13. AN INTRODUCTION TO FOUNDATION ENGINEERING

13.1 TYPES OF FOUNDATIONS

The foundation is that portion of a structure that transmits the loads from the structure to the underlying foundation material. There are two major requirements to be satisfied in the design of foundations:

(a) Provision of an adequate factor of safety against failure of the foundation material. Failure of the foundation material may lead to failure of the foundation and may also lead to failure of the entire structure.

(b) Adequate provision against damage to the structure which may be caused by total or differential settlements of the foundations.

In order to satisfy these requirements it is necessary to carry out a thorough exploration of the foundation materials together with an investigation of the properties of these materials by means of laboratory or field testing. Using these physical properties of the foundation materials the foundations may be designed to carry the loads from the structure with an adequate margin of safety. In doing this, much use may be made of soil mechanics but to a large extent foundation engineering still remains an art. This chapter will be largely concerned with the contributions that may be made by soil mechanics to foundation engineering.

There are four major types of foundations which are used to transmit the loads from the structure to the underlying material. These foundations types are illustrated in Fig. 13.1. The most common type of foundation is the footing which consists of an enlargement of the base of a column or wall so that the pressure transmitted to the foundation material will not cause failure or excessive settlement. In order to reduce the bearing pressure transmitted to the foundation material the area of the footing may be increased. As the size of the footing increases however, the deeper the effect of footing pressure extends as was illustrated in Geomechanics 1.

If the foundation material cannot withstand the pressure transmitted by the footings the pressure may be reduced by combining all of the footings into a single slab or raft covering the entire plan area of the structure as illustrated in Fig. 13.1. Raft foundations are also used to bridge localised weak or compressible areas in the foundation material. They are also used where it is desirable to reduce the differential settlements that may occur between adjacent columns.

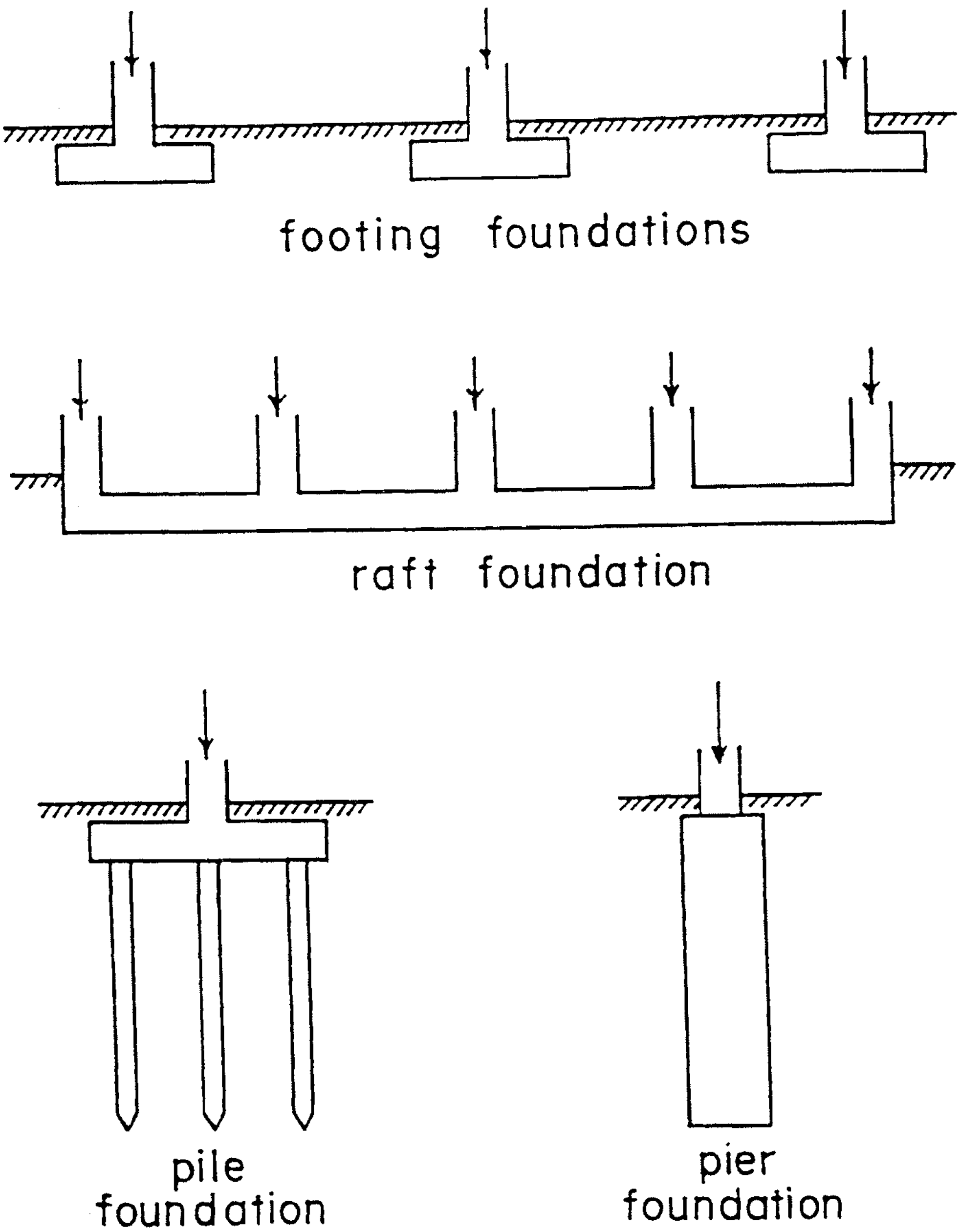

FIGURE 13.1 FOUNDATION TYPES

At a building site, firm foundation material may be overlain by strata of weak or compressible soil. In this case a pile foundation may be a satisfactory solution to transmit the structural loads through the weak material to the firmer underlying material. The piles may be driven steel, concrete or timber sections or may be made of cast in place concrete. These foundations are often classified either as end (or point) bearing piles or friction piles depending upon the major source of the support.

Piers are sometimes used for the transmission of large loads to firm foundation material which may be overlain by poorer material. Piers may be considered generally as large diameter cast in place piles. Piers are sometimes constructed as caissons in which the foundation members are sunk through the soil. With open caissons the hole is advanced by means of internal dredging and in the case of pneumatic caissons excavation is carried out under compressed air to prevent the entry of water and mud into the working chamber.

13.2 SETTLEMENT ANALYSES

Foundations of structures may experience movements through a number of causes, among which may be listed:

(a) elastic and inelastic compression of the sub-soil due to the weight of the structure,
(b) ground water lowering, producing an increase in effective stress beneath the foundation,
(c) vibration due to pile driving, machinery vibrations etc. which is of particular importance in granular soils,
(d) seasonal swelling and shrinking of expansive clays,
(e) adjacent excavation and construction which may cause movement of the foundations,
(f) regional subsidence or movement.

This section will be concerned mainly with settlement which is caused by changes in load such as the weight of a building.

For conditions of one dimensional compression the calculation of the amount of settlement and the rate at which it occurs have been covered in Geomechanics 1. This technique, which is based upon the results of oedometer tests will be referred to as the conventional method in the following discussion. Unfortunately, in foundation engineering practice, one dimensional compression conditions are not frequently encountered and consequently the use of the conventional method for the calculation of the amount and rate of settlement may not be reliable. This is illustrated in Figs. 13.2 and 13.3 which show comparisons between calculated and observed settlements for the Waterloo Bridge, London, and the Monadnock Block, Chicago. In these two instances both the amount and rate of settlement calculated by means of a conventional method considerably underestimate the values observed.

In calculating the rate of settlement of a structure it is necessary to determine the boundary conditions in relation to drainage, then to use the appropriate theoretical solution that satisfies the actual boundary conditions.

Regarding the calculation of the amount of settlement it is widely observed, as illustrated in Figs. 13.2 and 13.3, that a settlement occurs during the period of construction which is not predicted by the conventional one dimensional method. This type of observation has led to the two components of settlement being considered separately. The first component is an immediate settlement which occurs immediately following the application of load and the second component is a consolidation settlement which occurs as the pore pressure dissipates. The stress changes which take place in the soil as the immediate and consolidation settlements occur are quite different from those when settlement takes place under one dimensional conditions only.

The stress path followed by a sample of soil undergoing one dimensional consolidation is illustrated in Fig. 13.4. The stress paths that are drawn in this figure are for a representative element of soil beneath a foundation and beneath the water table. Points M and N indicate the initial effective and total stress points separated by an initial value of pore pressure u_i. Following erection of the structure a vertical stress increase represented by the distance NP will be applied to the soil. This means that the total stress path will move from point N to point P. If the time of construction is very short in relation to the consolidation time then point M will remain unmoved. This means that the effective stresses will be initially unchanged and the pore pressure in the soil will increase from the initial value u_i to a value represented by the distance MP.

As consolidation proceeds the value of the total major principal stress σ_1 will remain constant since it is governed by the externally applied foundation load, and the minor principal stress σ_3 will decrease. That is, the total stress path will move from point P to point R. At the same time the effective stress on the sample will increase as the pore pressure dissipates. The effective stress path will move along what is referred to as the K_o line from point M to point Q. A K_o line is a locus of the tops of the effective Mohr circles which possess a constant ratio between the minor and major principal stresses. It is along this line that effective stress paths move when a soil is being compressed under one dimensional conditions. Consolidation is complete when the excess pore pressure has fully dissipated the the pore pressure in the sample returns to the initial value.

