Lec-07

Pile Groups

11.1 INTRODUCTION

Chapter 9 has dealt with single vertical piles subjected to vertical loads only and Chapter 10 with the behaviour of single vertical and batter piles subjected to lateral loads only. This chapter deals with the behaviour of pile groups with or without batter piles subjected to vertical / lateral loads.

11.2 NUMBER AND SPACING OF PILES IN A GROUP

Very rarely structures are founded on single piles. Normally, there will be a minimum of three piles under a column or a foundation element because of alignment problems and inadvertent eccentricities. The spacing of piles in a group depends upon many factors such as

1. overlapping of stresses of adjacent piles,
2. cost of foundation,
3. efficiency of pile group.

The pressure isobars of a single pile with load Q acting on the top is shown in Fig. 11.1*a*. When piles are placed in a group, there is a possibility of pressure isobars of adjacent piles overlapping each other as shown in Fig. 11.1*b*. The soil is highly stressed in the zones of overlapping of pressures. With sufficient overlap, either the soil will fail or the pile group will settle excessively since the combined pressure bulb extend to a considerable depth below the base of the piles. It is possible to avoid overlap by installing the piles at considerable distances apart as shown in Fig. 11.1*c*. Large spacings are not recommended sometimes, since this would result in a bigger size of pile cap which would increase the cost of the foundation.

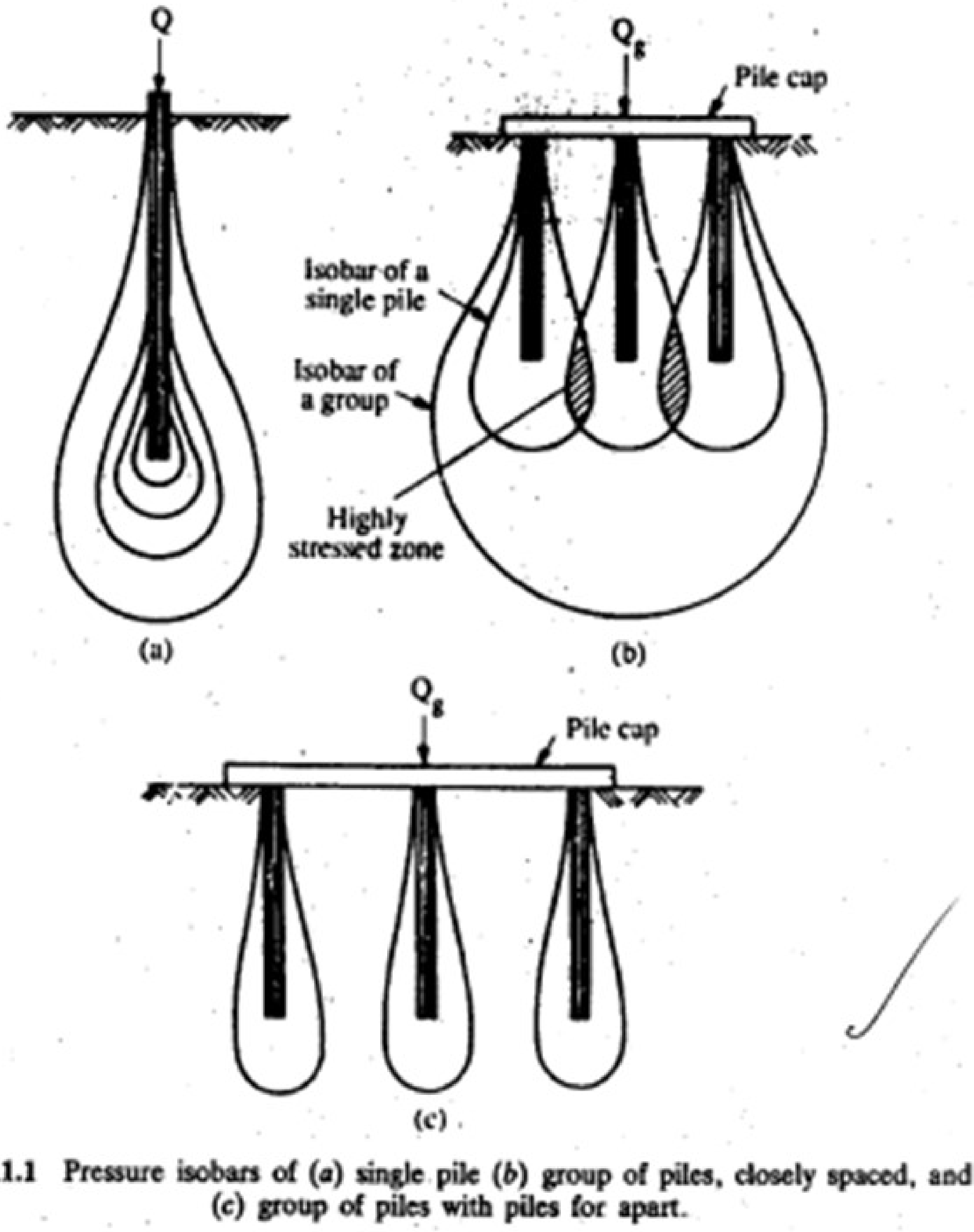


Fig. 11.1 Pressure isobars of (*a*) single pile (*b*) group of piles, closely spaced, and (*c*) group of piles with piles for apart.

The spacing of pile depends upon the method of installing a pile and the type of soil. The piles can be driven piles or *cast-in-situ* piles.

When the piles are driven there will be greater overlapping of stresses due to the displacement of soil. If the displacement of soil compacts the soil in between the piles just as in the case of loose sandy soils, the piles may be placed at closer intervals. But if the piles are driven into saturated clay or silty soils, the displaced soil will not compact the soil between the piles. As a result the soil between the piles may move upwards and in this process lift the pile cup. Greater spacing between piles is required in soils of this type to avoid lifting of piles. When piles are *cast-in-situ*, the soils adjacent to the piles are not stressed to that extent and as such smaller spacings are permitted.

Generally, the spacing for point bearing piles, such as piles founded on rock, can be much less than that friction piles since the high-point-bearing stresses and the superposition effect of overlap of the point stresses will most likely not overstress the underlying material nor cause excessive settlements.

The minimum allowable spacing of piles is usually stipulated in building codes. The spacings for straight uniform diameter piles may vary from 2 to 6 times the diameter of the shaft. For friction piles, the minimum spacing recommended is $3d$ where d is the diameter of the pile. For end bearing piles passing through relatively compressible strata the spacing of piles shall not be less than $2{\cdot}5d$. Whereas for end bearing piles passing through compressible strata and resting in stiff clay, the spacing may preferably be increased to $3{\cdot}5d$. For compaction piles, the spacing may be $2d$. Typical arrangements of piles in groups are shown in Fig. 11.2.

