Example 11.11

Consider a 20-m-long steel pile driven by a Bodine Resonant Driver (Section HP 310 × 125) in a medium dense sand. If $H_p = 350$ horsepower, $v_p = 0.0016$ m/s, and $f = 115$ Hz, calculate the ultimate pile capacity, Q_u.

Solution

From Eq. (11.122),

$$Q_u = \frac{0.746H_p + 98v_p}{v_p + S_L f}$$

For an HP pile in medium dense sand, $S_L \approx 0.762 \times 10^{-3}$ m/cycle. So

$$Q_u = \frac{(0.746)(350) + (98)(0.0016)}{0.0016 + (0.762 \times 10^{-3})(115)} = \mathbf{2928\ kN}$$ ■

11.23 Negative Skin Friction

Negative skin friction is a downward drag force exerted on a pile by the soil surrounding it. Such a force can exist under the following conditions, among others:

1. If a fill of clay soil is placed over a granular soil layer into which a pile is driven, the fill will gradually consolidate. The consolidation process will exert a downward drag force on the pile (see Figure 11.42a) during the period of consolidation.
2. If a fill of granular soil is placed over a layer of soft clay, as shown in Figure 11.42b, it will induce the process of consolidation in the clay layer and thus exert a downward drag on the pile.

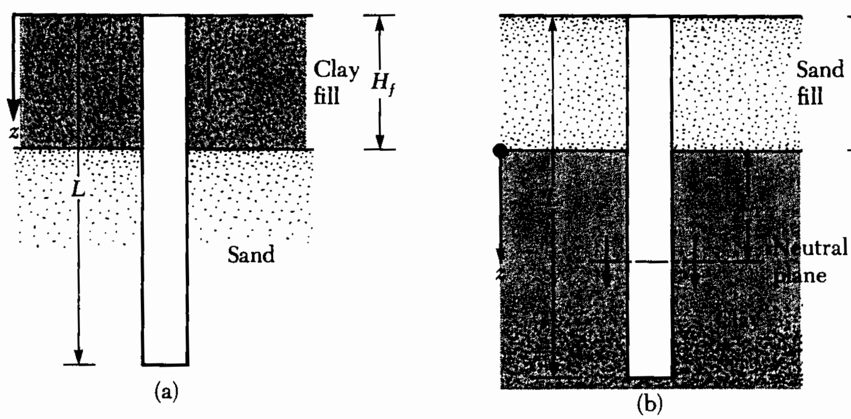


Figure 11.42 Negative skin friction

3. Lowering of the water table will increase the vertical effective stress on the soil at any depth, which will induce consolidation settlement in clay. If a pile is located in the clay layer, it will be subjected to a downward drag force.

In some cases, the downward drag force may be excessive and cause foundation failure. This section outlines two tentative methods for the calculation of negative skin friction.

Clay Fill over Granular Soil (Figure 11.42a)

Similar to the β method presented in Section 11.13, the negative (downward) skin stress on the pile is

$$f_n = K'\sigma'_o \tan \delta \tag{11.125}$$

where K' = earth pressure coefficient = $K_o = 1 - \sin \phi'$
σ'_o = vertical effective stress at any depth $z = \gamma'_f z$
γ'_f = effective unit weight of fill
δ = soil–pile friction angle ≈ 0.5–$0.7\phi'$

Hence, the total downward drag force on a pile is

$$Q_n = \int_0^{H_f} (pK'\gamma'_f \tan \delta) z\, dz = \frac{pK'\gamma'_f H_f^2 \tan \delta}{2} \tag{11.126}$$

where H_f = height of the fill

If the fill is above the water table, the effective unit weight, γ'_f, should be replaced by the moist unit weight.

Granular Soil Fill over Clay (Figure 11.42b)

In this case, the evidence indicates that the negative skin stress on the pile may exist from $z = 0$ to $z = L_1$, which is referred to as the *neutral depth*. (See Vesic, 1977, pp. 25–26.) The neutral depth may be given as (Bowles, 1982)

$$L_1 = \frac{(L - H_f)}{L_1}\left[\frac{L - H_f}{2} + \frac{\gamma'_f H_f}{\gamma'}\right] - \frac{2\gamma'_f H_f}{\gamma'} \tag{11.127}$$

where γ'_f and γ' = effective unit weights of the fill and the underlying clay layer, respectively

For end-bearing piles, the neutral depth may be assumed to be located at the pile tip (i.e., $L_1 = L - H_f$).

Once the value of L_1 is determined, the downward drag force is obtained in the following manner: The unit negative skin friction at any depth from $z = 0$ to $z = L_1$ is

$$f_n = K'\sigma'_o \tan \delta \tag{11.128}$$

where $K' = K_o = 1 - \sin\phi'$
$\sigma'_o = \gamma'_f H_f + \gamma' z$
$\delta = 0.5\text{–}0.7\phi'$

$$Q_n = \int_0^{L_1} pf_n\,dz = \int_0^{L_1} pK'(\gamma'_f H_f + \gamma' z)\tan\delta\,dz$$

$$= (pK'\gamma'_f H_f \tan\delta)L_1 + \frac{L_1^2 pK'\gamma' \tan\delta}{2} \quad (11.129)$$

If the soil and the fill are above the water table, the effective unit weights should be replaced by moist unit weights. In some cases, the piles can be coated with bitumen in the downdrag zone to avoid this problem.

A limited number of case studies of negative skin friction is available in the literature. Bjerrum et al. (1969) reported monitoring the downdrag force on a test pile at Sorenga in the harbor of Oslo, Norway (noted as pile G in the original paper). The study of Bjerrum et al. (1969) was also discussed by Wong and Teh (1995) in terms of the pile being driven to bedrock at 40 m. Figure 11.43a shows the soil profile and the pile. Wong and Teh estimated the following quantities:

- *Fill:* Moist unit weight, $\gamma_f = 16\ \text{kN/m}^3$
  Saturated unit weight, $\gamma_{sat(f)} = 18.5\ \text{kN/m}^3$

So

$$\gamma'_f = 18.5 - 9.81 = 8.69\ \text{kN/m}^3$$

and

$$H_f = 13\ \text{m}$$

- *Clay:* $K' \tan\delta \approx 0.22$
  Saturated effective unit weight, $\gamma' = 19 - 9.81 = 9.19\ \text{kN/m}^3$
- *Pile:* $L = 40$ m
  Diameter, $D = 500$ m