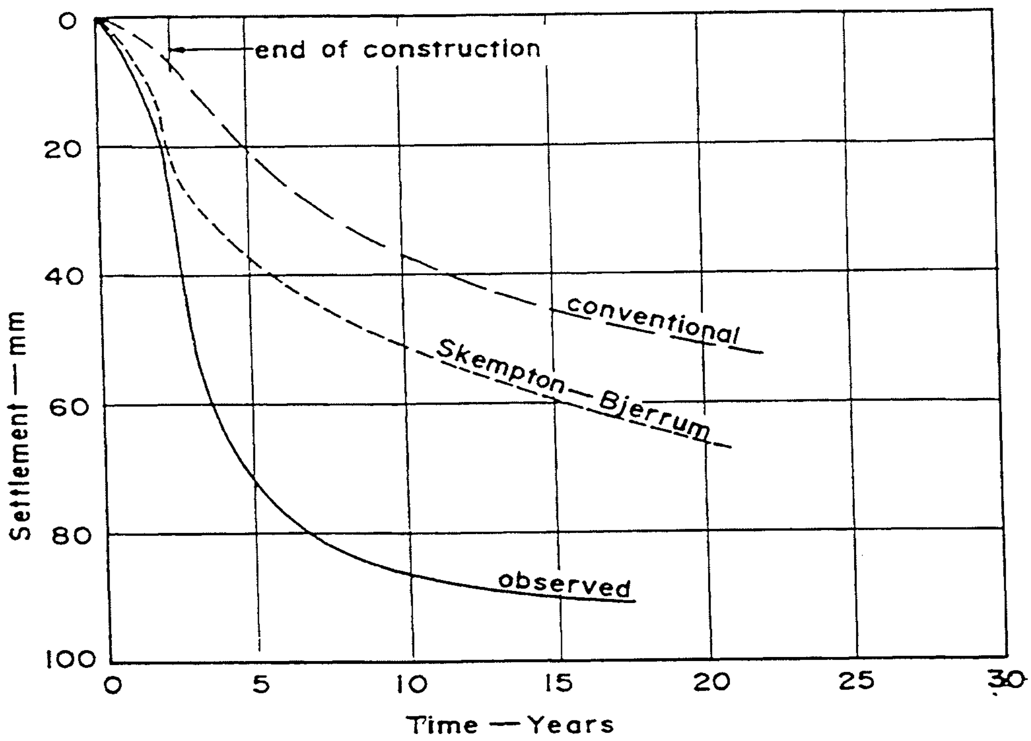

Fig 13.2 Settlement of Waterloo Bridge, London

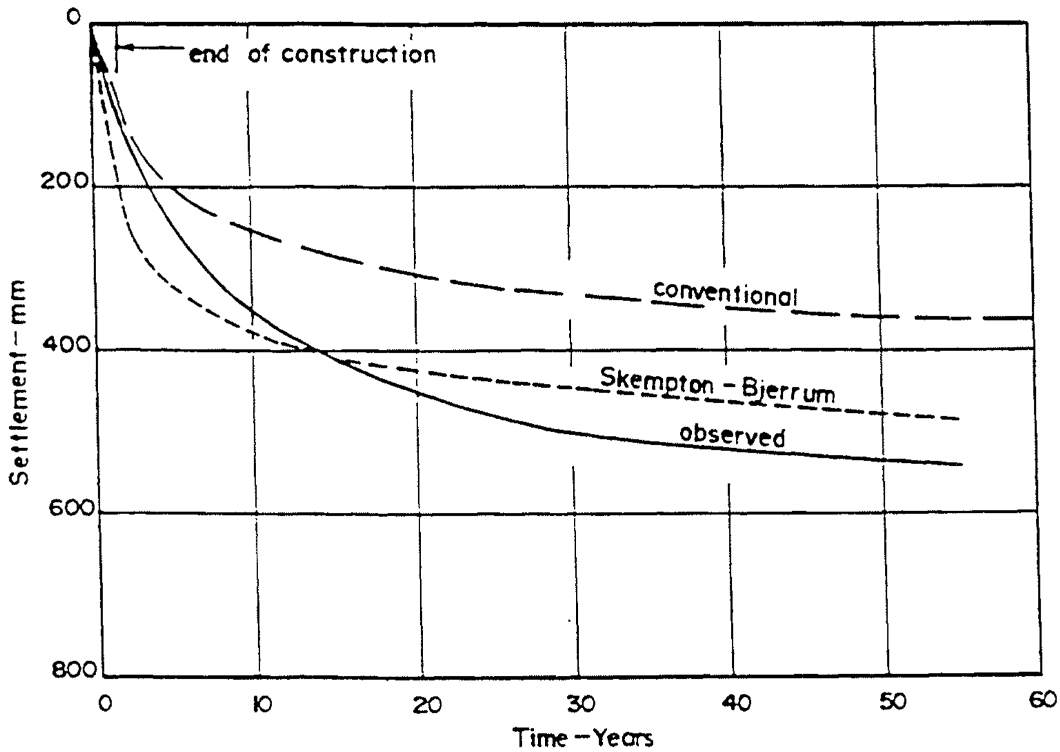

Fig 13.3 Settlement of Monadnock Block, Chicago

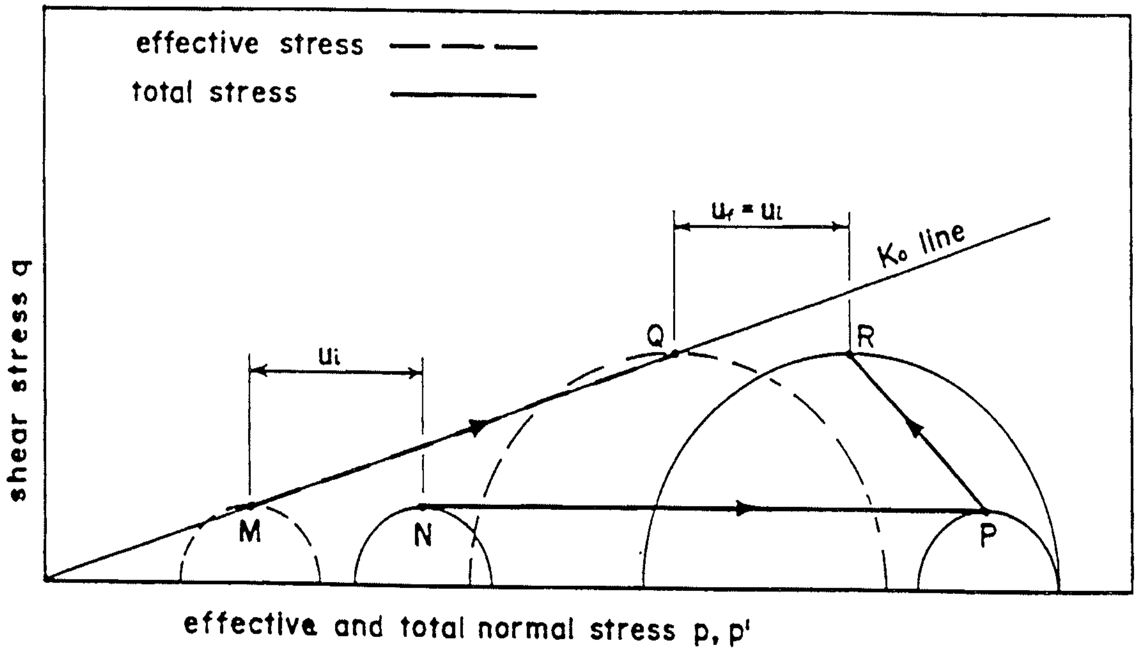

Fig 13.4 Stress Paths for One Dimensional Consolidation

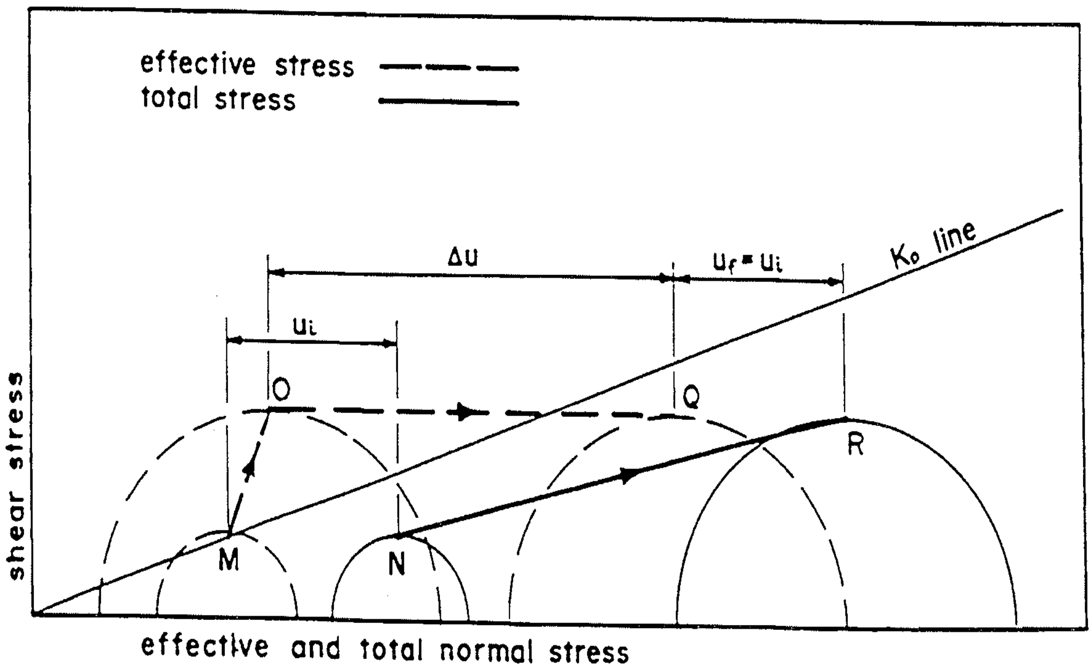

Fig 13.5 Stress Paths for Typical Settlement Situation

For a typical settlement situation in which one dimensional compression conditions are not present, the stress paths are illustrated in Fig. 13.5. Again, points M and N represent the initial effective and total stress points which are separated by an initial pore pressure, u_i. Following erection of a structure both principal stresses in the soil will change and, in general, the total stress path will move along a line represented by NR.

Immediately following erection, the effective stress path will move from point M to some point O. At the same time the pore pressure will increase from the initial value u_i to a value represented by the distance OR. In contrast to the one dimensional case the effective stresses change immediately following the application of the load. In response to this change in stresses the soil will undergo a settlement known as the immediate settlement.

From point O the effective stress path will move horizontally to point Q as the excess pore pressure Δu dissipates. Consolidation will be complete when the pore pressure in the soil returns to the initial value. During the movement of the effective stress path from point O to point Q the consolidation settlement will take place.

EXAMPLE

A circular pervious footing having a radius of 1.0m is located on the surface of a clay stratum 10m thick, which is underlain by a rigid impermeable base. Compare the value of t_{90} for one dimensional consolidation with the value based upon the more realistic assumption that the consolidation is three dimensional.

one dimensional $c_v = 10^{-3}$ cm^2/sec

The time factors for the one dimensional and three dimensional cases may be taken from Fig. 10.16. Geomechanics 1.

One Dimensional

$$T_{90} = 0.848 = c_v t_{90}/H^2$$

$$\therefore t_{90} = \frac{0.848 \text{ x } 10^2}{10^{-3} \text{ x } 10^{-4}} \text{ sec} = 26.9 \text{ yr}$$

Three Dimensional

The curve corresponding to a (h/a) value of 20 should be used (Fig. 10.16).

$$T_{90} = 2 \times 10^{-2} = c_v t_{90}/h^2$$

$$\therefore t_{90} = \frac{2 \times 10^{-2} \times 10^{2}}{10^{-3} \times 10^{-4}} \text{ sec} = 0.63 \text{ yr}$$

This indicates that the assumption that the consolidation occurs under one dimensional conditions leads to a significant over estimate of consolidation time.

13.3 THE SKEMPTON-BJERRUM METHOD OF SETTLEMENT ANALYSIS

As discussed in Chapter 9, Geomechanics 1, the settlement which occurs in a soil layer of thickness h when the compression takes place under one dimensional conditions, may be expressed as

$$\rho = \rho_{oed} = \int_o^h m_v \Delta\sigma_1 dz \qquad (13.1)$$

where m_v is the one dimensional compressibility as determined in the oedometer test

$\Delta\sigma_1$ is the stress change at depth z imposed by the applied load.

This one dimensional (oedometer) settlement may be considered as the total final settlement provided the loading conditions are strictly one dimensional. As discussed in section 13.2, loading conditions are commonly three dimensional, so the final settlement should be re-expressed as follows:

$$\rho_{final} = \rho_i + \rho_c \qquad (13.2)$$

where ρ_i is the immediate settlement which occurs immediately following the application of load, and ρ_c is the consolidation settlement which occurs as the pore pressure dissipates.

Note: A more general expression for the final settlement includes the secondary compression component ρ_s which occurs after consolidation is complete. Secondary compression has been described in section 10.7.1. Geomechanics 1. The expression for final settlement would then read

$$\rho_{final} = \rho_i + \rho_c + \rho_s$$

The settlement ρ_t which occurs at any time t between the time of construction and the completion of consolidation is given by the following expression

$$\rho_t = \rho_i + U\rho_c \qquad (13.3)$$

where U is the degree of settlement which was discussed in section 10.6. Geomechanics 1.

For the calculation of the immediate settlement (ρ_i) the use of elastic theory is considered to be applicable. The elastic displacement equation which may be used for this calculation and which was discussed in section 13.2 Geomechanics 1 is:

$$\rho_i = \frac{qB(1-\nu^2)I_\rho}{E} \qquad (13.4)$$

where q is the pressure applied by the footing to the foundation material

- B is the width or diameter of the footing
- ν is the Poisson's ratio of the soil
- I_ρ is the appropriate influence coefficient for settlement
- E is the relevant value of the Young's modulus for the soil.

