$$Q_{gu} = cN_cA_g + P_gL\bar{c} \qquad (10.4)$$

where, c = cohesive strength of clay beneath the pile group,

$\bar{c}$ = average cohesive strength of clay around the group,

L = length of pile,

P_g = perimeter of pile group,

A_g = sectional Area of group,

N_c = bearing capacity factor which may be assumed as 9 for deep foundation.

Bearing capacity of pile group on the basis of individual pile failure may be written as

$$Q_{gu} = nQ_u \qquad (10.5)$$

where, n = number of piles in the group,

Q_u = bearing capacity of an individual pile.

Terzaghi and Peck recommend that bearing capacity of a pile group may be taken as the smaller of the two given by Eqs (10.4) and (10.5).

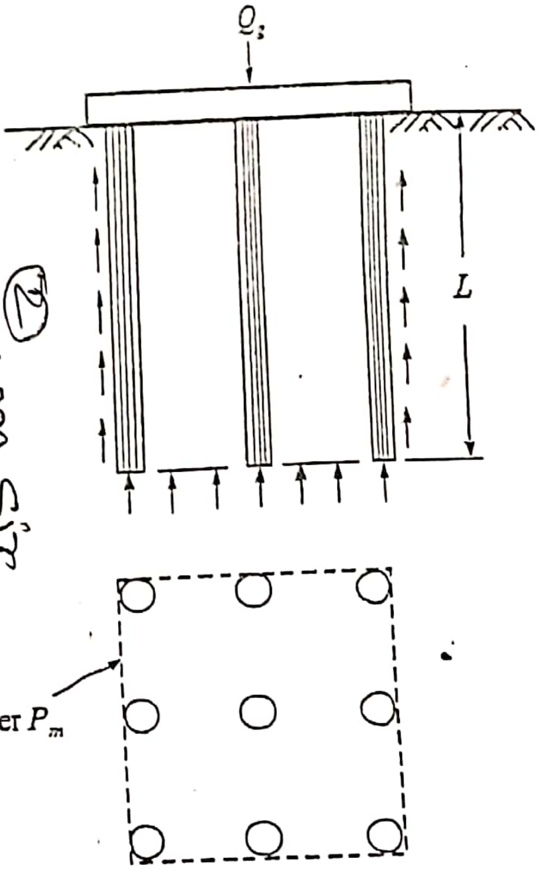

Fig. 10.4 Block failure of a pile group in clay soil

10.9 SETTLEMENT OF PILES AND PILE GROUPS IN SANDS AND GRAVELS

The present knowledge is not sufficient to evaluate the settlements of piles and pile groups. For most engineering structures, the loads to be applied to a pile group will be governed by consideration of consolidation settlement rather than by bearing capacity of the group divided by an arbitrary factor of safety of 2 or 3. It has been found from field observation that the settlement of a pile group is many times the settlement of a single pile at the corresponding working load. The settlement of a group is affected by the shape and size of group, length of piles, method of installation of piles and possibly many other factors.

Vesic has proposed an equation to determine the settlement of a single pile. The equation has been developed on the basis of the experimental results he obtained from tests on piles. Tests on piles of diameters ranging from 2 to 18 inches were carried out in sands of different relative densities D_r. Tests were also carried out on driven piles, jacked piles, and bored piles (jacked piles are those that are pushed into the ground by using a jack). The equation for total settlement of a single pile may be expressed as

$$S = S_p + S_f \qquad (10.6)$$

where, S = total settlement,

S_p = settlement of the pile tip,

S_f = settlement due to the deformation of the pile shaft.