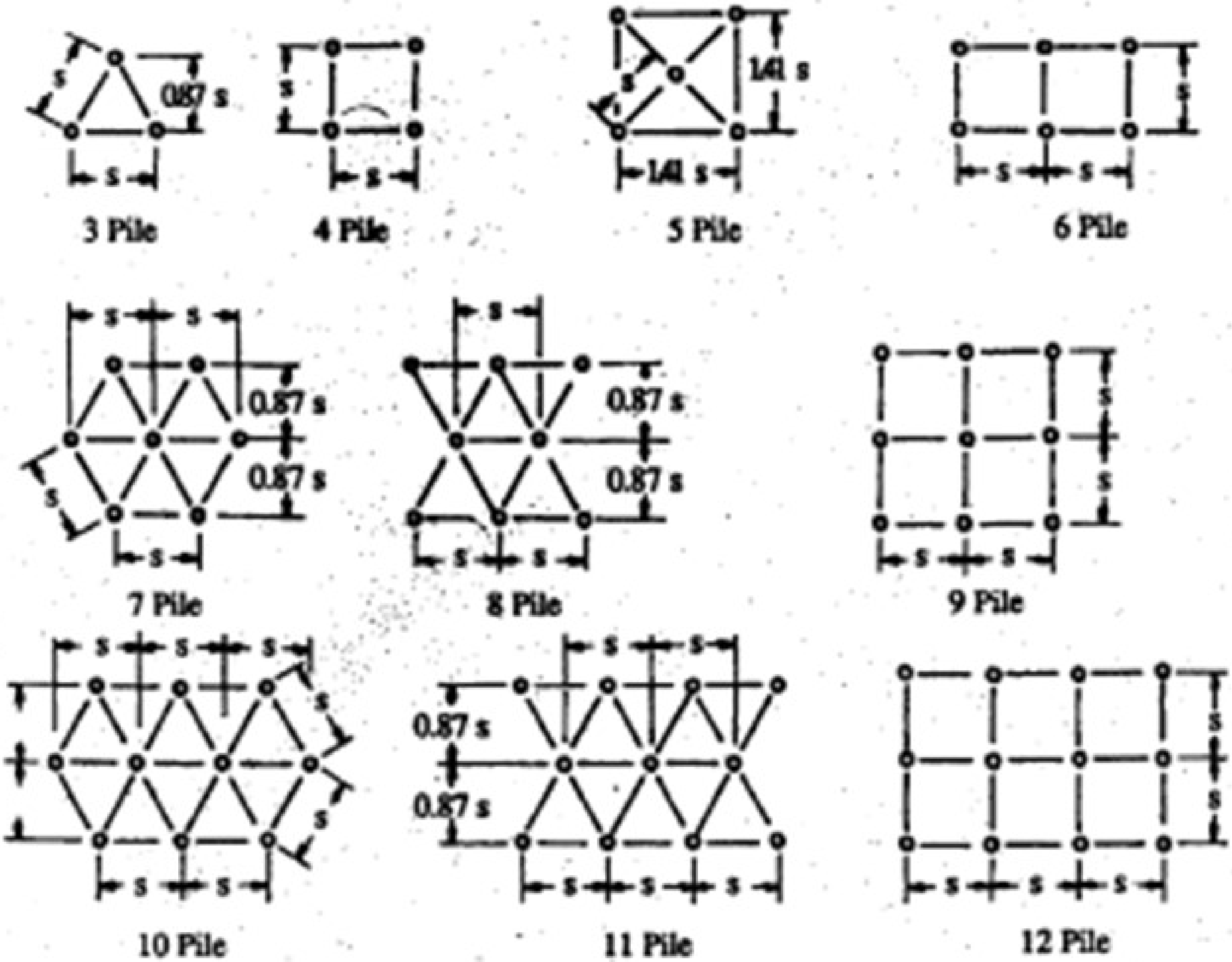


Fig. 11.2 Typical arrangements of piles in groups

11.3 PILE GROUP EFFICIENCY

The spacing of piles is usually predetermined by practical and economical considerations. The design of a pile foundation subjected to vertical loads comprises of :

1. The determination of the ultimate load bearing capacity of the group Q_{gu}.
2. Determination of the settlement of the group, S_g under an allowable load Q_{ga}.

It is well known that the ultimate load of the group is generally different from the sum of the ultimate loads of individual piles Q_u.

The factor

$$E_g = \frac{Q_{gu}}{\Sigma Q_u} \tag{11.1}$$

is called group efficiency which depends on parameters such as

1. type of soil in which the piles are embedded,
2. method of installation of piles *i.e.* either driven or *cast-in-situ* piles, and
3. spacing of piles.

11.4 EFFICIENCY OF PILE GROUPS IN SAND

Vesic (1967) carried out tests on 4 and 9 pile groups driven into sand under controlled conditions. Piles of spacings 2, 3, 4, and 6 times the diameter were used in the tests. The tests were conducted in homogeneous and medium dense sand. His findings are given in Fig. 11.3. The figure gives the following :

1. The efficiencies of 4 and 9 pile groups when the pile caps do not rest on the surface.
2. The efficiencies of 4 and 9 pile groups when the pile caps rest on the surface.
3. The skin efficiency of 4 and 9 pile groups.
4. The average point efficiency of all the pile groups.

It may be mentioned here that a pile group with pile cap resting on the surface takes more load than the one with free standing piles above the surface. In the former case, a part of the load is taken by the soil directly under the cap and the rest is taken by the piles. The pile cap behaves just the same way as a shallow foundation of the same size. Though the percentage of load taken by the group is quite considerable, building codes have not so far considered the contribution made by the cap.

It may be seen from the Fig. 11.3 that the overall efficiency of a four pile group with cap resting on the surface increases to a maximum of about 1·7 at pile spacings of 3 to 4 pile diameters, becoming somewhat lower with the further increase of spacing. A sizable part of the increased bearing capacity comes from the caps. If the loads transmitted by the caps are deduced, the group efficiency drops to a maximum of about 1·3.

Very similar results are indicated from tests with 9 pile groups. Since these tests in this case were carried out only up to spacing of 3 diameter of piles, the full picture of the curve is not available. However, it may be seen that the contribution of the cap for the bearing capacity is relatively smaller.

Vesic measured the skin loads of all the piles. The skin efficiences for both the 4 and 9-pile groups indicate an increasing trend. For the 4-pile group series the efficiency increases from about 1·8 at 2 pile diameter to a maximum of about 3 at 5 piles diameters and beyond. In constrast to this, the average point load efficiency for the groups is about 1·01. Vesic showed for the first time that the increasing bearing capacity of a pile group for piles driven in sand comes primarily from increase in skin loads. The point loads seem to be virtually unaffected by group action.

