17.1 INTRODUCTION

Chapter 15 dealt with piles subjected to vertical loads and Chapter 16 with piles subjected to lateral loads. Drilled pier foundations, the subject matter of this chapter, belong to the same category as pile foundations. Because piers and piles serve the same purpose, no sharp deviations can be made between the two. The distinctions are based on the method of installation. A pile is installed by driving, a pier by excavating. Thus, a foundation unit installed in a drill-hole may also be called a bored cast-in-situ concrete pile. Here, distinction is made between a small diameter pile and a large diameter pile. A pile, cast-in-situ, with a diameter less than 0.75 m (or 2.5 ft) is sometimes called a small diameter pile. A pile greater than this size is called a large diameter bored-cast-in-situ pile. The latter definition is used in most non-American countries whereas in the USA, such large-diameter bored piles are called drilled piers, drilled shafts, and sometimes drilled caissons. Chapter 15 deals with small diameter bored-cast-in situ piles in addition to driven piles.

17.2 TYPES OF DRILLED PIERS

Drilled piers may be described under four types. All four types are similar in construction technique, but differ in their design assumptions and in the mechanism of load transfer to the surrounding earth mass. These types are illustrated in Figure 17.1.

- **Straight-shaft end-bearing piers** develop their support from end-bearing on strong soil, “hardpan” or rock. The overlying soil is assumed to contribute nothing to the support of the load imposed on the pier (Fig. 17.1(a)).
Figure 17.1 Types of drilled piers and underream shapes (Woodward et al., 1972)

Straight-shaft side wall friction piers pass through overburden soils that are assumed to carry none of the load, and penetrate far enough into an assigned bearing stratum to develop design load capacity by side wall friction between the pier and bearing stratum (Fig. 17.1(b)).

Combination of straight shaft side wall friction and end bearing piers are of the same construction as the two mentioned above, but with both side wall friction and end bearing assigned a role in carrying the design load. When carried into rock, this pier may be referred to as a socketed pier or a “drilled pier with rock socket” (Fig. 17.1(c)).

Belled or underreamed piers are piers with a bottom bell or underream (Fig. 17.1(d)). A greater percentage of the imposed load on the pier top is assumed to be carried by the base.
17.3 ADVANTAGES AND DISADVANTAGES OF DRILLED PIER FOUNDATIONS

Advantages

1. Pier of any length and size can be constructed at the site
2. Construction equipment is normally mobile and construction can proceed rapidly
3. Inspection of drilled holes is possible because of the larger diameter of the shafts
4. Very large loads can be carried by a single drilled pier foundation thus eliminating the necessity of a pile cap
5. The drilled pier is applicable to a wide variety of soil conditions
6. Changes can be made in the design criteria during the progress of a job
7. Ground vibration that is normally associated with driven piles is absent in drilled pier construction
8. Bearing capacity can be increased by underreaming the bottom (in non-caving materials)

Disadvantages

1. Installation of drilled piers needs a careful supervision and quality control of all the materials used in the construction
2. The method is cumbersome. It needs sufficient storage space for all the materials used in the construction
3. The advantage of increased bearing capacity due to compaction in granular soil that could be obtained in driven piles is not there in drilled pier construction
4. Construction of drilled piers at places where there is a heavy current of ground water flow due to artesian pressure is very difficult

17.4 METHODS OF CONSTRUCTION

Earlier Methods

The use of drilled piers for foundations started in the United States during the early part of the twentieth century. The two most common procedures were the Chicago and Gow methods shown in Fig. 17.2. In the Chicago method a circular pit was excavated to a convenient depth and a cylindrical shell of vertical boards or staves was placed by making use of an inside compression ring. Excavation then continued to the next board length and a second tier of staves was set and the procedure continued. The tiers could be set at a constant diameter or stepped in about 50 mm. The Gow method, which used a series of telescopic metal shells, is about the same as the current method of using casing except for the telescoping sections reducing the diameter on successive tiers.

Modern Methods of Construction

Equipment

There has been a phenomenal growth in the manufacture and use of heavy duty drilling equipment in the United States since the end of World War II. The greatest impetus to this development occurred in two states, Texas and California (Woodward et al., 1972). Improvements in the machines were made responding to the needs of contractors. Commercially produced drilling rigs of sufficient size and capacity to drill pier holes come in a wide variety of mountings and driving arrangements. Mountings are usually truck crane, tractor or skid. Fig. 17.3 shows a tractor mounted rig. Drilling machine ratings as presented in manufacturer’s catalogs and technical data sheets are
usually expressed as maximum hole diameter, maximum depth, and maximum torque at some particular rpm.

Many drilled pier shafts through soil or soft rock are drilled with the open-helix auger. The tool may be equipped with a knife blade cutting edge for use in most homogeneous soil or with hard-surfaced teeth for cutting stiff or hard soils, stony soils, or soft to moderately hard rock. These augers are available in diameters up to 3 m or more. Fig. 17.4 shows commercially available models.

Underreaming tools (or buckets) are available in a variety of designs. Figure 17.5 shows a typical 30° underreamer with blade cutter for soils that can be cut readily. Most such underreaming tools are limited in size to a diameter three times the diameter of the shaft.

When rock becomes too hard to be removed with auger-type tools, it is often necessary to resort to the use of a core barrel. This tool is a simple cylindrical barrel, set with tungsten carbide teeth at the bottom edge. For hard rock which cannot be cut readily with the core barrel set with hard metal teeth, a calyx or shot barrel can be used to cut a core of rock.