The equation for S_p is

$$S_p = \frac{C_w Q_p}{\left(1 + D_r^2\right) q_{pu}} \tag{10.7}$$

The equation for S_f is

$$S_f = (Q_p + \alpha Q_f)\,\frac{L}{AE} \tag{10.8}$$

where, Q_p = point load,

d = diameter of pile at the base,

q_{pu} = ultimate point resistance per unit area,

D_r = relative density of sand,

C_w = settlement coefficient = 0.04 for driven piles
= 0.05 for jacked piles
= 0.18 for bored piles,

Q_f = friction load,

L = length of pile,

A = cross-sectional area of pile,

E = modulus of deformation of pile shaft,

α = coefficient which depends on the distribution of skin friction along the shaft and can be taken equal to 0.6.

Settlement of piles cannot be predicted accurately by making use of equations such as the one given here. One should use such equations with caution. It is better to rely on load tests for piles in sand.

Settlement of Pile Groups in Sand

The relation between settlement of a group and a single pile at corresponding working loads may be expressed as

$$F_g = \frac{S_g}{S} \tag{10.9}$$

where, F_g = group settlement factor,

S_g = settlement of group,

S = settlement of a single pile.

Vesic has obtained the curve given in Fig. 10.5 which is obtained by plotting F_g against B/d where d is the diameter of the pile and B, the distance between the centre to centre of outer piles in the group (only square pile groups are considered). It should be remembered here that the curve is based on the results obtained from the tests on groups of piles embedded in medium dense sand. It is possible that groups in much looser or much denser deposits might give somewhat different behaviour. Also the group settlement ratio is very likely be affected by the ratio of the pile point settlement S_p to total pile settlement.

Skempton, Yassin and Gibson (1953) have published curves relating F_g with the width of pile groups as shown in Fig. 10.6. These curves can be taken as applying to driven or bored piles.

Since the abscissa for the curve in Fig. 10.6 is not expressed as a ratio, this curve cannot directly be compared with Vesic's curve given in Fig. 10.5. According to Fig. 10.6 a pile group of 3 m wide would settle 5 times that of a single test pile.

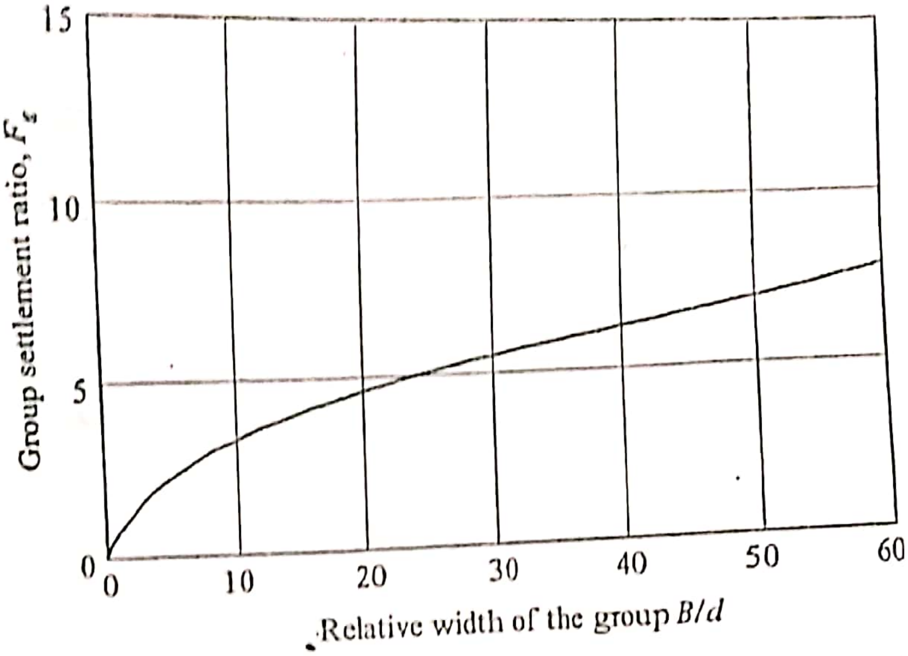

Fig. 10.5 Curve showing the relationship between group settlement ratio and relative widths of pile groups in sand (Vesic, 1967)