Thus, the maximum downdrag force on the pile can be estimated from Eq. (11.129). Since in this case the pile is a point bearing pile, the magnitude of $L_1 = 27$ m, and

$$Q_n = (p)(K'\tan\delta)[\gamma_f \times 2 + (13 - 2)\gamma'_f](L_1) + \frac{L_1^2 p\gamma'(K'\tan\delta)}{2}$$

or

$$Q_n = (\pi \times 0.5)(0.22)[(16 \times 2) + (8.69 \times 11)](27) + \frac{(27)^2(\pi \times 0.5)(9.19)(0.22)}{2}$$

$$= 2348\ \text{kN}$$

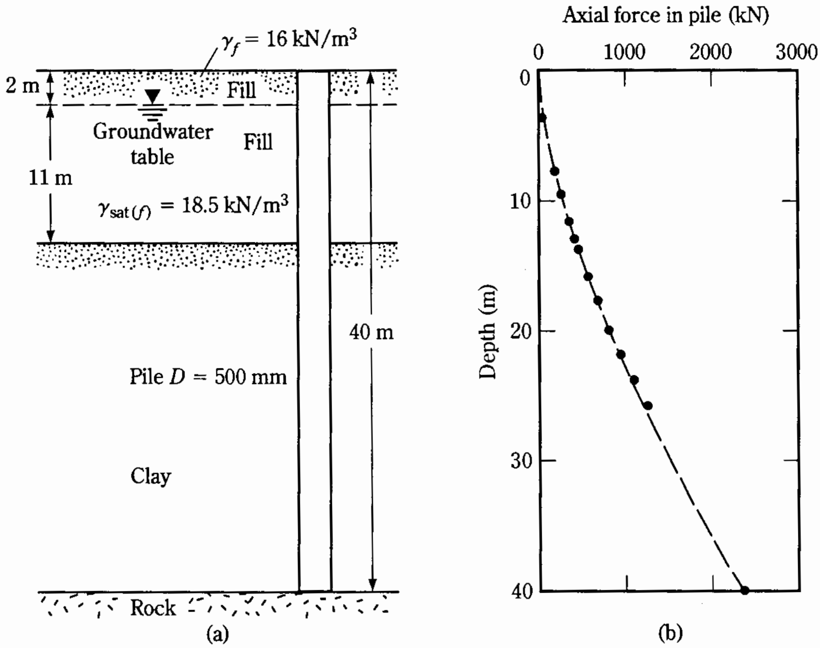


Figure 11.43 Negative skin friction on a pile in the harbor of Oslo, Norway [based on Bjerrum et al. (1969) and Wong and Teh (1995)]

The measured value of the maximum Q_n was about 2500 kN (Figure 11.43b), which is in good agreement with the calculated value.

Example 11.12

In Figure 11.42a, let $H_f = 2$ m. The pile is circular in cross section with a diameter of 0.305 m. For the fill that is above the water table, $\gamma_f = 16$ kN/m^3 and $\phi' = 32°$. Determine the total drag force. Use $\delta = 0.6\,\phi'$.

Solution

From Eq. (11.126),

$$Q_n = \frac{pK'\gamma_f H_f^2 \tan\delta}{2}$$

with

$$p = \pi(0.305) = 0.958 \text{ m}$$

$$K' = 1 - \sin\phi' = 1 - \sin 32 = 0.47$$

and

$$\delta = (0.6)(32) = 19.2°$$

Thus,

$$Q_n = \frac{(0.958)(0.47)(16)(2)^2 \tan 19.2}{2} = \mathbf{5.02\ kN}$$

■

Example 11.13

In Figure 11.42b, let $H_f = 2$ m, pile diameter $= 0.305$ m, $\gamma_f = 16.5\ \text{kN/m}^3$, $\phi'_{\text{clay}} = 34°$, $\gamma_{\text{sat(clay)}} = 17.2\ \text{kN/m}^3$, and $L = 20$ m. The water table coincides with the top of the clay layer. Determine the downward drag force. Assume that $\delta = 0.6\phi'_{\text{clay}}$.

Solution

The depth of the neutral plane is given in Eq. (11.127) as

$$L_1 = \frac{L - H_f}{L_1}\left(\frac{L - H_f}{2} + \frac{\gamma_f H_f}{\gamma'}\right) - \frac{2\gamma_f H_f}{\gamma'}$$

Note that γ'_f in Eq. (11.127) has been replaced by γ_f because the fill is above the water table, so

$$L_1 = \frac{(20 - 2)}{L_1}\left[\frac{(20 - 2)}{2} + \frac{(16.5)(2)}{(17.2 - 9.81)}\right] - \frac{(2)(16.5)(2)}{(17.2 - 9.81)}$$

or

$$L_1 = \frac{242.4}{L_1} - 8.93;\ L_1 = 11.75\ \text{m}$$

Now, from Eq. (11.129), we have

$$Q_n = (pK'\gamma_f H_f \tan\delta)L_1 + \frac{L_1^2 pK'\gamma' \tan\delta}{2}$$

with

$$p = \pi(0.305) = 0.958\ \text{m}$$

and

$$K' = 1 - \sin 34° = 0.44$$

Hence,

$$Q_n = (0.958)(0.44)(16.5)(2)[\tan(0.6 \times 34)](11.75) + \frac{(11.75)^2(0.958)(0.44)(17.2 - 9.81)[\tan(0.6 \times 34)]}{2}$$

$$= 60.78 + 79.97 = \mathbf{140.75\ kN}$$

■

Group Piles

11.24 Group Efficiency

In most cases, piles are used in groups, as shown in Figure 11.44, to transmit the structural load to the soil. A *pile cap* is constructed over *group piles*. The cap can be in contact with the ground, as in most cases (see Figure 11.44a), or well above the ground, as in the case of offshore platforms (see Figure 11.44b).

Determining the load-bearing capacity of group piles is extremely complicated and has not yet been fully resolved. When the piles are placed close to each other, a reasonable assumption is that the stresses transmitted by the piles to the soil will overlap (see Figure 11.44c), reducing the load-bearing capacity of the piles. Ideally, the piles in a group should be spaced so that the load-bearing capacity of the group is not less than the sum of the bearing capacity of the individual piles. In practice, the minimum center-to-center pile spacing, d, is $2.5D$ and, in ordinary situations, is actually about 3–$3.5D$.

The efficiency of the load-bearing capacity of a group pile may be defined as

$$\eta = \frac{Q_{g(u)}}{\Sigma Q_u} \tag{11.130}$$

where η = group efficiency
$Q_{g(u)}$ = ultimate load-bearing capacity of the group pile
Q_u = ultimate load-bearing capacity of each pile without the group effect

Many structural engineers use a simplified analysis to obtain the group efficiency for *friction piles*, particularly in sand. This type of analysis can be explained with the aid of Figure 11.44a. Depending on their spacing within the group, the piles may act in one of two ways: (1) as a *block*, with dimensions $L_g \times B_g \times L$, or (2) as *individual piles*. If the piles act as a block, the frictional capacity is $f_{av}p_gL \approx Q_{g(u)}$. [*Note:* p_g = perimeter of the cross section of block $= 2(n_1 + n_2 - 2)d + 4D$, and f_{av} = average unit frictional resistance.] Similarly, for each pile acting individually, $Q_u \approx pLf_{av}$. (*Note:* p = perimeter of the cross section of each pile.) Thus,

$$\eta = \frac{Q_{g(u)}}{\Sigma Q_u} = \frac{f_{av}[2(n_1 + n_2 - 2)d + 4D]L}{n_1 n_2 p L f_{av}} = \frac{2(n_1 + n_2 - 2)d + 4D}{p n_1 n_2} \tag{11.131}$$