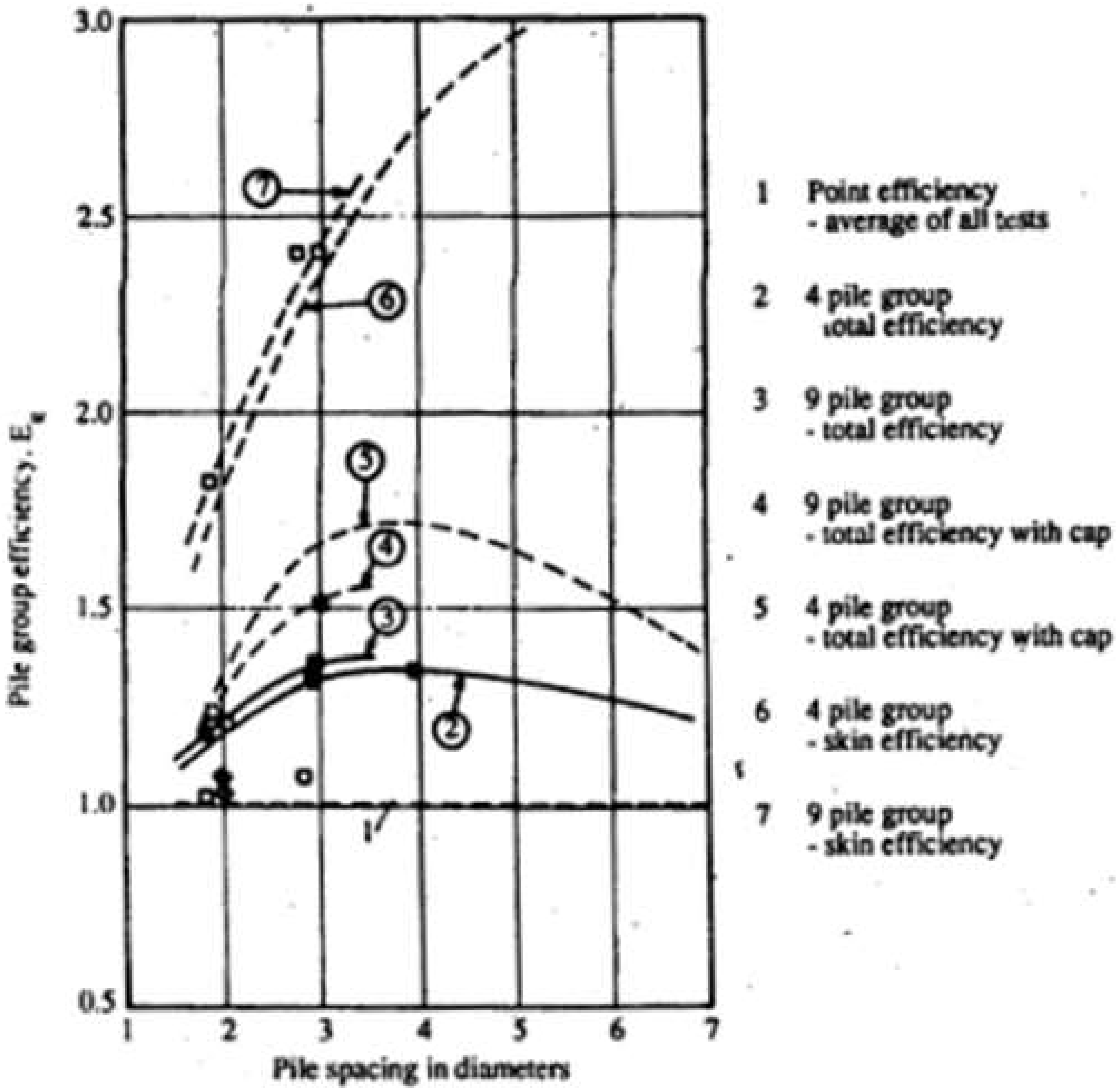


Fig. 11.3 Efficiency of pile groups in sand (Vesic, 1967)

11.6 VERTICAL BEARING CAPACITY OF PILE GROUPS EMBEDDED IN SANDS AND GRAVELS

Driven piles. If piles are driven into loose sands and gravel, the soil around the piles to a radius of at least three times the pile diameter gets compacted. When piles are, therefore, driven in a group at close spacing, the soil around and between them becomes highly compacted. When the group is loaded, the piles and the soil between them move together as a unit. Thus, the pile group acts as a pier foundation having a base area equal to the gross plan area contained by the piles. The efficiency of the pile group will be greater than unity as explained earlier. It is normally assumed that the efficiency falls to unity when the spacing is increased to five or six diameters. Since the present knowledge is not sufficient to evaluate the efficiency for different spacing of piles, it is quite conservative to assume an efficiency factor of unity for all practical purposes. We may, therefore, write

$$Q_{gn} = nQ_u, \tag{11.3}$$

where, n = the number of piles in the group.

The procedure explained above is not applicable if pile tips rest on compressible soil such as silts or clays. When the pile tips rest on compressible soils, the stresses transferred so the compressible soils from the pile group might result in over-stressing or extensive consolidation. The carrying capacity of pile groups under these conditions is governed by the shear strength and compressibility of the soil, rather than by the '*efficiency*' of the group within the sand or gravel structure.

11.7 BORED PILE GROUPS IN SAND AND GRAVEL

Bored piles are *cast-in-situ* concrete piles. The method of installation involves

1. boring a hole of the required diameter and depth,
2. pouring in of concrete.

There will always be a general loosening of the soil during boring and that too when the boring has to be done below water table. Though benotonite slurry (what is sometimes called as *drilling mud*) is used for stabilising the sides and bottom of the bores, loosening of the soil cannot be avoided. Cleaning of the bottom of the bore hole prior to concreting is always a problem which will never be achieved quite satisfactorily. Since bored piles do not compact the soil between the piles, the efficiency factor will never be greater than unity. However for all practical purposes, the efficiency may be taken as unity.

11.8 PILE GROUPS IN COHESIVE SOILS

The effect of driving piles into cohesive soils (clays and silts) is very different from that of cohesionless soils. It has already been explained earlier that when piles are driven into clay soils, particularly when the soil is soft and sensitive, there will be considerable remoulding of the soil. Besides, there will be heaving of the soil between the piles since compaction during driving cannot be achieved in soils of such low permeability. There is every possibility of lifting of the pile also during this process of heaving of the soil. Bored piles are, therefore, preferred to driven piles in cohesive soils. In case driven piles are to be used, the following steps should be kept in view:

1. Piles should be spaced at greater distances apart.
2. Piles should be driven from the centre of the group towards the edges, and
3. the rate of driving of each pile should be adjusted as to minimise the development of porewater pressure.

Experimental results have indicated that when a pile group installed in cohesive soils is loaded, it may fail by any one of the following ways:

1. May fail as a block called as *block failure*.
2. Individual piles in the group may fail.

When piles are spaced at closer intervals, the soil contained between the piles move downward with the piles and at failure, piles and soil move together to give the typical '*block failure*'. Normally this type of failure occurs when piles are placed within 2 to 3 pile diameters. But for wider spacings, the piles fail individually. The efficiency ratio is less than unity at closer spacings and may reach unity at a spacing of about 8 diameters.

The equation for block failure may be written as (Fig. 11.4).