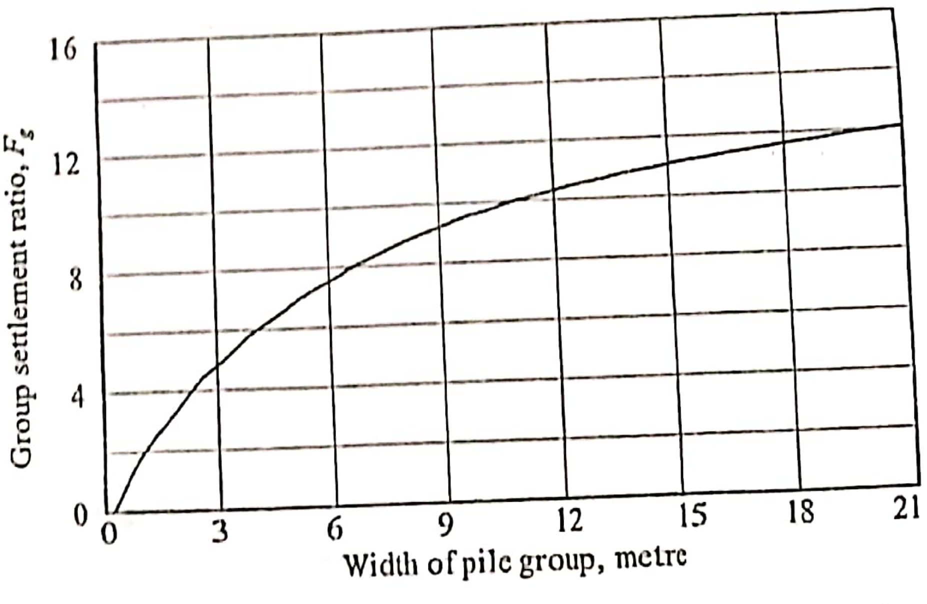

Fig. 10.6 Curve showing relationship between F_g and pile group width (Skempton Yassin and Gibson, 1953)

10.10 SETTLEMENT OF PILE GROUPS IN COHESIVE SOILS

The total settlement of pile groups may be calculated by making use of consolidation settlement equations. The problem involved here is to evaluate the increase in stress Δp beneath a pile group when the group is subjected to a vertical load Q_g. The computation of stresses depends on the type of soil through which the pile passes. The methods of computing the stresses are explained below:

1. The soil in the first group given in (a) of the Fig. 10.7 is homogeneous clay. The load Q_g is assumed to act on a fictitious footing at a depth (2/3)L from the surface and distributed over the sectional area of the group. The load on the pile group acting at this level is assumed to spread out at 2 : 1 slope. The stress Δp at any depth z below the fictitious footing may be found out as explained in Chapter 2.