For the calculation of the immediate settlement of a saturated fine grained soil the relevant value of the Poisson's ratio is 0.5. The relevant value of the influence factor I_ρ may be obtained from a number of tabulations or figures which are available (for example, see Terzaghi (1943), Fox (1948), Egorov (1958), Scott (1963), Harr (1966), Christian & Carrier (1978), Das (1984)). Influence factors have been evaluated for smooth or rough rigid footings of various shapes and the centre, corner and average of flexibly loaded areas. Some information on influence factors has been provided in Chapter 9 Geomechanics 1 and further data is contained in Table 13.1.

For the calculation of the consolidation settlement (ρ_c) in equations (13.2) and (13.3), Skempton and Bjerrum (1957) proposed the following expression

$$\rho_c = \int_o^h m_v \Delta u \, dz \qquad (13.5)$$

where m_v is the one dimensional compressibility of the soil

- h is the layer thickness
- Δu is the pore pressure change developed by the application of load and may be determined from equation (7.9) Geomechanics 1 ($\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)]$).

TABLE 13.1

INFLUENCE FACTORS FOR SETTLEMENT OF SMOOTH RIGID FOOTINGS ON THE SURFACE OF AN ELASTIC SOLID - RIGID BASE AT DEPTH H

Length of Footing = L
Width of Footing = B

$$\rho = \frac{qBI_\rho (1-\nu^2)}{E}$$

VALUES OF $I\rho$

H/B	Circle (Diameter = B)	RECTANGLE					Infinite Strip L/B = ∞
		L/B = 1	**L/B = 2**	**L/B = 3**	**L/B = 5**	**L/B = 10**	
0	0.000	0.000	0.00	0.000	0.000	0.000	0.000
0.1	0.096	0.096	0.098	0.098	0.099	0.099	0.100
0.25	0.225	0.226	0.231	0.233	0.236	0.238	0.239
0.5	0.396	0.403	0.427	0.435	0.441	0.446	0.452
1.0	0.578	0.609	0.698	0.727	0.748	0.764	0.784
1.5	0.661	0.711	0.856	0.910	0.952	0.982	1.018
2.5	0.740	0.800	1.010	1.119	1.201	1.256	1.323
3.5	0.776	0.842	1.094	1.223	1.346	1.442	1.532
5.0	0.818	0.873	1.155	1.309	1.475	1.619	1.758
∞	0.849	0.946	1.300	1.527	1.826	2.246	∞

(after Egorov, 1958)

For a saturated soil the pore pressure parameter B is unity so equation (7.9) simplifies to:

$$\Delta u = \Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)$$

$$= \Delta\sigma_1\{(A + \frac{\Delta\sigma_3}{\Delta\sigma_1} (1 - A))\} \qquad (13.6)$$

If equation (13.6) is substituted into equation (13.5) the consolidation settlement may then be expressed as:

$$\rho_c = \int_o^h m_v \Delta\sigma_1 (A + \frac{\Delta\sigma_3}{\Delta\sigma_1}(1-A))dz \qquad (13.7)$$

That is:

$$\rho_c = \mu \rho_{oed} \qquad (13.8)$$

where

$$\mu = \frac{\int_o^h m_v \Delta\sigma_1 \left(A + \frac{\Delta\sigma_3}{\Delta\sigma_1}(1-A)\right)dz}{\int_o^h m_v \Delta\sigma_1 dz} \qquad (13.9)$$

If m_v and A are assumed to be constant with depth

$$\mu = A + \alpha (1 - A)$$

where

$$\alpha = \frac{\int_o^h \Delta\sigma_3 \, dz}{\int_o^h \Delta\sigma_1 \, dz}$$

$$\therefore \quad \rho_{final} = \rho_i + \mu \rho_{oed} \qquad (13.10)$$

The tabulation of α values for circular and strip footings is given in Table 13.2

TABLE 13.2

VALUES OF α IN SKEMPTON-BJERRUM SETTLEMENT ANALYSIS
(H = depth to rigid stratum)

H/B	**Circular Footing (diameter = B)**	**Strip Footing (width = B)**
0	1.00	1.00
0.25	0.67	0.74
0.5	0.50	0.53
1.0	0.38	0.37
2.0	0.30	0.26
4.0	0.28	0.20
10.0	0.26	0.14
∞	0.25	0.00

Skempton and Bjerrum have prepared a plot relating the μ factor to the pore pressure co-efficient A for circular and strip footings and for various ratios of layer depth to footing width. This plot has been modified by Scott (1963) and this modified diagram has been reproduced as Fig. 13.6.

If the foregoing procedures are used for the calculation of the immediate settlement (ρ_i) and the consolidation settlement (ρ_c), equations (13.2) and (13.3) respectively may be used for the calculation of the final settlement (ρ_{final}) and the settlement (ρ_t) at any time during consolidation. From comparisons between observed and calculated settlements, it is possible to determine whether equation (13.2) provides a better prediction of settlement that that provided by the conventional oedometer method in equation (13.1). Skempton and Bjerrum (1957) showed that their method (equation (13.2)) gave, on average, a calculated settlement almost equal to the observed settlement for structures on both normally and overconsolidated clay deposits. In contrast the conventional method (equation (13.1)) tended to underestimate settlement for normally consolidated clay deposits and overestimate the observed settlement for overconsolidated clay deposits. However there was a noticeable scatter of results of the calculated to observed settlement ratio with both methods.

While the Skempton-Bjerrum method does provide an improved technique for the calculation of settlement when compared with the conventional method, it is not, strictly speaking, a truly three-dimensional method and therefore cannot be expected to yield accurate estimates of settlement in all situations. More valid and potentially more accurate methods for calculating settlement have been presented by Lambe (1964) and by Davis and Poulos (1968).

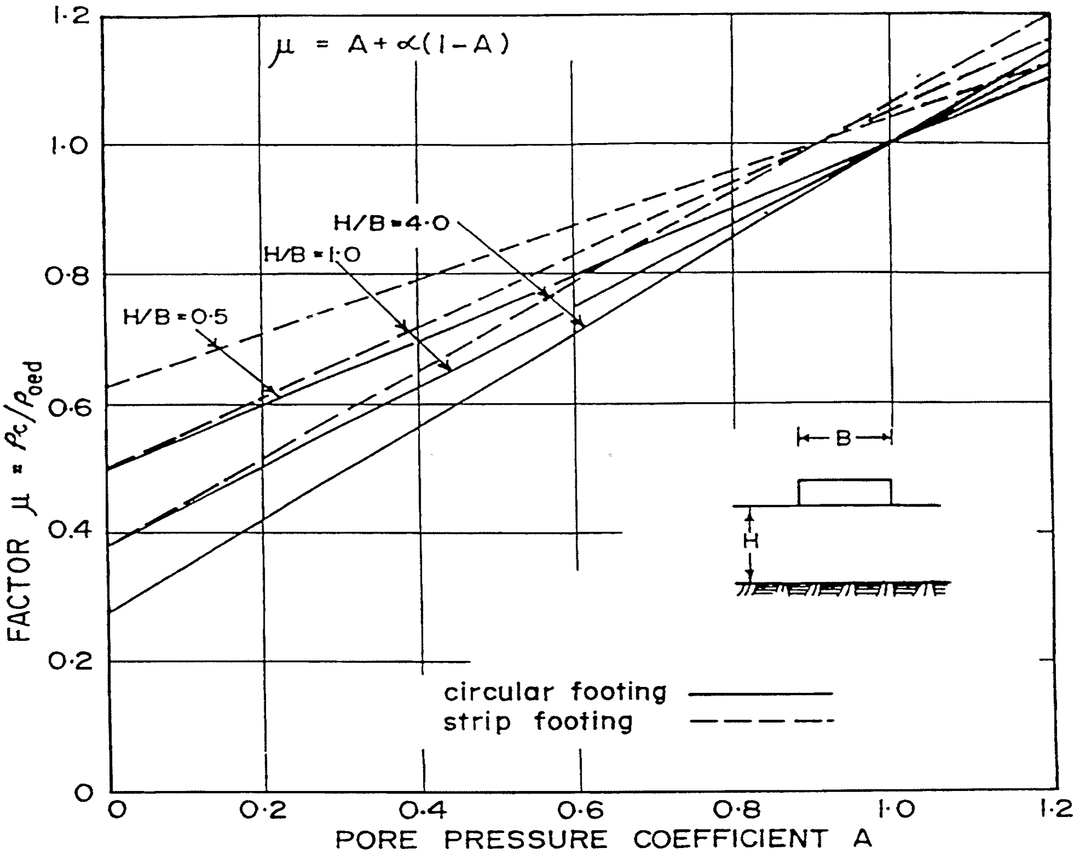

Fig 13.6 Values of Factor μ (After Scott, 1963)

EXAMPLE

Estimate the immediate and total final settlement of a 2m square rigid concrete footing supporting a load of 280 kN. The footing is underlain by a uniform deposit of saturated clay with solid rock at a depth of 6m. The properties of the clay are:

one dimensional compressibility	m_v	=	$0.6 m^2/MN$
pore pressure parameter	A	=	0.5
undrained Youngs modulus	E_u	=	$1.5 MN/m^2$

IMMEDIATE SETTLEMENT

$$\rho_i = \frac{q\, B\, I\rho\, (1 - \nu^2)}{E_u}$$

since the clay is saturated $\nu = 0.5$

and from Table 13.1 for H/B = 3 and L/B = 1,

I_ρ = 0.82

q = 280/(2 x 2) = 70kPa

$$\therefore \rho_i = \frac{70 \times 2 \times 0.82\,(1 - 0.25)}{1.5 \times 1000}$$

= 57.4mm

CONSOLIDATION SETTLEMENT

This will be calculated by means of equation (13.8). The oedometer settlement (ρ_{oed}) will be calculated by means of the 2:1 stress transmission approximation.

stress change at depth z

$\Delta\sigma_z$ = $280/(z + B)^2$

ρ_{oed} = $\Sigma\, m_v\, \Delta\, \sigma_z\, h$

where h is the layer thickness

The calculations based on three 2m thick clay layers are tabulated below.

Depth (z) (m)	**(z + B) (m)**	**$(z + B)^2$**	**$\Delta\sigma_z$ (kPa)**	**ρ_{oed} (mm)**
1.00	3.0	9	31.1	37.3
3.0	5.0	25	11.2	13.4
5.0	7.0	49	5.7	6.8
			Total	**57.5**

A more accurate assessment of ρ_{oed} may be made if the calculation is carried is carried out by integration

$$\rho_{oed} = \int_0^6 m_v\ \Delta\sigma_z\, dz$$

$$= \int_0^6 0.6 \times (280/(2 + z)^2)\ dz$$

$$= -0.6\,[280/(2 + z)]_0^6$$

= 63.0mm

From Table 13.2, the value of α for a circular footing may be used for the square footing in this problem.

For (H/B) = 6/2 = 3.0

α = 0.29 approx.

$\therefore \mu = A + \alpha(1 - A)$

$= 0.5 + 0.29(1 - 0.5)$

$= 0.65$

$\therefore \rho_c = 0.65\,\rho_{oed} = 0.65 \times 63.0$

$= 40.6$mm

total final settlement $= \rho_i + \rho_c$

$= 57.4 + 40.6$

$= 98$mm

13.4 SETTLEMENT OF STRUCTURES ON SANDY SOILS

Because of the difficulty of obtaining undisturbed samples of sandy soil and carrying out laboratory tests to measure the soil compressibility, calculations of settlement of structures on sandy soil are usually based upon field tests. Some techniques are based upon penetration resistance readings of the soil. One commonly used measure of penetration resistance is the so called standard penetration resistance, N, which is obtained in a dynamic penetration test in which the penetrometer consists of a split barrel sampler. With this test the sampler is driven by means of a 63.5 kilogram mass falling a distance of 760mm to the top of the drill rods, the number of blows required to drive the sampler a distance of 300mm being recorded as the standard penetration resistance N.