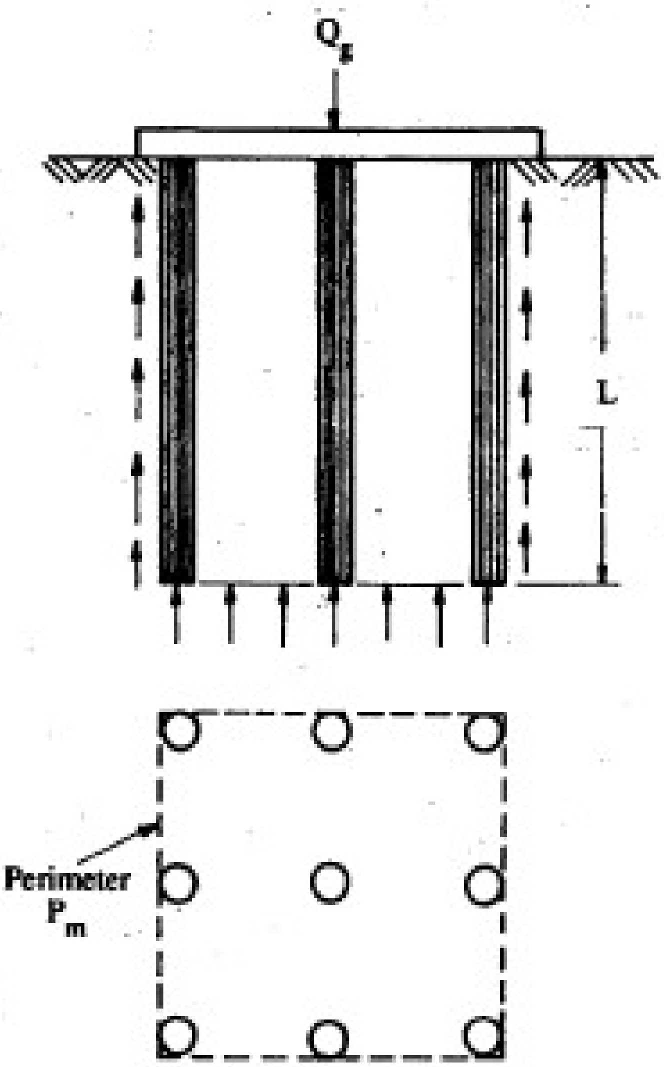


Fig. 11.4 Block failure of a pile group in clay soil

9.15 SETTLEMENT OF A SINGLE PILE

9.15.1 Introduction

Normally it is not necessary to compute the settlement of a single pile as this settlement under working load will be within the tolerable limits. However, settlement analysis of pile group is very much essential. The total settlement of a pile group does not bear any relationship with that of a single pile since in a group the settlement is very much affected due to the interaction of stresses between piles and the stressed zone below the tips of piles.

Settlement analysis of single piles by Poulus (1980) indicate that immediate settlement contributes the major part of the final settlement (which includes the consolidation settlement for saturated clay soils) even for piles in clay. So far piles in sand are concerned, the immediate settlement is almost equal to the final settlement.

However, it may be noted here that consolidation settlement becomes more important for pile groups in saturated clay soils.

Meyerhof (1959) proposed that the settlement of single piles in sand could be estimated from the equation

$$S = \frac{d_b}{30F_s}, \tag{9.56}$$

where, d_b = diameter of pile base,

F_s = factor of safety on ultimate load $Q_u (F_s > 3)$.

In recent years, with the advent of computers, more sophisticated methods of analysis have been developed to predict the settlement and load distribution in a single pile. The following three methods are very much in the news.

1. 'Load transfer' method which is also called as the 't-z' method.
2. Elastic method based on Mindlin's (1936) equations for the effects of subsurface loadings within a semi-infinite mass.
3. The finite element method.

11.9 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

$$S = S_p + S_f, \tag{11.6}$$

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

$$S_p = \frac{C_w Q_p}{(1+D_r^2)\, q_{pu}} \,. \tag{11.7}$$

The equation for S_f is

$$S_f = (Q_p + \alpha Q_f)\frac{L}{AE}, \tag{11.8}$$

where, Q_p = point load,
d = diamter 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

$$F_g = \frac{S_g}{S}, \tag{11.9}$$

where, F_g = group settlement factor,
S_g = settlement of group,
S = settlement of a single pile.

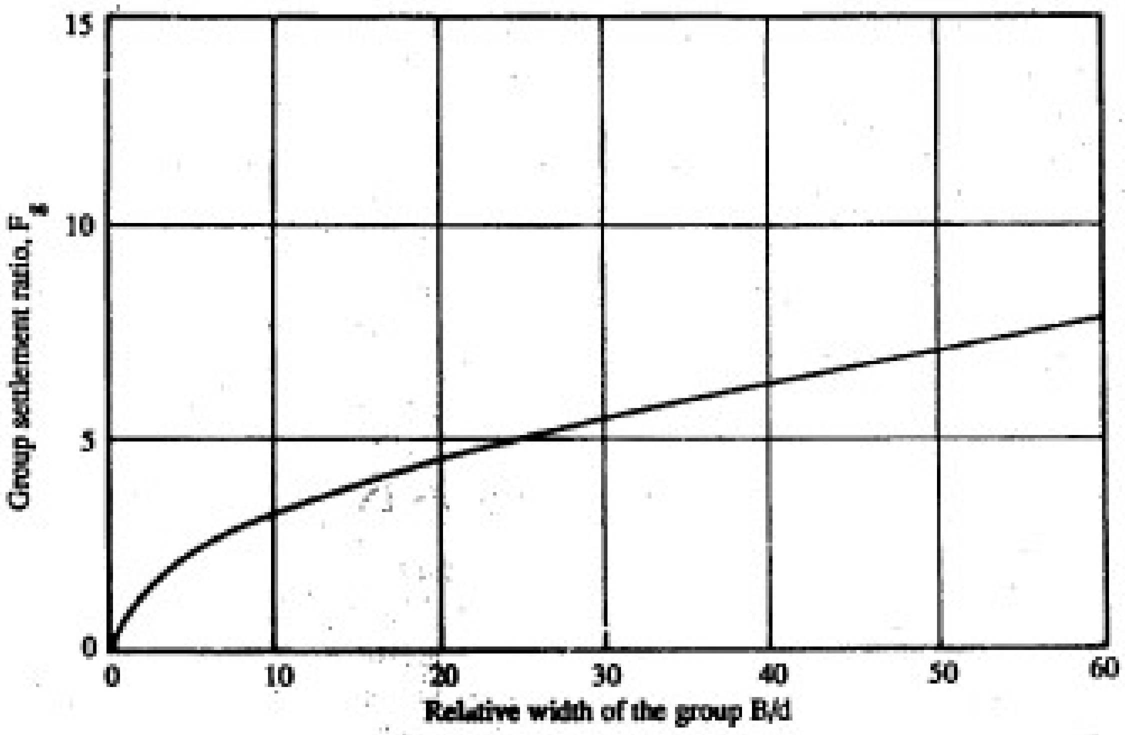


Fig. 11.5 Curve showing the relationship between group settlement ratio and relative widths of pile groups in sand (Vesic, 1967)

Vesic has obtained the curve given in Fig. 11.5 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

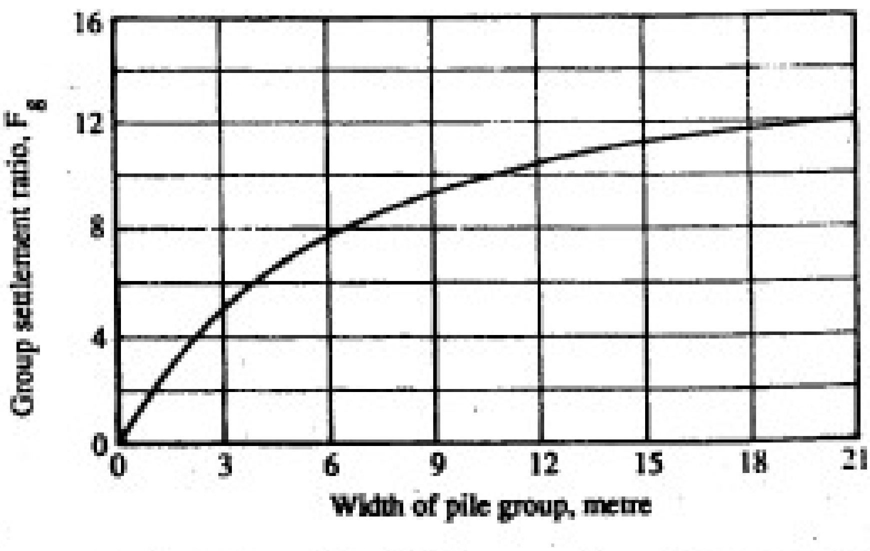


Fig. 11.6 Curve showing relationship between F_g and pile group width (Skempton Yassin and Gibson, 1953)

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. 11.6. These curves can be taken as applying to driven or bored piles.