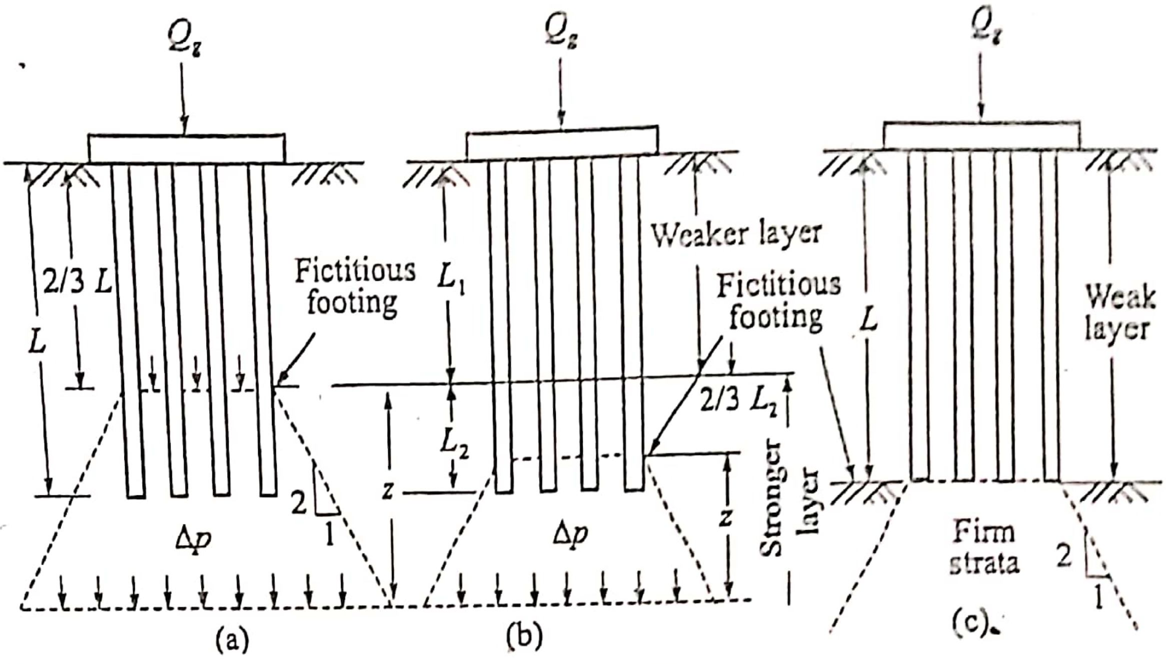

Fig. 10.7 Settlement of pile groups in clay soils

2. In the second group given in (b) of the figure, the pile passes through a very weak layer of depth L_1 and the lower portion of length L_2 is embedded in a strong layer. In this case, the load Q_g is assumed to act at a depth equal to 2/3 L_2 below the surface of the strong layer and spreads at 2 : 1 slope as before.
3. In the third case shown in (c) of the figure, the piles are point bearing piles. The load in this case is assumed to act at the level of the firm strata and spreads out at 2 : 1 slope.

10.11 ALLOWABLE LOADS ON GROUPS OF PILES

The basic criterion governing the design of a pile foundation should be the same as that of a shallow foundation, that is, the settlement of the foundation must not exceed some permissible value. The permissible values of settlements assumed for shollow foundations in Chapter 6 are also applicable to pile foundations. The allowable load on a group of piles should be the least of the values computed on the basis of the following two criterions.

1. Shear failure criterion.
2. Settlement criterion.

Procedures have been given in the earlier chapters as to how to compute allowable loads on the basis of shear failure criterion. The settlement of pile groups should not exceed the permissible limits under these loads.

10.12 NEGATIVE FRICTION ON PILES

Figure 10.8 (a) shows a single pile and (b) a group of piles passing through a recently filled cohesive soil. The soil below the fill had completely consolidated under its own overburden pressure.

When the filled up soils starts consolidating under its own overburden pressure, it develops a drag on the surface of the pile. This drag on the surface of the pile is called as *negative friction.* Negative friction may also be developed if the fill material is loose cohesionless soil. Negative friction can also occur when fill is placed over peat or soft clay strata as shown in Fig. 10.8 (c). The superimposed loading on such compressible strata causes heavy settlement of the fill with consequent drag on piles.

Negative friction may also be developed [illegible] of the ground water which increases the effective stress causing consolidation of the soil [illegible] [illegible]ltant settlement and friction forces being developed on the pile.

Negative friction must be allowed for when considering the fac[illegible] [illegible]ty on the ultimate carrying capacity of pile. The factor of safety, F_s, where negative friction is likely [illegible] [illegible]ur may be written as

$$F_s = \frac{\text{Ultimate carrying capacity of a single or group of piles}}{\text{Working load + Negative skin friction load}}$$

Computation of Negative Friction on Single Piles

The magnitude of negative friction F_n for a single pile in filled up soils may be taken as [Fig. 10.8 (a)].