Using this standard penetration resistance, Terzaghi and Peck (1948) prepared a design chart based on the assumption that the allowable maximum settlement of a footing was 25mm. It is now appreciated that this design chart was too conservative and Peck, Hanson and Thornburn (1974) have prepared a more realistic design chart which is shown in Figure 13.7. The horizontal lines in the figure indicate the soil pressure corresponding to a settlement of 25mm. The chart is based on the observed behaviour of footings located at depths of 3 to 5 metres below the ground surface. The N values governing the footing behaviour therefore corresponded to an effective overburden pressure of approx. 0.1MPa. Some engineers apply a factor of 1.5 to the allowable soil pressures obtained from Fig. 13.7, arguing that this figure is still overconservative. In the design of proposed footings, if the overburden pressure differs greatly from 0.1MPa the N values should be corrected before the correlation in Figure 13.7 is used.

Gibbs and Holtz (1957) in an extensive laboratory study showed the way in which the standard penetration resistance is influenced by overburden pressure (Fig. 13.8). Gibbs and Holtz suggested that N values could be corrected for the effects of oberburden pressure by means of this diagram. If the N value at a depth corresponding to an effective overburden pressure of 0.1MPa is considered to be a standard, the correction factor, C_N, recommended by Peck, Hanson and Thornburn (1974) is given by:

$$C_N = 0.77 \log_{10} (2/p') \quad . \qquad (13.11)$$

where p' is the effective overburden pressure in MPa. The equation is valid for $p' \geq 25$ kPa. Equation (13.11) with a modification for low overburden pressures, is illustrated graphically in Fig. 13.9 Peck, Hanson and Thornburn (1974) have suggested that correction factors within the range 0.8 to 1.2 may be ignored without serious error.

The lines in Fig. 13.7 are drawn for the condition that the water table is at great depth. If the water table is located at a depth below the ground surface of less than (D + B), Peck, Hanson and Thornburn (1974) have recommended that the soil pressures must be multiplied by a correction factor (C_w) as follows:

$$C_w = 0.5 + 0.5 \left(\frac{D_w}{D + B}\right) \qquad (13.12)$$

where D_w is the depth to the water table
D is the depth to the underside of the footing
B is the footing width.

Another measure of penetration resistance which is widely used is that obtained from a cone penetration test. (AS1289 F13.1). In this test, the point resistance is measured as the conical point of the penetrometer is gradually pushed into the ground. Meyerhof (1956) has suggested that the relationship between the cone penetration resistance (q_c) and the standard penetration resistance N is given by the following expression

$$q_c = 400N \text{ KPa} \qquad (13.13)$$

where N is the uncorrected SPT value. In contrast Meigh and Nixon (1961) have shown that q_c varies from 430 N to 1930 N kPa.

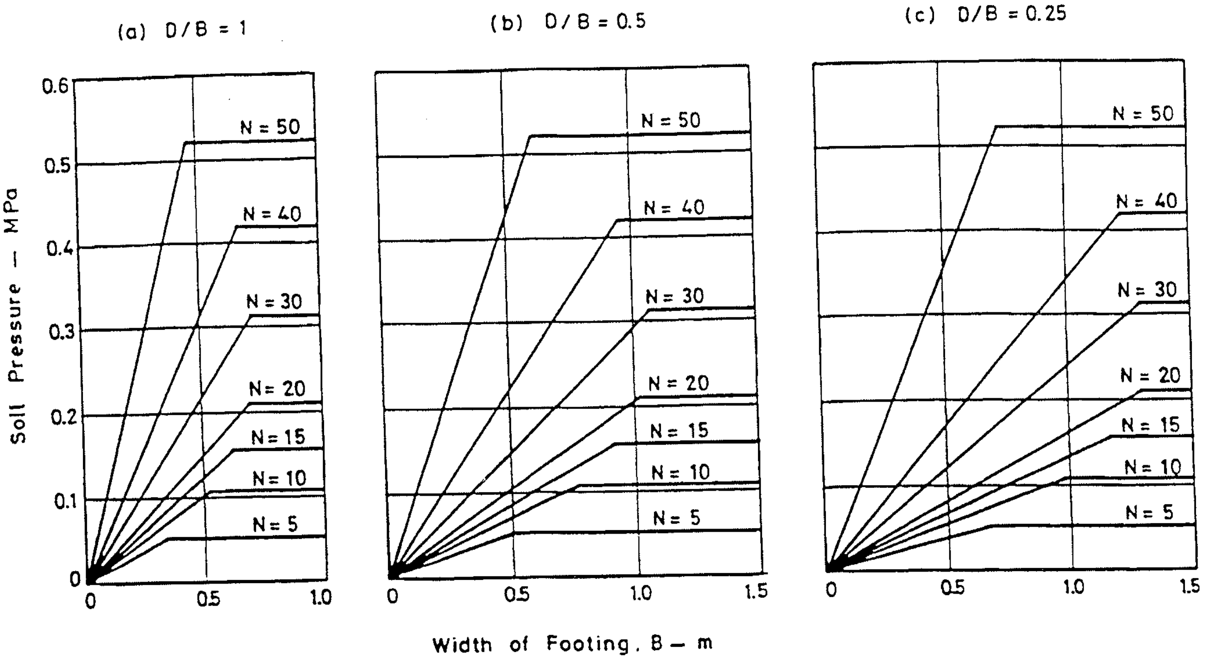

Fig 13.7 Design Chart for Proportioning Shallow Footing on Sand

(after peck, hanson and thornburn, 1974)

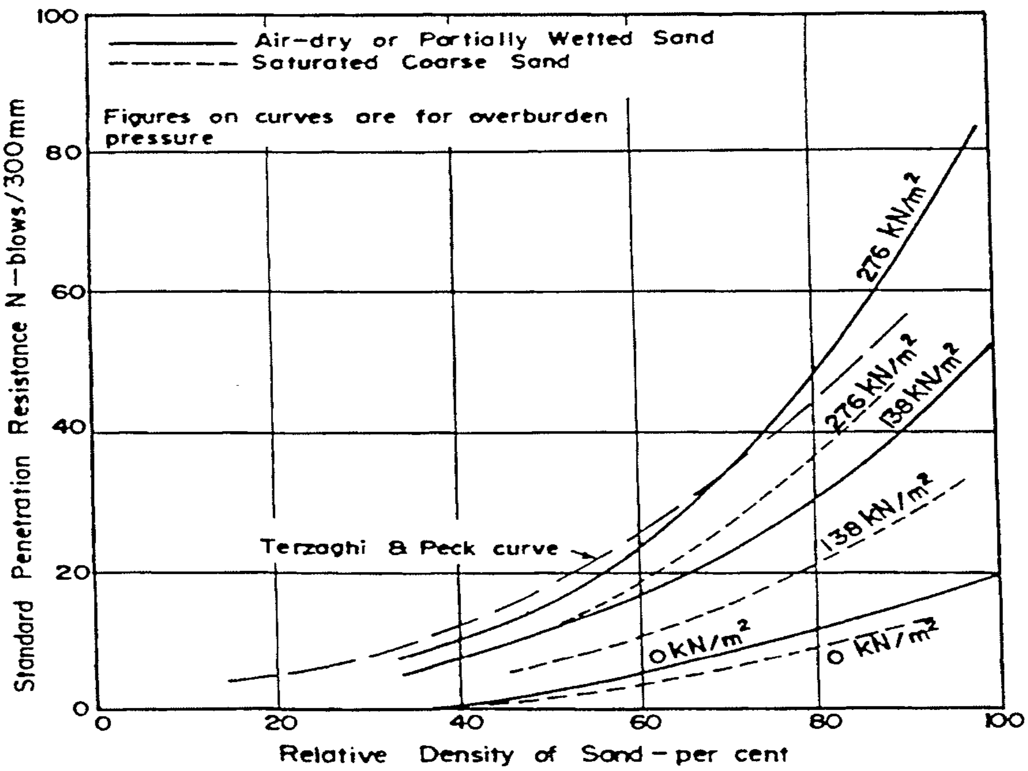

Fig 13.8 Influence of Overburden Pressure on Penetration Resistance in Sands

(After Gibbs and Holtz, 1957)

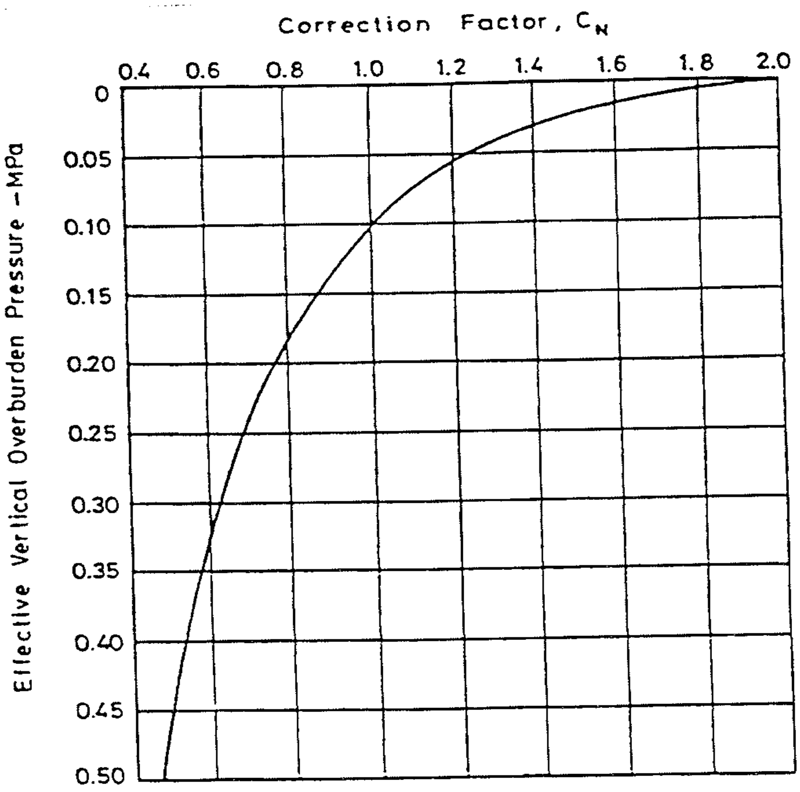

Fig 13.9 Chart for Correction Of N - Values in sand for influence of Over-Burden Pressure (After Peck, Hanson and Thornburn, 1974)

If the Young's modulus of a sand deposit can be evaluated or estimated, settlement may be determined by means of the elastic displacement equation (13.4), the same one that was used for the calculation of immediate settlement. From the cone penetration resistance (q_c) the Young's modulus (E_s) of sandy soil may be determined by means of the following empirical expression

$$E_s = 2.5q_c \qquad (13.16)$$

With this value of E_s the settlement of a structure, supported on square footings on sand may also be determined by means of a semi-empirical procedure described by Schmertmann (1970, 1978).

Correlations have been developed between the compressibility (m_v) and the cone penetration resistance (q_c). Expressions relating the two are often of the form

$$m_v = 1/(\alpha q_c) \qquad (13.17)$$

where α is a constant depending largely on the type of soil. This form of expression is widely used for clay soils. Tabulations of α values to be used with this expression have been given by Sanglerat (1972).