Since the abscissa for the curve in Fig. 11.6 is not expressed as a ratio, this curve cannot directly be compared with Vesic's curve given in Fig. 11.5. According to Fig. 11.6 a pile group of 3 m wide would settle 5 times that of a single test pile.

11.10 SETTLEMENT OF PILE GROUPS IN COHESIVE SOILS

The total settlements 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. 11.7 is homogeneous clay. The load Q_g is assumed to act on a fictitious footing at a depth 2/3L 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

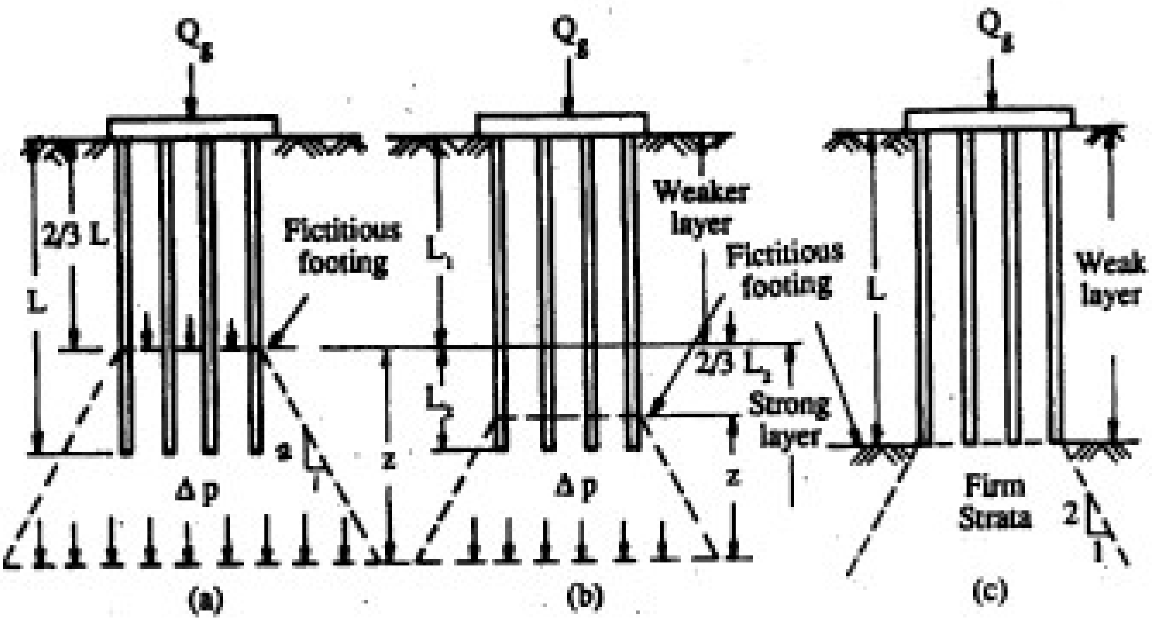


Fig. 11.7 Settlement of pile groups in clay soils

slope. The stress Δp at any depth z below the fictitious footing may be found out as explained in Chapter 2.

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.

11.11 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.

11.12 NEGATIVE FRICTION

Figure 11.8 (*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 '*negetive 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. 11.8(*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 by the lowering of the ground water which increases the effective stress causing consolidation of the soil with the resultant settlement and friction forces being developed on the pile..

Negative friction must be allowed for when considering the factor of safety on the ultimate carrying capacity of pile. The factor of safety, F_s, where negative friction is likely to occur may be written as

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

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. 11.8*a*].

(*a*) For cohesive soils

$$F_n = PL_n s . \tag{11.10}$$

(*b*) For cohesionless soils

$$F_n = \tfrac{1}{2} PL_n^2 \gamma K \tan \delta , \tag{11.11}$$

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. 11.8(*b*)].

Max. of (a) & (b) should be used

(*a*) $$F_{ng} = nF_n , \tag{11.12}$$

(*b*) $$F_{ng} = sL_nP_g + \gamma L_n A_g , \tag{11.13}$$

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

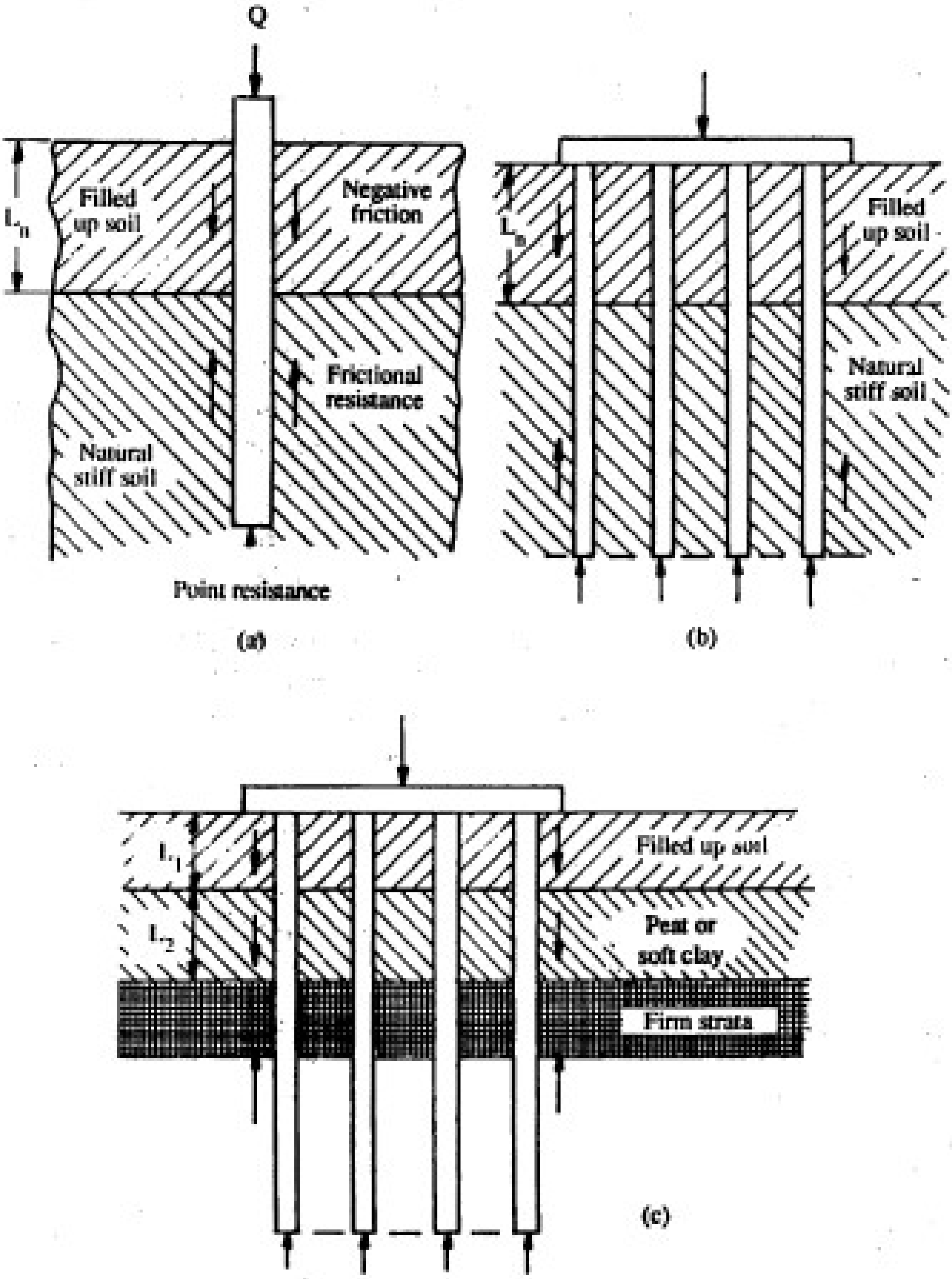


Fig. 11.8 Negative friction on piles

Eq. (11.13) 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. (11.12) or (11.13) should be used.