(*a*) For cohesive soils

$$F_n = PL_n s \tag{10.10}$$

(*b*) For cohesionless soils

$$F_n = \frac{1}{2} PL_n^2 \gamma K \tan \delta \tag{10.11}$$

where, L_n = length of piles in the compressible material,

s = shear strength of cohesive soils in the filled up zone,

P = perimeter of pile,

K = earth pressure coefficient which lies between the active and the passive earth pressure coefficients,

δ = angle of wall friction which may vary from $\phi/2$ to ϕ.

Negative Friction on Pile Groups

When a group of piles passes through compressible filled up soil, the negative friction, F_{ng}, on the group may be found by any of the following methods [Fig. 10.8 (b)].

(*a*) $F_{ng} = nF_n$ (10,12)

(*b*) $F_{ng} = sL_n P_g + \gamma L_n A_g$ (10.13)

where, n = Number of piles in the group,

γ = unit weight of soil within the pile group upto depth L_n,

P_g = perimeter of pile group,

A_g = area of pile group within the perimeter P_g,

s = shear strength of soil along the perimeter of the group.

Equation (10.12) gives the negative friction forces of the group as equal to sum of the friction forces of all the single piles.

Equation (10.13) assumes the possibility of block shear failure along the perimeter of the group which includes the volume of the soil $\gamma L_n A_g$ enclosed in the group. The maximum value from Eqs (10.12) or (10.13) should be used.

When the fill is underlain by a compressible stratum as shown in Fig. 10.8 (c), the total negative friction may be found out as follows:

By suitably anchoring the pile in the ground it may be prevented from being pulled out. Such piles, are called as *anchor piles* or *tension piles*. If long piles of uniform diameter are used as anchor piles, the length should be sufficient to resist the up-lift force with a suitable factor of safety.

For some of the structures under-reamed piles are quite suitable as anchor piles. Anchor piles are normally required for the foundations of structures such as transmission line towers, gas storage tanks, tall chimneys, etc. The tensile forces beneath such structures are normally caused by moment due to wind. Broken wire condition is also responsible for producing high moments on transmission line towers. Hydrostatic pressures on underground structures also develop uplift forces which may have to be resisted by anchor piles.

10.16 UPLIFT CAPACITY OF A PILE GROUP

The uplift capacity of a pile group, when the vertical piles are arranged in a closely spaced groups may not be equal to the sum of the uplift resistances of the individual piles. This is because, at ultimate load conditions, the block of soil enclosed by the pile group gets lifted. The manner in which the load is transferred from the pile to the soil is quite complex. A simplified way of calculating the uplift capacity of a pile group embedded in cohesionless soil is shown in Fig. 10.13 (a). A spread of load of 1 Horiz.: 4 Vert from the pile group base to the ground surface may be taken as the volume of the soil to be lifted by the pile group (Tomlinson, 1977). For simplicity in calculation, the weight of the pile embedded in the ground is assumed to be equal to that of the volume of soil it displaces. If the pile group is partly or fully submerged, the submerged weight of soil below the water table has to be taken.

In the case of cohesive soil, the uplift resistance of the block of soil in undrained shear enclosed by the pile group given in Fig. 10.13 (b) has to be considered. The equation for the total uplift capacity P_{gu} of the group may be expressed by