Hence,

$$Q_{g(u)} = \left[\frac{2(n_1 + n_2 - 2)d + 4D}{p n_1 n_2}\right]\Sigma Q_u \tag{11.132}$$

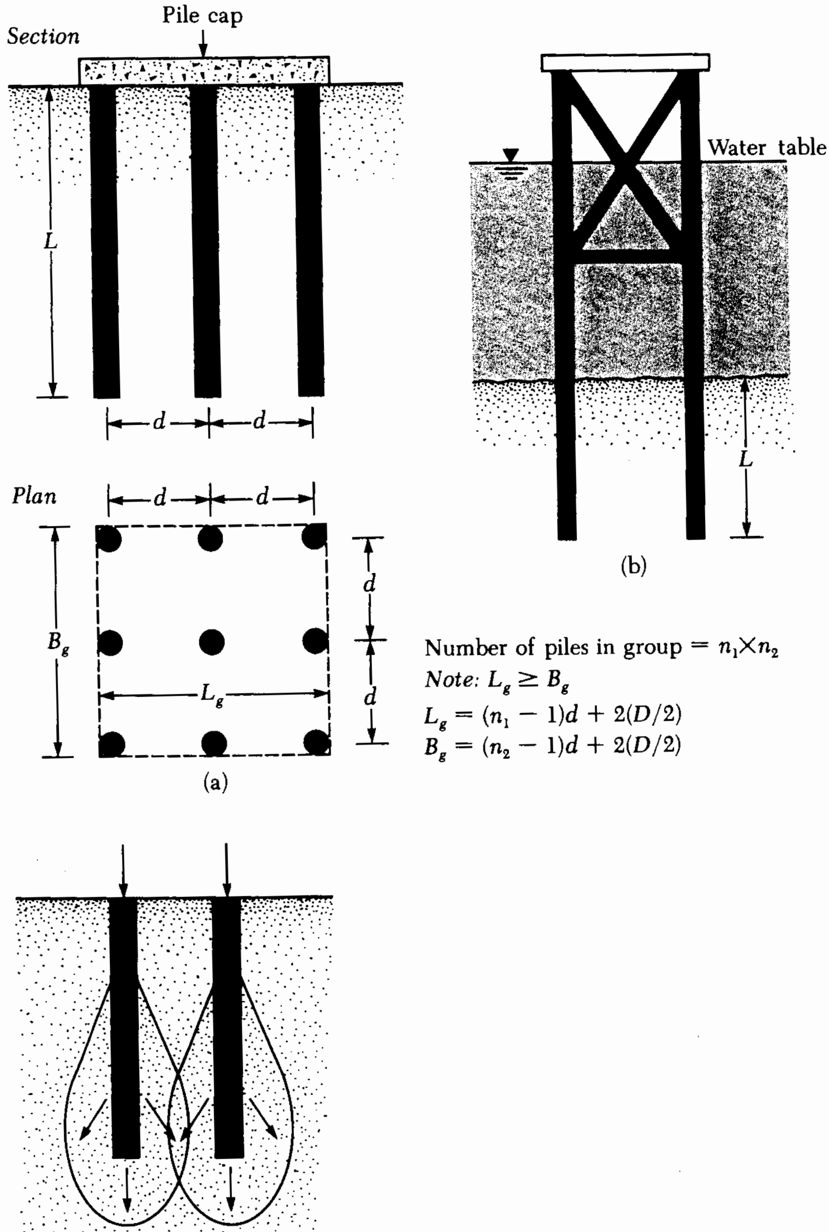


Figure 11.44 Group piles

From Eq. (11.132), if the center-to-center spacing d is large enough, $\eta > 1$. In that case, the piles will behave as individual piles. Thus, in practice, if $\eta < 1$, then

$$Q_{g(u)} = \eta \Sigma Q_u$$

and if $\eta \geqslant 1$, then

$$Q_{g(u)} = \Sigma Q_u$$

Table 11.15 Equations for Group Efficiency of Friction Piles

Name	Equation
Converse–Labarre equation	$\eta = 1 - \left[\frac{(n_1 - 1)n_2 + (n_2 - 1)n_1}{90 n_1 n_2}\right]\theta$ where $\theta(\text{deg}) = \tan^{-1}(D/d)$
Los Angeles Group Action equation	$\eta = 1 - \frac{D}{\pi d n_1 n_2}[n_1(n_2 - 1) + n_2(n_1 - 1)] + \sqrt{2}(n_1 - 1)(n_2 - 1)]$
Seiler–Keeney equation (Seiler and Keeney, 1944)	$\eta = \left\{1 - \left[\frac{11d}{7(d^2 - 1)}\right]\left[\frac{n_1 + n_2 - 2}{n_1 + n_2 - 1}\right]\right\} + \frac{0.3}{n_1 + n_2}$ where d is in ft

There are several other equations like Eq. (11.132) for calculating the group efficiency of friction piles. Some of these are given in Table 11.15.

Feld (1943) suggested a method by which the load capacity of individual piles (when only frictional resistance is considered) in a group embedded in sand could be assigned. According to this method, the ultimate capacity of a pile is reduced by one-sixteenth by each adjacent diagonal or row pile. The technique can be explained if one examines Figure 11.45, which shows the plan of a group pile. For pile type *A*, there are eight adjacent piles, for pile type *B*, there are five, and for pile type *C*, there are three. With this in mind, the following table can be prepared:

Pile type	No. of Piles	No. of adjacent piles/pile	Reduction factor for each pile	Ultimate capacity[a]
A	1	8	$1 - \frac{8}{16}$	$0.5Q_u$
B	4	5	$1 - \frac{5}{16}$	$2.75Q_u$
C	4	3	$1 - \frac{3}{16}$	$3.25Q_u$
				$\Sigma\, 6.5Q_u = Q_{g(u)}$

[a] (No of piles) (Q_u) (reduction factor)
Q_u = ultimate capacity for an isolated pile

