12

Drilled-Shaft Foundations

12.1 Introduction

The terms *caisson, pier, drilled shaft,* and *drilled pier* are often used interchangeably in foundation engineering; all refer to a *cast-in-place pile generally having a diameter of about 750 mm* (≈2.5 ft) or more, with or without steel reinforcement and with or without an enlarged bottom. Sometimes the diameter can be as small as 305 mm (≈1 ft).

To avoid confusion, we use the term *drilled shaft* for a hole drilled or excavated to the bottom of a structure's foundation and then filled with concrete. Depending on the soil conditions, casings may be used to prevent the soil around the hole from caving in during construction. The diameter of the shaft is usually large enough for a person to enter for inspection.

The use of drilled-shaft foundations has several advantages:

1. A single drilled shaft may be used instead of a group of piles and the pile cap.
2. Constructing drilled shafts in deposits of dense sand and gravel is easier than driving piles.
3. Drilled shafts may be constructed before grading operations are completed.
4. When piles are driven by a hammer, the ground vibration may cause damage to nearby structures. The use of drilled shafts avoids this problem.
5. Piles driven into clay soils may produce ground heaving and cause previously driven piles to move laterally. This does not occur during the construction of drilled shafts.
6. There is no hammer noise during the construction of drilled shafts; there is during pile driving.
7. Because the base of a drilled shaft can be enlarged, it provides great resistance to the uplifting load.
8. The surface over which the base of the drilled shaft is constructed can be visually inspected.
9. The construction of drilled shafts generally utilizes mobile equipment, which, under proper soil conditions, may prove to be more economical than methods of constructing pile foundations.
10. Drilled shafts have high resistance to lateral loads.

There are also a couple of drawbacks to the use of drilled-shaft construction. For one thing, the concreting operation may be delayed by bad weather and always needs close supervision. For another, as in the case of braced cuts, deep excavations for drilled shafts may induce substantial ground loss and damage to nearby structures.

12.2 Types of Drilled Shafts

Drilled shafts are classified according to the ways in which they are designed to transfer the structural load to the substratum. Figure 12.1a shows a drilled *straight shaft*. It extends through the upper layer(s) of poor soil, and its tip rests on a strong load-bearing soil layer or rock. The shaft can be cased with steel shell or pipe when required (as it is with cased, cast-in-place concrete piles; see Figure 11.4). For such shafts, the resistance to the applied load may develop from end bearing and also from side friction at the shaft perimeter and soil interface.

A *belled shaft* (see Figures 12.1b and c) consists of a straight shaft with a bell at the bottom, which rests on good bearing soil. The bell can be constructed in the shape of a dome (see Figure 12.1b), or it can be angled (see Figure 12.1c). For angled bells, the underreaming tools that are commercially available can make 30° to 45° angles with the vertical. For the majority of drilled shafts constructed in the United States, the entire load-carrying capacity is assigned to the end bearing only. However, under certain circumstances, the end-bearing capacity and the side friction are taken into account. In Europe, both the side frictional resistance and the end-bearing capacity are always taken into account.

Straight shafts can also be extended into an underlying rock layer. (See Figure 12.1d.) In the calculation of the load-bearing capacity of such shafts, the end bearing and the shear stress developed along the shaft perimeter and rock interface can be taken into account.

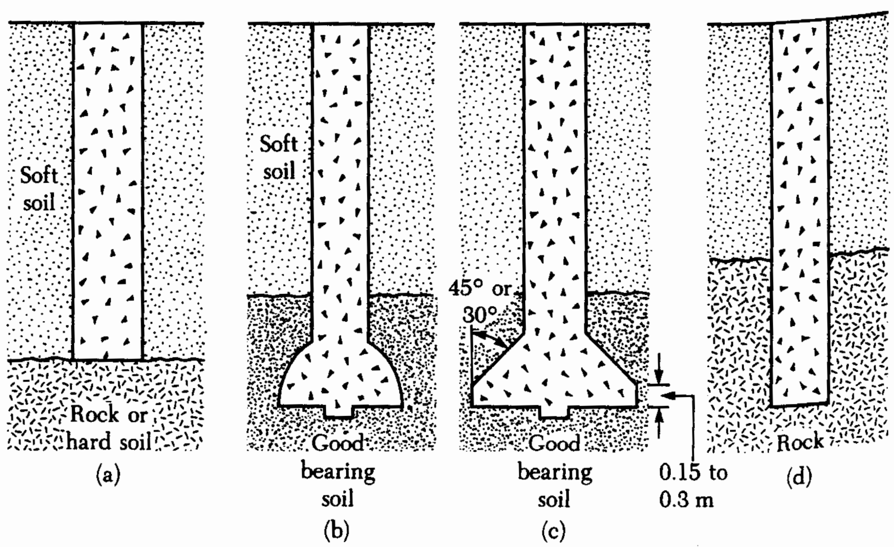

Figure 12.1 Types of drilled shaft: (a) straight shaft; (b) and (c) belled shaft; (d) straight-shaft socketed into rock

12.3 Construction Procedures

The most common construction procedure used in the United States involves rotary drilling. There are three major types of construction methods: the dry method, the casing method, and the wet method.

Dry Method of Construction

This method is employed in soils and rocks that are above the water table and that will not cave in when the hole is drilled to its full depth. The sequence of construction, shown in Figure 12.2, is as follows:

1. The excavation is completed (and belled if desired), using proper drilling tools, and the spoils from the hole are deposited nearby. (See Figure 12.2a.)
2. Concrete is then poured into the cylindrical hole. (See Figure 12.2b.)
3. If desired, a rebar cage is placed in the upper portion of the shaft. (See Figure 12.2c.)
4. Concreting is then completed, and the drilled shaft will be as shown in Figure 12.2d.

Casing Method of Construction

This method is used in soils or rocks in which caving or excessive deformation is likely to occur when the borehole is excavated. The sequence of construction is shown in Figure 12.3 and may be explained as follows:

1. The excavation procedure is initiated as in the case of the dry method of construction. (See Figure 12.3a.)
2. When the caving soil is encountered, bentonite slurry is introduced into the borehole. (See Figure 12.3b.) Drilling is continued until the excavation goes past the caving soil and a layer of impermeable soil or rock is encountered.
3. A casing is then introduced into the hole. (See Figure 12.3c.)
4. The slurry is bailed out of the casing with a submersible pump. (See Figure 12.3d.)
5. A smaller drill that can pass through the casing is introduced into the hole, and excavation continues. (See Figure 12.3e.)
6. If needed, the base of the excavated hole can then be enlarged, using an underreamer. (See Figure 12.3f.)
7. If reinforcing steel is needed, the rebar cage needs to extend the full length of the excavation. Concrete is then poured into the excavation and the casing is gradually pulled out. (See Figure 12.3g.)
8. Figure 12.3h shows the completed drilled shaft.

Wet Method of Construction

This method is sometimes referred to as the *slurry displacement method.* Slurry is used to keep the borehole open during the entire depth of excavation. (See Figure 12.4.) Following are the steps involved in the wet method of construction:

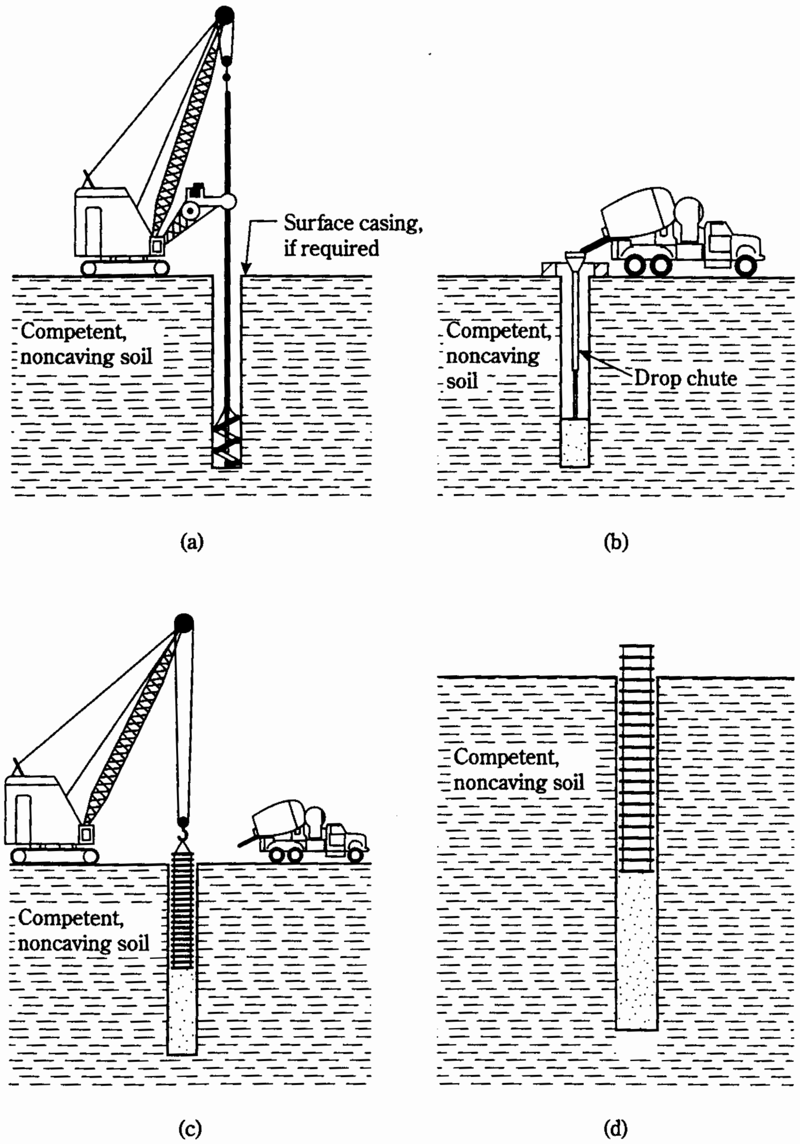

Figure 12.2 Dry method of construction: (a) initiating drilling; (b) starting concrete pour; (c) placing rebar cage; (d) completed shaft (after O'Neill and Reese, 1999)

1. Excavation continues to full depth with slurry. (See Figure 12.4a.)
2. If reinforcement is required, the rebar cage is placed in the slurry. (See Figure 12.4b.)
3. Concrete that will displace the volume of slurry is then placed in the drill hole. (See Figure 12.4c.)
4. Figure 12.4d shows the completed drilled shaft.

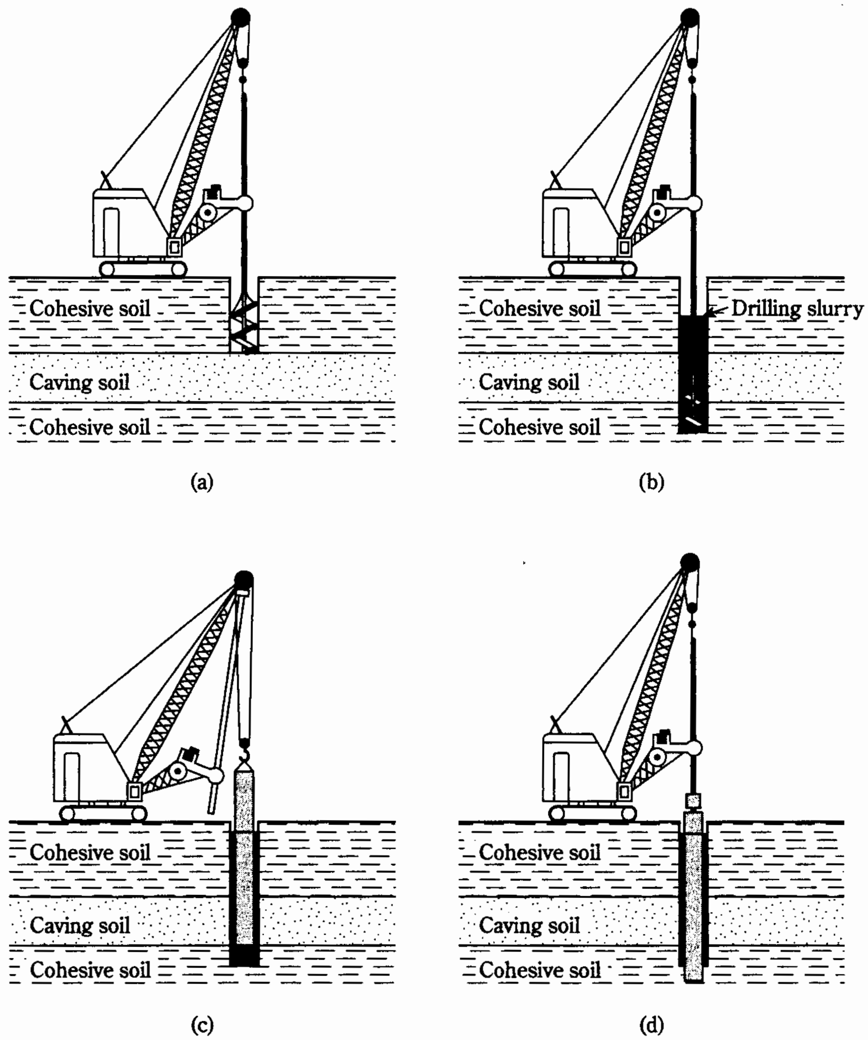

Figure 12.3 Casing method of construction: (a) initiating drilling; (b) drilling with slurry; (c) introducing casing; (d) casing is sealed and slurry is being removed from interior of casing; (e) drilling below casing; (f) underreaming; (g) removing casing; (h) completed shaft (after O'Neill and Reese, 1999)

12.4 Other Design Considerations

For the design of ordinary drilled shafts without casings, a minimum amount of vertical steel reinforcement is always desirable. Minimum reinforcement is 1% of the gross cross-sectional area of the shaft. In California, a reinforcing cage having a length of about 3.65 m (12 ft) is used in the top part of the shaft, and no reinforcement is provided at the bottom. This procedure helps in the construction process, because the cage is placed after the concreting is almost complete.