$$P_{gu} = 2L(\bar{L} + \bar{B})\bar{c}_u + W \tag{10.42}$$

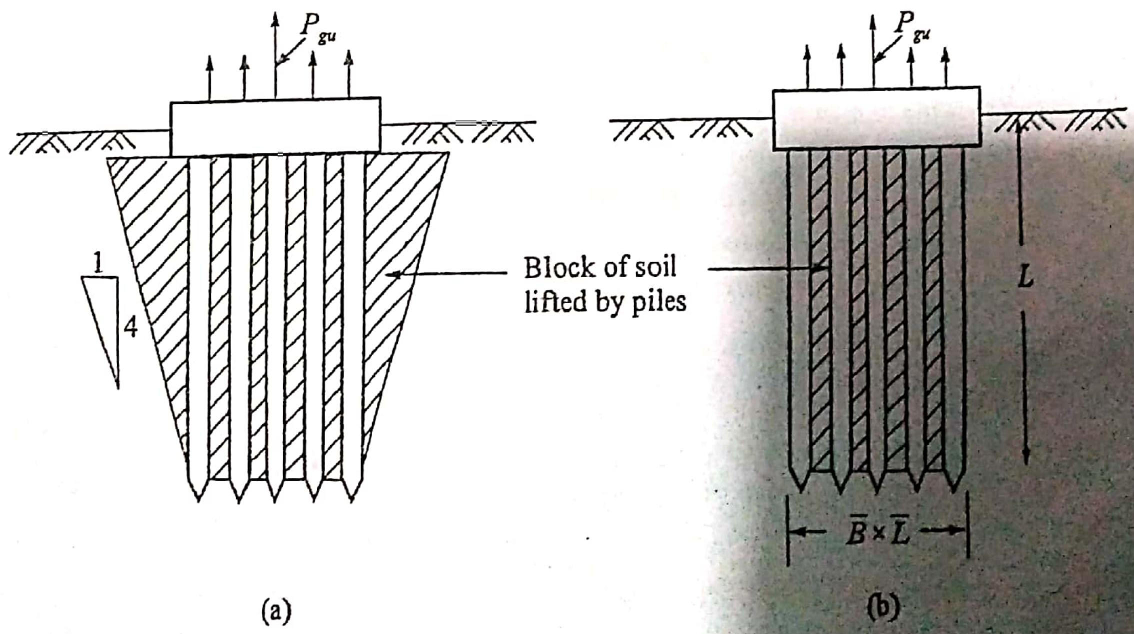

Fig. 10.13 Uplift capacity of a pile group: (a) Uplift of a group of closely-spaced piles in cohesonless soils, (b) uplift of a group of piles in cohesive soils

where, L = depth of the pile block,

$\bar{L}$ and $\bar{B}$ = overall length and width of the pile group,

$\bar{c}_u$ = average undrained shear strength of soil around the sides of the group,

W = combined weight of the block of soil enclosed by the pile group plus the weight of the piles and the pile cap.

A factor of safety of 2 may be used in both cases of piles in sand and clay.

The uplift efficiency E_{gu} of a group of piles may be expressed as

$$E_{gu} = \frac{P_{gu}}{nP_{us}} \tag{10.43}$$

where P_{us} = uplift capacity of a single pile,

n = number of piles in the group.

The efficiency E_{gu} varies with the method of installation of the piles, length and spacing and the type of soil. The available data indicate that E_{gu} increases with the spacing of piles. Meyerhof and Adams (1968) presented some data on uplift efficiency of groups of two and four model circular footings in clay. The results indicate that the uplift efficiency increases with the spacing of the footings or bases and as the depth of embedment decreases, but decreases as the number of footings or bases in the group increases. How far the footings would represent the piles is a debatable point. For uplift loading on pile groups in sand, there appears to be little data from full scale field tests.

10.17 EXAMPLES

Example 10.1

A group of 9 piles with 3 piles in a row were driven into a soft clay extending from ground level to a great depth. The diameter and length of the piles were 30 cm and 10 m respectively. The unconfined compressive strength of clay is 70 kPa. If the piles were spaced at 90 cm centre to centre, compute the allowable load on the pile group on the basis of shear failure criteria for a factor of safety of 2.5.

Solution

Allowable load on the group are to be calculated for two conditions,

(*a*) block failure,

(*b*) individual pile failure.

The least of the two gives the allowable load on the group.