Hence,

$$\eta = \frac{Q_{g(u)}}{\Sigma Q_u} = \frac{6.5Q_u}{9Q_u} = 72\%$$

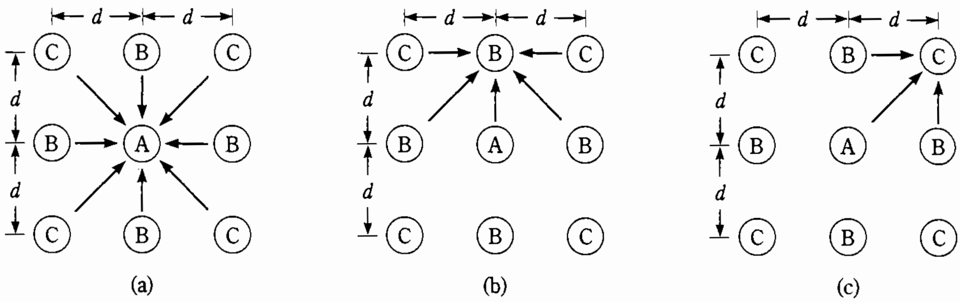


Figure 11.45 Feld's method for estimating the group capacity of friction piles

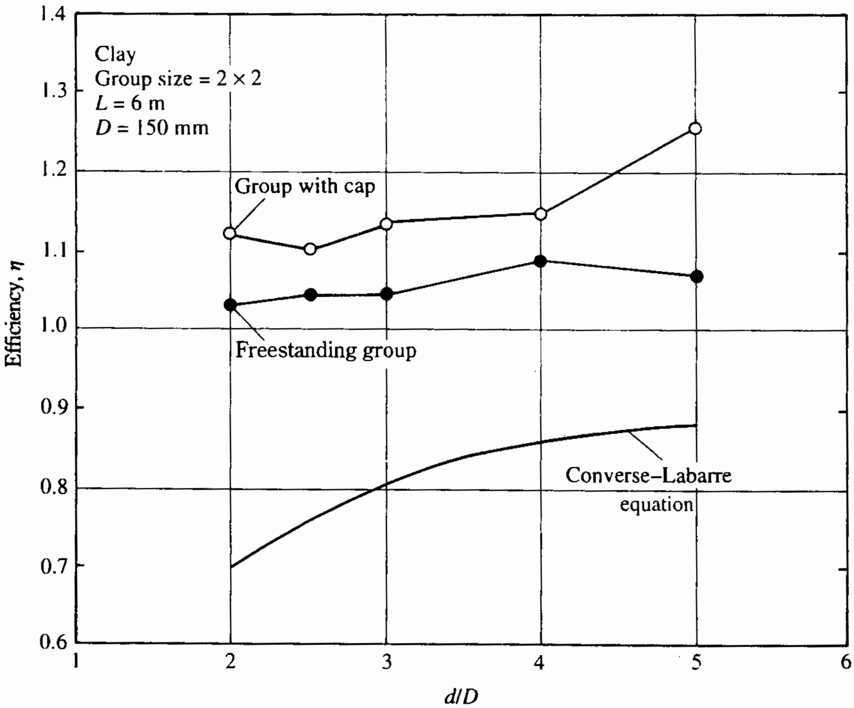


Figure 11.46 Variation of group efficiency with d/D (after Brand et al., 1972)

Figure 11.46 shows a comparison of field-test results in clay with the theoretical group efficiency calculated from the Converse–Labarre equation. (See Table 11.15.) Reported by Brand et al. (1972), the tests were conducted in soil, the details of which are given in Figure 3.7. Other parameters include the following:

- Length of piles = 6 m
- Diameter of piles = 150 mm
- Pile groups tested = 2 × 2
- Location of pile head = 1.5 m below the ground surface

Pile tests were conducted with and without a cap. (In the latter state, the group is a freestanding group.) Note that for $d/D \geq 2$, the magnitude of η was greater than 1.0. Also, for similar values of d/D, the group efficiency was greater with the pile cap than without the cap. Figure 11.47 shows the pile group settlement at various stages of the load test.

Figure 11.48 shows the variation of the group efficiency η for a 3 × 3 group pile in sand (Kishida and Meyerhof, 1965). It can be seen that, for loose and medium sands, the magnitude of the group efficiency is larger than unity. This is due primarily to the densification of sand surrounding the pile.

Liu et al. (1985) reported the results of field tests on 58 pile groups and 23 single piles embedded in granular soil. Included in the report were the following details:

- Pile length, $L = 8D–23D$
- Pile diameter, D = 125 mm–330 mm
- Type of pile installation = bored
- Spacing of piles in group, $d = 2D–6D$

Figure 11.49 shows the behavior of 3 × 3 pile groups with low-set and high-set pile caps in terms of the average skin friction, f_{av}. Figure 11.50 shows the variation of the average skin friction, based on the location of a pile in the group.

Based on the experimental observations of the behavior of group piles in sand to date, the following general conclusions may be drawn:

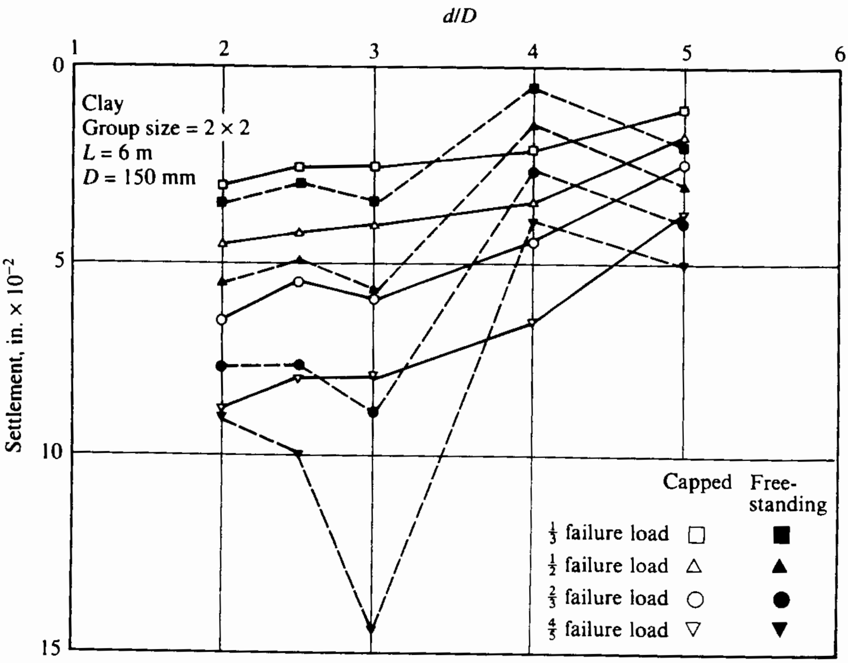


Figure 11.47 Variation of group pile settlement at various stages of load (after Brand et al., 1972)

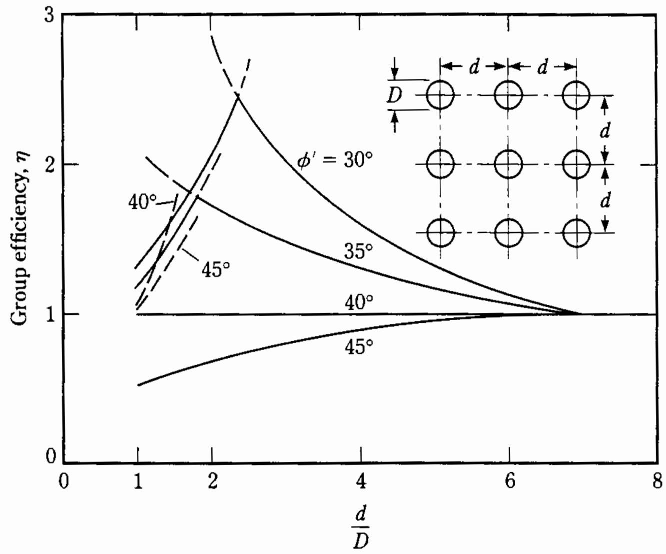


Figure 11.48 Variation of efficiency of pile groups in sand (based on Kishida and Meyerhof, 1965)