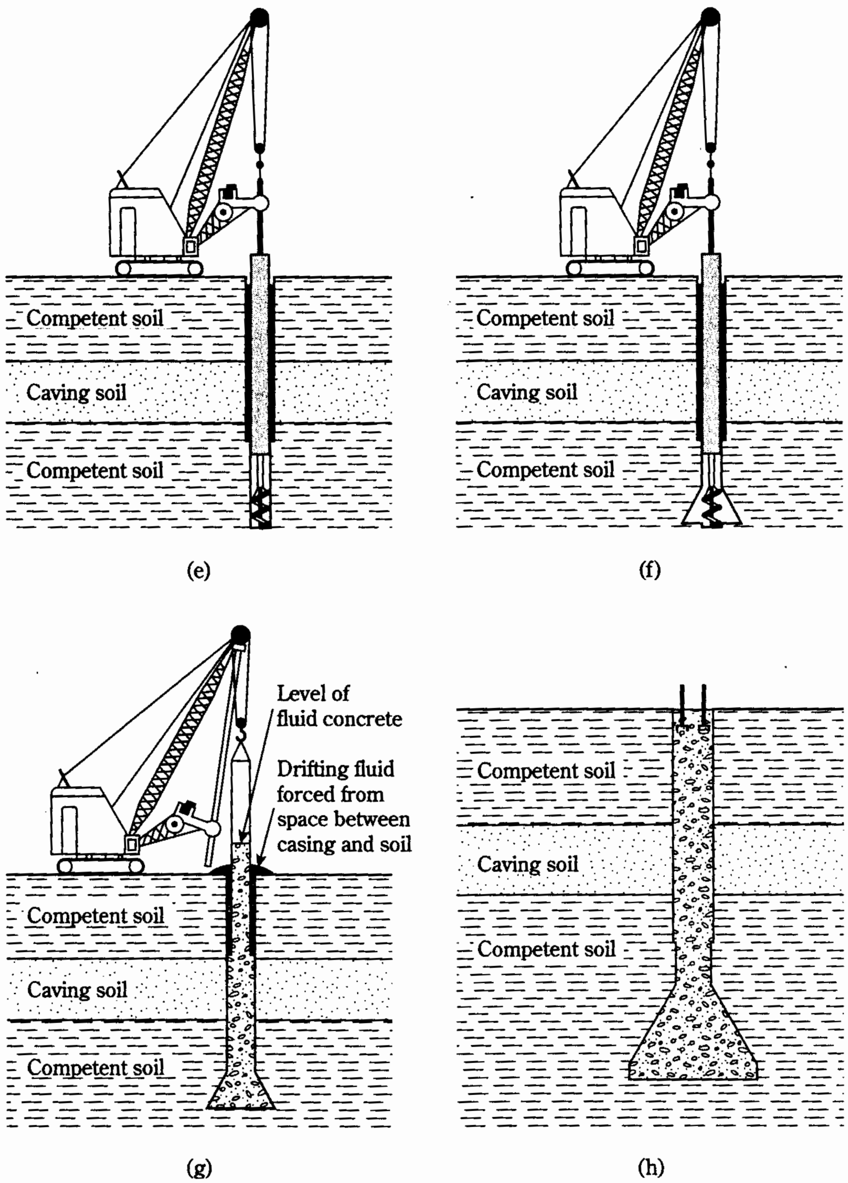

Figure 12.3 (Continued)

For drilled shafts with nominal reinforcement, most building codes suggest using a design concrete strength, f_c, on the order of $f'_c/4$. Thus, the minimum shaft diameter becomes

$$f_c = 0.25f'_c = \frac{Q_w}{A_{gs}} = \frac{Q_w}{\frac{\pi}{4}D_s^2}$$

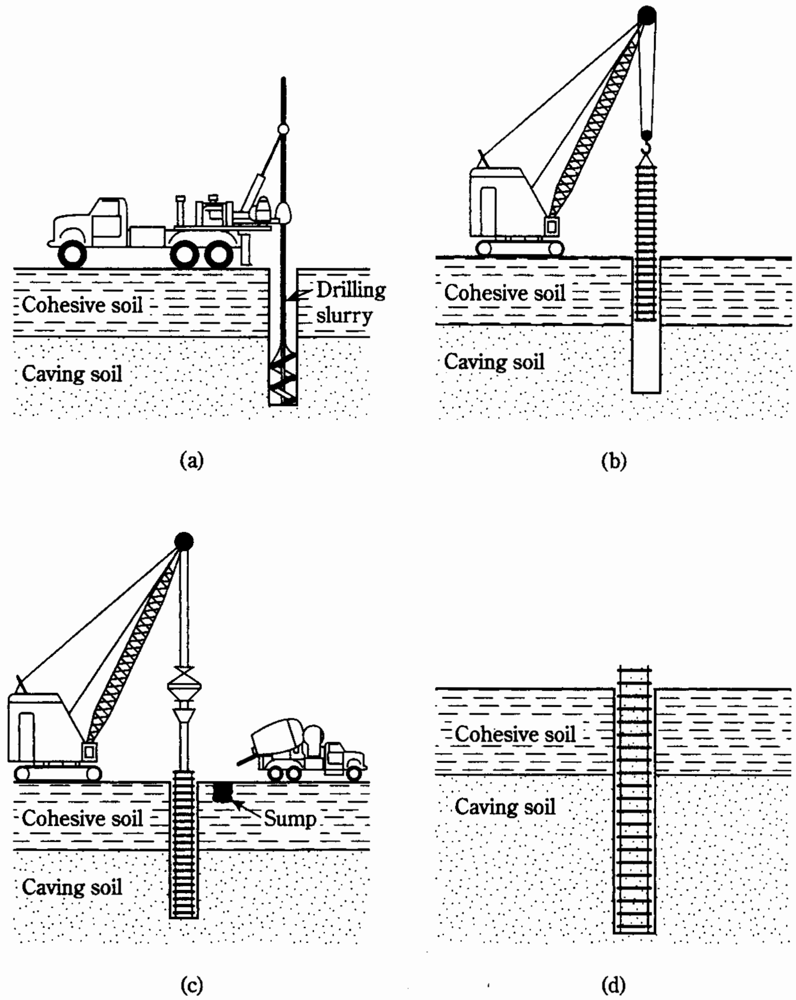

Figure 12.4 Slurry method of construction: (a) drilling to full depth with slurry; (b) placing rebar cage; (c) placing concrete; (d) completed shaft (after O'Neill and Reese, 1999)

or

$$D_s = \sqrt{\frac{Q_w}{\left(\frac{\pi}{4}\right)(0.25)f'_c}} = 2.257\sqrt{\frac{Q_u}{f'_c}} \tag{12.1}$$

where D_s = diameter of the shaft
f'_c = 28-day concrete strength
Q_w = working load of the drilled shaft

A_{gs} = gross cross-sectional area of the shaft

If drilled shafts are likely to be subjected to tensile loads, reinforcement should be continued for the entire length of the shaft.

Concrete Mix Design

The concrete mix design for drilled shafts is not much different from that for any other concrete structure. When a reinforcing cage is used, consideration should be given to the ability of the concrete to flow through the reinforcement. In most cases, a concrete slump of about 15.0 mm (6 in.) is considered satisfactory. Also, the maximum size of the aggregate should be limited to about 20 mm (0.75 in.).

12.5 Load Transfer Mechanism

The load transfer mechanism from drilled shafts to soil is similar to that of piles, as described in Section 11.5. Figure 12.5 shows the load test results of a drilled shaft in a clay soil in Houston, Texas (Reese, Touma, and O'Neill, 1976). This drilled shaft had a diameter of 0.76 m (2.5 ft) and a depth of penetration of 7.04 m (23.1 ft). The soil profile at the site is shown in Figure 12.5a. Figure 12.5b shows the load–settlement curves. It can be seen that the total load carried by the drilled shaft was 1246 kN (140 tons). The load carried by side resistance was about 800 kN (90 tons), and the rest was carried by point bearing. It is interesting to note that, with a downward movement of about 6.35 mm (0.25 in.), full side resistance was mobilized. However, about 25 mm ($\approx$1 in.) of downward movement was required for mobilization of full point resistance. This situation is similar to that observed in the case of piles. Figure 12.5c shows the load-distribution curves for different stages of the loading.

12.6 Estimation of Load-Bearing Capacity

The ultimate load-bearing capacity of a drilled shaft (see Figure 12.6) is

$$Q_u = Q_p + Q_s \tag{12.2}$$

where Q_u = ultimate load
Q_p = ultimate load-carrying capacity at the base
Q_s = frictional (skin) resistance

The ultimate base load Q_p can be expressed in a manner similar to the way it is expressed in the case of shallow foundations [Eq. (3.21)], or

$$Q_p = A_p\left(c'N_cF_{cs}F_{cd}F_{cc} + q'N_qF_{qs}F_{qd}F_{qc} + \frac{1}{2}\gamma'N_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma c}\right) \tag{12.3}$$

where c' = cohesion
N_c, N_q, N_γ = bearing capacity factors (Table 3.4)
$F_{cs}, F_{qs}, F_{\gamma s}$ = shape factors
$F_{cd}, F_{qd}, F_{\gamma d}$ = depth factors

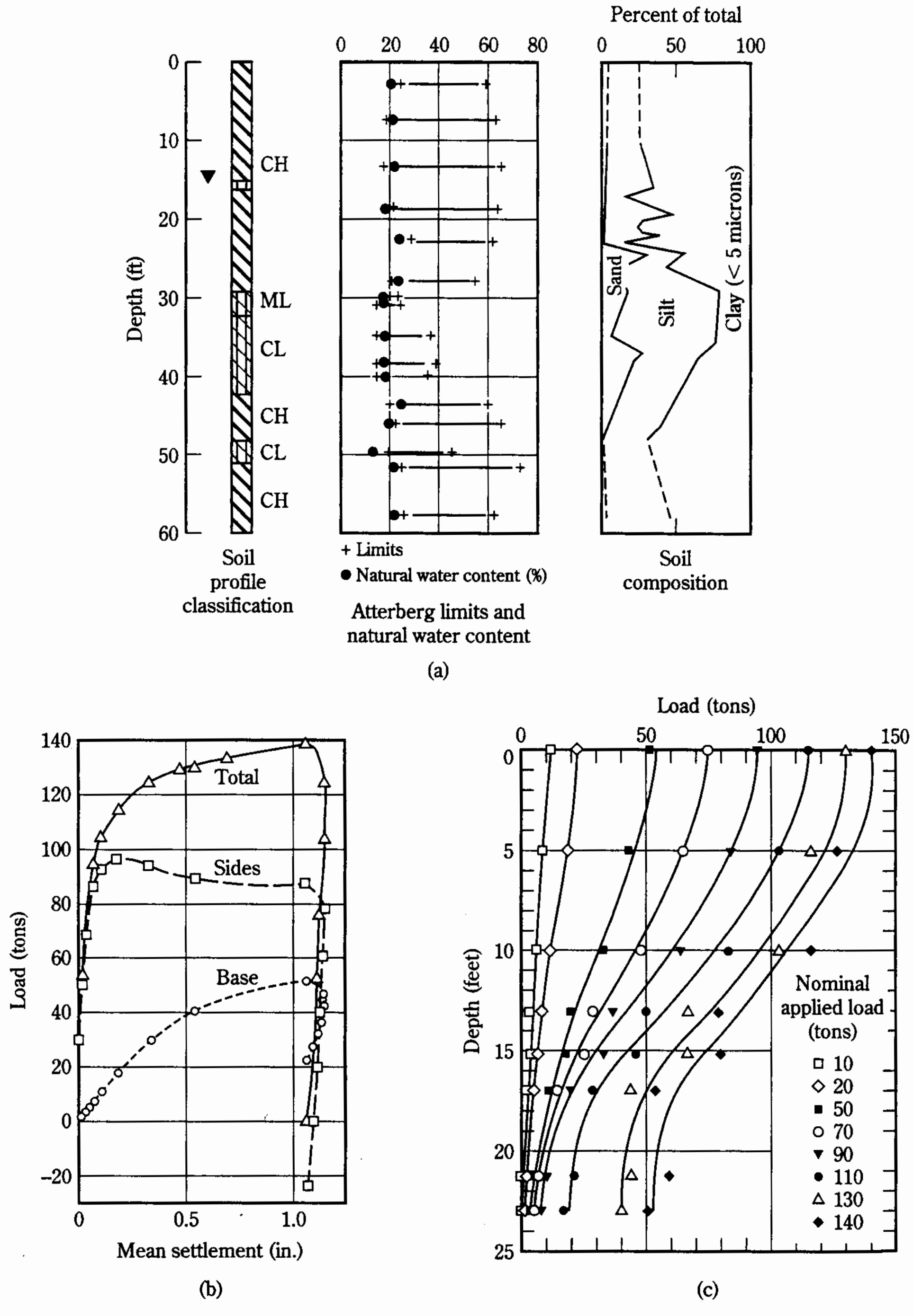

Figure 12.5 Load test results for a drilled shaft in Houston, Texas: (a) soil profile; (b) load–displacement curves; (c) load-distribution curves at various stages of loading (after Reese, Touma, and O'Neill, 1976)

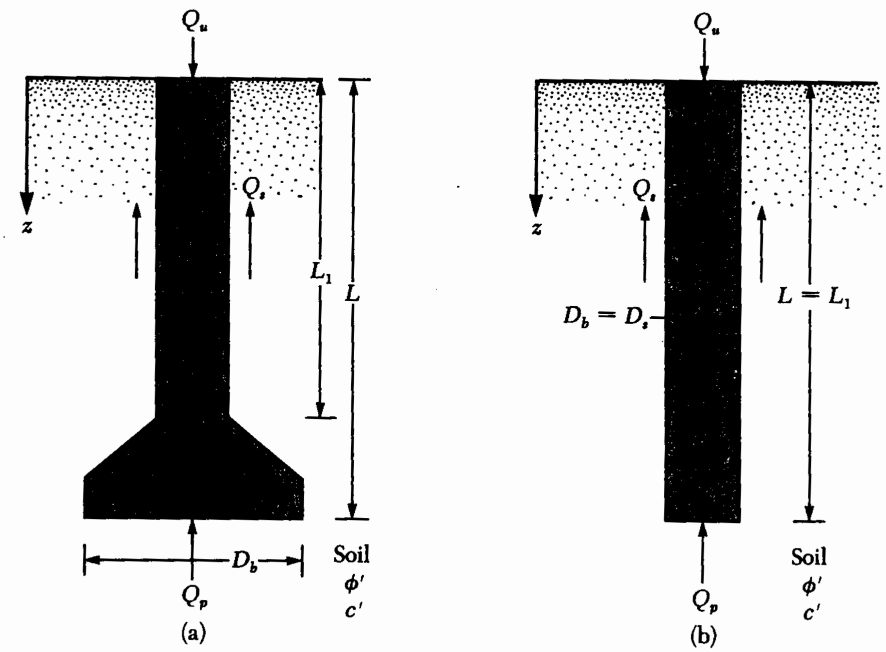

Figure 12.6 Ultimate bearing capacity of drilled shafts: (a) with bell; (b) straight shaft

$F_{cc}, F_{qc}, F_{\gamma c}$ = compressibility factors

γ' = effective unit weight of soil at the base of the shaft

q' = effective vertical stress at the base of the shaft

A_p = area of the base $= \dfrac{\pi}{4}D_b^2$

In most instances, the last term (the one containing N_γ) is neglected, except in the case of a relatively short drilled shaft. With this assumption, the net load-carrying capacity at the base (i.e., the gross load minus the weight of the drilled shaft) may be approximated as

$$Q_{p(\text{net})} = A_p[c'N_cF_{cs}F_{cd}F_{cc} + q'(N_q - 1)F_{qs}F_{qd}F_{qc}] \tag{12.4}$$

Using Eqs. (3.25), (3.26), and (3.32), we obtain

$$F_{cs} = 1 + \frac{N_q}{N_c} \tag{12.5}$$

$$F_{qs} = 1 + \tan\phi' \tag{12.6}$$

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan\phi'} \tag{12.7}$$

and

$$F_{qd} = 1 + 2 \tan \phi'(1 - \sin \phi')^2 \underbrace{\tan^{-1}\left(\frac{L}{D_b}\right)}_{\text{radians}} \tag{12.8}$$

According to Chen and Kulhawy (1994), F_{cc} and F_{qc} can be calculated in the following manner:

1. Calculate the critical rigidity index as

$$I_{rc} = 0.5\exp[2.85\cot(45 - \tfrac{\phi'}{2})] \tag{12.9}$$

2. Calculate the reduced rigidity index as

$$I_{rr} = \frac{I_r}{1 + I_r\Delta} \tag{12.10}$$

where I_r = soil rigidity index

$$= \frac{E_s}{2(1 + \mu_s)q' \tan \phi'} \tag{12.11}$$

in which E_s = drained modulus of elasticity of soil
μ_s = drained Poisson's ratio of soil
Δ = volumetric strain within the plastic zone during loading

3. If $I_{rr} \geq I_{rc}$ then

$$F_{cc} = F_{qc} = 1 \tag{12.12}$$

However, if $I_{rr} < I_{rc}$, then

$$F_{cc} = F_{qc} - \frac{1 - F_{qc}}{N_c \tan \phi'} \tag{12.13}$$

and

$$F_{qc} = \exp\left\{(-3.8 \tan \phi') + \left[\frac{(3.07 \sin \phi')(\log_{10} 2I_{rr})}{1 + \sin \phi'}\right]\right\} \tag{12.14}$$

The expression for the frictional, or skin, resistance Q_s is similar to that for piles; that is,

$$Q_s = \int_0^{L_1} pf\,dz \tag{12.15}$$

where p = shaft perimeter = πD_s
f = unit frictional (or skin) resistance

The next two sections describe the procedures for obtaining the *ultimate* and *allowable* load-bearing capacities of drilled shafts in sand and saturated clay ($\phi = 0$).