**General Construction Methods of Drilled Pier Foundations**

The rotary drilling method is the most common method of pier construction in the United States. The methods of drilled pier construction can be classified in three categories as

1. The dry method
2. The casing method
3. The slurry method
Dry Method of Construction
The dry method is applicable to soil and rock that are above the water table and that will not cave or slump when the hole is drilled to its full depth. The soil that meets this requirement is a homogeneous, stiff clay. The first step in making the hole is to position the equipment at the desired location and to select the appropriate drilling tools. Fig. 17.6(a) gives the initial location. The drilling is next carried out to its fill depth with the spoil from the hole removed simultaneously.

After drilling is complete, the bottom of the hole is underreamed if required. Fig. 17.6(b) and (c) show the next steps of concreting and placing the rebar cage. Fig 17.6(d) shows the hole completely filled with concrete.
Figure 17.4  (a) Single-flight auger bit with cutting blade for soils, (b) single-flight auger bit with hard-metal cutting teeth for hard soils, hardpan, and rock, and (c) cast steel heavy-duty auger bit for hardpan and rock (Source: Woodward et al., 1972)

Figure 17.5  A 30° underreamer with blade cutters for soils that can be cut readily (Source: Woodward et al., 1972)
Casing Method of Construction

The casing method is applicable to sites where the soil conditions are such that caving or excessive soil or rock deformation can occur when a hole is drilled. This can happen when the boring is made in dry soils or rocks which are stable when they are cut but will slough soon afterwards. In such a

![Diagram of Casing Method of Construction]

Figure 17.6 Dry method of construction: (a) initiating drilling, (b) starting concrete pour, (c) placing rebar cage, and (d) completed shaft (O’Neill and Reese, 1999)
in the bore hole is drilled, and a steel pipe casing is quickly set to prevent sloughing. Casing is also required if drilling is required in clean sand below the water table underlain by a layer of impermeable stones into which the drilled shaft will penetrate. The casing is removed soon after the concrete is deposited. In some cases, the casing may have to be left in place permanently. It may be noted here that until the casing is inserted, a slurry is used to maintain the stability of the hole. After the casing is seated, the slurry is bailed out and the shaft extended to the required depth. Figures 17.7(a) to (h) give the sequence of operations. Withdrawal of the casing, if not done carefully, may lead to voids or soil inclusions in the concrete, as illustrated in Fig. 17.8.

Figure 17.7  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 (continued)
**Slurry Method of Construction**

The slurry method of construction involves the use of a prepared slurry to keep the bore hole stable for the entire depth of excavation. The soil conditions for which the slurry displacement method is applicable could be any of the conditions described for the casing method. The slurry method is a

---

**Figure 17.7 (continued)** casing method of construction: (e) drilling below casing, (f) underreaming, (g) removing casing, and (h) completed shaft (O'Neill and Reese, 1999)
Figure 17.8 Potential problems leading to inadequate shaft concrete due to removal of temporary casing without care (D'Appolonia, et al., 1975)

viable option at any site where there is a caving soil, and it could be the only feasible option in a permeable, water bearing soil if it is impossible to set a casing into a stratum of soil or rock with low permeability. The various steps in the construction process are shown in Fig. 17.9. It is essential in this method that a sufficient slurry head be available so that the inside pressure is greater than that from the GWT or from the tendency of the soil to cave.

Bentonite is most commonly used with water to produce the slurry. Polymer slurry is also employed. Some experimentation may be required to obtain an optimum percentage for a site, but amounts in the range of 4 to 6 percent by weight of admixture are usually adequate.

The bentonite should be well mixed with water so that the mixture is not lumpy. The slurry should be capable of forming a filter cake on the side of the bore hole. The bore hole is generally not underreamed for a bell since this procedure leaves unconsolidated cuttings on the base and creates a possibility of trapping slurry between the concrete base and the bell roof.

If reinforcing steel is to be used, the rebar cage is placed in the slurry as shown in Fig 17.9(b). After the rebar cage has been placed, concrete is placed with a tremie either by gravity feed or by pumping. If a gravity feed is used, the bottom end of the tremie pipe should be closed with a closure plate until the base of the tremie reaches the bottom of the bore hole, in order to prevent contamination of the concrete by the slurry. Filling of the tremie with concrete, followed by subsequent slight lifting of the tremie, will then open the plate, and concreting proceeds. Care must be taken that the bottom of the tremie is buried in concrete at least for a depth of 1.5 m (5 ft). The sequence of operations is shown in Fig 17.9(a) to (d).
17.5 DESIGN CONSIDERATIONS

The process of the design of a drilled pier generally involves the following:

1. The objectives of selecting drilled pier foundations for the project.
2. Analysis of loads coming on each pier foundation element.
3. A detailed soil investigation and determining the soil parameters for the design.
4. Preparation of plans and specifications which include the methods of design, tolerable settlement, methods of construction of piers, etc.
5. The method of execution of the project.
In general the design of a drilled pier may be studied under the following headings.

1. Allowable loads on the piers based on ultimate bearing capacity theories.
2. Allowable loads based on vertical movement of the piers.
3. Allowable loads based on lateral bearing capacity of the piers.

In addition to the above, the uplift capacity of piers with or without underreams has to be evaluated.

The following types of strata are considered.

1. Piers embedded in homogeneous soils, sand or clay.
2. Piers in a layered system of soil.
3. Piers socketed in rocks.

It is better that the designer select shaft diameters that are multiples of 150 mm (6 in) since these are the commonly available drilling tool diameters.

