = 127.5\,\mathrm{kN/m^2}$

Shear $= 127.5 \times 0.3 \times 1.0 = 38.2\,\mathrm{kN}$

Shear stress $= \dfrac{38.2 \times 10^3}{1500 \times 1000} = 0.025\,\mathrm{N/mm^2}$ negligible OK

Bending stress need not be checked.

6.9 FIGURES FOR CHAPTER 6

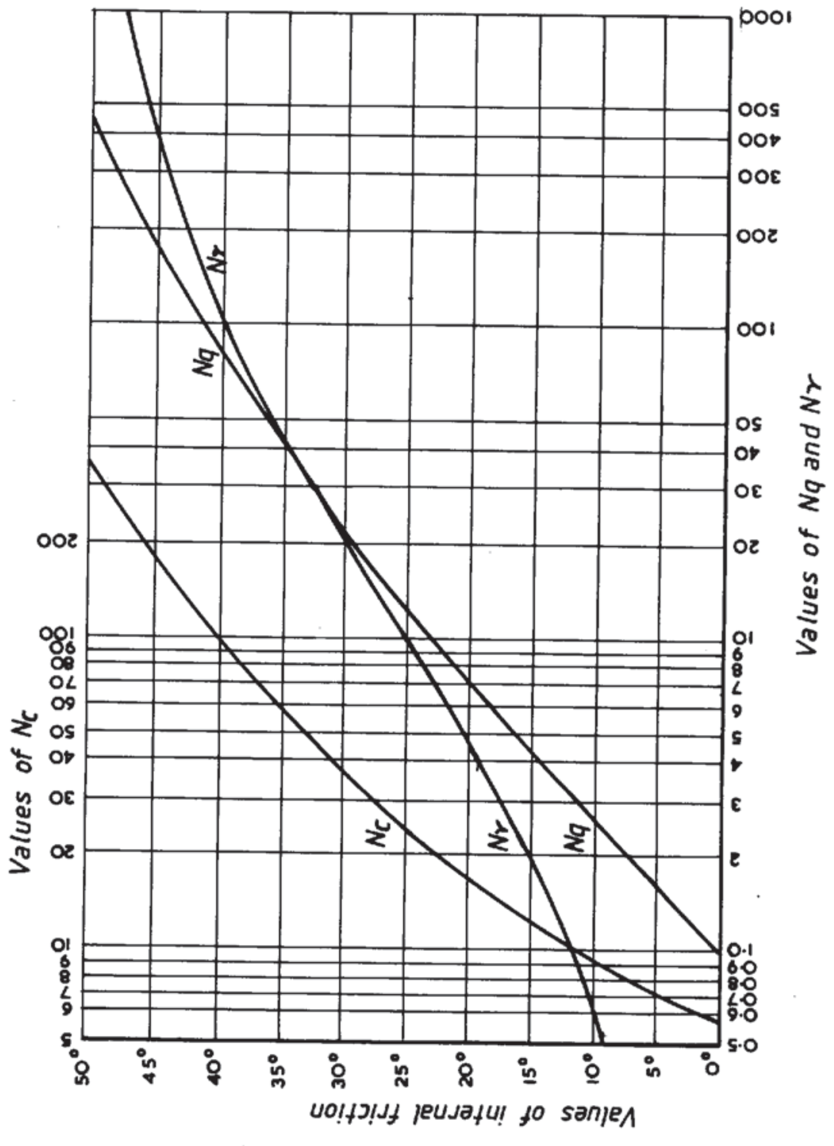


Fig. 6.1 Values of N_c, N_q and N_γ.