12.7 Drilled Shafts in Sand: Load-Bearing Capacity

For drilled shafts in sand, $c' = 0$; hence, Eq. (12.4) simplifies to

$$Q_{p(\text{net})} = A_p[q'(N_q - 1)F_{qs}F_{qd}F_{qc}] \tag{12.16}$$

The value of N_q for a given soil friction angle ϕ' can be determined from Table 3.4. The shape factor F_{qs} and depth factor F_{qd} can be evaluated from Eqs. (12.6) and (12.8), respectively. To calculate the compressibility factor F_{qc}, Eqs. (12.9), (12.10), (12.11), (12.12), (12.13) and (12.14) will have to be used. The terms E_s, μ_s, and Δ can be estimated by the relationship (Chen and Kulhawy, 1994)

$$\frac{E_s}{p_a} = m \tag{12.17}$$

where p_a = atmospheric pressure ($\approx$100 kN/m^2 or 2000 lb/ft^2)

$$m = \begin{cases} 100 \text{ to } 200 \text{ (loose soil)} \\ 200 \text{ to } 500 \text{ (medium dense soil)} \\ 500 \text{ to } 1000 \text{ (dense soil)} \end{cases}$$

$$\mu_s = 0.1 + 0.3\left(\frac{\phi' - 25}{20}\right) \quad (\text{for } 25° \leq \phi' \leq 45°) \tag{12.18}$$

and the formula

$$\Delta = 0.005\left(1 - \frac{\phi' - 25}{20}\right)\left(\frac{q'}{p_a}\right) \tag{12.19}$$

The magnitude of $Q_{p(\text{net})}$ can also be reasonably estimated from a relationship based on the analysis of Berezantzev et al. (1961) that can be expressed as

$$Q_{p(\text{net})} = A_p q'(\omega N_q^* - 1) \tag{12.20}$$

where N_q^* = bearing capacity factor = $0.21e^{0.17\phi'}$ (12.21)

ω = correction factor = $f(L/D_b)$

In Eq. (12.21), ϕ' is in degrees. The variation of ω with L/D_b is given in Figure 12.7.

The frictional resistance at ultimate load, Q_s, developed in a drilled shaft may be calculated from the relation given in Eq. (12.15), in which

$$p = \text{shaft perimeter} = \pi D_s$$

$$f = \text{unit frictional (or skin) resistance} = K\sigma'_o \tan\delta \tag{12.22}$$

where K = earth pressure coefficient $\approx K_o = 1 - \sin\phi'$

σ'_o = effective vertical stress at any depth z

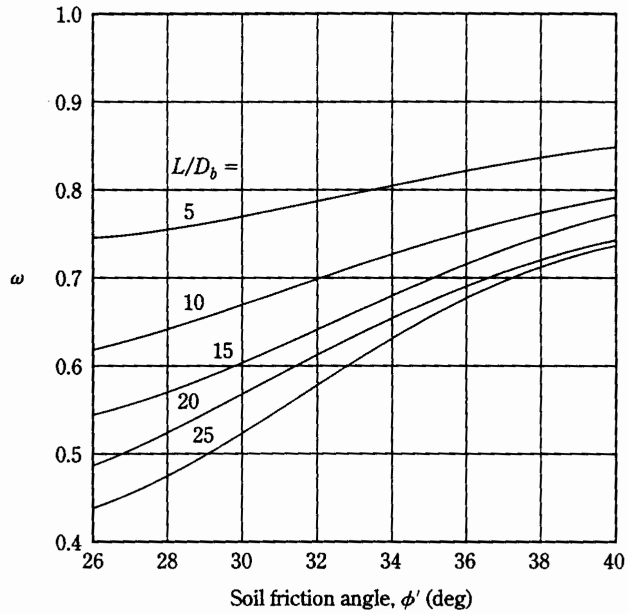

Figure 12.7 Variation of ω with ϕ' and L/D_b

Thus,

$$Q_s = \int_0^{L_1} pf\,dz = \pi D_s(1 - \sin\phi')\int_0^{L_1} \sigma_o' \tan\delta\,dz \tag{12.23}$$

The value of σ_o' will increase to a depth of about $15D_s$ and will remain constant thereafter, as shown in Figure 11.18.

An appropriate factor of safety should be applied to the ultimate load to obtain the net allowable load, or

$$Q_{\text{all(net)}} = \frac{Q_{p(\text{net})} + Q_s}{\text{FS}} \tag{12.24}$$

Load-Bearing Capacity Based on Settlement

On the basis of the performance of bored piles in sand with an average diameter of 750 mm (2.5 ft), Touma and Reese (1974) suggested a procedure for calculating the allowable load-carrying capacity. Their procedure, which is also applicable to drilled shafts in sand, is as follows:

For $L > 10D_b$ and a *base movement* of 25.4 mm (1 in.), the allowable net point load,

$$Q_{p\text{-all(net)}} = \frac{0.508A_p}{D_b} q_p \tag{12.25}$$

where $Q_{p\text{-all(net)}}$ is in kN, A_p is in m^2, D_b is in m, and q_p is the unit point resistance in kN/m^2

In English units,

$$Q_{p\text{-all(net)}} = \frac{A_p}{0.6D_b} q_p \tag{12.26}$$

where $Q_{p\text{-all(net)}}$ is in lb, A_p is in ft^2, D_b is in ft, and q_p is in lb/ft^2

The values of q_p, as recommended by Touma and Reese, are given in the following table:

Sand type	q_p (kN/m^2)	q_p (lb/ft^2)
Loose	0	0
Medium	1530	32,000
Very dense	3830	80,000

For sands of intermediate densities, linear interpolation can be used. The shaft friction resistance can be calculated as

$$Q_s = \int_0^{L_1} (0.7) p\sigma_o' \tan\phi' dz = 0.7(\pi D_s)\int_0^{L_1} \sigma_o' \tan\phi' \, dz = 2.2 D_s \int_0^{L_1} \sigma_o' \tan\phi' \, dz \tag{12.27}$$

where ϕ' = effective soil friction angle
σ_o' = vertical effective stress at a depth z

For the definition of L_1, see Figure 12.6. Thus,

$$Q_{\text{all(net)}} = Q_{p\text{-all(net)}} + \frac{Q_s}{\text{FS}} \quad \text{[for a base movement of 25.4 mm (1 in.)]} \tag{12.28}$$

where FS = factor of safety (≈ 2)

On the basis of a database of 41 loading tests, Reese and O'Neill (1989) proposed a method for calculating the load-bearing capacity of drilled shafts that is based on settlement. The method is applicable to the following ranges:

1. Shaft diameter: D_s = 0.52 m to 1.2 m (1.7 ft to 3.93 ft)
2. Bell depth: L = 4.7 m to 30.5 m (15.4 ft to 100 ft)
3. Field standard penetration resistance: N_{60} = 5 to 60
4. Concrete slump = 100 mm to 225 mm (4 in. to 9 in.)

Reese and O'Neill's procedure (see Figure 12.8) gives

$$Q_{u(\text{net})} = \sum_{i=1}^{N} f_i p \Delta L_i + q_p A_p \tag{12.29}$$

where f_i = ultimate unit shearing resistance in layer i
p = perimeter of the shaft = πD_s
q_p = unit point resistance
A_p = area of the base = $(\pi/4) D_b^2$

Following are the relationships for determining $Q_{u(\text{net})}$ from Eq. (12.29) for granular soils. We have

$$f_i = \beta \sigma'_{ozi} \leq 4\ \text{kip/ft}^2 \tag{12.30}$$

where σ'_{ozi} = vertical effective stress at the middle of layer i
$\beta = 1.5 - 0.135 z_i^{0.5} \quad (0.25 \leq \beta \leq 1.2)$ (12.31)
z_i = depth to the middle of layer i (ft)

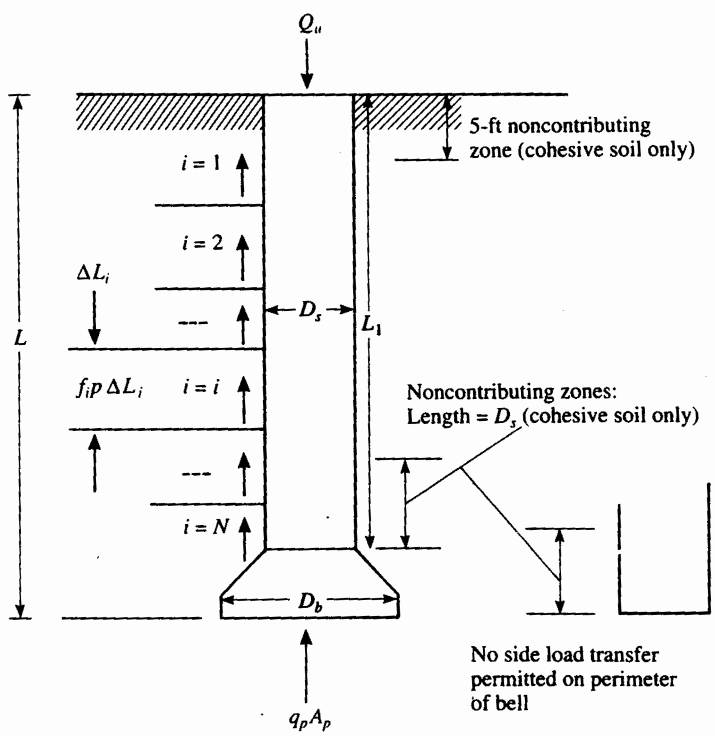

Figure 12.8 Development of Eq. (12.29)

The point bearing capacity is

$$q_p\ (\text{kip/ft}^2) = 1.2N_{60} \leqslant 90\ \text{kip/ft}^2 \qquad (\text{for } D_b < 50\ \text{in.}) \tag{12.32}$$

where N_{60} = mean *uncorrected* standard penetration number within a distance of $2D_b$ below the base of the drilled shaft

If D_b is equal to or greater than 50 in., excessive settlement may occur. In that case, q_p may be replaced by

$$q_{pr} = \frac{50}{D_b(\text{in.})} q_p \qquad (\text{for } D_b \geqslant 50\ \text{in.}) \tag{12.33}$$

Figures 12.9 and 12.10 may now be used to calculate the allowable load $Q_{all(net)}$, based on the desired level of settlement.

In SI units, Eqs. (12.30) through (12.33) will be of the form

$$f_i = \beta\sigma'_{ozi} \leqslant \quad 192\ \text{kN/m}^2 \tag{12.34}$$

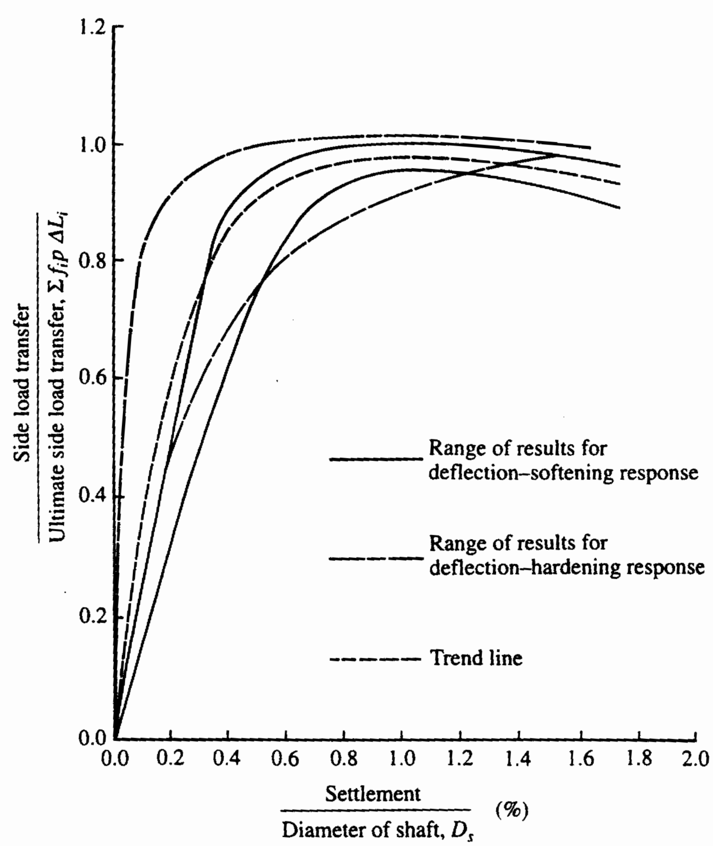

Figure 12.9 Normalized side load transfer vs. settlement for cohesionless soil (after Reese and O'Neill, 1989)

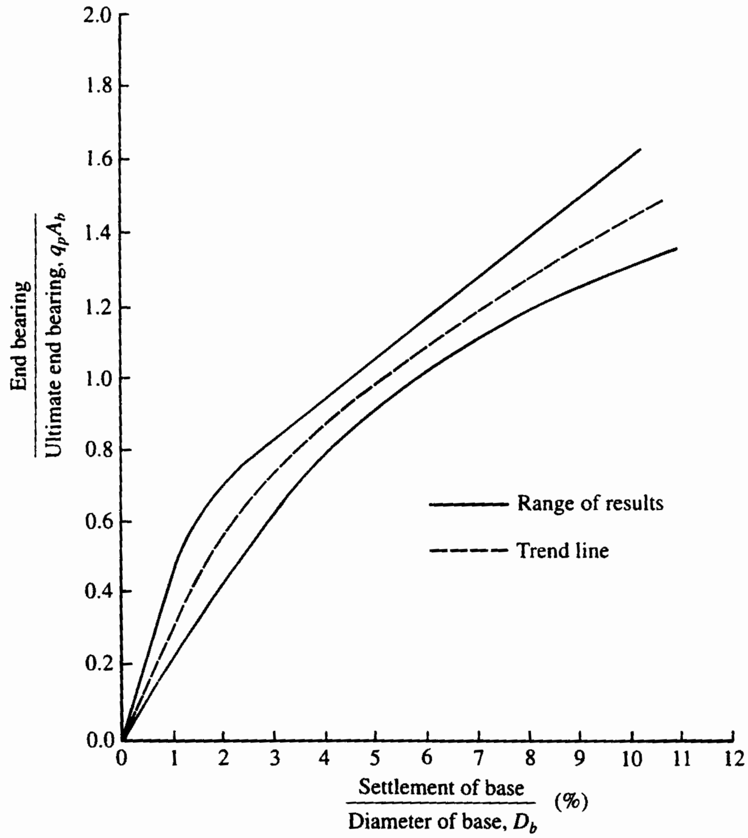

Figure 12.10 Normalized base load transfer vs. settlement for cohesionless soil (after Reese and O'Neill, 1989)

$$\beta = 1.5 - 0.244 z_i^{0.5} \qquad (0.25 \leq \beta \leq 1.2) \tag{12.35}$$

(where z_i is in m)

$$q_p\ (\text{kN/m}^2) = 57.5 N_{60} \leq 4310\ \text{kN/m}^2 \qquad (\text{for} D_b < 1.27\ \text{m}) \tag{12.36}$$

and

$$q_{pr} = \frac{1.27}{D_b(\text{m})} q_p \tag{12.37}$$

Example 12.1

A soil profile is shown in Figure 12.11. A point bearing drilled shaft with a bell is placed in a layer of dense sand and gravel. Determine the allowable load the drilled shaft could carry. Use Eq. (12.16) and a factor of safety of 4. Take $D_s = 1$ m and $D_b = 1.75$ m. For the dense sand layer, $\phi' = 36°$; $E_s = 500 p_a$. Ignore the frictional resistance of the shaft.