17.6 LOAD TRANSFER MECHANISM

Figure 17.10(a) shows a single drilled pier of diameter \( d \), and length \( L \) constructed in a homogeneous mass of soil of known physical properties. If this pier is loaded to failure under an ultimate load \( Q_u \), a part of this load is transmitted to the soil along the length of the pier and the balance is transmitted to the pier base. The load transmitted to the soil along the pier is called the ultimate friction load or skin load, \( Q_f \) and that transmitted to the base is the ultimate base or point load \( Q_b \). The total ultimate load, \( Q_u \), is expressed as (neglecting the weight of the pier)

\[
Q_u = Q_b + Q_f = q_b A_b + \sum_{i=1}^{N} f_{i} P_{i} \Delta z_i
\]

where

- \( q_b \) = net ultimate bearing pressure
- \( A_b \) = base area
- \( f_{i} \) = unit skin resistance (ultimate) of layer \( i \)
- \( P_{i} \) = perimeter of pier in layer \( i \)
- \( \Delta z_i \) = thickness of layer \( i \)
- \( N \) = number of layers

If the pier is instrumented, the load distribution along the pier can be determined at different stages of loading. Typical load distribution curves plotted along a pier are shown in Fig 17.10(b) (O’Neill and Reese, 1999). These load distribution curves are similar to the one shown in Fig. 15.5(b). Since the load transfer mechanism for a pier is the same as that for a pile, no further discussion on this is necessary here. However, it is necessary to study in this context the effect of settlement on the mobilization of side shear and base resistance of a pier. As may be seen from Fig. 17.11, the maximum values of base and side resistance are not mobilized at the same value of displacement. In some soils, and especially in some brittle rocks, the side shear may develop fully at a small value of displacement and then decrease with further displacement while the base resistance is still being mobilized (O’Neill and Reese, 1999). If the value of the side resistance at point \( A \) is added to the value of the base resistance at point \( B \), the total resistance shown at level \( D \) is overpredicted. On the other hand, if the designer wants to take advantage primarily of the base resistance, the side resistance at point \( C \) should be added to the base resistance at point \( B \) to evaluate \( Q_u \). Otherwise, the designer may wish to design for the side resistance at point \( A \) and disregard the base resistance entirely.
Figure 17.10  Typical set of load distribution curves (O’Neill and Reese, 1999)

Figure 17.11  Condition in which $Q_b + Q_f$ is not equal to actual ultimate resistance
17.7 VERTICAL BEARING CAPACITY OF DRILLED PIERS

For the purpose of estimating the ultimate bearing capacity, the subsoil is divided into layers (Fig. 17.12) based on judgment and experience (O’Neill and Reese, 1999). Each layer is assigned one of four classifications.

1. Cohesive soil (clays and plastic silts with undrained shear strength $c_u \leq 250$ kN/m$^2$ (2.5 t/ft$^2$)).
2. Granular soil (cohesionless geomaterial, such as sand, gravel or nonplastic silt with uncorrected SPT(N) values of 50 blows per 0.3/m or less).
3. Intermediate geomaterial (cohesive geomaterial with undrained shear strength $c_u$ between 250 and 2500 kN/m$^2$ (2.5 and 25 tsf), or cohesionless geomaterial with SPT(N) values > 50 blows per 0.3 m).
4. Rock (highly cemented geomaterial with unconfined compressive strength greater than 5000 kN/m$^2$ (50 tsf)).

The unit side resistance $f_s (= f_{max})$ is computed in each layer through which the drilled shaft passes, and the unit base resistance $q_b (= q_{max})$ is computed for the layer on or in which the base of the drilled shaft is founded.

The soil along the whole length of the shaft is divided into four layers as shown in Fig. 17.12.

**Effective Length for Computing Side Resistance in Cohesive Soil**

O’Neill and Reese (1999) suggest that the following effective length of pier is to be considered for computing side resistance in cohesive soil.

![Figure 17.12 Idealized geomaterial layering for computation of compression load and resistance (O’Neill and Reese, 1999)](image)
**Straight shaft:** One diameter from the bottom and 1.5 m (5 feet) from the top are to be excluded from the embedded length of pile for computing side resistance as shown in Fig. 17.13(a).

**Belled shaft:** The height of the bell plus the diameter of the shaft from the bottom and 1.5 m (5 ft) from the top are to be excluded as shown in Fig. 17.13(b).

### 17.8 The General Bearing Capacity Equation for the Base Resistance \( q_b \) (= \( q_{\text{max}} \))

The equation for the ultimate base resistance may be expressed as

\[
q_b = s_c d_c N_c c + s_q d_q (N_q - 1) q'_o + \frac{1}{2} \gamma d s_q d_q N_\gamma
\]

(17.2)

where

- \( N_c, N_q \) and \( N_\gamma \) = bearing capacity factors for long footings
- \( s_c, s_q \) and \( s_\gamma \) = shape factors
- \( d_c, d_q \) and \( d_\gamma \) = depth factors
- \( q'_o \) = effective vertical pressure at the base level of the drilled pier
- \( \gamma \) = effective unit weight of the soil below the bottom of the drilled shaft to a depth = 1.5 \( d \) where \( d \) = width or diameter of pier at base level
- \( c \) = average cohesive strength of soil just below the base.