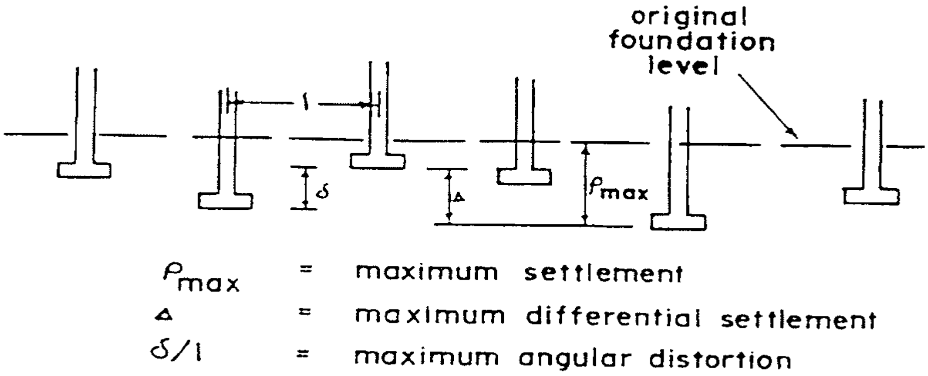

Fig 13.10 Definitions of Settlement

EXAMPLE

The results of standard penetration tests in a medium-coarse sand are:

HOLE NO.	1	2	3	4
DEPTH (m)	**Corrected N values**			
1.0	3	5	4	7
2.0	15	12	13	20
3.0	14	18	14	22
4.0	21	24	18	18
5.0	27	22	20	31
6.0	30	34	38	36
10.0	52	47	60	42

(water table at 12 m)

Comment upon the proposal to place a 3 m by 4 m footing at a depth of 2.0 m to carry a load of 1.0 MN, if the settlement of the footing is to be limited to 25 mm.

For each hole the average N value over a depth below the proposed founding level equal to the footing width B should be determined. The average N values between depths of 2 m and 5 m are 19, 19, 16 and 22 for holes 1, 2, 3 and 4 respectively. The minimum N value of 16 is the one on which the assessment of the allowable soil pressure should be based.

From Fig. 13.7 for B = 3 m, D/B = 0.67 and N = 16 allowable soil pressure = 0.16 MPa.

∴ load which can be safely carried by the footing

= 0.16 x 3 x 4
= 1.92 MN

Since the allowable load exceeds the actual load, the footing settlement should not exceed 25mm. Based upon settlement considerations the footing could be reduced in size.

13.5 ALLOWABLE SETTLEMENT

The stability and safety of a structure depends much more critically upon the distortion of the structure as a result of differential movements of the foundations than upon the absolute magnitude of the overall settlement of the foundation. Many buildings have experienced large settlements (even meters of settlement) without having undergone significant structural damage. In discussing allowable settlement of structures four measures of settlement are frequently used:

(a) ρ_{max} the maximum settlement of any portion of the foundation
(b) Δ the maximum differential settlement between any two portions of the foundation
(c) (δ/l) the maximum angular distortion of buildings with columns, where δ is the differential settlement between the adjacent column footings and l is the column spacing
(d) relative deflection which is the ratio of the maximum settlement to the length of the structure.

Some of these definitions are illustrated in Fig. 13.10.

A number of observational studies have been carried out in an attempt to define the allowable settlement for buildings. From the results of a study of a large number of buildings, Skempton and MacDonald (1956) have recommended a range of maximum allowable settlements for structures. This information has been set out in Table 13.3. In this same table, Polshin and Tokar (1957) have quoted the recommended maximum average settlement values which are given in the Russian building code. The maximum settlements quoted in Table 13.3 are greater than the 25mm value on which Fig. 13.7 is based. Although Fig. 13.7 may provide a conservative design, differential settlement rather than maximum settlement provides a more logical criterion for design as mentioned earlier.

TABLE 13.3

MAXIMUM ALLOWABLE SETTLEMENTS FOR BUILDINGS AND LOAD BEARING WALLS

		Footings	Rafts
Maximum Settlement (mm)	**Clays**	75	75 to 125
	Sands	50	50 to 75
Maximum Differential Settlement (mm)	**Clays**	45	
	Sands	30	

(after Skempton & MacDonald, 1956)

Structure	Maximum Average Settlement (mm)
Buildings with plain brick walls on continuous and separate foundations (L = wall length, H = wall height)	
L/H ≥ 2.5	80
L/H ≤ 1.5	100
Buildings with brick walls reinforced with reinforced concrete or reinforced brick	150
Framed buildings	100
Solid reinforced concrete foundations of blast furnaces, smoke stacks, silos, water towers, etc.	300

(after Polshin & Tokar, 1957)

The maximum settlement of a foundation is relatively easy to determine by means of one of the techniques previously discussed. The maximum differential settlement, however, is quite difficult to quantify. Fig. 13.11 has been prepared from a study of a large number of buildings located on both sandy and clayey soils. This diagram presents upper boundaries to the observed maximum differential settlements for particular values of observed maximum settlements. This figure indicates that in situations where the seat of settlement is located immediately beneath the foundation as in the case of sands the maximum differential settlement is almost identical with the maximum settlement. Smaller relative values of maximum differential settlement, however, are observed for structures founded on clay soils.

Observations of building settlements have indicated that the maximum angular distortion may be related in an approximate way to the maximum differential settlement as illustrated in Fig. 13.12. By making use of Figs. 13.11 and 13.12 a very rough idea of the maximum angular distortion corresponding to a particular calculated value of the maximum settlement of a foundation may be obtained.

Regarding the maximum allowable values of angular distortion a number of recommendations have been made. Terzaghi (1935) from a study of the settlement of six buildings with load bearing brick walls concluded that the limiting angular distortion was in the vicinity of 1/280. Skempton and McDonald (1956) have suggested that cracking of the panels in frame buildings is likely to occur if the angular distortion exceeds 1/300. Other values of the angular distortion corresponding to a range of criteria are presented in Table 13.4. These figures apply to a sagging mode of distortion. Burland et al (1979) showed that if the deformation mode was hogging cracking occurred at much lower values of angular distortion.

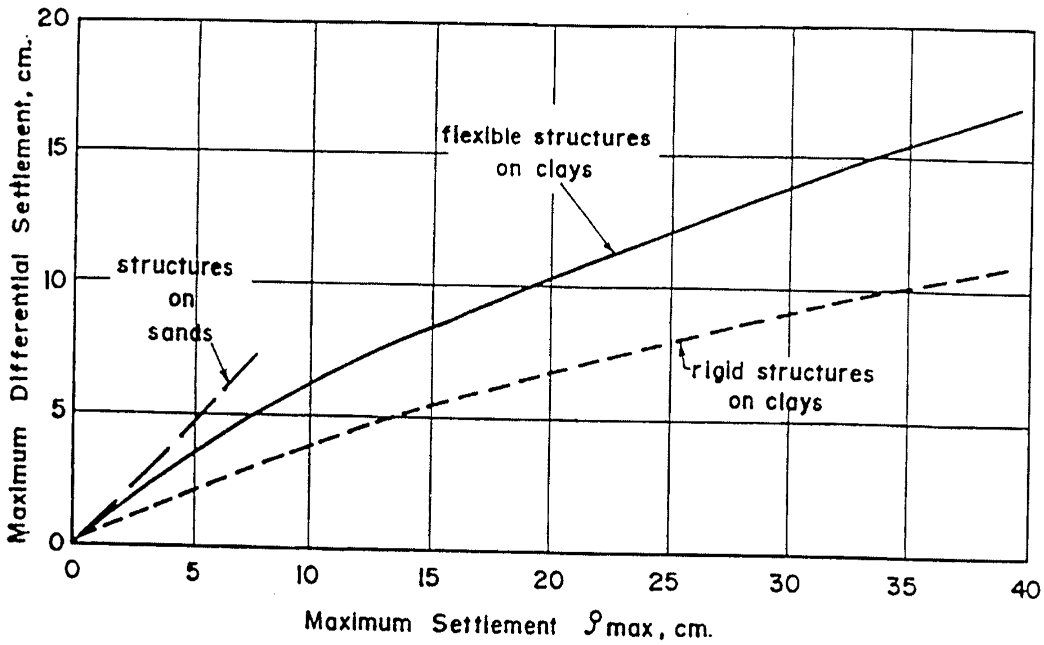

Fig 13.11 Envelopes of Maximum Observed Differential Settlements

(After Bjerrum, 1962)

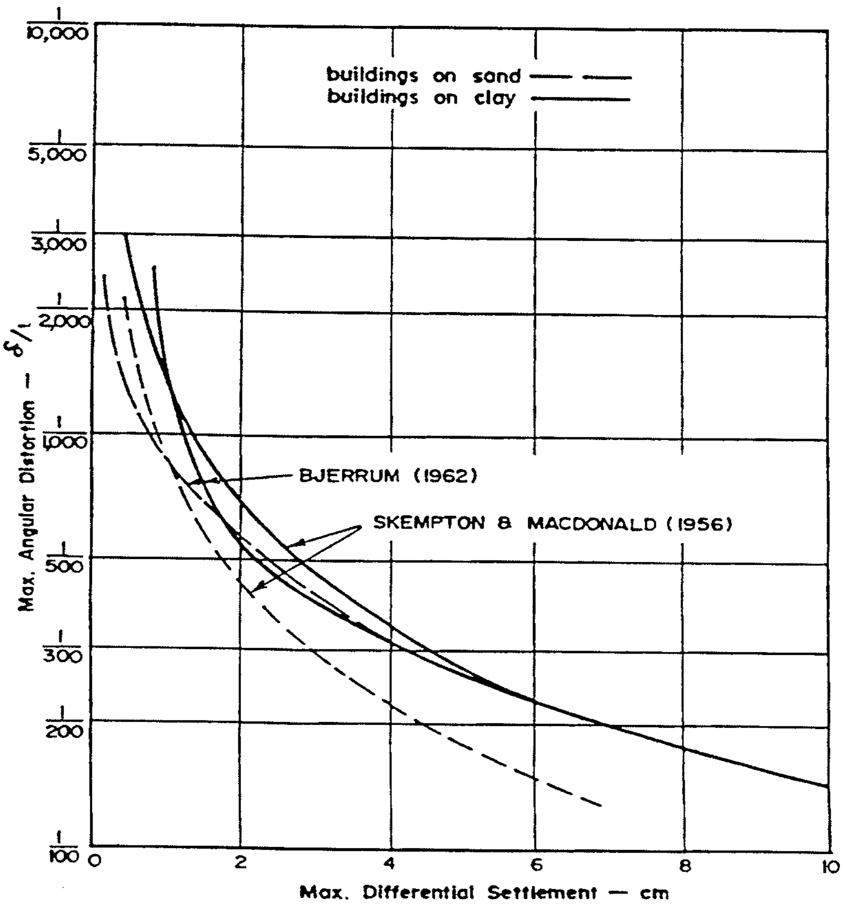

Fig 13.12 Observed relationship between Max. Differential Settlement and Max. Angular Distortion

TABLE 13.4
LIMITS OF ANGULAR DISTORTION OF BUILDINGS
(after Bjerrum, 1963)

Criterion	**Angular Distortion (δ/1)**
Structural damage to building	1/150
Safe limit for flexible brick walls $\left(H/L < \frac{1}{4}\right)$	1/150
Considerable cracking in panel walls and brick walls	1/150
Difficulties with overhead cranes to be expected	1/300
First cracking in panel walls to be expected	1/300
Safe limit for buildings where cracking is not permissible	1/500
Danger limit for frames with diagonals	1/600
Limit where difficulties with machinery sensitive to settlements to be feared	1/750

Values of the recommended maximum allowable angular distortion taken from the Russian building code have been presented in Table 13.5.