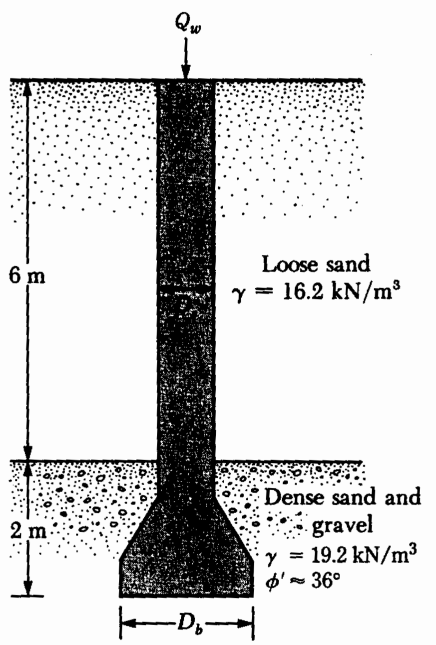

Figure 12.11 Allowable load of drilled shaft

Solution

We have

$$Q_{p(\text{net})} = A_p[q'(N_q - 1)F_{qs}F_{qd}F_{qc}]$$

and

$$q' = (6)(16.2) + (2)(19.2) = 135.6 \text{ kN/m}^2$$

For $\phi' = 36°$, from Table 3.4, $N_q = 37.75$. Also,

$$F_{qs} = 1 + \tan\phi' = 1 + \tan 36 = 1.727$$

and

$$F_{qd} = 1 + 2\tan\phi'(1 - \sin\phi')^2 \tan^{-1}\left(\frac{L}{D_b}\right)$$

$$= 1 + 2\tan 36(1 - \sin 36)^2 \tan^{-1}\left(\frac{8}{1.75}\right) = 1.335$$

From Eq. (12.9),

$$I_{rc} = 0.5\exp\left[2.85\cot\left(45 - \frac{\phi'}{2}\right)\right] = 134.3$$

From Eq. (12.17), $E_s = mp_a$. With $m = 500$, we have

$$E_s = (500)(100) = 50{,}000 \text{ kN/m}^2$$

From Eq. (12.18),

$$\mu_s = 0.1 + 0.3\left(\frac{\phi' - 25}{20}\right) = 0.1 + 0.3\left(\frac{36 - 25}{20}\right) = 0.265$$

So

$$I_r = \frac{E_s}{2(1 + \mu_s)(q')(\tan\phi')} = \frac{50{,}000}{2(1 + 0.265)(135.6)(\tan 36)} = 200.6$$

From Eq. (12.10),

$$I_{rr} = \frac{I_r}{1 + I_r\Delta}$$

with

$$\Delta = 0.005\left(1 - \frac{\phi' - 25}{20}\right)\frac{q'}{p_a} = 0.005\left(1 - \frac{36 - 25}{20}\right)\left(\frac{135.6}{100}\right) = 0.0031$$

it follows that

$$I_{rr} = \frac{200.6}{1 + (200.6)(0.0031)} = 123.7$$

I_{rr} is less than I_{rc}. So, from Eq. (12.14),

$$F_{qc} = \exp\left\{(-3.8\tan\phi') + \left[\frac{(3.07\sin\phi')(\log_{10} 2I_{rr})}{1 + \sin\phi'}\right]\right\}$$

$$= \exp\left\{(-3.8\tan 36) + \left[\frac{(3.07\sin 36)\log(2 \times 123.7)}{1 + \sin 36}\right]\right\} = 0.958$$

Hence,

$$Q_{p(\text{net})} = \left[\left(\frac{\pi}{4}\right)(1.75)^2\right](135.6)(37.75 - 1)(1.727)(1.335)(0.958) = 26{,}474 \text{ kN}$$

and

$$Q_{p(\text{all})} = \frac{Q_{p(\text{net})}}{FS} = \frac{26{,}474}{4} \approx \mathbf{6619\ kN}$$ ■

Example 12.2

Solve Example 12.1 using Eq. (12.20).

Solution

Equation (12.20) asserts that

$$Q_{p(\text{net})} = A_p q'(\omega N_q^* - 1)$$

We have

$$N_q^* = 0.21e^{0.17\phi'} = 0.21e^{(0.17)(36)} = 95.52$$

and

$$\frac{L}{D_b} = \frac{8}{1.75} = 4.57$$

From Figure 12.7, for $\phi' = 36°$ and $L/D_b = 4.57$, the value of ω is about 0.83. So

$$Q_{p(\text{net})} = \left[\left(\frac{\pi}{4}\right)(1.75)^2\right](135.6)[(0.83)(95.52) - 1] = 25{,}532 \text{ kN}$$

and

$$Q_{p(\text{all})} = \frac{25{,}532}{4} = \mathbf{6383\ kN}$$ ■

Example 12.3

A drilled shaft is shown in Figure 12.12. The uncorrected average standard penetration number (N_{60}) within a distance of $2D_b$ below the base of the shaft is about 30. Determine

- **a.** The ultimate load-carrying capacity
- **b.** The load-carrying capacity for a settlement of 0.5 in. Use Reese and O'Neill's method.

Solution

Part a

From Eqs. (12.30) and (12.31),

$$f_i = \beta\sigma'_{ozi}$$

and

$$\beta = 1.5 - 0.135z_i^{0.5}$$

For this problem, $z_i = 20/2 = 10$ ft, so

$$\beta = 1.5 - (0.135)(10)^{0.5} = 1.07$$

and

$$\sigma'_{ozi} = \gamma z_i = (100)(10) = 1000 \text{ lb/ft}^2$$

Thus,

$$f_i = (1000)(1.07) = 1070 \text{ lb/ft}^2$$

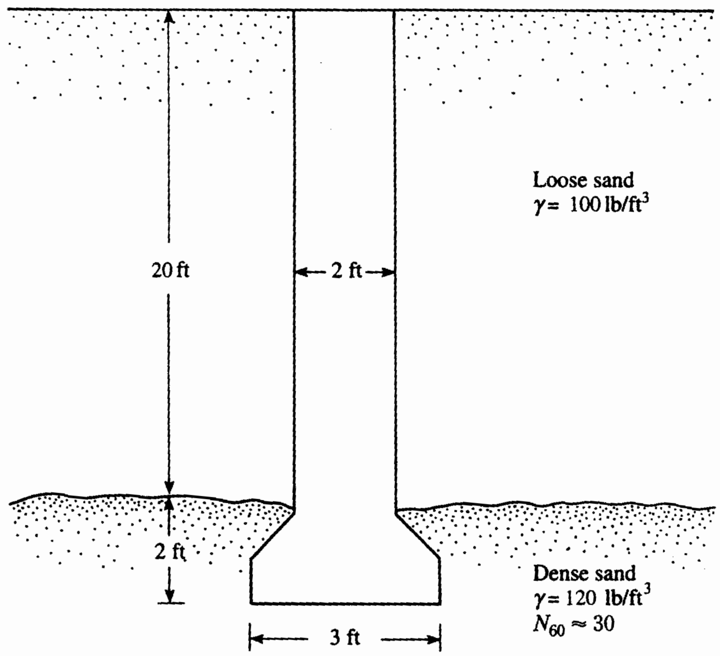

Figure 12.12 Drilled shaft supported by a dense layer of sand

and

$$\Sigma f_i p \,\Delta L_i = (1070)(\pi \times 2)(20) = 134{,}460 \text{ lb} = 134.46 \text{ kip}$$

From Eq. (12.32)

$$q_p = 1.2N_{60} = (1.2)(30) = 36 \text{ kip/ft}^2$$

so

$$q_p A_p = (36)\left[\frac{\pi}{4}(3)^2\right] = 254.47 \text{ kip}$$

Hence,

$$Q_{u(\text{net})} = q_p A_p + \Sigma f_i p \,\Delta L_i = 254.47 + 134.46 = \mathbf{388.9 \text{ kip}}$$

Part b

We have

$$\frac{\text{Allowable settlement}}{D_s} = \frac{0.5}{(2)(12)} = 0.021 = 2.1\%$$

The trend line shown in Figure 12.9 indicates that, for a normalized settlement of 2.1%, the normalized side load is about 0.9. Thus, the side load transfer is (0.9)(134.46) ≈ 121 kip. Similarly,

$$\frac{\text{Allowable settlement}}{D_b} = \frac{0.5}{(3)(12)} = 0.014 = 1.4\%$$

The trend line shown in Figure 12.10 indicates that, for a normalized settlement of 1.4%, the normalized base load is 0.312. So the base load is (0.312)(254.47) = 79.4 kip. Hence, the total load is

$$Q = 121 + 79.4 \approx \mathbf{200\ kip}$$ ■

12.8 Drilled Shafts in Clay: Load-Bearing Capacity

For saturated clays with $\phi = 0$, the bearing capacity factor N_q in Eq. (12.4) is equal to unity. Thus, for this case, Eq. (12.4) will be of the form

$$Q_{p(\text{net})} = A_p c_u N_c F_{cs} F_{cd} F_{cc} \tag{12.38}$$

where c_u = undrained cohesion

Assuming that $L \geqslant 3D_b$, we can rewrite Eq. (12.38) as

$$Q_{p(\text{net})} = A_p c_u N_c^* \tag{12.39}$$

where $N_c^* = N_c F_{cs} F_{cd} F_{cc} = 1.33[(\ln I_r) + 1]$ (12.40)

in which I_r = soil rigidity index

The soil rigidity index was defined in Eq. (12.11). For $\phi = 0$,

$$I_r = \frac{E_s}{3c_u} \tag{12.41}$$

O'Neill and Reese (1999) provided an approximate relationship between c_u and $E_s/3c_u$. This relationship is shown in Figure 12.13. For all practical purposes, if c_u/p_a is equal to or greater than unity (p_a = atmospheric pressure $\approx$ 100 kN/m^2 or 2000 lb/ft^2), then the magnitude of N_c^* can be taken to be 9.

Experiments by Whitaker and Cooke (1966) showed that, for belled shafts, the full value of $N_c^* = 9$ is realized with a base movement of about 10%–15% of D_b. Similarly, for straight shafts ($D_b = D_s$), the full value of $N_c^* = 9$ is obtained with a base movement of about 20% of D_b.

The expression for the skin resistance of drilled shafts in clay is similar to Eq. (11.54), or

$$Q_s = \sum_{L=0}^{L=L_1} \alpha^* c_u p \, \Delta L \tag{12.42}$$

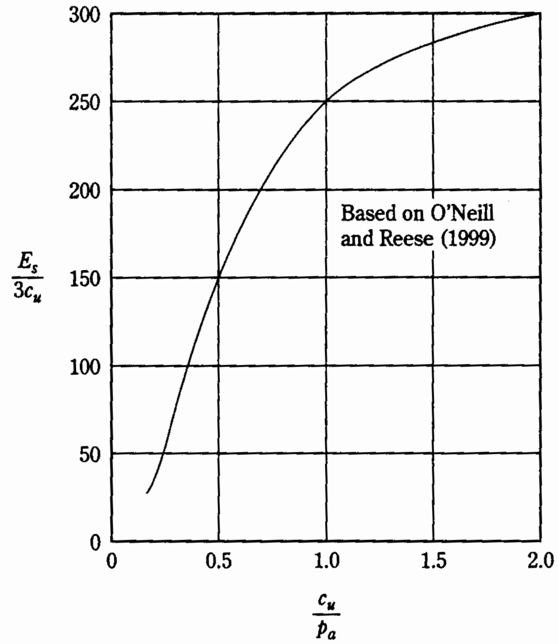

Figure 12.13 Approximate variation of $\frac{E_s}{3c_u}$ with c_u/p_a (*Note:* p_a = atmospheric pressure)

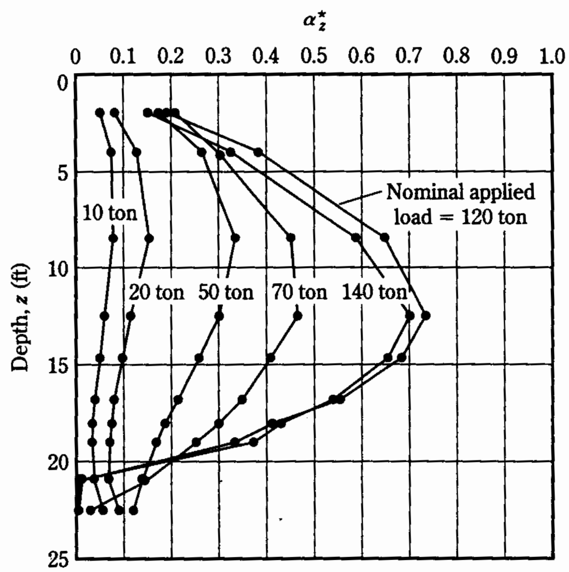

Figure 12.14 Variation of α_z^* with depth for the drilled shaft load test shown in Figure 12.5 (after Reese, Touma, and O'Neill, 1976)

where p = perimeter of the shaft cross section

The value of α^* that can be used in Eq. (12.42) has not yet been fully established. However, the field-test results available at this time indicate that α^* may vary between 1.0 and 0.3. Figure 12.14 shows the variation of α^* with depth (α_z^*) at various stages of loading for the case of the drilled shaft presented in Figure 12.5. The values of α_z^* were derived from Figure 12.5c. At ultimate load, the peak value of α_z^* is about 0.7, with an average of $\alpha^* \approx 0.5$.

Kulhawy and Jackson (1989) reported the field-test result of 106 straight drilled shafts—65 in uplift and 41 in compression. The magnitudes of α^* obtained from these tests are shown in Figure 12.15. The best correlation obtained from the results is

$$\alpha^* = 0.21 + 0.25\left(\frac{p_a}{c_u}\right) \leq 1 \tag{12.43}$$

where p_a = atmospheric pressure $\approx$ 1 ton/ft^2 ($\approx$100 kN/m^2)
So, conservatively, we may assume that

$$\alpha^* = 0.4 \tag{12.44}$$

Load-Bearing Capacity Based on Settlement

Reese and O'Neill (1989) suggested a procedure for estimating the ultimate and allowable (based on settlement) bearing capacities for drilled shafts in clay. According to this procedure, we can use Eq. (12.29) for the net ultimate load, or

$$Q_{u(\text{net})} = \sum_{i=1}^{n} f_i p \,\Delta L_i + q_p A_p$$

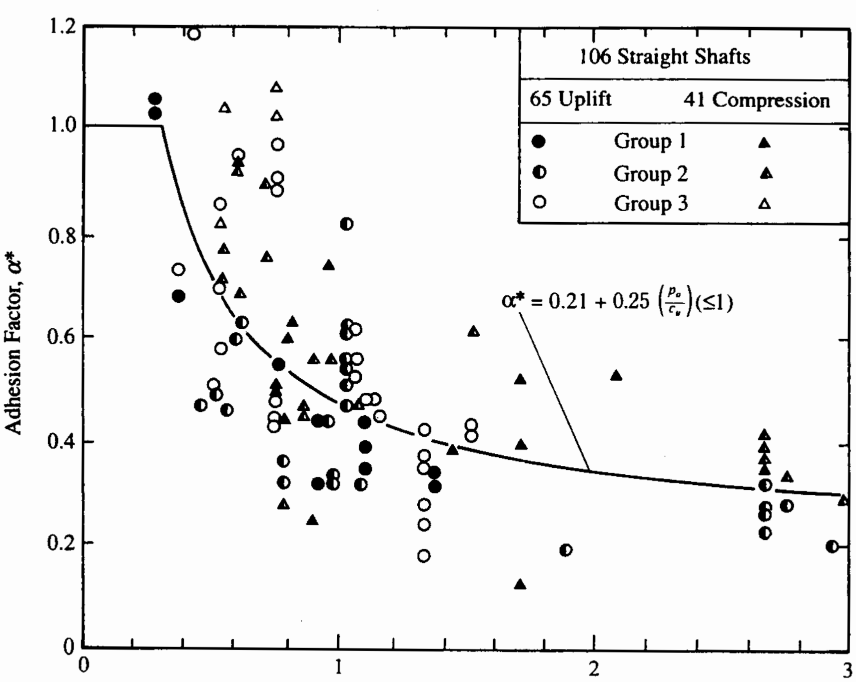

Figure 12.15 Variation of α^* with c_u/p_a (after Kulhawy and Jackson, 1989)

The unit skin friction resistance can be given as

$$f_i = \alpha_i^* c_{u(i)} \tag{12.45}$$

The following values are recommended for α_i^*:

$\alpha_i^* = 0$ for the top 1.5 m (5 ft) and bottom 1 diameter, D_s, of the drilled shaft. (*Note*: If $D_b > D_s$, then $\alpha^* = 0$ for 1 diameter above the top of the bell and for the peripheral area of the bell itself.)