For deep foundations the last term in Eq. (17.2) becomes insignificant and may be ignored. Now Eq. (17.2) may be written as

\[
q_b = s_c d_c N_c c + s_q d_q (N_q - 1) q'_o
\]

(17.3)

\[\text{Figure 17.13 Exclusion zones for estimating side resistance for drilled shafts in cohesive soils}\]
17.9 BEARING CAPACITY EQUATIONS FOR THE BASE IN COHESIVE SOIL

When the Undrained Shear Strength, \( c_u \leq 250 \text{ kN/m}^2 \ (2.5 \text{ t/ft}^2) \)

For \( \phi = 0 \), \( N_q = 1 \) and \( (N_q - 1) = 0 \), here Eq. (17.3) can be written as (Vesic, 1972)

\[
q_b = N_c^* c_u
\]  
(17.4)
in which

\[
N_c^* = \frac{4}{3} (\ln I_r + 1)
\]  
(17.5)

\( I_r \) = rigidity index of the soil

Eq. (17.4) is applicable for \( c_u \leq 96 \text{ kPa} \) and \( L \geq 3d \) (base width)

For \( \phi = 0 \), \( I_r \) may be expressed as (O’Neill and Reese, 1999)

\[
I_r = \frac{E_s}{3c_u}
\]  
(17.6)
where \( E_s \) = Young’s modulus of the soil in undrained loading. Refer to Section 13.8 for the methods of evaluating the value of \( E_s \).

Table 17.1 gives the values of \( I_r \) and \( N_c^* \) as a function of \( c_u \).

If the depth of base \( (L) < 3d \) (base)

\[
q_b (= q_{\text{max}}) = \frac{2}{3} \left( 1 + \frac{L}{6d} \right) \ N_c^* c_u
\]  
(17.7)

When \( c_u \geq 96 \text{ kPa} \ (2000 \text{ lb/ft}^2) \), the equation for \( q_b \) may be written as

\[
q_b = 9c_u
\]  
(17.8)
for depth of base \( (= L) \geq 3d \) (base width).

17.10 BEARING CAPACITY EQUATION FOR THE BASE IN GRANULAR SOIL

Values \( N_c \) and \( N_q \) in Eq. (17.3) are for strip footings on the surface of rigid soils and are plotted as a function of \( \phi \) in Fig. 17.14. Vesic (1977) explained that during bearing failure, a plastic failure zone develops beneath a circular loaded area that is accompanied by elastic deformation in the surrounding elastic soil mass. The confinement of the elastic soil surrounding the plastic soil has an effect on \( q_b (= q_{\text{max}}) \). The values of \( N_c \) and \( N_q \) are therefore dependent not only on \( \phi \), but also on \( I_r \). They must be corrected for soil rigidity as given below.

<table>
<thead>
<tr>
<th>( c_u ) (kPa)</th>
<th>( I_r )</th>
<th>( N_c^* )</th>
</tr>
</thead>
<tbody>
<tr>
<td>24 (500 lb/ft²)</td>
<td>50</td>
<td>6.5</td>
</tr>
<tr>
<td>48 (1000 lb/ft²)</td>
<td>150</td>
<td>8.0</td>
</tr>
<tr>
<td>≥ 96 (2000 lb/ft²)</td>
<td>250–300</td>
<td>9.0</td>
</tr>
</tbody>
</table>
\[ N_c \text{ (corrected)} = N_c C_c \]
\[ N_q \text{ (corrected)} = N_q C_q \] (17.9)

where \( C_c \) and \( C_q \) are the correction factors. As per Chen and Kulhawy (1994)

Eq (17.3) may now be expressed as

\[ q_b = c N_c s_c d_c C_c + (N_q - 1)q'_o s_q d_q C_q \] (17.10)

\[ C_c = C_q \frac{1 - C_q}{N_c \tan \phi} \] (17.11a)

\[ C_q = \exp \left\{ -3.8 \tan \phi + \left[ (3.07 \sin \phi) \log_{10} \frac{2I_{rr}}{1 + \sin \phi} \right] \right\} \] (17.11b)

where \( \phi \) is an effective angle of internal friction. \( I_{rr} \) is the reduced rigidity index expressed as [Eq. (15.28)]

\[ I_{rr} = \frac{I_r}{1 + \Delta I_r} \] (17.12)

and

\[ I_r = \frac{E_d}{2(1 + \mu_d)q'_o \tan \phi} \] (17.13)

by ignoring cohesion, where,

\[ N_q = \tan^2 \left( \frac{45 + \phi/2}{2} \right) e^{\pi \tan \phi} \]
\[ N_c = (N_q - 1) \cot \phi \]
\[ N_f = 2(N_q + 1) \tan \phi \]

\[ N_c \]
\[ N_q \]
\[ N_f \]

Figure 17.14  Bearing capacity factors (Chen and Kulhawy, 1994)
\( E_d \) = drained Young's modulus of the soil  
\( \mu_d \) = drained Poisson’s ratio  
\( \Delta \) = volumetric strain within the plastic zone during the loading process

The expressions for \( \mu_d \) and \( \Delta \) may be written as (Chen and Kulhawy, 1994)

\[
\mu_d = 0.1 + 0.3 \phi_{rel} \tag{17.14}
\]

\[
\Delta = \frac{0.005(1 - \phi_{rel})q'_{o}}{p_o} \tag{17.15}
\]

where \( \phi_{rel} = \frac{(\phi^o - 25^o)}{45^o - 25^o} \) for \( 25^o \leq \phi^o \leq 45^o \) \( \tag{17.16} \)

= relative friction angle factor, \( p_o \) = atmospheric pressure = 101 kPa.