(*a*) *Block failure (Fig. 10.4)*

$$Q_{gu} = cN_c A_g + P_g L\bar{c}$$

$$N_c = 9$$

where, $c = \bar{c} = \frac{70}{2} = 35 \text{ kN/m}^2$,

$$A_g = 2.1 \times 2.1 = 4.4 \text{ m}^2,$$

$$P_g = 4 \times 2.1 = 8.4 \text{ m},$$

$$L = 10 \text{ m},$$

$$Q_{gu} = 35 \times 9 \times 4.4 + 8.4 \times 10 \times 35 = 1390 + 2950$$

$$= 4340 \text{ kN},$$

$$Q_a = \frac{4340}{2.5} = 1740 \text{ kN}.$$

(b) *Individual pile failure*

$$Q_u = Q_b + Q_f = q_b A_b + \alpha \bar{c} A_s$$

Assume $\alpha = 1$, Now $q_b = cN_c = 35 \times 9 = 315$ kN/m^2

$$A_b = 0.07 \text{ m}^2, A_s = 3.14 \times 0.3 \times 10 = 9.42 \text{ m}^2$$

Substituting, $Q_u = 315 \times 0.07 + 1 \times 35 \times 0.42$

$$= 22 + 330 = 352 \text{ kN}$$

$$Q_{gu} = nQ_u = 9 \times 352 = 3168 \text{ kN}$$

$$Q_a = \frac{3168}{2.5} \approx 1267 \text{ kN}$$

The allowable load is 1267 kN.

Example 10.2

A square pile group similar to the one shown in Fig. 10.4 passes through a recently filled up fill. The depth of fill $L_n = 3$ m. The diameter of pile is 30 cm, and they are spaced at 90 cm apart. If the soil is cohesive with $q_u = 60$ kN/m^2, $\gamma = 15$ kN/m^3, compute the negative frictional load on the pile group.

Solution

The negative friction on the group is the maximum of the following

(a) $F_{ng} = nF_n$

(b) $F_{ng} = sL_n P_g + \gamma L_n A_g$

where, $P_g = 4 \times 3 = 12$ m, $A_g = 3 \times 3 = 9$ m^2

(a) $F_{ng} = 9 \times 3.14 \times 0.3 \times 3.0 \times 30 = 763$ kN

(b) $F_{ng} = 30 \times 3 \times 12 + 15 \times 3 \times 9 = 1080 + 405 = 1485$ kN

The negative friction of the group is 1485 kN.

Example 10.3

A tall retaining wall is supported on piles comprising vertical and batter piles as shown in Fig. Ex. 10.3. The piles are rigidly fixed to the foundation. The piles are spaced at 1.0 m apart parallel to the face of the wall. The resultant vertical (V), horizontal (H) forces and moment (M) acting on the foundation per metre length of the wall are shown in the figure. The number of piles per metre length of wall is 5 comprising of 3 batter piles all of equal batter of 18° 26′ and two vertical piles and the spacings are shown on the figure. The piles are reinforced concrete piles of 30 cm diameter driven into very loose sand to a depth of 9 m and rests on rocky strata. The coefficient of modulus variation n_h, is assumed as equal to 2.5 MN/m^3 ($\approx$ 0.25 kg/cm^3).

Lateral displacement

$$y_t = \Delta x \sin\theta_1 - \Delta z \cos\theta_1 - \alpha x_1 \cos\theta_1$$

$$= (-0.394)(0.94869) - (0.0313)(-0.31623) - (0.000042)(-150)(-0.31623)$$

$$= -0.366 \text{ cm.}$$

The displacements of other piles may be calculated in the same way.

5. *Pile loads*

Computation of pile loads for pile number 1

Axial load $Q_g = K_a\delta_a = 157 \times 0.1495 = 23.47$ tonnes.