$\alpha_i^* = 0.55$ elsewhere.

$$q_p = 6c_{ub}\left(1 + 0.2\frac{L}{D_b}\right) \leq 9c_{ub} \leq 80 \text{ kip/ft}^2 \tag{12.46}$$

where c_{ub} = average undrained cohesion within $2D_b$ below the base.

If D_b is large, excessive settlement will occur at the ultimate load per unit area, q_p, as given by Eq. (12.46). Thus, for $D_b > 1.91$ m (75 in.), q_p may be replaced by

$$q_{pr} = F_r q_p \tag{12.47}$$

where $$F_r = \frac{2.5}{\psi_1 D_b \text{ (in.)} + \psi_2} \leq 1 \tag{12.48}$$

in which

$$\psi_1 = 0.0071 + 0.0021\left(\frac{L}{D_b}\right) \leq 0.015 \tag{12.49}$$

and

$$\psi_2 = \underset{\substack{\uparrow \\ \text{kip/ft}^2}}{0.45(c_{ub})^{0.5}} \quad (0.5 \leq \psi_2 \leq 1.5) \tag{12.50}$$

In SI units, Eqs. (12.46), (12.48), (12.49), and (12.50) can be expressed as

$$q_p = 6c_{ub}\left(1 + 0.2\frac{L}{D_b}\right) \leq 9c_{ub} \leq 3830 \text{ kN/m}^2 \tag{12.51}$$

$$F_r = \frac{2.5}{\psi_1 D_b \text{ (mm)} + \psi_2} \leq 1 \tag{12.52}$$

$$\psi_1 = 2.78 \times 10^{-4} + 8.26 \times 10^{-5}\left(\frac{L}{D_b}\right) \leq 5.9 \times 10^{-4} \tag{12.53}$$

and

$$\psi_2 = 0.065[c_{ub}\ (\text{kN/m}^2)]^{0.5} \tag{12.54}$$

Figures 12.16 and 12.17 may now be used to evaluate the allowable load-bearing capacity, based on settlement. (Note that the ultimate bearing capacity in Figure 12.17 is q_p, not q_{pr}.) To do so,

1. Select a value of settlement, s.
2. Calculate $\sum_{i=1}^{N} f_i p\ \Delta L_i$ and $q_p A_p$.
3. Using Figures 12.16 and 12.17 and the calculated values in Step 2, determine the *side load* and the *end bearing load.*
4. The sum of the side load and the end bearing load gives the total allowable load.

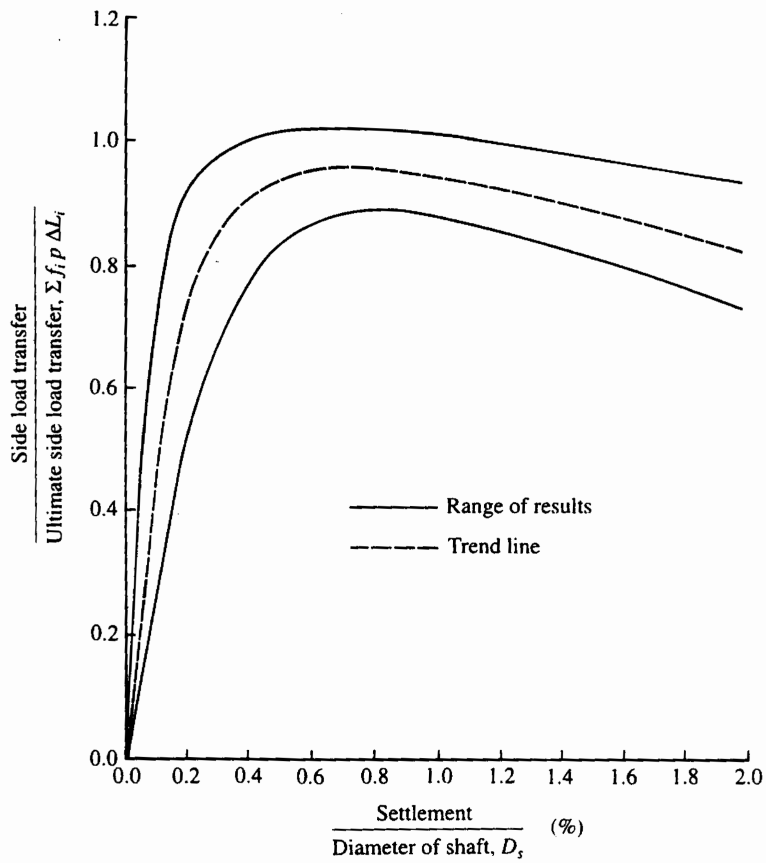

Figure 12.16 Normalized side load transfer vs. settlement for cohesive soil (after Reese and O'Neill, 1989)

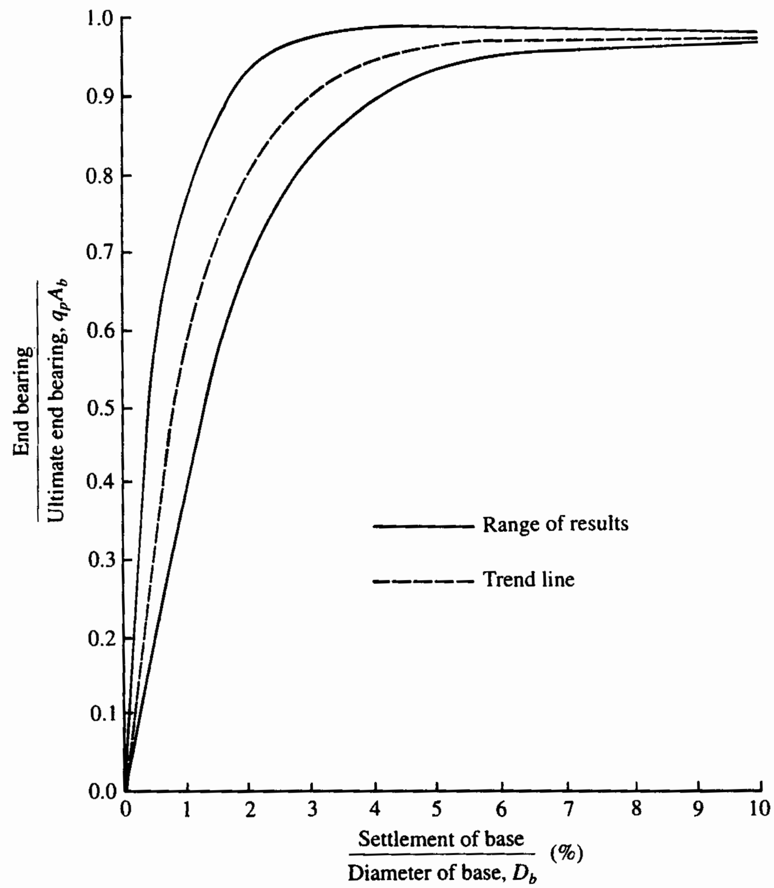

Figure 12.17 Normalized base load transfer vs. settlement for cohesive soil (after Reese and O'Neill, 1989)

Example 12.4

Figure 12.18 shows a drilled shaft without a bell. Here, $L_1 = 27$ ft, $L_2 = 8.5$ ft, $D_s = 3.3$ ft, $c_{u(1)} = 1000$ lb/ft^2, and $c_{u(2)} = 2175$ lb/ft^2. Determine

a. The net ultimate point bearing capacity
b. The ultimate skin resistance
c. The working load, Q_w (FS = 3)

Use Eqs. (12.39), (12.42), and (12.44).

Solution

Part a

From Eq. (12.39),

$$Q_{p(\text{net})} = A_p c_u N_c^* = A_p c_{u(2)} N_c^* = \left[\left(\frac{\pi}{4}\right)(3.3)^2\right](2175)(9)$$

$$= 167{,}425 \text{ lb} \approx \mathbf{167.4 \text{ kip}}$$

(*Note:* Since $c_u/p_a > 1$, $N_c^* \approx 9$.)

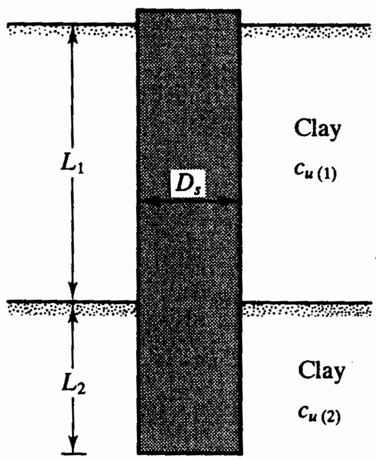

Figure 12.18 A drill shaft without a bell

Part b

From Eq. (12.42),

$$Q_s = \Sigma \alpha^* c_u p \Delta L$$

From Eq. (12.44),

$$\alpha^* = 0.4$$

$$p = \pi D_s = (3.14)(3.3) = 10.37 \text{ ft}$$

and

$$Q_s = (0.4)(10.37)[(1000 \times 27) + (2175 \times 8.5)]$$

$$= 188{,}682 \text{ lb} \approx \mathbf{188.7 \text{ kip}}$$

Part c

$$Q_w = \frac{Q_{p(\text{net})} + Q_s}{\text{FS}} = \frac{167.4 + 188.7}{3} = \mathbf{118.7 \text{ kip}}$$ ■

Example 12.5

A drilled shaft in a cohesive soil is shown in Figure 12.19. Use Reese and O'Neill's method to determine

a. The ultimate load-carrying capacity (Eqs. 12.45 through 12.50)
b. The load-carrying capacity for an allowable settlement of 0.5 in.

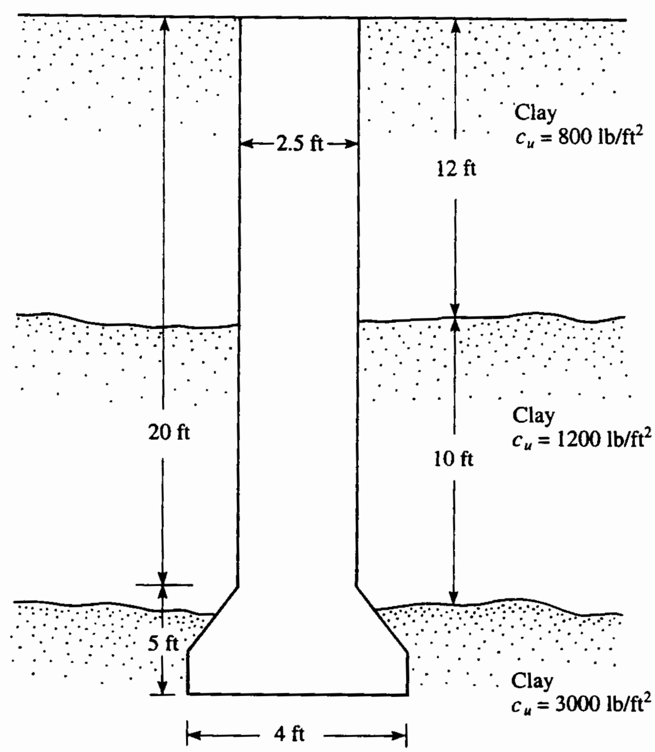

Figure 12.19 A drilled shaft in layered clay

Solution

Part a

From Eq. (12.45),

$$f_i = \alpha_i^* c_{u(i)}$$

From Figure 12.19

$$\Delta L_1 = 12 - 5 = 7 \text{ ft}$$

$$\Delta L_2 = (20 - 12) - D_s = (20 - 12) - 2.5 = 5.5 \text{ ft}$$

$$c_{u(1)} = 800 \text{ lb/ft}^2$$

and

$$c_{u(2)} = 1200 \text{ lb/ft}^2$$

Hence,

$$\Sigma f_i p \Delta L_i = \Sigma \alpha_i^* c_{u(i)} p \Delta L_i$$

$$= (0.55)(800)(\pi \times 2.5)(7) + (0.55)(1200)(\pi \times 2.5)(5.5)$$

$$= 52{,}700 \text{ lb} = 52.7 \text{ kip}$$

Again, from Eq. (12.46),

$$q_p = 6c_{ub}\left(1 + 0.2\frac{L}{D_b}\right) = (6)(3000)\left[1 + 0.2\left(\frac{20+5}{4}\right)\right] = 40{,}500 \text{ lb/ft}^2$$

$$= 40.5 \text{ kip/ft}^2$$

A check reveals that

$$q_p = 9c_{ub} = (9)(3000) = 27{,}000 \text{ lb/ft}^2 = 27 \text{ kip/ft}^2 < 40.5 \text{ kip/ft}^2$$

So we use $q_p = 27$ kip/ft²:

$$q_pA_p = q_p\left(\frac{\pi}{4}D_b^2\right) = (27)\left[\left(\frac{\pi}{4}\right)(4)^2\right] \approx 339.3 \text{ kip}$$

Hence,

$$Q_u = \Sigma\alpha_i^* c_{u(i)} p\Delta L_i + q_pA_p = 52.7 + 339.3 = \mathbf{392 \text{ kip}}$$

Part b

We have

$$\frac{\text{Allowable settlement}}{D_s} = \frac{0.5}{(2.5)(12)} = 0.167 = 1.67\%$$

The trend line shown in Figure 12.16 indicates that, for a normalized settlement of 1.67%, the normalized side load is about 0.89. Thus, the side load is

$$(0.89)(\Sigma f_i p\Delta L_i) = (0.89)(52.7) = 46.9 \text{ kip}$$

Again,

$$\frac{\text{Allowable settlement}}{D_b} = \frac{0.5}{(4)(12)} = 0.0104 = 1.04\%$$

The trend line shown in Figure 12.17 indicates that, for a normalized settlement of 1.04%, the normalized end bearing is about 0.57, so

$$\text{Base load} = (0.57)(q_pA_p) = (0.57)(339.3) = 193.4 \text{ kip}$$

Thus, the total load is

$$Q = 46.9 + 193.4 = \mathbf{240.3 \text{ kip}}$$

■

12.9 Settlement of Drilled Shafts at Working Load

The settlement of drilled shafts at working load is calculated in a manner similar to that outlined in Section 11.18. In many cases, the load carried by shaft resistance is small compared with the load carried at the base. In such cases, the contribution of

s_3 may be ignored. Note that in Eqs. (11.74) and (11.75) the term D should be replaced by D_b for drilled shafts.

Example 12.6

For the drilled shaft of Example 12.4, estimate the elastic settlement at working loads (i.e., $Q_w = 118.7$ kip). Use Eqs. (11.73), (11.75), and (11.76). Take $\xi = 0.65$, $E_p = 3 \times 10^6$ lb/in², $E_s = 2000$ lb/in², $\mu_s = 0.3$, and $Q_{wp} = 24.35$ kip.

Solution

From Eq. (11.73),

$$s_{e(1)} = \frac{(Q_{wp} + \xi Q_{ws})L}{A_p E_p}$$

Now,

$$Q_{ws} = 118.7 - 24.35 = 94.35 \text{ kip}$$

so

$$s_{e(1)} = \frac{[24.35 + (0.65 \times 94.35)](35.5)}{\left(\frac{\pi}{4} \times 3.3^2\right)\left(\frac{3 \times 10^6 \times 144}{1000}\right)} = 0.000823 \text{ ft} = 0.0099 \text{ in.}$$

From Eq. (11.75),

$$s_{e(2)} = \frac{Q_{wp} C_p}{D_b q_p}$$

From Table 11.11, for stiff clay, $C_p \approx 0.04$; also,

$$q_p = c_{u(b)} N_c^* = (2.175 \text{ kip/ft}^2)(9) = 19.575 \text{ kip/ft}^2$$

Hence,

$$s_{e(2)} = \frac{(24.35)(0.04)}{(3.3)(19.575)} = 0.015 \text{ ft} = 0.18 \text{ in.}$$

Again, from Eqs. (11.76) and (11.77),

$$s_{e(3)} = \left(\frac{Q_{ws}}{pL}\right)\left(\frac{D_s}{E_s}\right)(1 - \mu_s^2) I_{ws}$$

where $I_{ws} = 2 + 0.35\sqrt{\frac{L}{D_s}} = 2 + 0.35\sqrt{\frac{35}{3.3}} = 3.15$

So

$$s_{e(3)} = \left[\frac{94.35}{(\pi \times 3.3)(35.5)}\right]\left(\frac{3.3}{\frac{2000 \times 144}{1000}}\right)(1 - 0.3^2)(3.15) = 0.0084 \text{ ft} = 0.1 \text{ in.}$$

The total settlement is

$$s_e = s_{e(1)} + s_{e(2)} + s_{e(3)} = 0.0099 + 0.18 + 0.1 \approx \mathbf{0.29\ in.}$$ ■

12.10 Lateral Load-Carrying Capacity

Several methods for analyzing the lateral load-carrying capacity of piles, as well as the load-carrying capacity of drilled shafts, were presented in Section 11.9; therefore, they will not be repeated here. In 1994, Duncan et al. developed a *characteristic load method* for estimating the lateral load capacity for drilled shafts that is fairly simple to use. We describe this method next.