Chen and Kulhawy (1994) suggest that, for granular soils, the following values may be considered.

- loose soil, \( E_d = 100 \) to \( 200p_o \) \( \tag{17.17} \)
- medium dense soil, \( E_d = 200 \) to \( 500p_o \)
- dense soil, \( E_d = 500 \) to \( 1000p_o \)

The correction factors \( C_c \) and \( C_q \) indicated in Eq. (17.9) need be applied only if \( l_{rr} \) is less than the critical rigidity index \( (l_{rr})_{crit} \) expressed as follows

\[
(l_{rr})_{crit} = \frac{1}{2} \exp \frac{2.85\cot 45^o - \phi^o}{2} \tag{17.18}
\]

The values of critical rigidity index may be obtained from Table 12.4 for piers circular or square in section.

If \( l_{rr} > (l_{rr})_{crit} \), the factors \( C_c \) and \( C_q \) may be taken as equal to unity.

The shape and depth factors in Eq. (17.3) can be evaluated by making use of the relationships given in Table 17.2.

**Table 17.2** Shape and depth factors (Eq. 17.3) (Chen and Kulhawy, 1994)

<table>
<thead>
<tr>
<th>Factors</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( s_c )</td>
<td>[ 1 + \frac{N_s}{N_c} ]</td>
</tr>
<tr>
<td>( s_d )</td>
<td>[ d_q - \frac{1 - d_q}{N_s \tan \phi} ]</td>
</tr>
<tr>
<td>( s_q )</td>
<td>[ 1 + \tan \phi ]</td>
</tr>
<tr>
<td>( d_q )</td>
<td>[ 1 + 2\tan \phi(1 - \sin \phi)^2 \frac{\pi}{180} \tan^{-1} \frac{L}{d} ]</td>
</tr>
</tbody>
</table>
Base in Cohesionless Soil

The theoretical approach as outlined above is quite complicated and difficult to apply in practice for drilled piers in granular soils. Direct and simple empirical correlations have been suggested by O’Neill and Reese (1999) between SPT $N$ value and the base bearing capacity as given below for cohesionless soils.

$$q_b (= q_{max}) = 57.5N \text{ kPa} \leq 2900 \text{ kN/m}^2$$ (17.19a)

$$q_b (= q_{max}) = 0.60N \text{ tsf} \leq 30 \text{ tsf}$$ (17.19b)

where $N = \text{SPT value} \leq 50 \text{ blows / 0.3 m.}$

Base in Cohesionless IGM

Cohesionless IGM’s are characterized by SPT blow counts if more than 50 per 0.3 m. In such cases, the expression for $q_b$ is

$$q_b (= q_{max}) = 0.60 N_{60} \frac{p_a}{q_o^0} q_o^0$$ (17.20)

where $N_{60} = \text{average SPT corrected for 60 percent standard energy within a depth of 2d (base) below the base. The value of } N_{60} \text{ is limited to 100. No correction for overburden pressure}$

$p_a = \text{atmospheric pressure in the units used for } q_o^0 (=101 \text{ kPa in the SI System})$

$q_o^0 = \text{vertical effective stress at the elevation of the base of the drilled shaft.}$

17.11 BEARING CAPACITY EQUATIONS FOR THE BASE IN COHESIVE IGM OR ROCK (O’NEILL AND REESE, 1999)

Massive rock and cohesive intermediate materials possess common properties. They possess low drainage qualities under normal loadings but drain more rapidly under large loads than cohesive soils. It is for these reasons undrained shear strengths are used for rocks and IGMs.

If the base of the pier lies in cohesive IGM or rock ($RQD = 100$ percent) and the depth of socket, $D_s$, in the IGM or rock is equal to or greater than $1.5d$, the bearing capacity may be expressed as

$$q_b (= q_{max}) = 2.5q_u$$ (17.21)

where $q_u = \text{unconfined compressive strength of IGM or rock below the base}$

For $RQD$ between 70 and 100 percent,

$$q_b (= q_{max}) = 4.83(q_u)^{0.51} \text{ MPa}$$ (17.22)

For jointed rock or cohesive IGM

$$q_b (= q_{max}) = [s^{0.5} + (m s^{0.5} + s^{0.5})]q_u$$ (17.23)

where $q_u$ is measured on intact cores from within 2d (base) below the base of the drilled pier. In all the above cases $q_b$ and $q_u$ are expressed in the same units and $s$ and $m$ indicate the properties of the rock or IGM mass that can be estimated from Tables 17.3 and 17.4.
Table 17.3 Descriptions of rock types

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Carbonate rocks with well-developed crystal cleavage (e.g., dolostone, limestone, marble)</td>
</tr>
<tr>
<td>B</td>
<td>Lithified argillaeous rocks (mudstone, siltstone, shale, slate)</td>
</tr>
<tr>
<td>C</td>
<td>Arenaceous rocks (sandstone, quartzite)</td>
</tr>
<tr>
<td>D</td>
<td>Fine-grained igneous rocks (andesite, dolerite, diabase, rhyolite)</td>
</tr>
<tr>
<td>E</td>
<td>Coarse-grained igneous and metamorphic rocks (amphibole, gabbro, gneiss, granite, norite, quartz-diorite)</td>
</tr>
</tbody>
</table>

Table 17.4 Values of $s$ and $m$ (dimensionless) based on rock classification (Carter and Kulhawy, 1988)