TABLE 13.5

MAXIMUM ALLOWABLE ANGULAR DISTORTION OF BUILDINGS AND LOAD BEARING WALLS

(after Polshin & Tokar, 1957)

	Sand and Hard Clay	**Plastic Clay**
Angular distortion for:		
(a) steel and reinforced concrete frame structures	.002	.002
(b) end rows of columns with brick cladding	.007	.001
Relative deflection of plain brick walls for:		
(a) multi storey buildings		
$L/H \leq 3$	.0003	.0004
$L/H \geq 5$	.0005	.0007
(L = length of deflected part of wall,		
H = wall height)		
(b) one storey mills	.001	.001

13.6 BEARING CAPACITY OF STRIP FOOTINGS

The maximum bearing pressure that a foundation can support before the foundation soil fails is widely referred to as the ultimate bearing capacity. General shear failure of the foundation soil for a continuous or strip footing is considered to occur as shown in Fig. 13.13. The surface of sliding is along line KLM with a similar surface of sliding on the opposite side of the footing. A number of solutions for the ultimate bearing capacity (q_{ult}), in which various simplifying assumptions are made, have been developed. Prandtl (1921) has produced a solution for a surface footing ($D = 0$) in which the foundation soil is assumed to be weightless. For this solution, which was carried out by means of the theory of plasticity, and angle ψ is equal to $(45^\circ + \phi/2)$.

With the Prandtl solution the underside HJ of the footing is smooth and three zones may be identified within the failure region. The zone indicated in Fig. 13.13 by the triangle JHK is an active Rankine zone, the zone indicated by the region KJL is a zone of radial shear with the line KL being a portion of a log spiral and the zone indicated by the triangle JLM is a passive Rankine zone.

If a surcharge pressure q is considered to act on the surface of the ground, level with the underside of the foundation, an additional component is added to the ultimate bearing capacity. Reissner (1924) provided a solution for this surcharge component, which was also based on the theory of plasticity. These two bearing capacity components have been represented by means of the expression

$$q_{ult} = cN_c + qN_q \qquad (13.18)$$

where N_c and N_q are bearing capacity factors which are given by the following expressions:

$$N_q = \tan^2 (45^\circ + \phi/2)\ e^{\pi \tan \phi} \qquad (13.19)$$

$$N_c = (N_q - 1) \cot \phi \qquad (13.20)$$

In the analyses by Prandtl and Reissner, it was assumed that the soil was weightless and the underside of the footing was perfectly smooth. In reality, however, soils possess weight and the underside of a footing is rarely smooth. These two aspects of the problem were taken into account in the solution presented by Terzaghi (1943). He assumed that the underside of the continuous footing was rough and the angle ψ in Fig. 13.13 was equal to the angle of shearing resistance ϕ. With the Terzaghi approach the wedge of soil JHK beneath the footing is treated as part of the footing. A surcharge or overburden pressure (q) will be present if the footing is located a distance below the ground surface. Terzaghi assumed that the soil above the foundation level HJM could be simply represented by the equivalent surcharge pressure q. The ultimate bearing capacity (q_{ult}) is determined

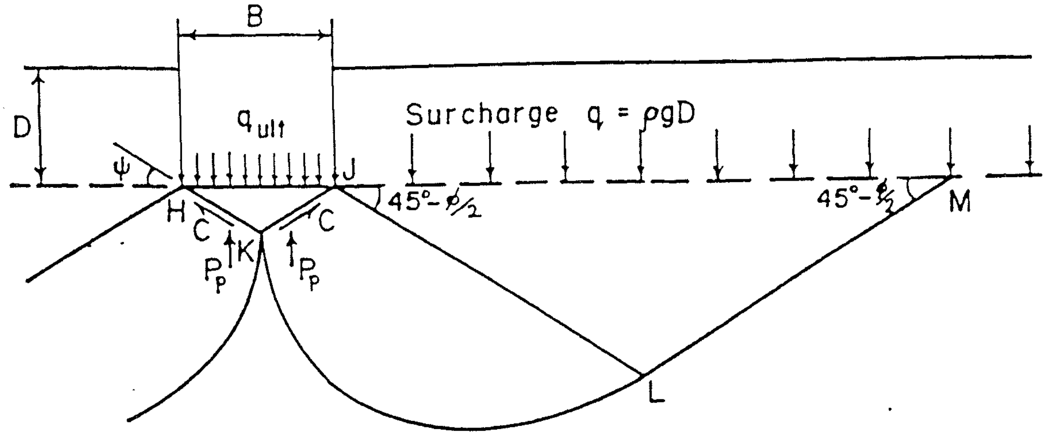

Fig 13.13 Assumed Mode of Failure for a Strip Footing

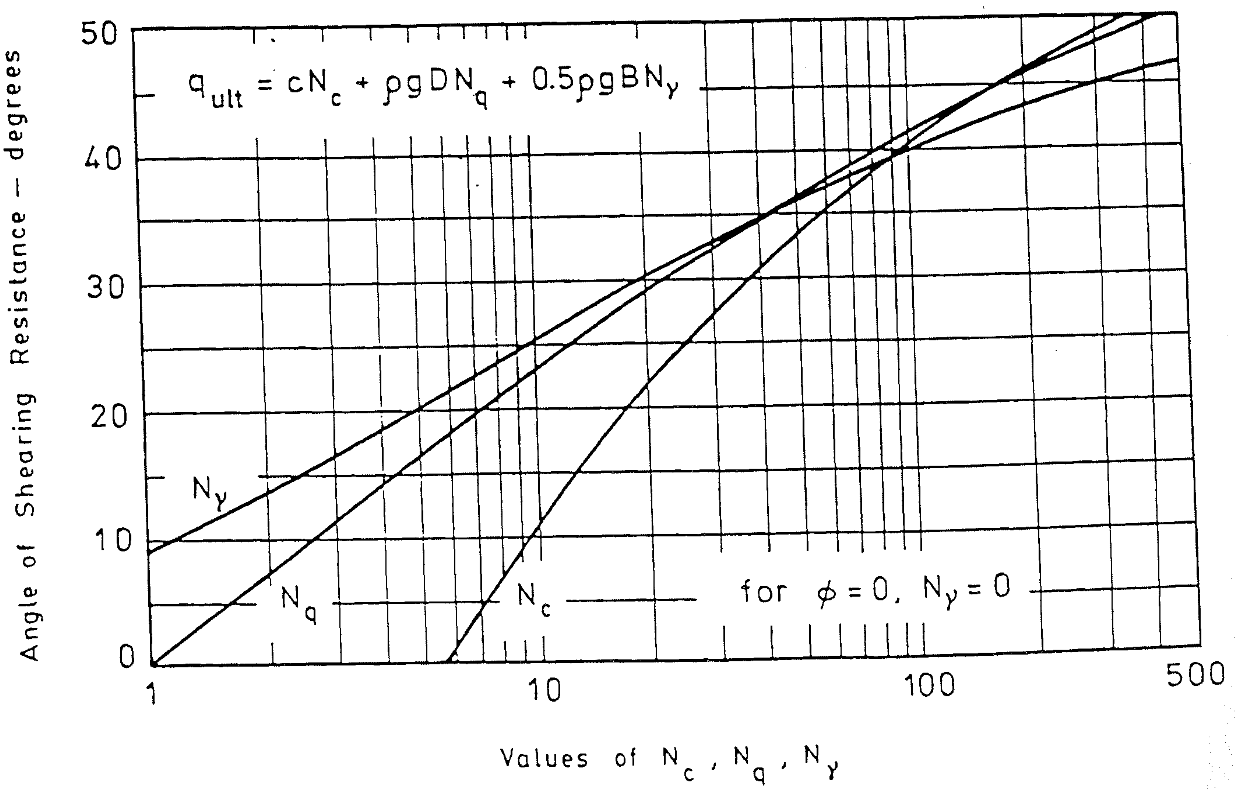

Fig 13.14 Terzaghi Bearing Capacity Factors for a Strip Footing

from considerations of vertical force equilibrium at the underside of the footing, and is usually made up of three components, a cohesion term, a surcharge term and a weight term, as follows

$$q_{ult} = cN_c + \rho g D N_q + 0.5\rho g B N_\gamma \tag{13.21}$$

The dimensionless bearing capacity factors N_c, N_q and N_γ are functions of the friction angle (ϕ), the Terzaghi values being given in Fig. 13.14. Several sets of bearing capacity factors are in current use in addition to the Terzaghi values. Vesic (1973) has identified many theoretical solutions for shallow foundations and a detailed discussion of some of these solutions has been presented by Bowles (1988).

Equation (13.21) may be used with either total or effective stresses. For short term bearing capacity calculations total stresses and undrained strength parameters, c_u and ϕ_u, should be used. For long term conditions effective stresses and drained strength parameters, c_d and ϕ_d, should be used. The density (ρ) in the surcharge term applies to the soil above foundation level, and the ρ in the weight term applies to the soil below foundation level.

The greatest shortcoming of available theories lies in the assumption of incompressibility of the foundation soil. This means that the theories should be applied only to soils which are dense or stiff. Only in this way will the general failure pattern depicted in Fig. 13.13 and referred to as a general shear failure, develop. For softer or looser soils the footing tends to punch into the soil and the general failure pattern shown in that figure does not develop. There is no reliable theory which adequately takes into account the effect of soil compressibility in the calculation of bearing capacity. However, a footing on compressible soil may settle significantly under the effects of the footing loads and it is quite possible that settlement rather than shear failure may become the criterion for the design of the footing.

13.7 EFFECT OF FOOTING SHAPE AND DEPTH ON BEARING CAPACITY

Equation (13.21), which applies only to strip footings, has to be modified when circular, square or rectangular footings are used. This is usually done by multiplying the bearing capacity factors N_c, N_q and N_γ by appropriate semi-empirical shape factors s_c, s_q and s_γ, respectively. There have been many suggestions regarding the magnitude of these shape factors. For example, Terzaghi and Peck (1967) have proposed the following expression for a circular footing of diameter B.

$$q_{ult} = 1.2\, cN_c + \rho g D N_q + 0.3\, \rho g\, B N_\gamma \qquad (13.22)$$

and the following for a square footing of width B

$$q_{ult} = 1.2\, cN_c + \rho g D N_q + 0.4\, \rho g\, B N_\gamma \qquad (13.23)$$

Shape factors for rectangular footings (B x L) according to Meyerhof (1963)

$$s_c = 1 + 0.2\, K_p\, (B/L) \qquad (13.24)$$

$$s_q = s_\gamma = 1 + 0.1\, K_p\ (B/L) \text{ for } \phi > 10^\circ \qquad (13.25)$$

$$s_q = s_\gamma = 1 \quad \text{for } \phi = 0 \qquad (13.26)$$

where $K_P = \tan^2 (45 + \frac{\phi}{2})$

For a strip footing on a saturated clay soil the short term bearing capacity factors may be evaluated for the value of ϕ_u being equal to zero. For this condition the value of N_q is equal to unity and the value of N_γ is zero. Equation (13.21) then simplifies to the following

$$q_{ult} = cN_c + q \qquad (13.27)$$

For square or rectangular footings, Skempton (1951) has proposed the following expression for the bearing capacity factor N_c for the case of $\phi = 0$.

$$N_c\ (\text{rect}) = \left(1 + \frac{B}{5L}\right)\left(1 + \frac{D}{5B}\right) N_c\ (\text{strip}) \qquad (13.28)$$

where B is the width of the footing
L is the length of the footing
D is the depth of the footing below the ground surface.

With this expression which contains both shape and depth factors, the maximum value of the ratio $\frac{D}{B}$ is 2.5 regardless of the actual value of D. Skempton has also proposed that with this expression the bearing capacity factor N_c for a strip footing may be rounded from the Prandtl value of 13.14 to a value of 5. The Skempton values of the bearing capacity factor N_c for circular and square footings are shown in Fig. 13.15.