According to the characteristic load method, the *characteristic load* Q_c and *moment* M_c form the basis for the dimensionless relationship that can be given by the following correlations:

Characteristic Load

$$Q_c = 7.34D_s^2\,(E_pR_I)\left(\frac{c_u}{E_pR_I}\right)^{0.68} \quad \text{(for clay)} \qquad (12.55)$$

$$Q_c = 1.57D_s^2\,(E_pR_I)\left(\frac{\gamma' D_s\phi' K_p}{E_pR_I}\right)^{0.57} \quad \text{(for sand)} \qquad (12.56)$$

Characteristic Moment

$$M_c = 3.86D_s^3\,(E_pR_I)\left(\frac{c_u}{E_pR_I}\right)^{0.46} \quad \text{(for clay)} \qquad (12.57)$$

$$M_c = 1.33D_s^3(E_pR_I)\left(\frac{\gamma' D_s\phi' K_p}{E_pR_I}\right)^{0.40} \quad \text{(for sand)} \qquad (12.58)$$

In these equations, D_s = diameter of drilled shafts
E_p = modulus of elasticity of drilled shafts
R_I = ratio of moment of inertia of drilled shaft section to moment of inertia of a solid section (*Note:* $R_I = 1$ for uncracked shaft without central void)
γ' = effective unit weight of sand
ϕ' = effective soil friction angle (degrees)
K_p = Rankine passive pressure coefficient = $\tan^2(45 + \phi'/2)$

Deflection Due to Load Q_g Applied at the Ground Line

Table 12.1 gives the variation of x_o/D_s with Q_g/Q_c and Figure 12.20 gives the plot of Q_g/Q_c versus x_o/D_s for drilled shafts in sand and clay due to the load Q_g applied at the ground surface. Note that x_o is the ground line deflection. If the magnitudes of Q_g and

Table 12.1 Variation of x_o/D_s with Q_g/Q_c due to load Q_g applied at the ground surface

	Clay		Sand	
$\frac{x_o}{D_s}$	Free head $\frac{Q_g}{Q_c}$	Fixed head $\frac{Q_g}{Q_c}$	Free head $\frac{Q_g}{Q_c}$	Fixed head $\frac{Q_g}{Q_c}$
0.0000	0.0000	0.0000	0.0000	0.0000
0.0025	0.0040	0.0088	0.0008	0.0016
0.0050	0.0065	0.0133	0.0013	0.0028
0.0075	0.0078	0.0168	0.0017	0.0039
0.0100	0.0091	0.0197	0.0021	0.0049
0.0150	0.0113	0.0247	0.0027	0.0065
0.0200	0.0135	0.0289	0.0033	0.0079
0.0300	0.0171	0.0359	0.0043	0.0104
0.0400	0.0200	0.0419	0.0052	0.0125
0.0500	0.0226	0.0471	0.0060	0.0144
0.0600	0.0250	—	0.0068	—
0.0800	0.0292	—	0.0083	—
0.1000	0.0332	—	0.0097	—
0.1500	0.0412	—	0.0124	—

After Duncan et al. (1994)

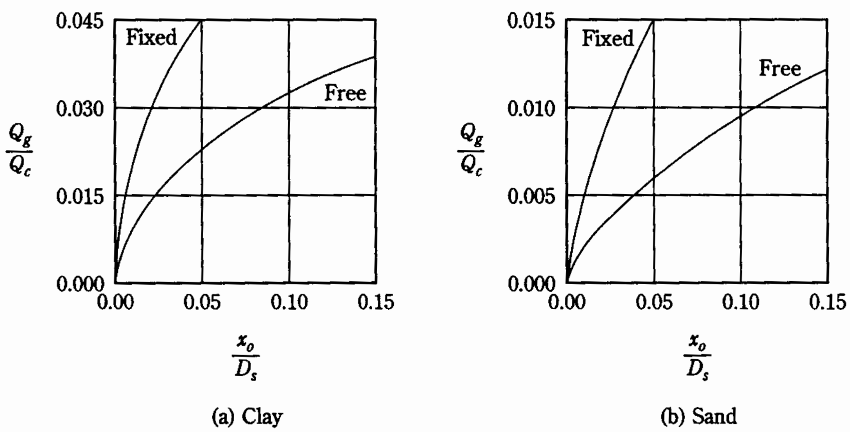

Figure 12.20 Plot of Q_g/Q_c vs. x_o/D_s (after Duncan et al., 1994)

Q_c are known, the ratio Q_g/Q_c can be calculated. The table or the figure can then be used to estimate the corresponding value of x_o/D_s and, hence, x_o.

Deflection Due to Moment Applied at the Ground Line

Table 12.2 gives the variation of x_o/D_s with M_g/M_c and Figure 12.21 gives the variation plot of M_g/M_c with x_o/D_s for drilled shafts in sand and clay due to an applied moment M_g at the ground line. Again, x_o is the ground line deflection. If the magnitudes of M_g, M_c, and D_s are known, the value of x_o can be calculated with the use of the table or the figure.

Table 12.2 Moment–deflection coefficients

	Applied Moment	
$\frac{x_o}{D_s}$	Clay M_g/M_c	Sand M_g/M_c
0.00	0.0000	0.0000
0.01	0.0048	0.0019
0.02	0.0074	0.0032
0.03	0.0097	0.0044
0.04	0.0119	0.0055
0.05	0.0139	0.0065
0.06	0.0158	0.0075
0.08	0.0193	0.0094
0.10	0.0226	0.0113
0.15	0.0303	0.0150

After Duncan et al. (1994)

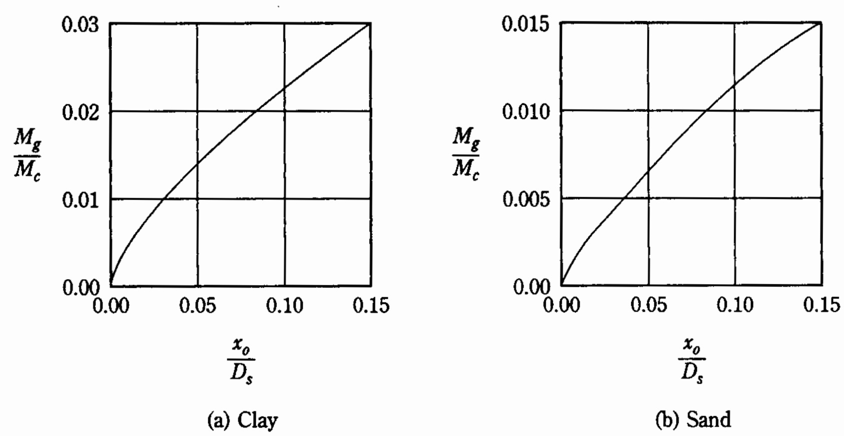

Figure 12.21 Plot of M_g/M_c vs. x_o/D_s (after Duncan et al., 1994)

Deflection Due to Load Applied Above the Ground Line

When a load Q is applied above the ground line, it induces both a load $Q_g = Q$ and a moment $M_g = Qe$ at the ground line, as shown in Figure 12.22. A superposition solution can now be used to obtain the ground line deflection. The step-by-step procedure is as follows:

1. Calculate Q_g and M_g.
2. Calculate the deflection x_{oQ} (see Figure 12.22a) that would be caused by the load Q_g acting alone. Use Table 12.1 or Figure 12.20.
3. Calculate the deflection x_{oM} (see Figure 12.22b) that would be caused by the moment acting alone. Use Table 12.2 or Figure 12.21.
4. Determine the value of a load Q_{gM} that would cause the same deflection as the moment (i.e., x_{oM}). Use Table 12.1 or Figure 12.20. This is schematically shown in Figure 12.22c.

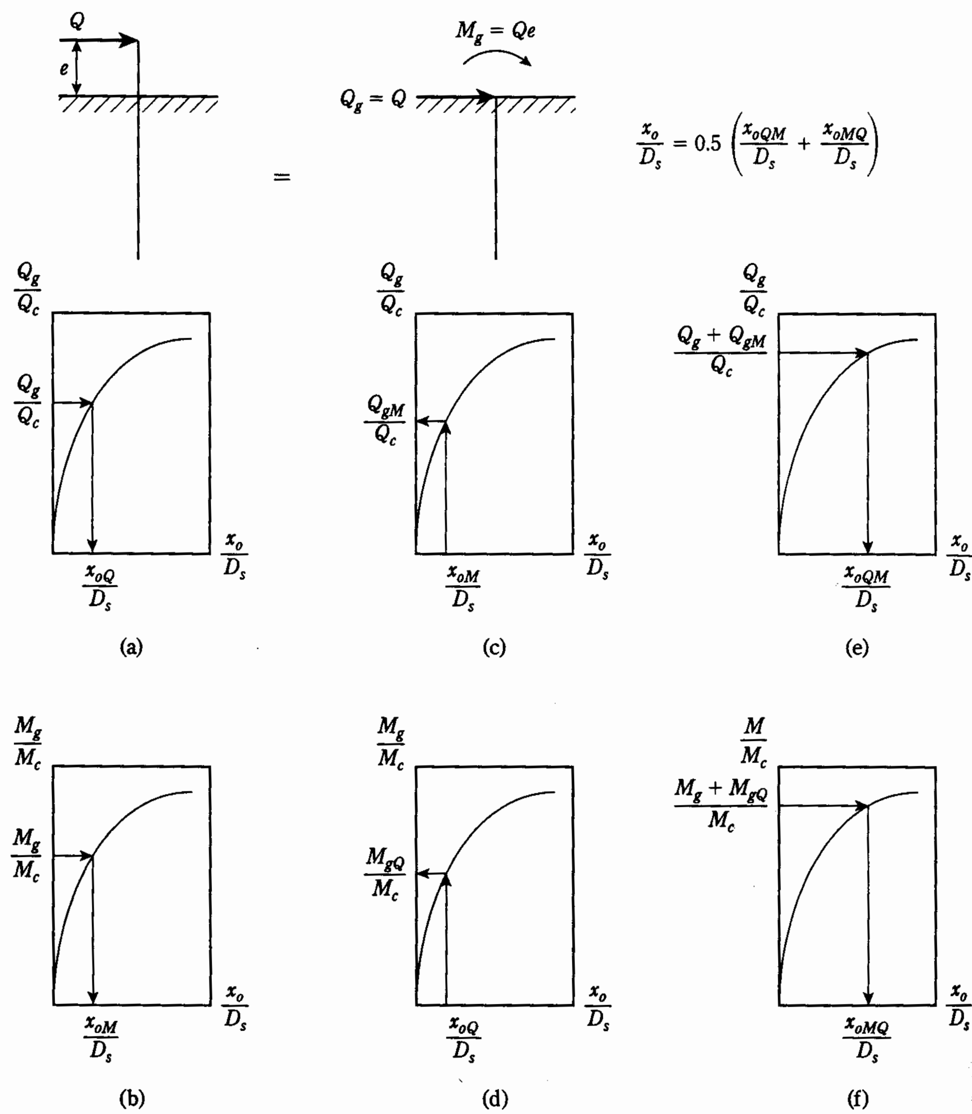

Figure 12.22 Nonlinear superposition of deflections due to load and moment (after Duncan et al., 1994)

5. Determine the value of a moment M_{gQ} that would cause the same deflection as the load (i.e, x_{oQ}), as shown in Figure 12.22d. Use Table 12.2 or Figure 12.21.
6. Calculate $(Q_g + Q_{gM})/Q_c$. Use Table 12.1 or Figure 12.20 to determine x_{oQM}/D_s. (See Figure 12.22e.)
7. Calculate $(M_g + M_{gQ})/M_c$. Use Table 12.2 or Figure 12.21 to determine x_{oMQ}/D_s. (See Figure 12.22f.)
8. Calculate the combined deflection: $x_{o(\text{combined})} = 0.5(x_{oQM} + x_{oMQ})$ (12.59)

Maximum Moment in Drilled Shaft Due to Ground Line Load Only

Figure 12.23 shows the plot of Q_g/Q_c with M_{max}/M_c and Table 12.3 shows the variation of M_{max}/M_c with Q_g/Q_c for fixed- and free-headed drilled shafts due only to the application of a ground line load Q_g. For fixed-headed shafts, the maximum moment in the shaft, M_{max}, occurs at the ground line. For this condition, if Q_c, M_c, and Q_g are known, the magnitude of M_{max} can be easily calculated.

Maximum Moment Due to Load and Moment at Ground Line

If a load Q_g and a moment M_g are applied at the ground line, the maximum moment in the drilled shaft can be determined in the following manner:

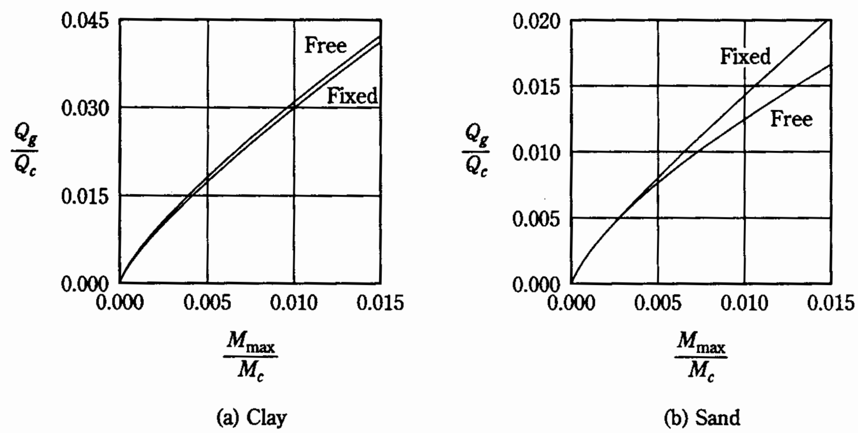

Figure 12.23 Variation of Q_g/Q_c with M_{max}/M_c (after Duncan et al., 1994)

Table 12.3 Load–moment coefficients

M_{max}/M_c	Clay: Free head $\frac{Q_g}{Q_c}$	Clay: Fixed head $\frac{Q_g}{Q_c}$	Sand: Free head $\frac{Q_g}{Q_c}$	Sand: Fixed head $\frac{Q_g}{Q_c}$
0.00	0.0000	0.0000	0.0000	0.0000
0.001	0.0050	0.0041	0.0021	0.0019
0.002	0.0090	0.0078	0.0038	0.0037
0.003	0.0125	0.0112	0.0052	0.0052
0.004	0.0157	0.0144	0.0065	0.0067
0.005	0.0185	0.0175	0.0076	0.0080
0.006	0.0212	0.0204	0.0087	0.0093
0.008	0.0264	0.0258	0.0107	0.0117
0.010	0.0319	0.0308	0.0126	0.0138
0.015	0.0432	0.0419	0.0168	0.0186

After Duncan et al. (1994)

1. Using the procedure described before, calculate $x_{o(\text{combined})}$ from Eq. (12.59).
2. To solve for the characteristic length T, use the following equation:

$$x_{o(\text{combined})} = \frac{2.43Q_g}{E_pI_p}T^3 + \frac{1.62M_g}{E_pI_p}T^2 \tag{12.60}$$

3. The moment in the shaft at a depth z below the ground surface can be calculated as

$$M_z = A_mQ_gT + B_mM_g \tag{12.61}$$

where A_m, B_m = dimensionless moment coefficients (Matlock and Reese, 1961); see Figure 12.24

The value of the maximum moment M_{max} can be obtained by calculating M_z at various depths in the upper part of the drilled shaft.