<table>
<thead>
<tr>
<th>Quality of rock mass</th>
<th>Joint description and spacing</th>
<th>$s$</th>
<th>Value of $m$ as function of rock type $(A-E)$ from $c_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excellent</td>
<td>Intact (closed); spacing &gt; 3 m (10 ft)</td>
<td>1</td>
<td>A: 7 B: 10 C: 15 D: 17 E: 25</td>
</tr>
<tr>
<td>Very good</td>
<td>Interlocking; spacing of 1 to 3 m (3 to 10 ft)</td>
<td>0.1</td>
<td>A: 3.5 B: 5 C: 7.5 D: 8.5 E: 12.5</td>
</tr>
<tr>
<td>Good</td>
<td>Slightly weathered; spacing of 1 to 3 m (3 to 10 ft)</td>
<td>$4 \times 10^{-2}$</td>
<td>A: 0.7 B: 1 C: 1.5 D: 1.7 E: 2.5</td>
</tr>
<tr>
<td>Fair</td>
<td>Moderately weathered; spacing of 0.3 to 1 m (1 to 3 ft)</td>
<td>$10^{-4}$</td>
<td>A: 0.14 B: 0.2 C: 0.3 D: 0.34 E: 0.5</td>
</tr>
<tr>
<td>Poor</td>
<td>Weathered with gouge (soft material); spacing of 30 to 300 mm (1 in. to 1 ft)</td>
<td>$10^{-5}$</td>
<td>A: 0.04 B: 0.05 C: 0.08 D: 0.09 E: 0.13</td>
</tr>
<tr>
<td>Very poor</td>
<td>Heavily weathered; spacing of less than 50 mm (2 in.)</td>
<td>0</td>
<td>A: 0.007 B: 0.01 C: 0.015 D: 0.017 E: 0.025</td>
</tr>
</tbody>
</table>

17.12 THE ULTIMATE SKIN RESISTANCE OF COHESIVE AND INTERMEDIATE MATERIALS

Cohesive Soil

The process of drilling a borehole for a pier in cohesive soil disturbs the natural condition of the soil all along the side to a certain extent. There is a reduction in the soil strength not only due to boring but also due to stress relief and the time spent between boring and concreting. It is very difficult to quantify the extent of the reduction in strength analytically. In order to take care of the disturbance, the unit frictional resistance on the surface of the pier may be expressed as

$$f_s = \alpha c_u$$ (17.24)

where $\alpha =$ adhesion factor

$c_u =$ undrained shear strength

Relationships have been developed between $c_u$ and $\alpha$ by many investigators based on field load tests. Fig 17.15 gives one such relationship in the form of a curve developed by Chen and Kulhawy (1994). The curve has been developed on the following assumptions (Fig. 17.15).
Deep Foundation III: Drilled Pier Foundations

\[ f_s = 0 \quad \text{up to } 1.5 \text{ m (} = 5 \text{ ft) from the ground level.} \]

\[ f_s = 0 \quad \text{up to a height equal to } (h + d) \text{ as per Fig 17.13} \]

O’Neill and Reese (1999) recommend the chart’s trend line given in Fig. 17.15 for designing drilled piers. The suggested relationships are:

\[ \alpha = 0.55 \quad \text{for } c_u / p_a \leq 1.5 \]

(17.25a)

\[ \text{and } \alpha = 0.55 - 0.1 \frac{c_u}{p_a} - 1.5 \quad \text{for } 1.5 \leq c_u / p_a \leq 2.5 \]

(17.25b)

Cohesive Intermediate Geomaterials

Cohesive IGM’s are very hard clay-like materials which can also be considered as very soft rock (O’Neill and Reese, 1999). IGM’s are ductile and failure may be sudden at peak load. The value of \( f_a \) (please note that the term \( f_a \) is used instead of \( f_s \) for ultimate unit resistance at infinite displacement) depends upon the side condition of the bore hole, that is, whether it is rough or smooth. For design purposes the side is assumed as smooth. The expression for \( f_a \) may be written as

\[ f_a = \alpha q_u \]

(17.26)

where, \( q_u \) = unconfined compressive strength

\( f_a \) = the value of ultimate unit side resistance which occurs at infinite displacement.

Figure 17.16 gives a chart for evaluating \( \alpha \). The chart is prepared for an effective angle of friction between the concrete and the IGM (assuming that the intersurface is drained) and \( S_t \) denotes the settlement of piers at the top of the socket. Further, the chart involves the use of \( \sigma_n / p_a \) where \( \sigma_n \) is the normal effective pressure against the side of the borehole by the drilled pier and \( p_a \) is the atmospheric pressure (101 kPa).
Figure 17.16  Factor $\alpha$ for cohesive IGM's (O’Neill and Reese, 1999)

Table 17.5  Estimation of $E_m/E_i$ based on ROD (Modified after Carter and Kulhawy, 1988)

<table>
<thead>
<tr>
<th>ROD (percent)</th>
<th>Closed joints</th>
<th>Open joints</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>1.00</td>
<td>0.60</td>
</tr>
<tr>
<td>70</td>
<td>0.70</td>
<td>0.10</td>
</tr>
<tr>
<td>50</td>
<td>0.15</td>
<td>0.10</td>
</tr>
<tr>
<td>20</td>
<td>0.05</td>
<td>0.05</td>
</tr>
</tbody>
</table>

Note: Values intermediate between tabulated values may be obtained by linear interpolation.