When the fill is underlain by a compressible stratum as shown in Fig. 11.8*c*, the total negative friction may be found out as follows:

When the fill is underlain by a compressible stratum as shown in Fig. 11.8*c*, the total negative friction may be found out as follows:

$$F_{ng} = n(F_{n1} + F_{n2}), \tag{11.14}$$

$$F_{ng} = s_1L_1P_g + s_2L_2P_g + \gamma_1L_2A_g + \gamma L_2A_g$$
$$= P_g(s_1L_1 + s_2L_2) + A_g(\gamma_1 L_1 + \gamma_2L_2), \tag{11.15}$$

wherein, L_1 = depth of filled up soil,

L_2 = depth of compressible natural soil,

s_1, s_2 = shear strengths of the fill and compressible soils respectively,

γ_1, γ_2 = unit weights of fill and compressible soils respectively,

F_{n1} = negative friction of a single pile in the fill,

F_{n2} = negative friction of a single pile in the compressible soil.

The maximum value of the negative friction obtained from Eqs. (11.14) or (11.15) should be used for the design of pile groups.

Lecture-08

11.14 PILE GROUPS SUBJECTED TO ECCENTRIC VERTICAL LOADS

The reactions exerted by piles in a group when it is subjected to direct vertical load and moments only may be determined on the following assumptions:

1. The pile cap is a rigid structure.
2. When the pile group is subjected to moment, the reactions exerted by the piles increase linearly with the distance of pile from the centre of gravity of pile group.
3. The resisting moment at pile heads due to the fixity condition between piles and the cap is either negligible or ignored.

Though the above assumptions are not strictly valid, it is considered sufficiently accurate for the purpose of design.

Consider the group of piles shown in Fig. 11.12*a*. O is the centroid of the group. XX and YY are the coordinate axes passing through O. The positive directions of the axes are shown by the arrows. O_x and O_y are points on the X and Y axes with eccentricities e_x and e_y respectively as shown in the figure. O_{xy} is another point with coordinates e_x and e_y.

When a vertical load, V, passes through, O, the centroid of the pile group, there will be no moment on the group and V is the vertical load. The reactions of all the piles, in such a case, are equal to each other and is equal to V/n where n is the number of piles in the group. However, when the load V passes through O_x, an eccentric condition develops with one way eccentricity. The pile group is then considered to have been subjected to the vertical load, V, passing through, O, and a moment $M_y = Ve_x$ about the Y-axis as shown in (*b*) of the figure. If the load passes through O_y instead of O_x similar condition as above develops, but in this case the moment $M_x = Ve_y$ is about the X-axis as shown in (*c*) of the figure.

When the load V passes through O_{xy}, there will be two way eccentricity. This condition is equivatent to the vertical load, V, passing through O and moments M_x and M_y acting simultaneously about the axes XX and YY respectively.

The reaction developed at pile heads due to V passing through O is equal to V/n and is as shown in (*d*) of the figure. When only moment M_y (or M_x) acts on the pile group, the reaction due to this at any pile head is assumed to vary linearly as shown in (*e*) of the figure. The combined reactions due to V and moment M_y are as shown in figure (*f*). If the pile group is subjected to a vertical load with one way eccentricity e_x, the total reaction R at the pile head may be obtained by the equation

$$R = \frac{V}{n} \pm \frac{M_y x}{\Sigma x^2}. \tag{11.37}$$

But, if the pile group is subjected to the vertical load V with two way eccentricity, the general equation for determining, R, is

$$R = \frac{V}{n} \pm \frac{M_y x}{\Sigma x^2} \pm \frac{M_x y}{\Sigma y^2}, \tag{11.38}$$

where, R = total reaction at the pile head,
V = total vertical load acting on the pile cap,
n = number of piles in the group,
M_x = total moment about X-axis = Ve_y,
M_y = total moment about Y-axis = Ve_x,
x = distance of the pile in question from the Y-axis,
y = distance of the pile in question from the X-axis,
Σx^2 = sum of all the squares of all the piles from the X-axis,
Σy^2 = sum of all the squares of the distances of all the piles from the X-axis.

Inspection of Eq. (11.38) indicates that this form is similar to Eq. (7.1*b*) which gives soil pressure at a given point under the base of a shallow foundation with two-way eccentric loading. The number of piles n is substituted for area, the terms Σx^2 and Σy^2 replaces the moments of inertia of the area about YY and XX axes respectively. For this reason Σx^2 or Σy^2 is sometimes called as the moment of inertia of the group of piles. The second or the third term in the Eq. (11.38) may be derived as follows.

Assume the pile group in Fig. (11.12*a*) is subjected to a moment M_y only. The reactions developed at the pile heads give rise to a resisting moment which is equal to the applied moment M_y. Let R_1, R_2, R_3 and R_4 be the reactions of the piles placed at distances of x_1, x_2, x_3 and x_4 respectively from the Y-axis [Fig. 11.12(*e*)]. We may write,