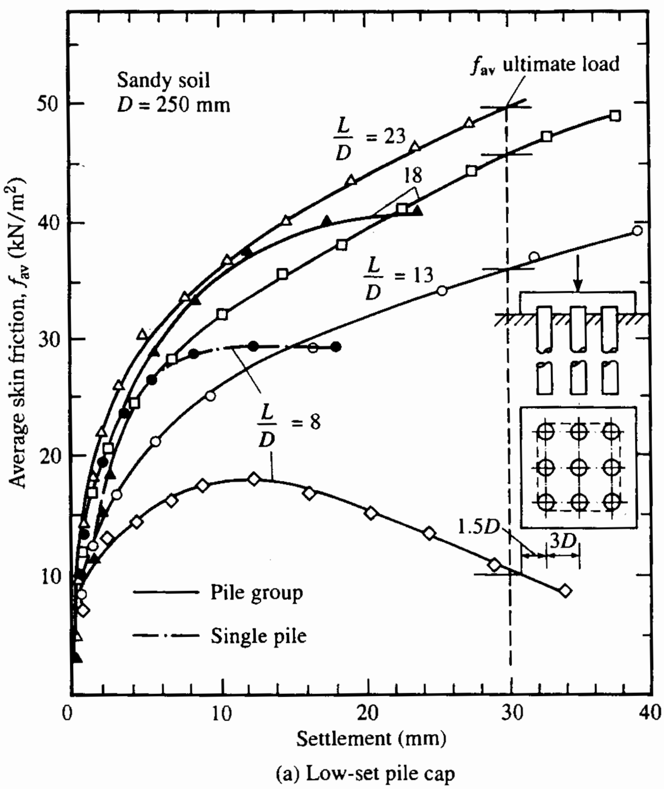
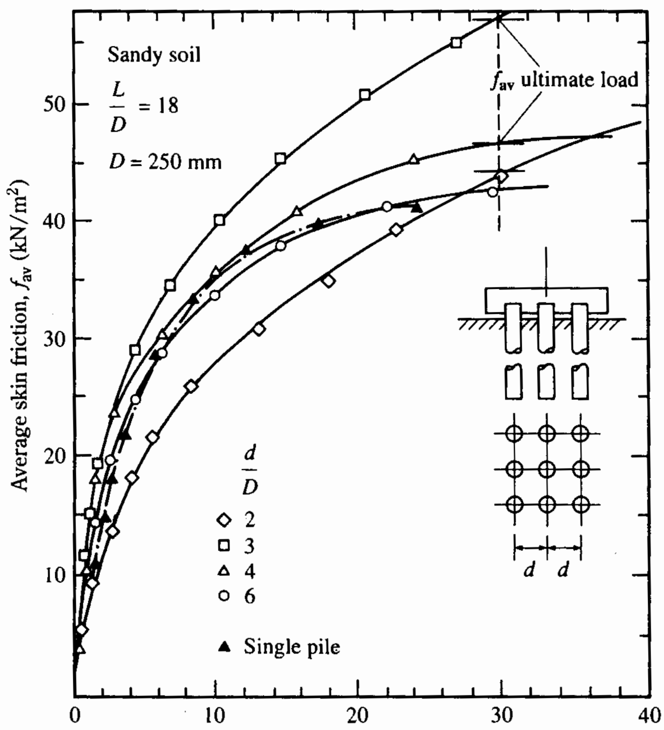


Figure 11.49 Behavior of low-set and high-set pile groups in terms of average skin friction (based on Liu et al., 1985)



(b) High-rise pile cap

Figure 11.49 (Continued)

1. For *driven* group piles in *sand* with $d \geqslant 3D$, $Q_{g(u)}$ may be taken to be $\Sigma\, Q_u$, which includes the frictional and the point bearing capacities of individual piles.
2. For *bored* group piles in *sand* at conventional spacings ($d \approx 3D$), $Q_{g(u)}$ may be taken to be $\frac{2}{3}$ to $\frac{3}{4}$ times $\Sigma\, Q_u$ (frictional and point bearing capacities of individual piles).

11.25 Ultimate Capacity of Group Piles in Saturated Clay

Figure 11.51 shows a group pile in saturated clay. Using the figure, one can estimate the ultimate load-bearing capacity of group piles in the following manner:

1. Determine $\Sigma\, Q_u = n_1 n_2 (Q_p + Q_s)$. From Eq. (11.19),

$$Q_p = A_p[9c_{u(p)}]$$

where $c_{u(p)}$ = undrained cohesion of the clay at the pile tip

Also, from Eq. (11.55),

$$Q_s = \Sigma\, \alpha p c_u \Delta L$$

So

$$\Sigma\, Q_u = n_1 n_2 [9 A_p c_{u(p)} + \Sigma\, \alpha p c_u \Delta L] \tag{11.133}$$

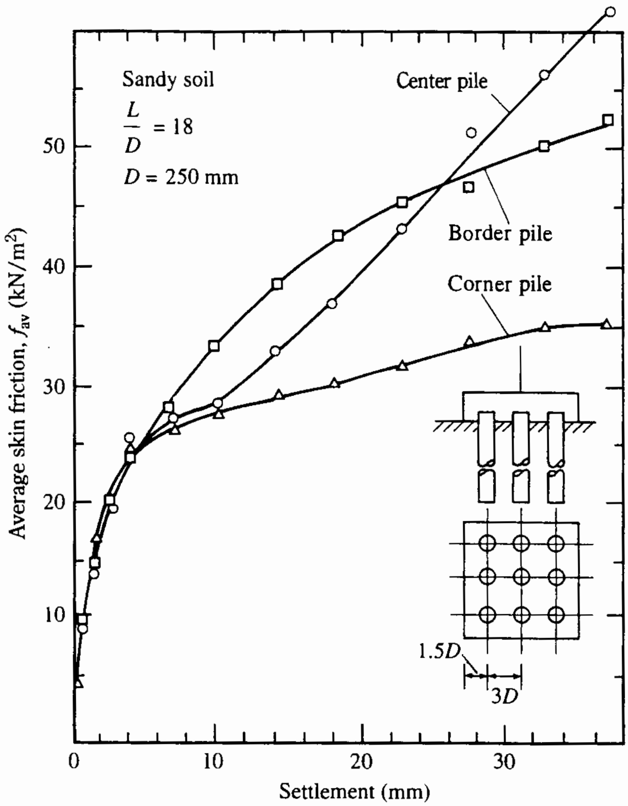


Figure 11.50 Average skin friction, based on pile location (after Liu et al., 1985)

2. Determine the ultimate capacity by assuming that the piles in the group act as a block with dimensions $L_g \times B_g \times L$. The skin resistance of the block is

$$\Sigma\, p_g c_u \Delta L = \Sigma\, 2(L_g + B_g) c_g \Delta L$$

Calculate the point bearing capacity:

$$A_p q_p = A_p c_{u(p)} N_c^* = (L_g B_g) c_{u(p)} N_c^*$$

Obtain the value of the bearing capacity factor N_c^* from Figure 11.52. Thus, the ultimate load is

$$\Sigma\, Q_u = L_g B_g c_{u(p)} N_c^* + \Sigma\, 2(L_g + B_g) c_u \Delta L \qquad (11.134)$$

3. Compare the values obtained from Eqs. (11.133) and (11.134). The *lower* of the two values is $Q_{g(u)}$.

11.26 Piles in Rock

For point bearing piles resting on rock, most building codes specify that $Q_{g(u)} = \Sigma\, Q_u$, provided that the minimum center-to-center spacing of the piles is D + 300 mm. For H-piles and piles with square cross sections, the magnitude of D is equal to the diagonal dimension of the cross section of the pile.