Various depth factors (d_c, d_q, d_γ) have been proposed and these are used as multipliers for the bearing capacity factors (N_c, N_q, N_γ) in a similar way to the shape factors. The Meyerhof (1963) values, for example, are

$$d_c = 1 + 0.2\sqrt{K_P}\,(D/B) \qquad (13.29)$$

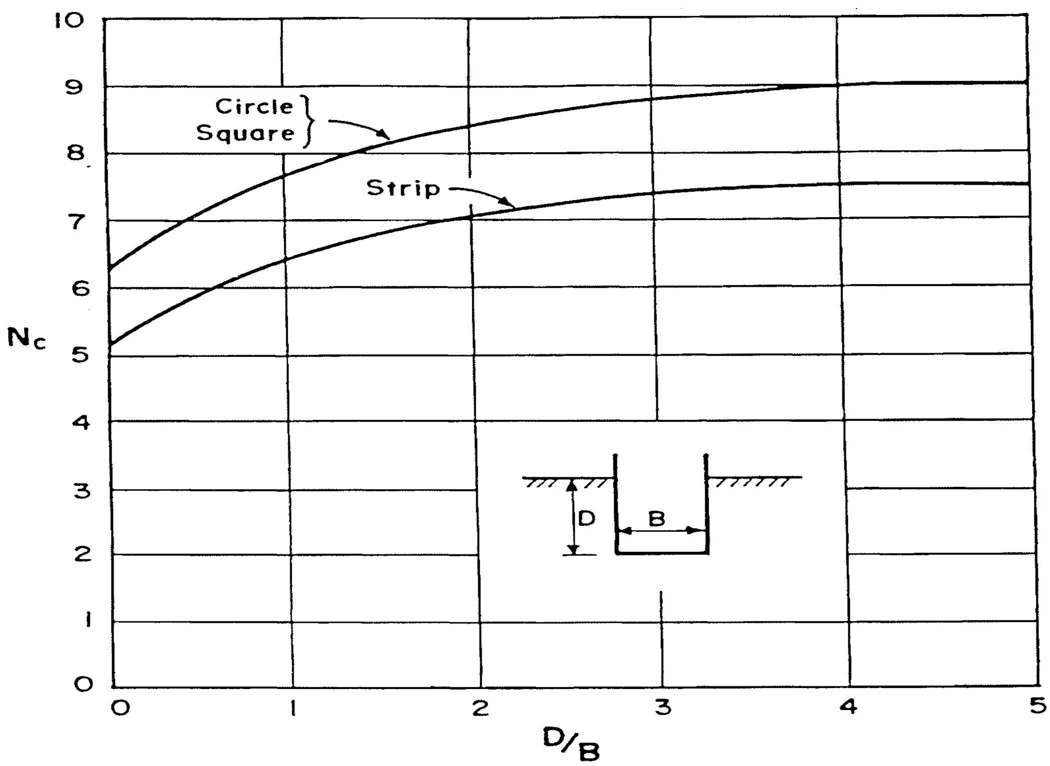

Fig 13.15 Short Term Bearing Capacity Factors for Foundations in Saturated Clay ($Ø_u$ = 0)
(After Skempton, 1951)

$$d_q = d_\gamma = 1 + 0.1\sqrt{K_P}\,(D/B) \quad \text{for } \phi > 10^\circ \tag{13.30}$$

$$d_q = d_\gamma = 1 \quad \text{for } \phi = 0 \tag{13.31}$$

13.8 ALLOWABLE BEARING PRESSURE

For a strip footing (for purposes of discussion) the allowable bearing pressure (q_{all}), which is the pressure used to proportion the footing to support a given load, can be expressed in various ways

(a) as the q_{ult} (eq. (13.19)), often referred to as the gross ultimate bearing capacity, divided by an appropriate factor of safety (F) (Bowles, 1988),

$$q_{all} = q_{ult}/F \tag{13.32}$$

(b) as the net ultimate bearing capacity, which is defined as the gross ultimate bearing capacity less the overburden pressure at foundation level (q), divided by an appropriate factor of safety (Das, 1984),

$$q_{all} = (q_{ult} - q)/F \qquad (13.33)$$

(c) as in (b) but with the addition of the unfactored overburden pressure (Skempton, 1951),

$$q_{all} = (q_{ult} - q)/F + q \qquad (13.34)$$

On logical grounds equation (13.34) is the preferred one to use. In foundation design a value of 3 for F is widely used. For long term conditions where the analysis is carried out in terms of effective stress, Skempton (1951) gives the allowable bearing pressure as

$$q_{all} = \frac{1}{F}\left(c'N_c + q'\,(N_q - 1) + 0.5\,\rho g B N_\gamma\right) + q \qquad (13.35)$$

where q' is the effective overburden pressure at foundation level

q is the total overburden pressure at foundation level

and the bearing capacity factors are determined from the effective friction angle (ϕ').

If the foundation level is beneath the water table then the terms q' and q will differ. The use of q at the end of equation (13.35) allows for uplift for the submergence or partial submergence of the foundation. Also the density (ρ) in the third term of the bearing capacity expression should be the buoyant density.

For short term conditions where the analysis is carried out in terms of total stresses, the allowable bearing pressure for a strip footing on a saturated cohesive soil may be written as

$$q_{all} = \frac{c_u N_c}{F} + q \qquad (13.36)$$

EXAMPLE

Determine the allowable bearing pressure that should be used for design of a square footing 3m square. The footing is to be placed a distance of 2.5m below the surface of a saturated clay soil. The water table is located a distance of 1.0m below the ground surface.

Soil properties:

saturated density		=	$1.9 Mg/m^3$
undrained cohesion	c_u	=	$110 kN/m^2$
undrained friction angle	ϕ_u	=	0°
drained cohesion	c_d	=	$15 kN/m^2$
drained friction angle	ϕ_d	=	36°

For determination of the allowable bearing pressure for short term conditions the undrained strength parameters should be used. A factor of safety of 3 will be used. If the Terzaghi & Peck equation (13.23) is used, the ultimate bearing capacity is

$$q_{ult} = 1.2\, c_u N_c + q N_q + 0.4\, \rho g B N_\gamma$$

for $\phi_u = 0^\circ$ from Fig. 14.16

$$N_\gamma = 0, \quad N_q = 1 \text{ and } N_c = 5.7$$

$$\therefore \quad q_{ult} = 1.2\, c_u\, N_c + q$$

$$q_{all} = \frac{1.2\, c_u N_c}{F} + q$$

$$= \frac{1.2 \times 110 \times 5.7}{3} + 1.9 \times 10 \times 2.5$$

$$= 251 + 48 = 299 \text{ kPa}$$

If, alternatively, the value of N_c was evaluated by means of the Skempton expression (13.28)

$$N_c = 5\left(1 + \frac{B}{5L}\right)\left(1 + \frac{D}{5B}\right)$$

$$= 5\,(1 + 0.2)\left(1 + \frac{2.5}{5 \times 3}\right)$$

$$= 7.0$$

$$\therefore \quad q_{all} = \frac{c_u\, N_c}{F} + q$$

$$= \frac{110 \times 7}{3} + 48$$

$$= 257 + 48 = \underline{305 \text{ kPa}}$$

This value of allowable bearing pressure is only slightly different from the value calculated from equation (13.23). The short term allowable bearing pressure may be taken as approx. 300 kPa.

For long term conditions the calculation should be carried out in terms of effective stresses using the drained strength parameters. For $\phi_d = 36^\circ$ from Fig. 13.14.

$N_c = 63$, $N_q = 47$, $N_\gamma = 51$

If the Meyerhof shape and depth factors are used

$$K_P = \tan^2\left(45 + \frac{\phi}{2}\right) = 3.85$$

$$s_c = 1 + 0.2 \times 3.85 \times 1 = 1.77$$

$$s_q = s_\gamma = 1 + 0.1 \times 3.85 \times 1 = 1.39$$

$$d_c = 1 + 0.2 \times \sqrt{3.85} \times (2.5/3) = 1.33$$

$$d_q = d_\gamma = 1 + 0.1 \times \sqrt{3.85} \times (2.5/3) = 1.16$$

The allowable bearing pressure is given by

$$q_{all} = \frac{1}{F}\left(c_d N_c s_c d_c + q' (Nq - 1) s_q d_q + 0.5 \rho_b g B N_\gamma s_\gamma d_\gamma\right) + q$$

$$= 1/3 (15 \times 63 \times 1.77 \times 1.33 + 33 (47 - 1) \times 1.39 \times 1.16 + 0.5 \times 0.9 \times 10 \times 3 \times 51 \times 1.39 \times 1.16) + 48$$

$$= 1/3 (2225 + 2448 + 1110) + 48$$

$$= 1976 \text{ kPa}$$

Clearly the footing should be designed for the short term allowable bearing pressure of 300kPa.

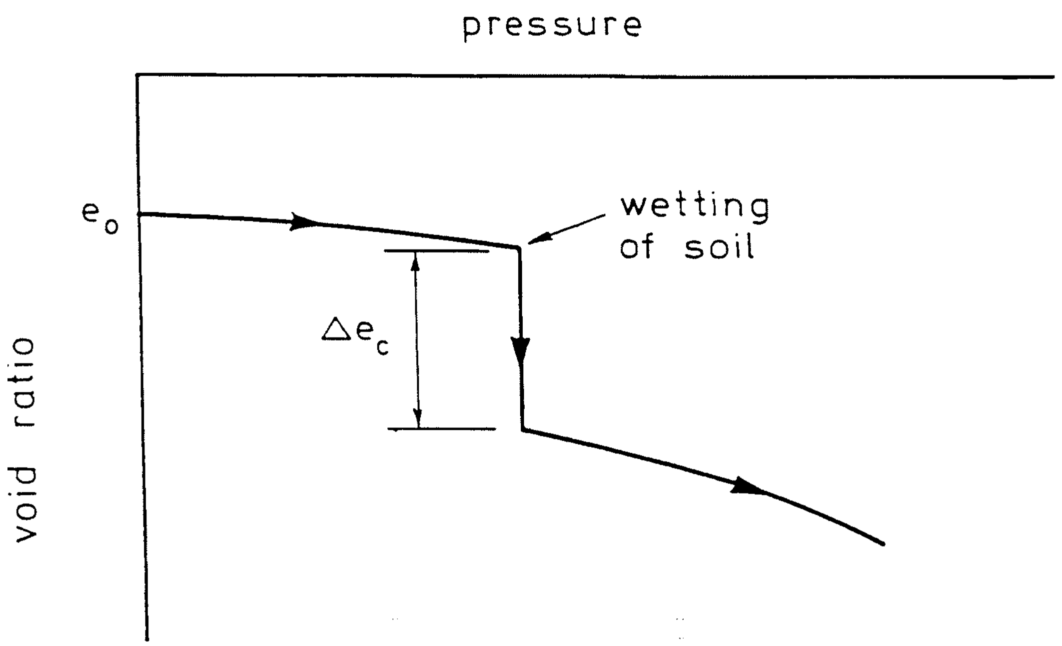

FIGURE 13.16 COLLAPSE OF SOIL AFTER WETTING

13.9 COLLAPSIBLE SOILS

Metastable or collapsible soils are defined as any unsaturated soil that goes through a radical rearrangement of particles and decrease in volume upon wetting with or without additional loading. These soils are generally wind blown (aeolian) deposits in a loose state and often found in arid or semi-arid regions. Typical collapse behaviour is illustrated on the pressure - void ratio plot in Fig. 13.16. As discussed by Clemence and Finbarr (1981) the collapse susceptibility of a soil may be determined by means of the collapse potential (CP) which is defined as

$$CP = \Delta e_c / (1 + e_o) \quad (13.36)$$

where the symbols are given in Fig. 13.16. A guide to the severity of the problem is given in Table 13.6.