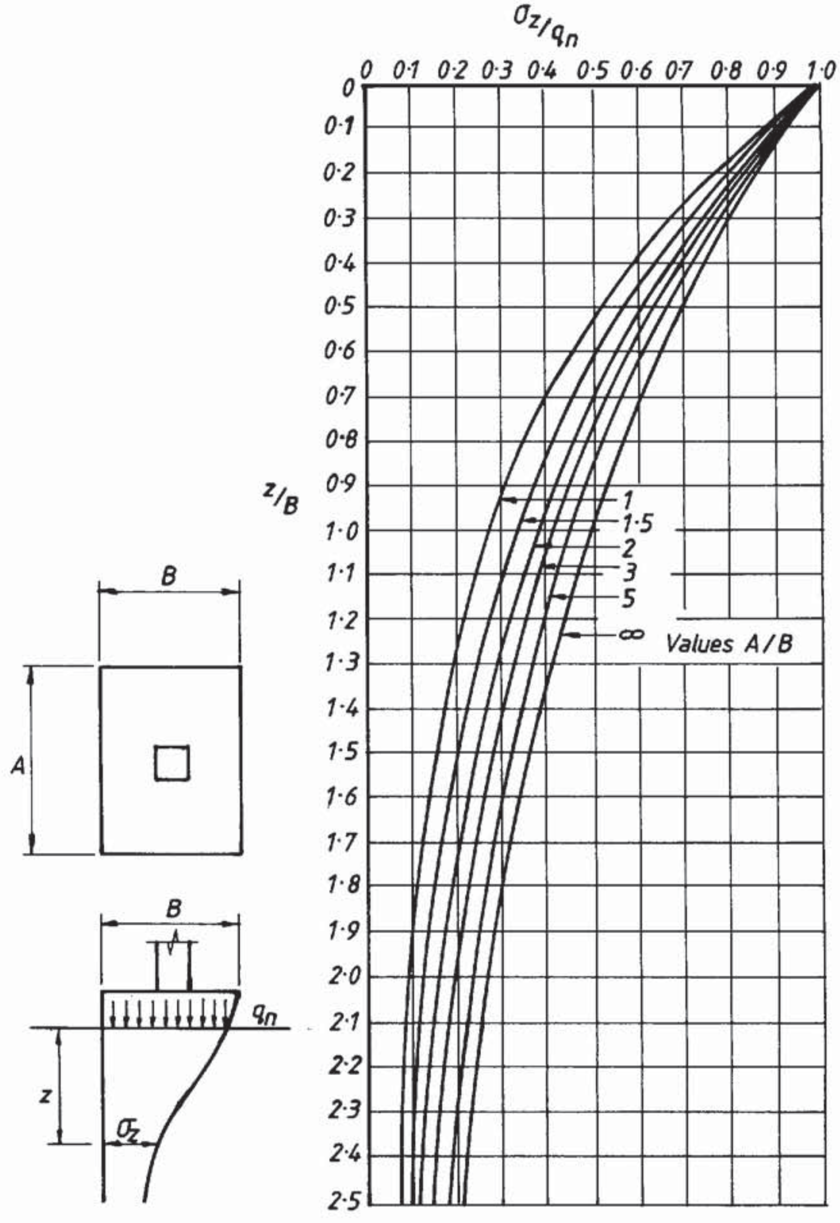


Fig. 6.2 Calculation of mean vertical stress (σ_z) at depth z beneath rectangular area $a \times b$ on surface, loaded at uniform pressure q_n.

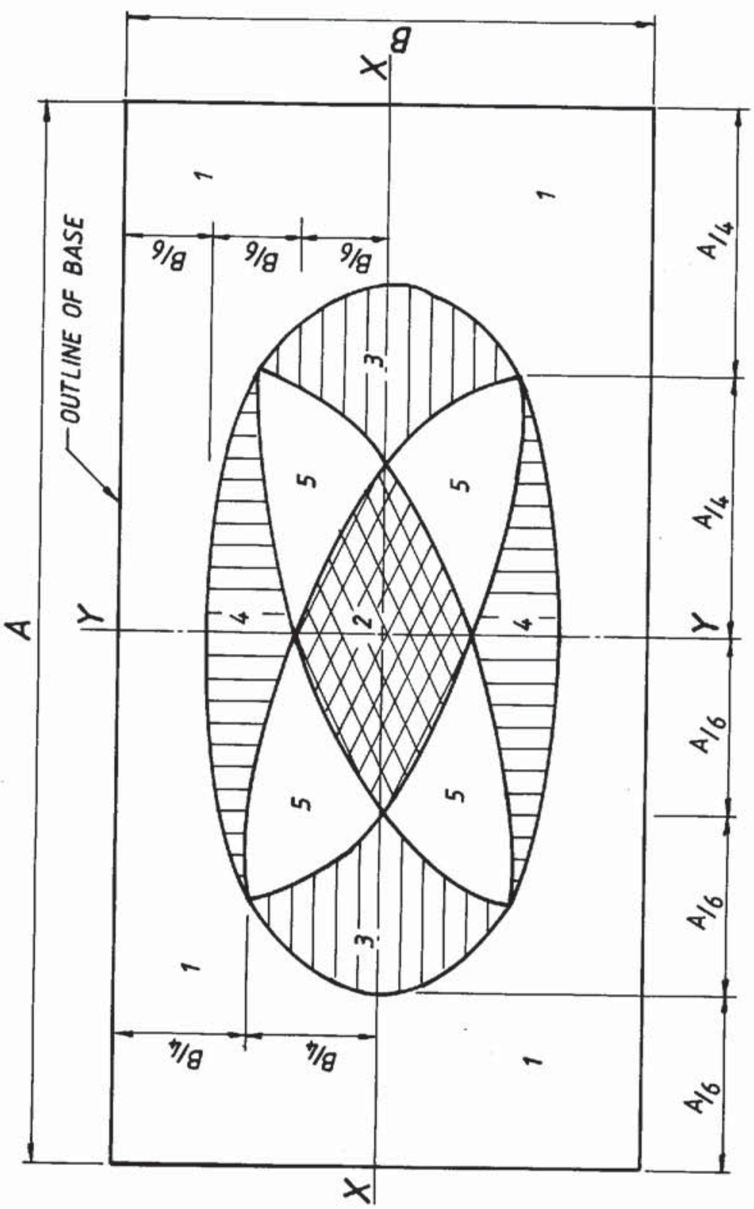


Fig. 6.3 Plan on base showing different zones.

Fig. 6.4 Chart for calculation of maximum pressure under a rectangular base subject to moments in two directions.