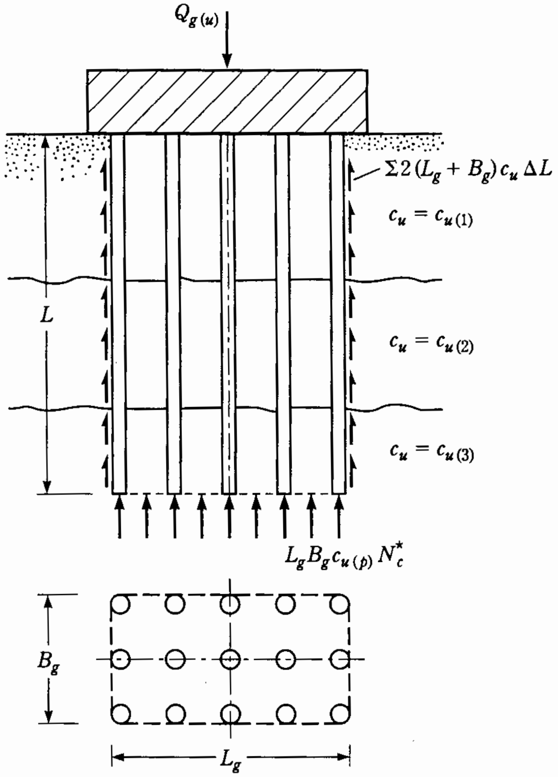


Figure 11.51 Ultimate capacity of group piles in clay

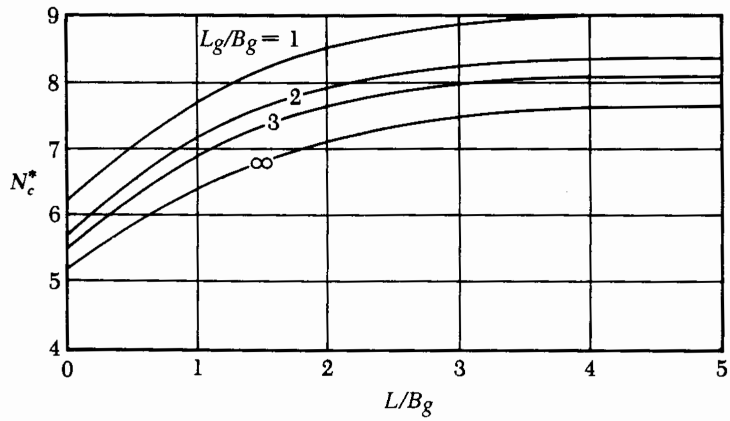


Figure 11.52 Variation of N_c^* with L_g/B_g and L/B_g

Example 11.14

In Figure 11.44a, let $n_1 = 4$, $n_2 = 3$, $D = 305$ mm, $d = 1220$ mm, and $L = 15$ m. Suppose the piles are square in cross section and are embedded in a homogeneous saturated clay with $c_u = 70$ kN/m^2. Using a factor of safety equal to 4, determine the allowable load-bearing capacity of the group pile. The unit weight of clay, $\gamma = 18.8\ \text{kN/m}^3$, and the groundwater table is located at a depth 18 m below the ground surface.

Solution

From Eq. (11.133),

$$\Sigma Q_u = n_1 n_2 [9 A_p c_{u(p)} + \Sigma \alpha p c_u \Delta L]$$

with

$$A_p = (0.305)(0.305) = 0.093\ \text{m}^2$$

and

$$p = (4)(0.305) = 1.22\ \text{m}$$

The average value of the effective overburden pressure is

$$\overline{\sigma}'_o = \left(\frac{15}{2}\right)(18.8) = 141\ \text{kN/m}^2$$

Also, with

$$c_u = 70\ \text{kN/m}^2$$

it follows that

$$\frac{c_u}{\overline{\sigma}'_o} = \frac{70}{141} = 0.496$$

From Figure 11.24, for $\dfrac{c_u}{\overline{\sigma}'_o} = 0.496$, the magnitude of α is about 0.7. So

$$\Sigma Q_u = (4)(3)[(9)(0.093)(70) + (0.7)(1.22)(70)(15)]$$
$$= 12(58.59 + 896.7) \approx 11{,}463\ \text{kN}$$

Again, from Eq. (11.134), the ultimate block capacity is

$$L_g B_g c_{u(p)} N_c^* + \Sigma 2(L_g + B_g) c_u \Delta L$$

Now,

$$L_g = (n_1 - 1)d + 2\left(\frac{D}{2}\right) = (4 - 1)(1.22) + 0.305 = 3.965\ \text{m}$$

and

$$B_g = (n_2 - 1)d + 2\left(\frac{D}{2}\right) = (3 - 1)(1.22) + 0.305 = 2.745\ \text{m}$$

so we have

$$\frac{L}{B_g} = \frac{15}{2.745} = 5.46$$

and

$$\frac{L_g}{B_g} = \frac{3.965}{2.745} = 1.44$$

From Figure 11.52, $N_c^* \approx 8.6$. So

$$\text{block capacity} = (3.965)(2.745)(70)(8.6) + 2(3.965 + 2.745)(70)(15)$$
$$= 6552 + 14{,}091 = 20{,}643 \text{ kN}$$

Hence,

$$Q_{g(u)} = 11{,}463 \text{ kN} < 20{,}643 \text{ kN}$$

and

$$Q_{g(\text{all})} = \frac{Q_{g(u)}}{\text{FS}} = \frac{11{,}463}{4} \approx \mathbf{2866\ kN}$$ ■

11.27 Elastic Settlement of Group Piles

In general, the settlement of a group pile under a similar working load per pile increases with the width of the group (B_g) and the center-to-center spacing of the piles (d). This fact is demonstrated in Figure 11.53, obtained from the experimental results of Meyerhof (1961) for group piles in sand. In the figure, $s_{g(e)}$ is the settlement of the group pile and s_e is the settlement of isolated piles under a similar working load.