The characteristic load method just described is valid only if L/D_s has a certain minimum value. If the actual L/D_s is less than $(L/D_s)_{min}$, then the ground line deflections will be underestimated and the moments will be overestimated. The values of $(L/D_s)_{min}$ for drilled shafts in sand and clay are given in the following table:

Clay	$\frac{E_pR_I}{c_u}$	$(L/D_s)_{min}$
	1×10^5	6
	3×10^5	10
	1×10^6	14
	3×10^6	18

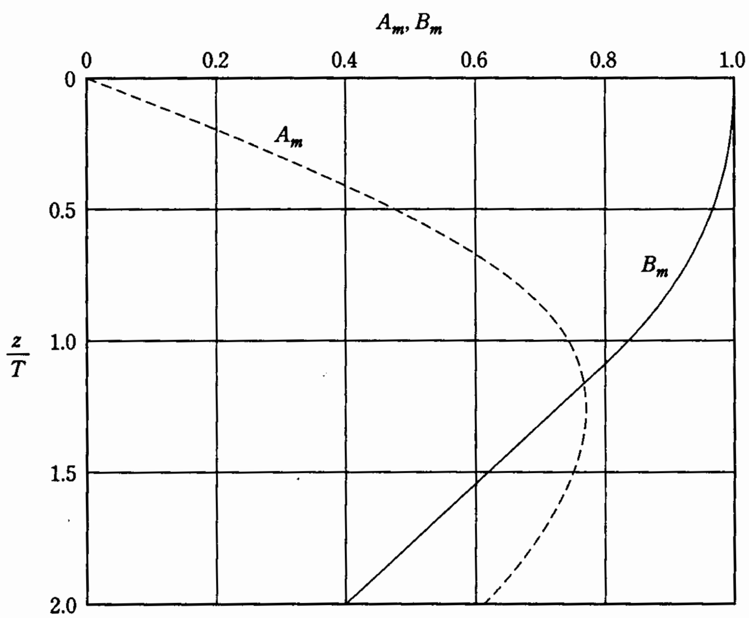

Figure 12.24 Variation of A_m and B_m with z/T

Sand	$\dfrac{E_p R_I}{\gamma' D_s \phi' K_p}$	$(L/D_s)_{min}$
	1×10^4	8
	4×10^4	11
	2×10^5	14

Example 12.7

Figure 12.25 shows a drilled shaft in sand. If $L = 6$ m, $D_s = 800$ mm, average horizontal soil modulus $E_s = 35 \times 10^3$ kN/m^2, and $E_p = 20.7 \times 10^6$ kN/m^2, estimate the ultimate lateral load, $Q_{u(g)}$, applied at the ground surface. Use Meyerhof's method given in Section 11.19, and check your results with Eq. (11.106).

Solution

From Eq. (11.104), the relative stiffness of the shaft is

$$K_r = \frac{E_p I_p}{E_s L^4}$$

with

$$I_p = \frac{\pi}{64} D_s^4 = \frac{\pi}{64}\left(\frac{800}{1000}\right)^4 = 0.02 \text{ m}^4$$

it follows that

$$K_r = \frac{(20.7 \times 10^6)(0.02)}{(35 \times 10^3)(6)^4} = 0.009$$

Since K_r is less than 0.01, this is a flexible drilled shaft. From Eq. (11.109),

$$\frac{L_e}{L} = 1.65\, K_r^{0.12}$$

so

$$L_e = (1.65)(0.009)^{0.12}(6) = 5.63 \text{ m}$$

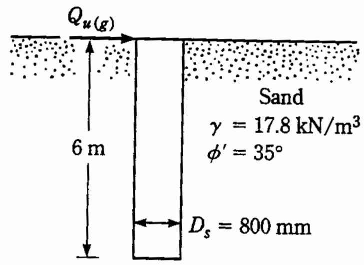

Figure 12.25 Ultimate lateral load of a drilled shaft

Thus, from Eq. (11.105)

$$Q_{u(g)} = 0.12\gamma\, D_s L_e^2 K_{br} \leq 0.4 p_l D L_e$$

Hence,

$$\frac{L_e}{D_s} = \frac{5.63}{0.8} = 7.04$$

From Figure 11.39, for $L_e/D_s = 7.04$ and $\phi' = 35°$, the value of $K_{br} \approx 10$, so

$$Q_{u(g)} = (0.12)(17.8)(0.8)(5.63)^2(10) = 541.6 \text{ kN}$$

As a check, we have

$$Q_{u(g)} = 0.4 p_l D_s L_e = (0.4)\underset{\substack{\uparrow \\ \text{Eq. 11.106}}}{(40 N_q \tan \phi')} D_s L_e$$

For $\phi' = 35°$, $N_q = 33.3$ (see Table 3.4); thus,

$$Q_{u(g)} = (0.4)(40)(33.3)(\tan 35)(0.8)(5.63) = 1680.3 \text{ kN}$$

Consequently,

$$Q_{u(g)} = \mathbf{541.6\ kN}$$ ■

Example 12.8

A free-headed drilled shaft in clay is shown in Figure 12.26. Let $E_p = 22 \times 10^6$ kN/m². Determine

a. the ground line deflection, $x_{o(\text{combined})}$
b. the maximum bending moment in the drilled shaft
c. the maximum tensile stress in the shaft
d. the minimum penetration of the shaft needed for this analysis

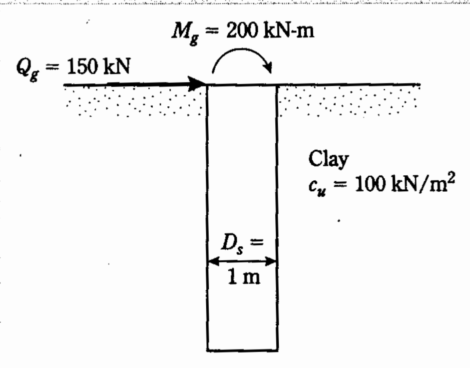

Figure 12.26 Free-headed drilled shaft

Solution

We are given

$$D_s = 1 \text{ m}$$
$$c_u = 100 \text{ kN/m}^2$$
$$R_I = 1$$
$$E_p = 22 \times 10^6 \text{ kN/m}^2$$

and

$$I_p = \frac{\pi D_s^4}{64} = \frac{(\pi)(1)^4}{64} = 0.049 \text{ m}^4$$

Part a

From Eq. (12.55),

$$Q_c = 7.34D_s^2\,(E_pR_I)\left(\frac{c_u}{E_pR_I}\right)^{0.68}$$
$$= (7.34)(1)^2[(22 \times 10^6)(1)]\left[\frac{100}{(22 \times 10^6)(1)}\right]^{0.68}$$
$$= 37{,}607 \text{ kN}$$

From Eq. (12.57),

$$M_c = 3.86D_s^3\,(E_pR_I)\left(\frac{c_u}{E_pR_I}\right)^{0.46}$$
$$= (3.86)(1)^3[(22 \times 10^6)(1)]\left[\frac{100}{(22 \times 10^6)(1)}\right]^{0.46}$$
$$= 296{,}139 \text{ kN-m}$$

Thus,

$$\frac{Q_g}{Q_c} = \frac{150}{37{,}607} = 0.004$$

From Table 12.1, $x_{oQ} \approx (0.0025)D_s = 0.0025 \text{ m} = 2.5 \text{ mm}$. Also,

$$\frac{M_g}{M_c} = \frac{200}{296{,}139} = 0.000675$$

From Table 12.2, $x_{oM} \approx (0.0014)D_s = 0.0014 \text{ m} = 1.4 \text{ mm}$, so

$$\frac{x_{oM}}{D_s} = \frac{0.0014}{1} = 0.0014$$

From Table 12.1, for $x_{oM}/D_s = 0.0014$, the value of $Q_{gM}/Q_c \approx 0.002$. Hence,

$$\frac{x_{oQ}}{D_s} = \frac{0.0025}{1} = 0.0025$$

From Table 12.2, for $x_{oQ}/D_s = 0.0025$, the value of $M_{gQ}/M_c \approx 0.0013$, so

$$\frac{Q_g}{Q_c} + \frac{Q_{gM}}{Q_c} = 0.004 + 0.002 = 0.006$$

From Table 12.1, for $(Q_g + Q_{gM})/Q_c = 0.006$, the value of $x_{oQM}/D_s \approx 0.0046$. Hence,

$$x_{oQM} = (0.0046)(1) = 0.0046 \text{ m} = 4.6 \text{ mm}$$

Thus, we have

$$\frac{M_g}{M_c} + \frac{M_{gQ}}{M_c} = 0.000675 + 0.0013 \approx 0.00198$$

From Table 12.2, for $(M_g + M_{gQ})/M_c = 0.00198$, the value of $x_{oMQ}/D_s \approx 0.0041$. Hence,

$$x_{oMQ} = (0.0041)(1) = 0.0041 \text{ m} = 4.1 \text{ mm}$$

Consequently,

$$x_{o\,(\text{combined})} = 0.5(x_{oQM} + x_{oMQ}) = (0.5)(4.6 + 4.1) = \mathbf{4.35\ mm}$$

Part b

From Eq. (12.60),

$$x_{o\,(\text{combined})} = \frac{2.43Q_g}{E_pI_p}T^3 + \frac{1.62M_g}{E_pI_p}T^2$$

so

$$0.00435 \text{ m} = \frac{(2.43)(150)}{(22 \times 10^6)(0.049)}T^3 + \frac{(1.62)(200)}{(22 \times 10^6)(0.049)}T^2$$

or

$$0.00435 \text{ m} = 338 \times 10^{-6}T^3 + 300.6 \times 10^{-6}T^2$$

and it follows that

$$T \approx 2.05 \text{ m}$$

From Eq. (12.61),

$$M_z = A_m Q_g T + B_m M_g = A_m(150)(2.05) + B_m(200) = 307.5A_m + 200\,B_m$$

Now the following table can be prepared:

$\frac{z}{T}$	A_m (Figure 12.24)	B_m (Figure 12.24)	M_z (kN-m) [Eq. (a)]
0	0	1.0	200
0.4	0.36	0.98	306.7
0.6	0.52	0.95	349.9
0.8	0.63	0.9	373.7
1.0	0.75	0.845	399.6
1.1	0.765	0.8	395.2
1.25	0.75	0.73	376.6

So the maximum moment is 399.4 kN-m ≈ 400 kN-m and occurs at $z/T \approx 1$. Hence,

$$z = (1)(T) = (1)(2.05\text{ m}) = \mathbf{2.05\text{ m}}$$

Part c

The maximum tensile stress is

$$\sigma_{\text{tensile}} = \frac{M_{\max}\left(\frac{D_s}{2}\right)}{I_p} = \frac{(400)\left(\frac{1}{2}\right)}{0.049} = \mathbf{4081.6\text{ kN/m}^2}$$

Part d

We have

$$\frac{E_p R_I}{c_u} = \frac{(22 \times 10^6)(1)}{100} = 2.2 \times 10^5$$

By interpolation, for $(E_p R_I)/c_u = 2.2 \times 10^5$, the value of $(L/D_s)_{\min} \approx 8.5$. So

$$L \approx (8.5)(1) = \mathbf{8.5\text{ m}}$$ ■

12.11 Drilled Shafts Extending into Rock

In Section 12.1, we noted that drilled shafts can be extended into rock. In the current section, we describe the principles of analysis of the load-bearing capacity of such drilled shafts, based on the procedure developed by Reese and O'Neill (1988, 1989). Figure 12.27 shows a drilled shaft whose depth of embedment in rock is equal to L. In the design process to be recommended, it is assumed that *there is either side resistance between the shaft and the rock or point resistance at the bottom, but not both.* Following is a step-by-step procedure for estimating the ultimate bearing capacity:

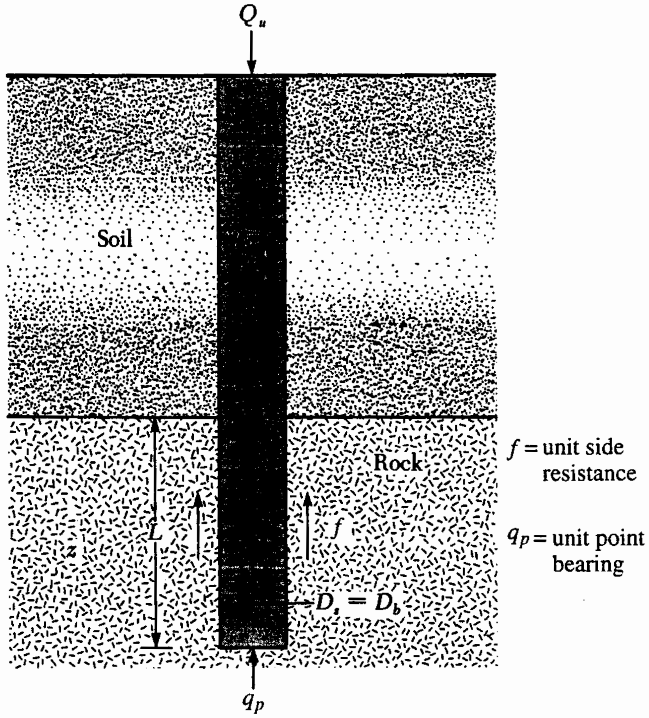

Figure 12.27 Drilled shaft socketed into rock

1. Calculate the ultimate unit side resistance as

$$f\,(\text{lb/in}^2) = 2.5q_u^{0.5} \leq 0.15q_u \tag{12.62}$$

where q_u = unconfined compression strength of a rock core of NW size or larger, or of the drilled shaft concrete, whichever is smaller (in lb/in²)

In SI units, Eq (12.62) can be expressed as

$$f\,(\text{kN/m}^2) = 6.564q_u^{0.5}(\text{kN/m}^2) \leq 0.15q_u\,(\text{kN/m}^2) \tag{12.63}$$

2. Calculate the ultimate capacity based on side resistance only, or

$$Q_u = \pi D_s L f \tag{12.64}$$

3. Calculate the settlement s_e of the shaft at the top of the rock socket, or

$$s_e = s_{e(s)} + s_{e(b)} \tag{12.65}$$

where $s_{e(s)}$ = elastic compression of the drilled shaft within the socket, assuming no side resistance

$s_{e(b)}$ = settlement of the base

However,

$$s_{e(s)} = \frac{Q_u L}{A_c E_c} \tag{12.66}$$

and

$$s_{e(b)} = \frac{Q_u I_f}{D_s E_{\text{mass}}} \tag{12.67}$$

where Q_u = ultimate load obtained from Eq. (12.62) or Eq. (12.63) (this assumes that the contribution of the overburden to the side shear is negligible)

A_c = cross-sectional area of the drilled shaft in the socket (12.68)

$= \frac{\pi}{4} D_s^2$

E_c = Young's modulus of the concrete and reinforcing steel in the shaft

E_{mass} = Young's modulus of the rock mass into which the socket is drilled

I_f = elastic influence coefficient (see Figure 12.28)

The magnitude of E_{mass} can be determined from the average plot shown in Figure 12.29. In this figure, E_{core} is the Young's modulus of intact specimens of rock cores of NW size or larger. However, unless the socket is very long (O'Neill, 1997),

$$s_e \approx s_{e(b)} = \frac{Q_u I_f}{D_s E_{\text{mass}}} \tag{12.69}$$

4. If s_e is less than 10 mm ($\approx$0.4 in.), then the ultimate load-carrying capacity is that calculated by Eq. (12.64). If $s_e \geqslant 10$ mm. (0.4 in.), then go to Step 5.