$$M_y = (4R_1x_1 + 4R_2x_2 + 4R_3x_3 + 4R_4x_4), \tag{11.39}$$

Since the variation of pile reactions is assumed to be linear, we have

$$R_1/x_1 = R_2/x_2 = R_3/x_3 = R_4/x_4$$

or $$R_2 = R_1x_2/x_1,$$

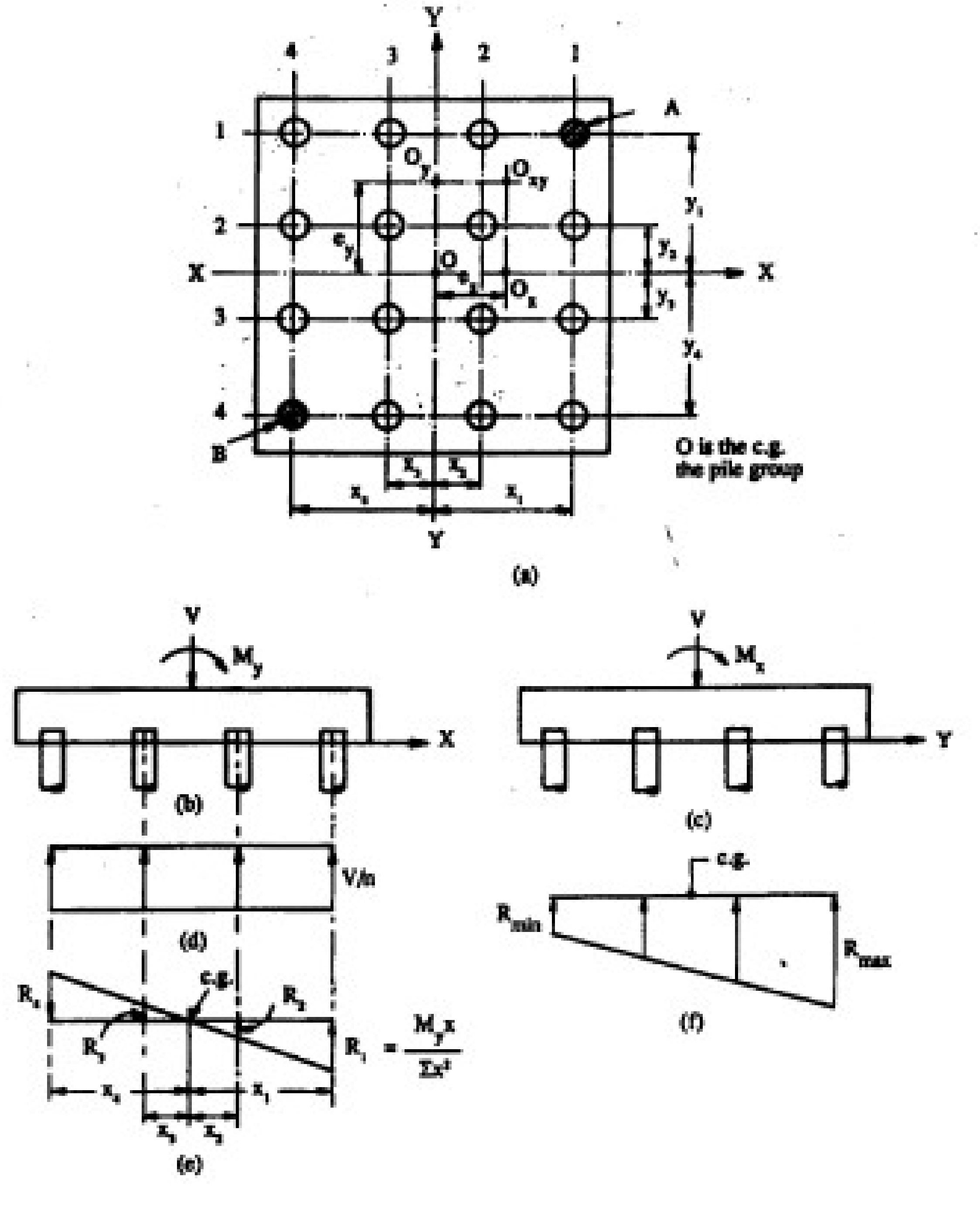


Fig. 11.12 Pile group subjected to eccentric vertical loads

$$R_3 = R_1 x_3 / x_1,$$

$$R_4 = R_1 x_4 / x_1.$$

Substituting these values of R_2, R_3 and R_4 in Eq. (11.39), we have

$$M_y = [4R_1 x_1^2/x_1 + 4R_1 x_2^2/x_1 + 4R_1 x_3^2/x_1 + 4R_1 x_4^2/x_1]$$

$$= \frac{R_1}{x_1} [4x_1^2 + 4x_2^2 + 4x_3^2 + 4x_4^2]$$

$$= \frac{R_1}{x_1} \Sigma x^2 .$$

Solving for R_1, we have,

$$R_1 = \frac{M_y x_1}{\Sigma x^2} . \tag{11.40}$$

Similarly, the reaction at any other pile head may be determined means of Eq. (11.40) by replacing x_1 by the distance of the pile from the y-axis.

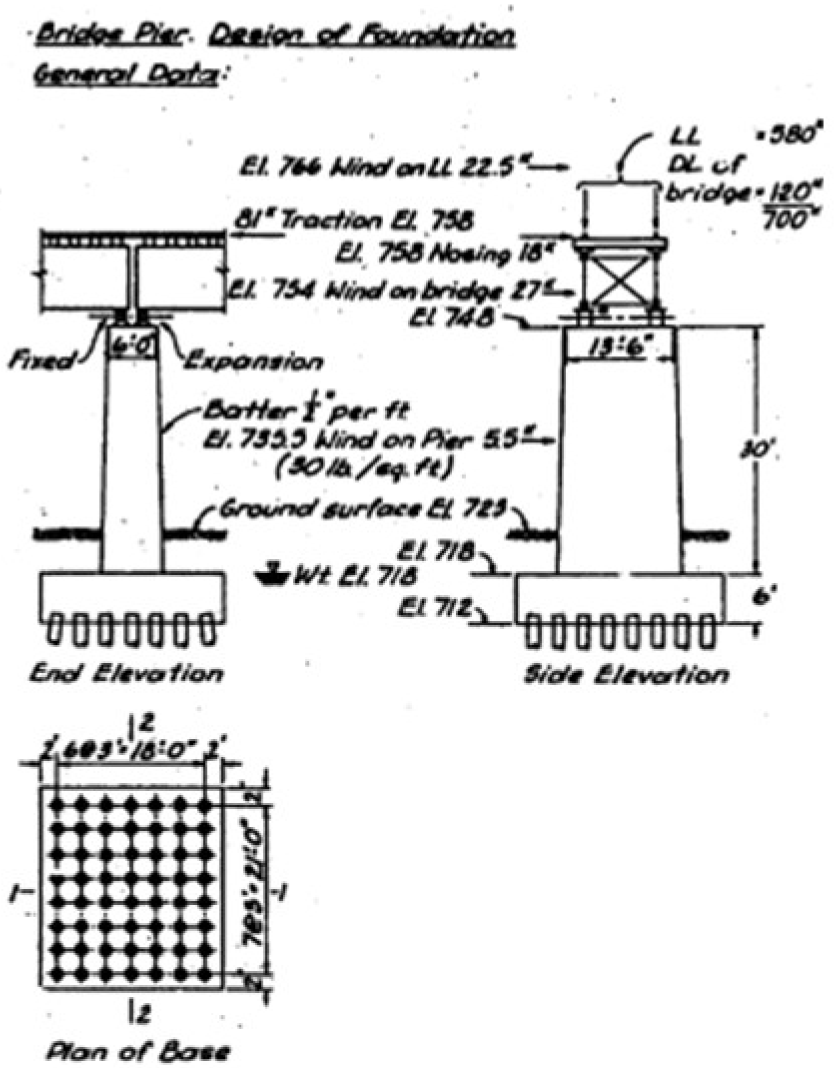
If the pile group in Fig. 11.12*a* is subjected to a vertical load passing through O_x only, then all the piles in Col. 1 will carry the maximum load and all the piles in Col. 4 will carry the minimum load. However, if the load V passes through O_{xy}, the pile A will carry the greatest load whereas pile B carries the least. Both M_x and M_y increases the reaction at A and decreases that at B. Thus, it is possible to select by inspection, the proper signs in the application of Eq. (11.38) to any pile.

The determination Σx^2 or Σy^2 for large groups of pile may be considerably simplified by the use of Eq. (11.41) which applies to a single row of piles with equal spacing.

$$\Sigma x^2 \text{ (one row)} = \frac{s^2}{12} n_1 (n_1^2 - 1), \tag{11.41}$$

where, s = spacings of piles in the row,

n_1 = number of piles in the row.



V =1724 k $\quad$ $M_{x\text{-}x}$ = 3303 k-ft $\quad$ $M_{y\text{-}y}$ = 3726 k-ft

Here n = 56

$\Sigma x^2 = \frac{3^2}{12} \times 7 \times (7^2 - 1) \times 8 = 2016$ sq.ft.

$\Sigma y^2 = \frac{3^2}{12} \times 8 \times (8^2 - 1) \times 7 = 2646$ sq.ft.