Several investigations relating to the settlement of group piles have been reported in the literature, with widely varying results. The simplest relation for the settlement of group piles was given by Vesic (1969), namely,

$$s_{g(e)} = \sqrt{\frac{B_g}{D}}\, s_e \tag{11.135}$$

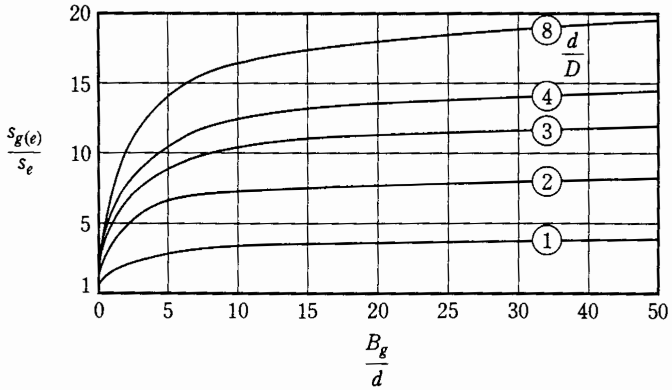


Figure 11.53 Settlement of group piles in sand (after Meyerhof, 1961)

where $s_{g(e)}$ = elastic settlement of group piles
B_g = width of group pile section
D = width or diameter of each pile in the group
s_e = elastic settlement of each pile at comparable working load (see Section 11.18)

For group piles in sand and gravel, for elastic settlement, Meyerhof (1976) suggested the empirical relation

$$s_{g(e)}(\text{in.}) = \frac{2q\sqrt{B_g}I}{(N_1)_{60}} \tag{11.136}$$

where $q = Q_g/(L_g B_g)$ (in U.S. ton/ft^2) (11.137)
L_g and B_g = length and width of the group pile section, respectively (ft)
$(N_1)_{60}$ = average corrected standard penetration number within seat of settlement ($\approx B_g$ deep below the tip of the piles)
I = influence factor $= 1 - L/8B_g \geqslant 0.5$ (11.138)
L = length of embedment of piles

In SI units,

$$S_{g(e)}(\text{mm}) = \frac{0.96q\sqrt{B_g}I}{(N_1)_{60}} \tag{11.139}$$

where q is in kN/m^2 and B_g and L_g are in m, and

$$I = 1 - \frac{L(\text{m})}{8B_g(\text{m})}$$

Similarly, the group pile settlement is related to the cone penetration resistance by the formula

$$S_{g(e)} = \frac{qB_gI}{2q_c} \tag{11.140}$$

where q_c = average cone penetration resistance within the seat of settlement. (Note that, in Eq. (11.140), all quantities are expressed in consistent units.)

11.28 Consolidation Settlement of Group Piles

The consolidation settlement of a group pile in clay can be estimated by using the 2:1 stress distribution method. The calculation involves the following steps (see Figure 11.54):

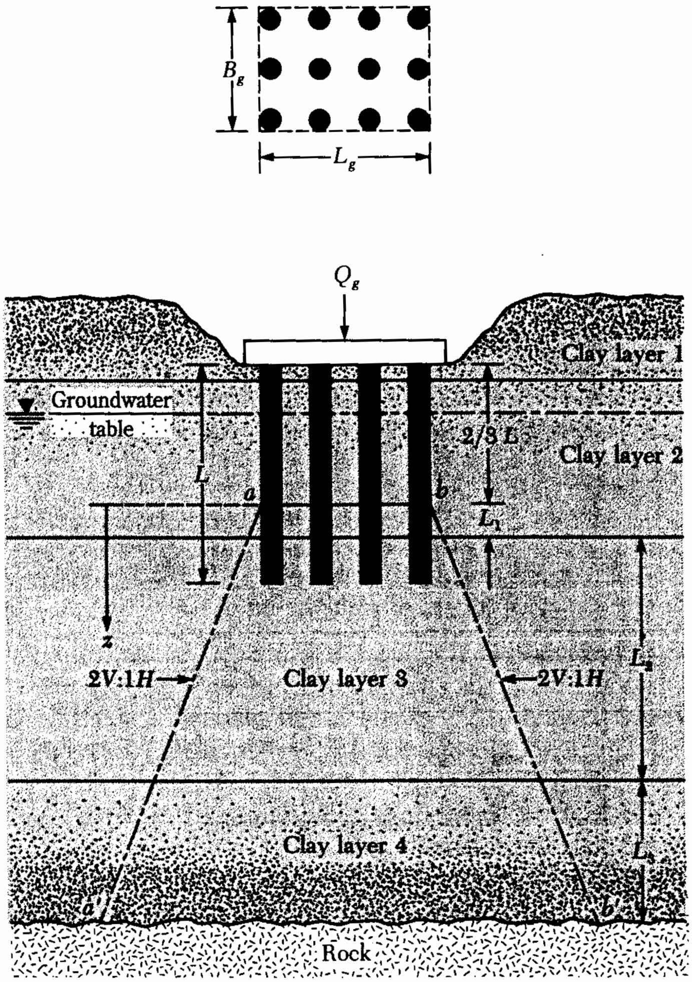


Figure 11.54 Consolidation settlement of group piles

1. Let the depth of embedment of the piles be L. The group is subjected to a total load of Q_g. If the pile cap is below the original ground surface, Q_g equals the total load of the superstructure on the piles, minus the effective weight of soil above the group piles removed by excavation.
2. Assume that the load Q_g is transmitted to the soil beginning at a depth of $2L/3$ from the top of the pile, as shown in the figure. The load Q_g spreads out along two vertical lines to one horizontal line from this depth. Lines aa' and bb' are the two 2:1 lines.
3. Calculate the increase in effective stress caused at the middle of each soil layer by the load Q_g. The formula is

$$\Delta\sigma_i' = \frac{Q_g}{(B_g + z_i)(L_g + z_i)} \tag{11.141}$$

where $\Delta\sigma'_i$ = increase in effective stress at the middle of layer i
L_g, B_g = length and width, respectively of the planned group piles
z_i = distance from $z = 0$ to the middle of the clay layer i

For example, in Figure 11.54, for layer 2, $z_i = L_1/2$; for layer 3, $z_i = L_1 + L_2/2$; and for layer 4, $z_i = L_1 + L_2 + L_3/2$. Note, however, that there will be no increase in stress in clay layer 1, because it is above the horizontal plane ($z = 0$) from which the stress distribution to the soil starts.

4. Calculate the consolidation settlement of each layer caused by the increased stress. The formula is

$$\Delta s_{c(i)} = \left[\frac{\Delta e_{(i)}}{1 + e_{o(i)}}\right] H_i \tag{11.142}$$

where $\Delta s_{c(i)}$ = consolidation settlement of layer i
$\Delta e_{(i)}$ = change of void ratio caused by the increase in stress in layer i
e_o = initial void ratio of layer i (before construction)
H_i = thickness of layer i (*Note:* In Figure 11.54, for layer 2, $H_i = L_1$; for layer 3, $H_i = L_2$; and for layer 4, $H_i = L_3$.)

Relationships involving $\Delta e_{(i)}$ are given in Chapter 1.

5. The total consolidation settlement of the group piles is then

$$\Delta s_{c(g)} = \Sigma \Delta s_{c(i)} \tag{11.143}$$

Note that the consolidation settlement of piles may be initiated by fills placed nearby, adjacent floor loads, or the lowering of water tables.

Example 11.15

A group pile in clay is shown in Figure 11.55. Determine the consolidation settlement of the piles. All clays are normally consolidated.