TABLE 13.6
COLLAPSE POTENTIAL VALUES
(after Clemence & Finbarr (1981))

CP (%)	Severity of Problem
0 - 1	No problem
1 - 5	Moderate trouble
5 - 10	Trouble
10 - 20	Severe trouble
20	Very severe trouble

13.10 EXPANSIVE SOILS

These soils which undergo volume changes upon wetting and drying have been discussed in Chapter 2. Some considerations which may need to be explored in designing foundations on these soils include

(a) replacing or chemically stablizing the expansive soil beneath the foundation,
(b) control the amount of swelling or shrinking by the use of moisture barriers,
(c) design the foundations and the structures to withstand the ground movements,
(d) use deep foundations extending to a depth below the active zone of movement,
(e) load the soil to a pressure intensity to balance the swell pressure.

13.11 SANITARY LANDFILL

Some of the problems associated with building on sanitary landfill material have been discussed by Sowers (1968). The physical properties of the material are quite difficult to quantify and the use of plate load testing has been found to be very helpful (Moore and Pedler, 1977). Sanitary landfills have been found to undergo large continuous settlements over a long period of time. Some empirical expression for settlement rate have been developed by Yen and Scanlon (1975) based on studies of several landfill sites in California. A more detailed listing of methods of predicting settlement may be found in Oweis and Khera (1990).

13.12 PILE FOUNDATIONS

Pile foundations are commonly divided into two types, end or point bearing piles and friction piles depending upon whether the source of support is at the tip of the pile or is derived from skin friction around the perimeter of the pile. Piles may be used to transfer loads to a stronger stratum, to compact loose sands, to resist lateral forces, to provide foundations below scour level (eg. for a bridge crossing a river) or to provide an economic alternative to surface footings. Piles are commonly less than one meter in diameter and when their diameters are greater than this figure they are often referred to piers. Pile lengths are found to vary from 10 to 60 meters. The loads that piles are called upon to carry usually falls within the range of 200kN to 2000kN.

Many techniques having varying degrees of reliability may be used for the determination of the load carrying capacity of piles. Probably the most reliable technique is by means of a full scale pile load test in which a pile is loaded to failure in the field. This procedure is extremely expensive and would only be seriously considered for large projects. The results of pile load tests are not always conclusive since, as discussed by Chellis (1961) and Fellenius (1980), there are many ways in which the results of these tests may be interpreted.

A much less expensive and, for this reason, more popular method for determination of the ultimate load capacity of a pile (Q_{ult}), is by consideration of the end bearing and skin friction components separately.

$$Q_{ult} = \text{skin friction component} + \text{end bearing component}$$
$$= \pi B \; D \; f_s + A_{tip} \, . \, q_u \qquad (13.37)$$

where the symbols are as given in Fig. 13.17. The skin friction (f_s) is not necessarily constant and may vary considerably over the depth of the pile. In this case the skin friction component would need to be obtained by means of integration over the embedded length of the pile. The ultimate bearing capacity (q_u) for the tip of the pile could be calculated by means of equation (13.22) assuming that the pile is circular is cross-section, and using N_c, N_q and N_γ values appropriate for deep foundation (Fig 13.18).

For a cohesive soil the skin friction (f_s) is equated to the adhesion (c_a) and this is commonly related to the undrained cohesion (c_u) by means of the expression

$$f_s = c_a = \alpha \; c_u \qquad (13.38)$$

where the adhesion factor (α) varies from unity for low c_u values to around 0.3 when c_u is equal to about 200kPa. More information is given in the SAA Piling Code (AS2159) and in Tomlinson

(1986). Building codes may also specify the allowable skin friction values which may be used for design.

With a uniform cohesive soil with a constant value of the skin friction over the length of the pile, the load in the pile is a maximum at the ground surface and decreases linearly with depth as illustrated in Fig. 13.17 (b). The load in the pile does not decrease linearly with depth in the case of a cohesionless soil as shown in part (c) of this figure. The reason for this is that the skin friction (f_s) is not constant over the entire length of the pile but increases with increasing depth.

For a cohesionless soil the skin friction (f_s) at any depth z below the ground surface may be expressed as follows

$$\begin{aligned} f_s &= \sigma_h \tan\delta \\ &= K\sigma_v \tan\delta \\ &= K\rho g z \tan\delta \end{aligned} \qquad (13.39)$$

where σ_h is the horizontal stress acting on the pile surface

σ_v is the vertical stress at a depth z below the ground surface

δ is the angle of friction for the pile surface

K is an earth pressure co-efficient relating the horizontal to the vertical stress.

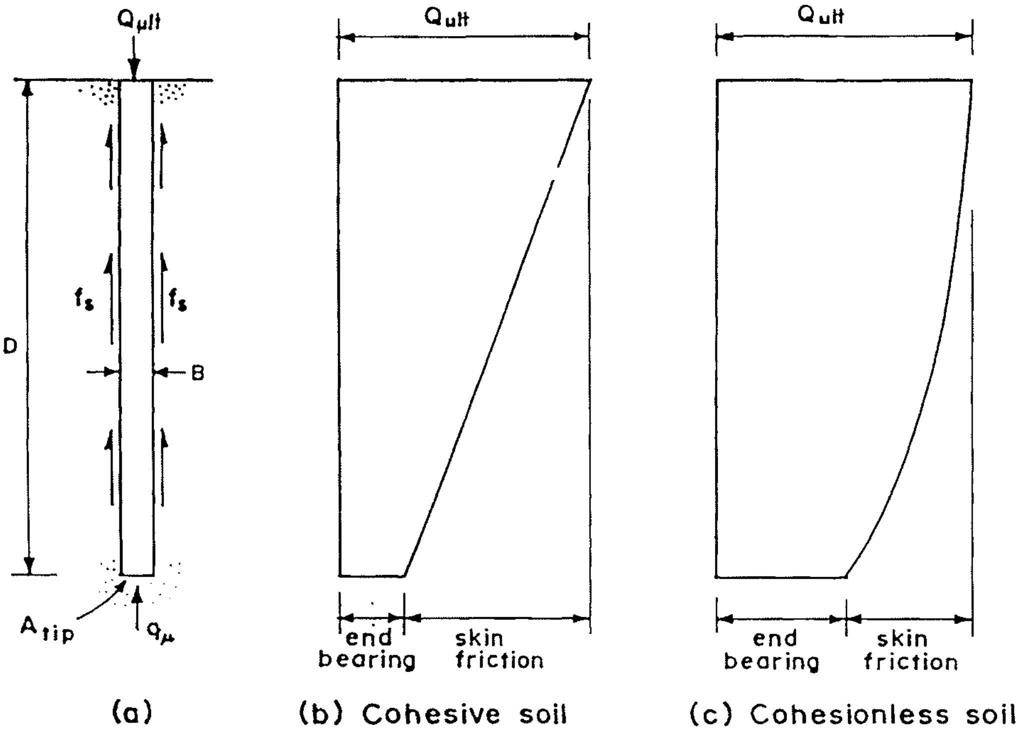

Fig 13.17 Load Distribution in Friction Piles

A number of proposals have been put forward regarding the evaluation of the parameters K and δ in equation (13.39). It has been suggested that the magnitude of the (K tan δ) varies from a value of 0.25 in loose sand to 1.0 in dense sand. Potyondy (1961) has shown that the angle δ may vary from approximately one-half of the angle of shearing resistance of the soil in the case of smooth steel piles in dry sand to a value equal to the angle of sharing resistance of the soil for a rough concrete pile in dry sand. Terzaghi and Peck (1967) have proposed a much simpler approach to the evaluation of skin friction and they have proposed values of 25 kN/m^2 for long piles in loose sand and 100 kN/m^2 for short piles in dense sand. For soil possessing both cohesional and frictional characteristics the skin friction may be evaluated by means of the following

$$f_s = c_a + \sigma_h \tan \delta \qquad (13.40)$$

The load capacity of a pile may also be determined by means of a dynamic formula in which the ultimate supporting capacity of the pile is assumed to be equal to the ultimate driving resistance of the pile with an appropriate allowance for energy loss.

$$WH = RS + \text{energy loss} \qquad (13.41)$$

where the term WH is the energy input in which for a drop hammer, W would be the weight of the ram, H would be the height of fall. R is the driving resistance of the pile which is assumed to be equal to the ultimate load capacity of the pile and S is the distance driven by the final blow.

Most dynamic formulas differ in the ways in which the energy loss is taken into account. The use of dynamic formulas for the calculation of the ultimate load capacity of a pile is not recommended for design although this procedure may be quite useful for construction control purposes.

For point bearing piles the skin friction component is commonly ignored although it is quite possible for a significant amount of the supporting capacity of the pile to be derived from this source. The total supporting capacity of a group of point bearing piles is often taken to be the sum of the individual supporting capacities of the piles making up the group. For a group of friction piles this is not found to be the case unless the pile spacing is very large in comparison with the pile diameter. For a group of friction piles driven into loose sand the group capacity is found to exceed the sum of the individual pile capacities due to the compacting effect of the pile driving on the sand. In the case of soft clay however, the group capacity is found to be less than the sum of the individual capacities.

This ratio of the group supporting capacity to the sum of the individual capacities of piles making up the group is referred to as the efficiency of the group. A number of efficiency formulas have been proposed in an attempt to take this effect into account. Terzaghi and Peck (1967) have proposed that the ultimate supporting capacity of the pile group, Q_g, may be checked by assuming that the pile group behaves as one solid block or pier. They propose that the following expression should be used.

$$Q_g = q_{ult} BL + sD (2B + 2L) \tag{13.42}$$

where B is the width of the pile group

L is the length of the pile group

s is the average shear strength of the soil over the embedded length D of the pile group.

This type of block failure is very rarely associated with sandy soils but is much more commonly experienced with friction piles in clay soils.

Techniques for the determination of settlement of single piles and pile groups where the soil is assumed to be an elastic solid have been presented by Poulos (1968). For the calculation of settlements of pile groups in clay soils a rough procedure which is often used is one in which the group load is assumed to be concentrated either at the location of the pile tips or at a depth equal to two-thirds of the pile lengths.

A detailed presentation of pile foundation behaviour has been given by Poulos (1989).

EXAMPLE

A group of nine timber piles (3 x 3) is driven 10m into a saturated clay stratum. The pile diameter is 0.5m and the pile spacing (centre to centre) in both directions is 1.0m. The undrained cohesion of the clay is 60 kN/m^2. Determine the ultimate load capacity of the pile group.

The capacity of a single pile is

$$Q_{ult} = \pi BDf_s + A_{tip} q_{ult} \tag{13.37}$$

For c_u =60 kPa, the SAA Piling Code suggests an α value of 0.8

$$\therefore \quad f_s = c_a = 0.8 \times 60 = 48 \text{ kPa}$$

The net ultimate bearing capacity of the tip of the pile is

$$q_{ult} = c_u N_c$$

$= \quad 60 \times 9 = 540 \text{ kPa}$

$$\therefore Q_{ult} = \pi \times 0.5 \times 10 \times 48 + \frac{\pi}{4} \times 0.5^2 \times 540$$

$= \quad 754 + 106 = 860 \text{ kN}$

Sum of the ultimate capacities of nine piles

$= \quad 9 \times 860 = 7740\text{kN}$

The ultimate capacity of the pile group assuming that the group behaves as a solid block should now be calculated in accordance with equation (13.42). The group has a square shape in plan with the length of the side of the square being

$= \quad 0.5 + 2 \times 1.0 = 2.5\text{m}$

$$\therefore Q_g = q_{ult} BL + sD (2B + 2L) \qquad (13.42)$$

$= \quad 540 \times 2.5^2 + 60 \times 10 (5 + 5)$

$= \quad 3380 + 6000 = 9380\text{kN}$

Since this calculation yields a capacity greater than that calculated on the basis of the piles being considered individually, the ultimate load capacity of the group is 7740kN.

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