The pile is in compression

The tranverse load,

$$P_g = -K_t y_t + m_t \alpha$$

$$= -4(-0.366) + 484\,(0.000042)$$

$$= 1.484 \text{ tonnes.}$$

Moment, $M_g = m_t y_t - m_\alpha \alpha$

$$= 484\,(-0.366) - 91916\,(0.000042)$$

$$= 181 \text{ T-cm}$$

The computation of pile loads moments of the other piles may be calculated in the same way.

The example worked out is only illustrative. The displacements of the foundation and the magnitudes of the pile loads depends upon the following factors.

(*a*) The accuracy of the value of n_h.

(*b*) The accuracy of the pile constant.

(*c*) The interaction between piles.

More research work is required in this field.

10.18 PROBLEMS

10.1 A group of nine friction piles arranged in a square pattern is to be proportioned in a depos of medium stiff clay. Assuming that the piles are 30 cm diameter and 10 m long, find th optimum spacing for the piles. Assume $\alpha = 0.8$ and $c_u = 50$ kN/m^2.

10.2 A group of 9 piles with 3 in a row was driven into sand at a site. The diameter and length the piles are 30 cm and 12 m respectively. The properties of the soil are: $\phi = 30$, $e = 0.7$, a $G = 2.64$.

If the spacing of the piles is 90 cm, compute the allowable load on the pile group on the ba of shear failure for $F_s = 2.0$ with respect to skin resistance, and $F_s = 2.5$ with respect to b resistance. For $\phi = 30°$, assume $N_q = 22.5$ and $N_\gamma = 19.7$. The water table is at ground lev

10.3 Nine RCC piles of diameter 30 cm each are driven in a square pattern at 90 cm centre centre to a depth of 12 m into a stratum of loose to medium dense sand. The bottom of pile cap embedding all the piles rests at a depth of 1.5 m below the ground surface. A depth of 15 m lies a clay stratum of thickness 3 m and below which lies sandy strata.

liquid limit of the clay is [illegible]%. The saturated unit weights of sand and clay are 18.5 kN/m^3 and [illegible] kN/m^3 respectively. The initial void ratio of the clay is 0.65. Calculate the consolidation settlement of the pile group under the allowable load. The allowable load Q_a = [illegible] kN

[illegible].4 A square pile group consisting of 16 piles of 40 cm diameter passes through two layers of compressible soils as shown in Fig. 15.32 (c). The thicknesses of the layers are : $L_1 = 2.5$ m and L_2 = 3 m. The piles are spaced at 100 cm centre to centre. The properties of the fill material are: top fill c_u = 25 kN/m^2; the bottom fill (peat), c_u = 30 kN/m^2. Assume γ = 14 kN/m^3 for both the fill materials. Compute the negative frictional load on the pile group.

0.5 9 precast concrete piles of 30 cm diameter were driven three in a row to sandy soil to form a square group. The distance between centre to centre of piles is 75 cm. The piles are connected by a rigid pile cap. The length of pile is 8 m. There is a soft clay strata of 0.5 m thick at a depth of one metre below the tips of the piles (depth to the top edge of the clay strata). Compute the following.

(*a*) The allowable load on the pile group.

(*b*) The settlement of the group at this load.

Given: Water table is at ground level. The submerged unit weight of sand = 8.50 kN/m^3; Average *N*-value of sandy strata = 18; liquid limit of clay soil, w_l = 45%, Initial void ratio of clay soil = 0.83; natural moisture content of clay w_n = 32%.

10.6 Assume that the pile group in Prob. 10.1 comprises both vertical and batter piles as follows.

(*a*) The centre row of three piles are vertical.

(*b*) The adjacent rows are batter piles inclining away from the centre row at a batter of 30°.

The forces acting on the pile cap level are as follows :

(*a*) Vertical load 2500 kN.

(*b*) Horizontal load 1500 kN.

(*c*) Moment = 2500 kN-m.

Assume n_h = 15 MN/m^3. $E = 2.1 \times 10^5$ kg-cm^2.

Compute:

(*a*) The pile cap movements.

(*b*) The pile loads.

iles for variou