$R = \frac{V}{n} \pm \frac{M_y x}{\sum x^2} \pm \frac{M_x y}{\sum y^2}$ =30.79±16.63±13.11

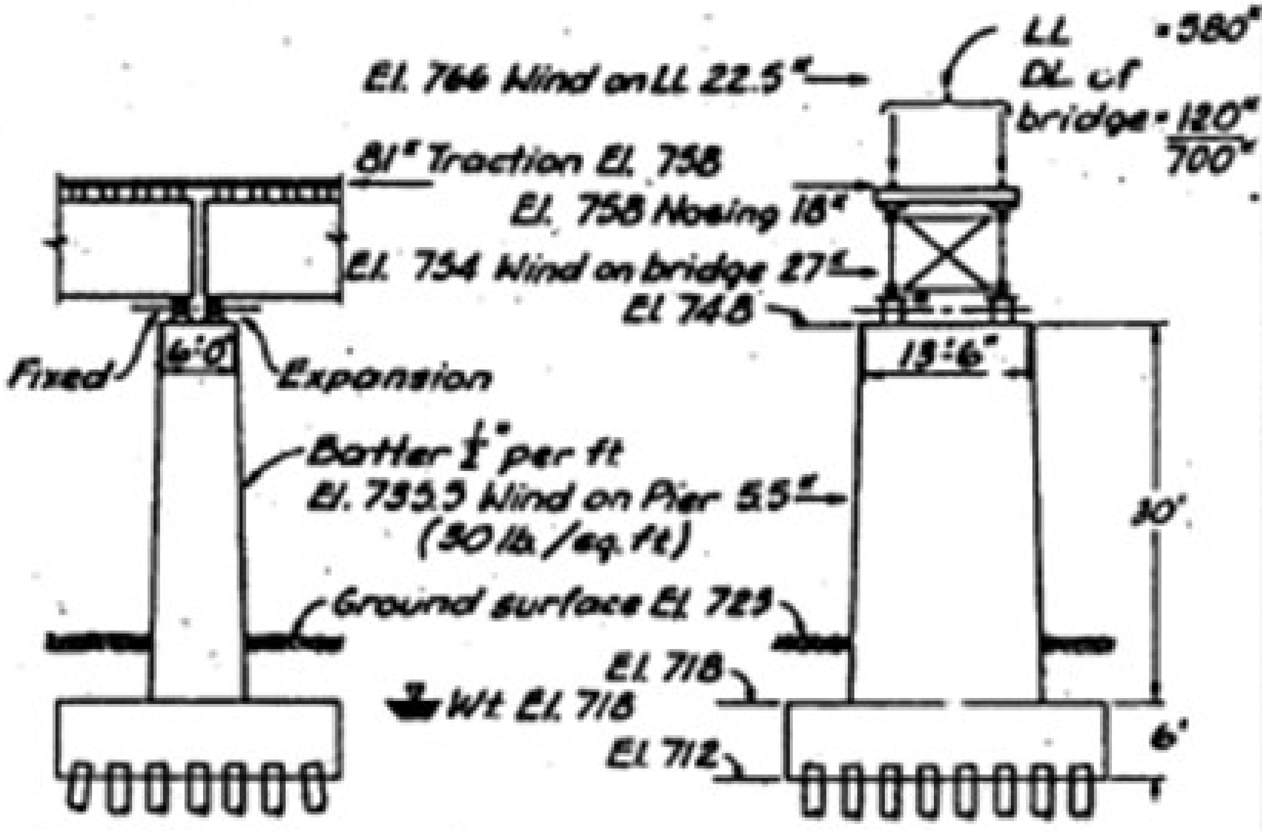
Max pile force =30.79 + 16.63 + 13.11 = 60.53 k

Min pile force =30.79 - 16.63 - 13.11 = 1.05 k

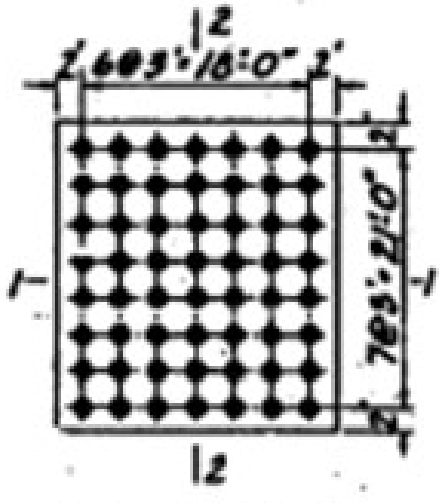
DP 2K-2
Bridge Pier Foundation
Sh. 1 of 3

Bridge Pier. Design of Foundation

General Data:



End Elevation Side Elevation



Plan of Base

56 Piles

Σd^2 = 7 rows

$\times \frac{3^2}{12}(8)(8^2-1)$ = 2646 pile ft.^{2}

Σd_2^2 = 8 rows

$\times \frac{3^2}{12}(7)(7^2-1)$ = 2016 pile ft.^{2}

Section Moduli:

Axis 1-1 = 2646 ÷ 10.5 = 252 pile ft

Axis 2-2 = 2016 ÷ 9.0 = 224 pile ft

Bridge Pier. Design of Foundation

Pile Reactions Due to Vertical Loads:

Shaft: Top area = 13.5 × 6 = 81.00 sq. ft
Mid " = 4 × 14.75 × 7.25 ÷ 4 = 427.75
Bott. " = 16.0 × 8.5 = 136.00
644.75 sq. ft
× 30/6 = 3224 cu. ft

3224 cu. ft × 0.15 = 484^k

Footing: 22 × 25 × 6 = 3300 cu. ft
× 0.15 = 495^k

Earth: 22 × 25 = 550 sq. ft
15.8 × 8.3 = 131
419 sq. ft
× 5 = 2095 cu. ft
× 0.12 = 251^k

DL (Superstructure): 120^k
1350^k

Buoyancy: 22 × 25 × 6 = 3300 cu. ft
× 0.0625 = 206^k
1144^k = total DL

LL: 580^k
1724^k = total DL + LL
÷ 56 = 30.8^k/pile

Pile Reactions Due to Moment:

Traction: 81^k × 46' = 3726^k'
÷ 224 = 16.6^k/pile

Transverse wind: On bridge 27^k × 42' = 1134^k'
÷ 252 = 4.5^k/pile
On LL 22.5^k × 54' = 1212^k'
÷ 252 = 4.8
On end of shaft
5.3^k × 23.5' = 124^k'
÷ 252 = 0.5
} 9.8^k/pile

Nosing: 18^k × 46' = 828^k'
÷ 252 = 3.3^k/pile

} 29.7^k/pile

Maximum pile reaction: 60.5^k/pile

Minimum pile reaction: 30.8^k − 29.7^k = 1.1^k/pile

Bridge Pier. Design of Foundation

Soil Pressure if Piles are Omitted:

Base 22' × 25'   Area = 550 sq. ft

Section Moduli: Axis 1-1 = $\frac{1}{6}(22)(25)^2 = 2290\ ft^3$

Axis 2-2 = $\frac{1}{6}(25)(22)^2 = 2020\ ft^3$

Vertical load on base = 1724^k

Moments on Base:

$1134^{'k}$
1212
129
826

$M_{1-1} = 3303^{'k}$ $M_{2-2} = 3726^{'k}$

Maximum soil pressure: $\frac{1724}{550} + \frac{3303}{2290} + \frac{3726}{2020} = 6.4^k/sq.ft.$ *

Minimum soil pressure: $3.13 - 1.44 - 1.84 = -.015^k/sq.ft.$ *

* Since the minimum soil pressure is negative indicating a small tension, the maximum soil pressure should be computed by other methods described in Art. 24.4. However, in this instance, greater accuracy is not warranted.