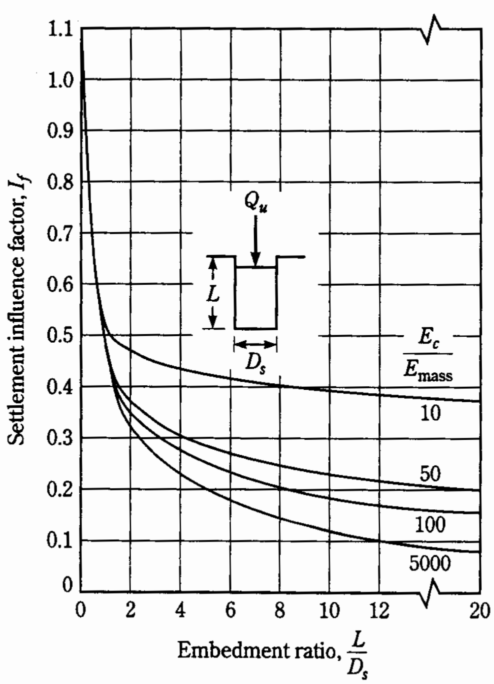

Figure 12.28 Variation of I_f (after Reese and O'Neill, 1989)

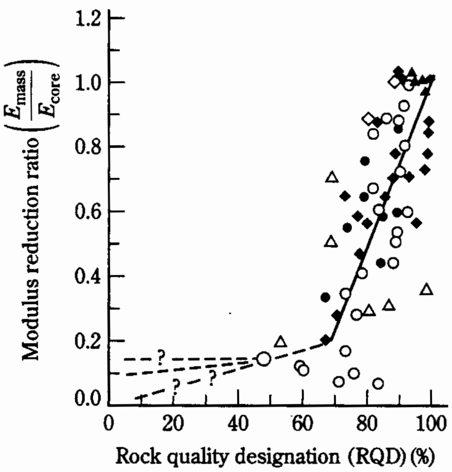

Figure 12.29 Plot of E_{mass}/E_{core} vs. RQD (after Reese and O'Neill, 1989)

5. If $s_e \geqslant 10$ mm (0.4 in.), there may be rapid, progressive side shear failure in the rock socket, resulting in a complete loss of side resistance. In that case, the ultimate capacity is equal to the point resistance, or

$$Q_u = 3A_p\left[\frac{3 + \dfrac{c_s}{D_s}}{10\left(1 + 300\dfrac{\delta}{c_s}\right)^{0.5}}\right]q_u \tag{12.70}$$

where c_s = spacing of discontinuities (same unit as D_s)
δ = thickness of individual discontinuity (same unit as D_s)
q_u = unconfined compression strength of the rock beneath the base of the socket, or the drilled shaft concrete, whichever is smaller

Note that Eq. (12.70) applies for horizontally stratified discontinuities with $c_s > 305$ mm (12 in.) and $\delta < 5$ mm (0.2 in.).

Example 12.9

Consider the case of a drilled shaft extending into rock, as shown in Figure 12.30. Let $L = 15$ ft, $D_s = 3$ ft, q_u (rock) $= 10{,}500$ lb/in², q_u (concrete) $= 3000$ lb/in², $E_c = 3 \times 10^6$ lb/in², RQD (rock) = 80%, E_{core} (rock) $= 0.36 \times 10^6$ lb/in², $c_s =$ 18 in., and $\delta = 0.15$ in. Estimate the allowable load-bearing capacity of the drilled shaft. Use a factor of safety (FS) = 3.

Solution

Step 1. From Eq. (12.62),

$$f\,(\text{lb/in}^2) = 2.5\,q_u^{0.5} \leqslant 0.15 q_u$$

Since q_u (concrete) $<$ q_u (rock), use q_u (concrete) in Eq. (12.62). Hence,

$$f = 2.5(3000)^{0.5} = 136.9 \text{ lb/in}^2$$

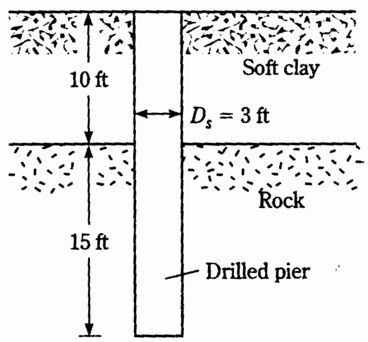

Figure 12.30 Drilled shaft extending into rock

As a check, we have

$$f = 0.15q_u = (0.15)(3000) = 450 \text{ lb/in}^2 > 136.9 \text{ lb/in}^2$$

So use $f = 136.9$ lb/in².

Step 2. From Eq. (12.64),

$$Q_u = \pi D_s L f = [(\pi)(3 \times 12)(15 \times 12)(136.9)]\frac{1}{1000} = 2787 \text{ kip}$$

Step 3. From Eqs. (12.65), (12.66), and (12.67),

$$s_e = \frac{Q_u L}{A_c E_c} + \frac{Q_u I_f}{D_s E_{\text{mass}}}$$

For RQD ≈ 80%, from Figure 12.29, the value of $E_{\text{mass}}/E_{\text{core}} \approx 0.5$; thus,

$$E_{\text{mass}} = 0.5E_{\text{core}} = (0.5)(0.36 \times 10^6) = 0.18 \times 10^6 \text{ lb/in}^2$$

so

$$\frac{E_c}{E_{\text{mass}}} = \frac{3 \times 10^6}{0.18 \times 10^6} \approx 16.7$$

and

$$\frac{L}{D_s} = \frac{15}{3} = 5$$

From Figure 12.28, for $E_c/E_{\text{mass}} = 16.7$ and $L/D_s = 5$, the magnitude of I_f is about 0.35. Hence,

$$s_e = \frac{(2787 \times 10^3 \text{ lb})(15 \times 12 \text{ in.})}{\frac{\pi}{4}(3 \times 12 \text{ in.})^2(3 \times 10^6 \text{ lb/in}^2)} + \frac{(2787 \times 10^3 \text{ lb})(0.35)}{(3 \times 12 \text{ in.})(0.18 \times 10^6 \text{ lb/in}^2)}$$

$$= 0.315 \text{ in.} < 0.4 \text{ in.}$$

Therefore,

$$Q_u = 2787 \text{ kip}$$

and

$$Q_{\text{all}} = \frac{Q_u}{\text{FS}} = \frac{2787}{3} = 929 \text{ kip}$$ ■

Problems

12.1 A drilled shaft is shown in Figure P12.1. Use Eq. (12.20) and determine the net allowable point bearing capacity. Assume the following values:

$D_b = 2$ m $\quad \gamma_c = 15.6$ kN/m^3
$D_s = 1.2$ m $\quad \gamma_s = 17.6$ kN/m^3
$L_1 = 6$ m $\quad \phi' = 35°$
$L_2 = 3$ m $\quad c_u = 35$ kN/m^3
Factor of safety = 4

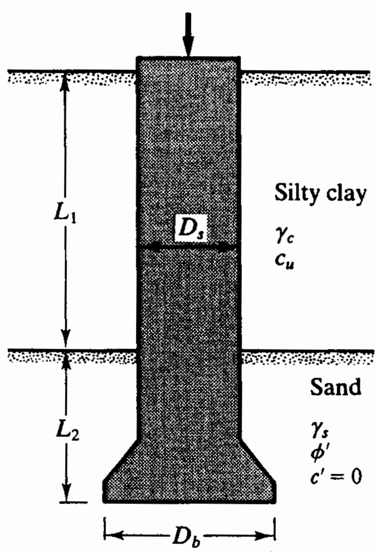

Figure P12.1

12.2 Redo Problem 12.1, this time using Eq. (12.16). Let $E_s = 600p_a$.

12.3 Redo Problem 12.1 with the following data:

$D_b = 1.75$ m $\quad \gamma_c = 17.8$ kN/m^3
$D_s = 1$ m $\quad \gamma_s = 18.2$ kN/m^3
$L_1 = 6.25$ m $\quad \phi' = 32°$
$L_2 = 2.5$ m $\quad c_u = 32$ kN/m^3
Factor of safety = 4

12.4 Solve Problem 12.3 using Eq. (12.16). Let $E_s = 400p_a$.

12.5 For the drilled shaft described in Problem 12.1, what skin resistance would develop in the top 6 m, which is in clay?
a. Use Eqs. (12.42) and (12.44).
b. Use Eq. (12.45).

12.6 For the drilled shaft described in Problem 12.3, what skin resistance would develop in the top 6.25 m?
a. Use Eqs. (12.42) and (12.44).
b. Use Eq. (12.45).

12.7 Figure P12.7 shows a drilled shaft without a bell. Assume the following values:

$$L_1 = 6\text{ m} \quad c_{u(1)} = 45\text{ kN/m}^2$$
$$L_2 = 5\text{ m} \quad c_{u(2)} = 74\text{ kN/m}^2$$
$$D_s = 1.5\text{ m}$$

Determine
a. The net ultimate point bearing capacity [use Eqs. (12.39) and (12.40)]
b. The ultimate skin friction [use Eqs. (12.42) and (12.44)]
c. The working load Q_w (factor of safety = 3)

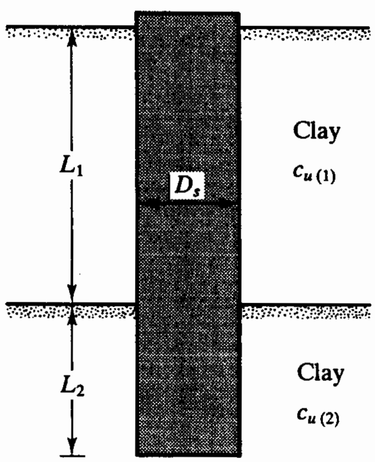

Figure P12.7

12.8 Repeat Problem 12.7 with the following data:

$$L_1 = 25\text{ ft} \quad c_{u(1)} = 1200\text{ lb/ft}^2$$
$$L_2 = 10\text{ ft} \quad c_{u(2)} = 2000\text{ lb/ft}^2$$
$$D_s = 3.5\text{ ft}$$

Use Eqs. (12.45) and (12.46).

12.9 A drilled shaft in a medium sand is shown in Figure P12.9. Using the method proposed by Reese and O'Neill, determine the following:
a. The net allowable point resistance for a base movement of 25 mm
b. The shaft frictional resistance for a base movement of 25 mm
c. The total load that can be carried by the drilled shaft for a total base movement of 25 mm

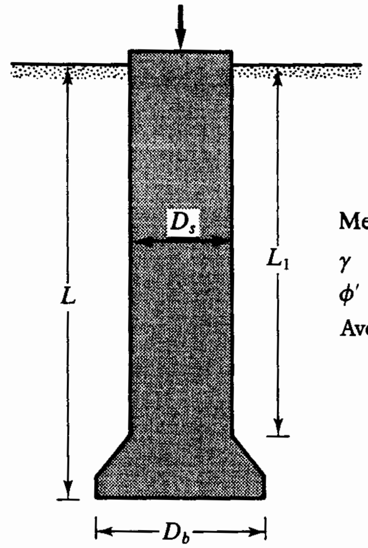

Figure P12.9

Assume the following values:

$$L = 12 \text{ m} \quad \gamma = 18 \text{ kN/m}^3$$
$$L_1 = 11 \text{ m} \quad \phi' = 38°$$
$$D_s = 1 \text{ m} \quad D_r = 65\% \text{ (medium sand)}$$
$$D_b = 2 \text{ m}$$

12.10 In Figure P12.9, let $L = 25$ ft, $L_1 = 20$ ft, $D_s = 3.5$ ft, $D_b = 5$ ft, $\gamma = 110 \text{ lb/ft}^3$, and $\phi' = 35°$. The average uncorrected standard penetration number (N_{60}) within $2D_b$ below the drilled shaft is 29. Determine
a. The ultimate load-carrying capacity
b. The load-carrying capacity for a settlement of 1 in.
Use Reese and O'Neill's method.

12.11 For the drilled shaft described in Problem 12.7, determine
a. The ultimate load-carrying capacity
b. The load carrying capacity for a settlement of 12 mm
Use the procedure outlined by Reese and O'Neill. (See Figures 12.16 and 12.17.)

12.12 For the drilled shaft described in Problem 12.7, estimate the total elastic settlement at working load. Use Eqs. (11.73), (11.75), and (11.76). Assume that $E_p = 20 \times 10^6 \text{ kN/m}^2$, $C_p = 0.03$, $\xi = 0.65$, $\mu_s = 0.3$, $E_s = 12{,}000 \text{ kN/m}^2$, and $Q_{ws} = 0.78\, Q_w$. Use the value of Q_w from Part (c) of Problem 12.7.

12.13 For the drilled shaft described in Problem 12.8, estimate the total elastic settlement at working load. Use Eqs. (11.73), (11.75), and (11.76). Assume that $E_p = 3 \times 10^6 \text{ lb/in}^2$, $C_p = 0.03$, $\xi = 0.65$, $\mu_s = 0.3$, $E_s = 2000 \text{ lb/in}^2$, and $Q_{ws} = 0.83\, Q_w$. Use the value of Q_w from Part (c) of Problem 12.8.

12.14 Suppose the drilled shaft shown in Figure P12.14 is a point bearing shaft with a working load of 650 kip. Calculate the drilled shaft settlement from Eqs. (11.73) and (11.74) for $E_p = 3 \times 10^6 \text{ lb/in}^2$, $\mu_s = 0.35$, and $E_s = 6000 \text{ lb/ft}^2$.

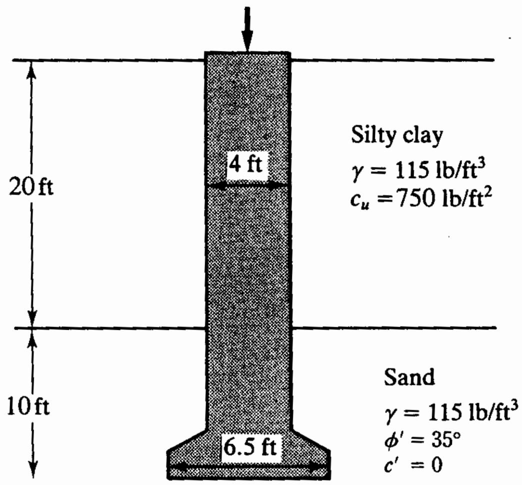

Figure P12.14

12.15 Figure P12.15 shows a drilled shaft extending to rock. Assume the following values:

$q_{u\,(\text{concrete})} = 24{,}000\ \text{kN/m}^2$ $\quad E_{(\text{concrete})} = 22\ \text{GN/m}^2$
$q_{u\,(\text{rock})} = 52{,}100\ \text{kN/m}^2$ $\quad E_{\text{core(rock)}} = 12.1\ \text{GN/m}^2$
$\text{RQD}_{(\text{rock})} = 75\%$
Spacing of discontinuity in rock = 550 mm
Thickness of individual discontinuity in rock = 2.5 mm
Estimate the allowable load-bearing capacity of the drilled shaft. Use FS = 4.

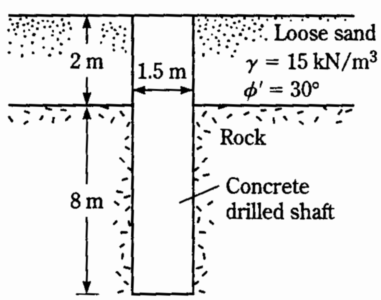

Figure P12.15

12.16 A free-headed drill shaft is shown in Figure P12.16. Let $Q_g = 260$ kN, $M_g = 0$, $\gamma = 17.5$ kN/m^3, $\phi' = 35°$, $c' = 0$, and $E_p = 22 \times 10^6$ kN/m^2. Determine

a. the ground line deflection, x_o
b. the maximum bending moment in the drilled shaft
c. the maximum tensile stress in the shaft
d. the minimum penetration of the shaft needed for this analysis

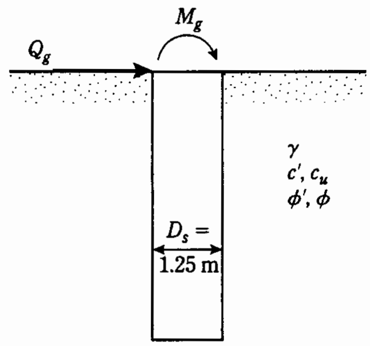

Figure P12.16

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