Table 17.6  $f_{ao}/f_a$ based on $E_m/E_i$ (O’Neill et al., 1996)

<table>
<thead>
<tr>
<th>$E_m/E_i$</th>
<th>$f_{ao}/f_a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>0.5</td>
<td>0.8</td>
</tr>
<tr>
<td>0.3</td>
<td>0.7</td>
</tr>
<tr>
<td>0.1</td>
<td>0.55</td>
</tr>
<tr>
<td>0.05</td>
<td>0.45</td>
</tr>
</tbody>
</table>
O’Neill and Reese (1999) give the following equation for computing \( \sigma_n \):

\[
\sigma_n = M \gamma_c z_c
\]  

(17.27)

where \( \gamma_c \) is the unit weight of the fluid concrete used for the construction, \( z_c \) is the depth of the point at which \( \sigma_n \) is required, and \( M \) is an empirical factor which depends on the fluidity of the concrete as indexed by the concrete slump.

Figure 17.17 gives the values of \( M \) for various slumps.

The mass modulus of elasticity of the IGM (\( E_m \)) should be determined before proceeding, in order to verify that the IGM is within the limits of Fig 17.16. This requires the average Young’s modulus of intact IGM core (\( E_i \)) which can be determined in the laboratory. Table 17.5 gives the ratios of \( E_m/E_i \) for various values of RQD. Values of \( E_m/E_i \) less than unity indicate that soft seams and/or joints exist in the IGM. These discontinuities reduce the value of \( f_a \). The reduced value of \( f_a \) may be expressed as

\[
f_{aa} = f_a R_a
\]  

(17.28)

where the ratio \( R_a = f_{aa}/f_a \) can be determined from Table 17.6.

If the socket is classified as smooth, it is sufficiently accurate to set \( f_a = f_{max} = f_{aa} \).

17.13 ULTIMATE SKIN RESISTANCE IN COHESIONLESS SOIL AND GRAVELLY SANDS (O’NEILL AND REESE, 1999)

In Sands

A general expression for total skin resistance in cohesionless soil may be written as [Eq. (17.1)]

\[
Q_{ti} = \sum_{i=1}^{N} P_i f_n \Delta z_i = \sum_{i=1}^{N} P_i \theta_i' K_n \tan \delta_i \Delta z_i
\]  

(17.29)
or \[ Q_{f_i} = \sum_{i=1}^{N} p_i \beta_i q_{oi} \Delta z_i \] (17.30)

where \[ f_{si} = \beta_i q_{oi} \] (17.31)
\[ \beta_i = K_{si} \tan \delta_i \] (17.31)
\[ \delta_i = \text{angle of skin friction of the } i\text{th layer} \]

The following equations are provided by O'Neill and Reese (1999) for computing \( \beta_i \).

For SPT \( N_{60} \) (uncorrected) \( \geq 15 \text{ blows / 0.3 m} \)
\[ \beta_i = 1.5 - 0.245(z_i)^{0.5} \] (17.32)

For SPT \( N_{60} \) (uncorrected) \( < 15 \text{ blows / 0.3 m} \)
\[ \beta_i = \frac{N_{60}}{15} \left[ 1.5 - 0.245(z_i)^{0.5} \right] \] (17.33)

In Gravelly Sands or Gravels
For SPT \( N_{60} \geq 15 \text{ blows / 0.3 m} \)
\[ \beta_i = 2.0 - 0.15(z_i)^{0.75} \] (17.34)

In gravelly sands or gravels, use the method for sands if \( N_{60} < 15 \text{ blows / 0.3 m} \).

The definitions of various symbols used above are
- \( \beta_i = \) dimensionless correlation factor applicable to layer \( i \). Limited to 1.2 in sands and 1.8 in gravelly sands and gravel. Minimum value is 0.25 in both types of soil; \( f_{si} \) is limited 200 kN/m\(^2\) (2.1 tsf)
- \( q_{oi} = \) vertical effective stress at the middle of each layer
- \( N_{60} = \) design value for SPT blow count, uncorrected for depth, saturation or fines corresponding to layer \( i \)
- \( z_i = \) vertical distance from the ground surface, in meters, to the middle of layer \( i \). The layer thickness \( \Delta z_i \) is limited to 9 m.

### 17.14 ULTIMATE SIDE AND TOTAL RESISTANCE IN ROCK (O’NEILL AND REESE, 1999)

**Ultimate Skin Resistance (for Smooth Socket)**

Rock is defined as a cohesive geomaterial with \( q_u > 5 \text{ MPa (725 psi)} \). The following equations may be used for computing \( f_s (= f_{max}) \) when the pier is socketed in rock. Two methods are proposed.

**Method 1**

\[ f_s (= f_{max}) = 0.65 p_a \left( \frac{q_u}{p_a} \right)^{0.5} \leq 0.65 \frac{f_c}{p_a} \] (17.35)

where,
- \( p_a = \) atmospheric pressure (= 101 kPa)
- \( q_u = \) unconfined compressive strength of rock mass
- \( f_c = \) 28 day compressive cylinder strength of concrete used in the drilled pier
Method 2

\[ f_s (= f_{\text{max}}) = 1.42 \sqrt{\frac{q_a}{p_a}} \]  

(17.36)

Carter and Kulhawy (1988) suggested equation (17.36) based on the analysis of 25 drilled shaft socket tests in a very wide variety of soft rock formations, including sandstone, limestone, mudstone, shale and chalk.