Solution

Because the lengths of the piles are 15 m each, the stress distribution starts at a depth of 10 m below the top of the pile. We are given that $Q_g = 2000$ kN.

Calculation of Settlement of Clay Layer 1
For normally consolidated clays,

$$\Delta s_{c(1)} = \left[\frac{C_{c(1)}H_1}{1 + e_{o(1)}}\right] \log \left[\frac{\sigma'_{o(1)} + \Delta\sigma'_{(1)}}{\sigma'_{o(1)}}\right]$$

$$\Delta\sigma'_{(1)} = \frac{Q_g}{(L_g + z_1)(B_g + z_1)} = \frac{2000}{(3.3 + 3.5)(2.2 + 3.5)} = 51.6 \text{ kN/m}^2$$

and

$$\sigma'_{o(1)} = 2(16.2) + 12.5(18.0 - 9.81) = 134.8 \text{ kN/m}^2$$

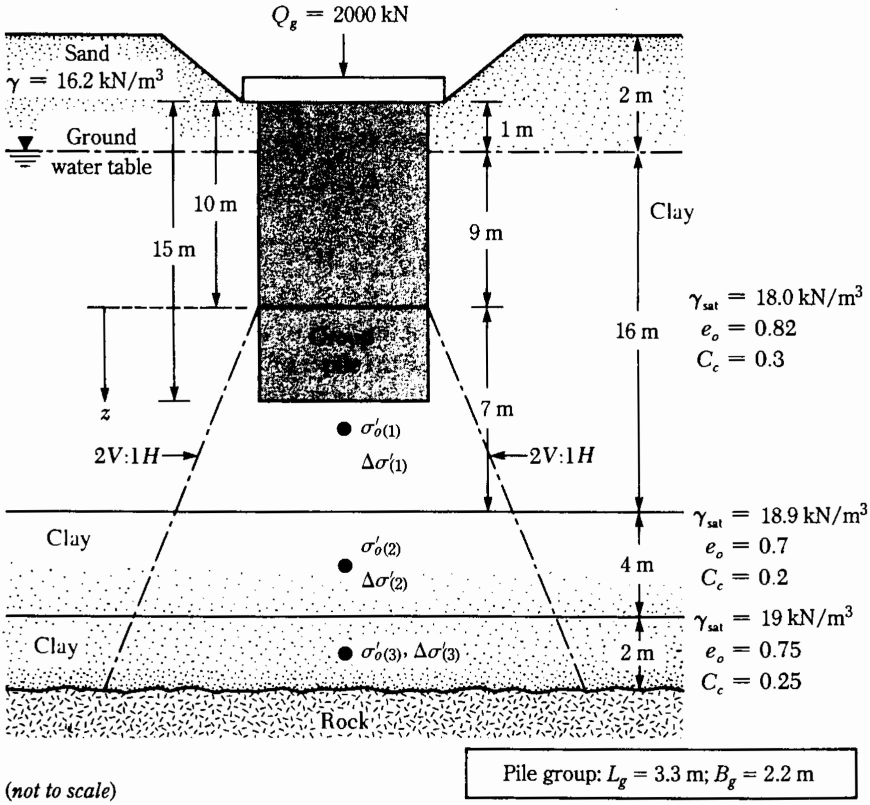


Figure 11.55 Consolidation settlement of a pile group

So

$$\Delta s_{c(1)} = \frac{(0.3)(7)}{1 + 0.82} \log\left[\frac{134.8 + 51.6}{134.8}\right] = 0.1624 \text{ m} = \mathbf{162.4 \text{ mm}}$$

Settlement of Layer 2

As with layer 1,

$$\Delta s_{c(2)} = \frac{C_{c(2)} H_2}{1 + e_{o(2)}} \log\left[\frac{\sigma'_{o(2)} + \Delta\sigma'_{(2)}}{\sigma'_{o(2)}}\right]$$

$$\sigma'_{o(2)} = 2(16.2) + 16(18.0 - 9.81) + 2(18.9 - 9.81) = 181.62 \text{ kN/m}^2$$

and

$$\Delta\sigma'_{(2)} = \frac{2000}{(3.3 + 9)(2.2 + 9)} = 14.52 \text{ kN/m}^2$$

Hence,

$$\Delta s_{c(2)} = \frac{(0.2)(4)}{1 + 0.7} \log\left[\frac{181.62 + 14.52}{181.62}\right] = 0.0157 \text{ m} = \mathbf{15.7 mm}$$

Settlement of Layer 3

Continuing analogously, we have

$$\sigma'_{o(3)} = 181.62 + 2(18.9 - 9.81) + 1(19 - 9.81) = 208.99 \text{ kN/m}^2$$

$$\Delta\sigma'_{(3)} = \frac{2000}{(3.3 + 12)(2.2 + 12)} = 9.2 \text{ kN/m}^2$$

$$\Delta s_{c(3)} = \frac{(0.25)(2)}{1 + 0.75} \log\left[\frac{208.99 + 9.2}{208.99}\right] = 0.0054 \text{ m} = \mathbf{5.4 mm}$$

Hence, the total settlement is

$$\Delta s_{c(g)} = 162.4 + 15.7 + 5.4 = \mathbf{183.5 mm}$$

■

Problems

11.1 A concrete pile is 25 m long and 305 mm × 305 mm in cross section. The pile is fully embedded in sand, for which $\gamma = 17.5$ kN/m³ and $\phi' = 35°$. Calculate

a. The ultimate point load, Q_p, by Meyerhof's method.

b. The total frictional resistance [Eqs. (11.14), (11.37), (11.38), and (11.39)] for $K = 1.3$ and $\delta = 0.8\phi'$.

11.2 Solve Problem 11.1, Part (a), using Coyle and Castello's method.

11.3 Solve Problem 11.1, Part (a), using Vesic's method [Eq. (11.20)]. Take $I_r = I_{rr} = 50$.

11.4 Solve Problem 11.1, Part (a), using Janbu's method [Eq. (11.31)]. Take $\eta' = 90°$.

11.5 Use the results of Problems 11.1–11.4 to estimate an allowable value for the point load. Take FS = 4.

11.6 Redo Problem 11.1 for $\gamma = 18.5$ kN/m³ and $\phi' = 40°$.

11.7 Solve Problem 11.6, Part (a), by Coyle and Castello's method.

11.8 A driven closed-ended pile, circular in cross section, is shown in Figure P11.8. Calculate

a. The ultimate point load, using Meyerhof's procedure.

b. The ultimate point load, using Vesic's procedure. Take $I_r = I_{rr} = 50$.

c. An approximate ultimate point load, on the basis of Parts (a) and (b).

d. The ultimate frictional resistance Q_s. [Use Eqs. (11.14), (11.37), (11.38), and (11.39), and take $K = 1.4$ and $\delta = 0.6\phi'$.]

e. The allowable load of the pile. (Use FS = 4)

11.9 A concrete pile 20 m long having a cross section of 381 mm × 381 mm is fully embedded in a saturated clay layer for which $\gamma_{sat} = 18.5$ kN/m³, $\phi = 0$,