**Ultimate Total Resistance** \( Q_u \)

If the base of the drilled pier rests on sound rock, the side resistance can be ignored. In cases where significant penetration of the socket can be made, it is a matter of engineering judgment to decide whether \( Q_s \) should be added directly to \( Q_b \) to obtain the ultimate value \( Q_u \). When the rock is brittle in shear, much side resistance will be lost as the settlement increases to the value required to develop the full value of \( q_b (=q_{\text{max}}) \). If the rock is ductile in shear, there is no question that the two values can be added directly (O’Neill and Reese, 1999).

### 17.15 ESTIMATION OF SETTLEMENTS OF DRILLED PIERS AT WORKING LOADS

O’Neill and Reese (1999) suggest the following methods for computing axial settlements for isolated drilled piers:

1. Simple formulas
2. Normalized load-transfer methods

The total settlement \( S_t \) at the pier head at working loads may be expressed as (Vesic, 1977)

\[ S_t = S_e + S_{bb} + S_{bs} \]  

(17.37)

where,

- \( S_e \) = elastic compression
- \( S_{bb} \) = settlement of the base due to the load transferred to the base
- \( S_{bs} \) = settlement of the base due to the load transferred into the soil along the sides.

The equations for the settlements are

\[ S_e = \frac{L(Q_a - 0.5Q_{fm})}{A_b E} \]

\[ S_{bb} = C_p \frac{Q_{mb}}{d q_b} \]  

(17.38)

\[ S_{bs} = 0.93 + 0.16 \sqrt{\frac{L}{d}} C_p \frac{Q_{fm}}{L q_b} \]  

(17.39)

where

- \( L \) = length of the drilled pier
- \( A_b \) = base cross-sectional area
- \( E \) = Young’s modulus of the drilled pier
- \( Q_a \) = load applied to the head
- \( Q_{fm} \) = mobilized side resistance when \( Q_a \) is applied
Table 17.7 Values of $C_p$ for various soils (Vesic, 1977)

<table>
<thead>
<tr>
<th>Soil</th>
<th>$C_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand (dense)</td>
<td>0.09</td>
</tr>
<tr>
<td>Sand (loose)</td>
<td>0.18</td>
</tr>
<tr>
<td>Clay (stiff)</td>
<td>0.03</td>
</tr>
<tr>
<td>Clay (soft)</td>
<td>0.06</td>
</tr>
<tr>
<td>Silt (dense)</td>
<td>0.09</td>
</tr>
<tr>
<td>Silt (loose)</td>
<td>0.12</td>
</tr>
</tbody>
</table>

\[ Q_{bm} = \text{mobilized base resistance} \]
\[ d = \text{pier width or diameter} \]
\[ C_p = \text{soil factor obtained from Table 17.7} \]

Normalized Load-Transfer Methods

Reese and O’Neill (1988) analyzed a series of compression loading test data obtained from full-sized drilled piers in soil. They developed normalized relations for piers in cohesive and cohesionless soils. Figures 17.18 and 17.19 can be used to predict settlements of piers in cohesive soils and Figs. 17.20 and 17.21 in cohesionless soils including soil mixed with gravel.

The boundary limits indicated for gravel in Fig. 17.20 have been found to be approximately appropriate for cemented fine-grained desert IGM’s (Walsh et al., 1995). The range of validity of the normalized curves are as follows:

![Normalized side load transfer for drilled shaft in cohesive soil (O’Neill and Reese, 1999)](image-url)
Figure 17.19 Normalized base load transfer for drilled shaft in cohesive soil (O’Neill and Reese, 1999)

Figure 17.20 Normalized side load transfer for drilled shaft in cohesionless soil (O’Neill and Reese, 1999)
Chapter 17

2.1.8 1.6 1.4 1.2 1.0 0.8 0.6 0.4 0.2 0.0

Range of results

1 2 3 4 5 6 7 8 9

10 11 12

E

D

0.8

0.6

0.4

0.2

0.0

Diameter of base

Range of results

Trend line

Figure 17.21 Normalized base load transfer for drilled shaft in cohesionless soil (O'Neill and Reese, 1999)

Figures 17.18 and 17.19
Normalizing factor = shaft diameter \( d \)
Range of \( d \) = 0.46 m to 1.53 m

Figures 17.20 and 17.21
Normalizing factor = base diameter
Range of \( d \) = 0.46 m to 1.53 m

The following notations are used in the figures:

\[ S_R = \text{Settlement ratio} = S_b/d \]

\[ S_a = \text{Allowable settlement} \]

\[ N_{fm} = \text{Normalized side load transfer ratio} = Q_{fm}/Q_f \]

\[ N_{bm} = \text{Normalized base load transfer ratio} = Q_{bm}/Q_b \]

Example 17.1
A multistory building is to be constructed in a stiff to very stiff clay. The soil is homogeneous to a great depth. The average value of undrained shear strength \( c_u \) is 150 kN/m\(^2\). It is proposed to use a drilled pier of length 25 m and diameter 1.5 m. Determine (a) the ultimate load capacity of the pier, and (b) the allowable load on the pier with \( F_s = 2.5 \). (Fig. Ex. 17.1)

Solution
Base load
When \( c_u \geq 96 \text{ kPa} \) (2000 lb/ft\(^2\)), use Eq. (17.8) for computing \( q_b \). In this case \( c_u > 96 \text{ kPa} \).

\[ q_b = 9c_u = 9 \times 150 = 1350 \text{ kN/m}^2 \]
Frictional load

The unit ultimate frictional resistance $f_s$ is determined using Eq. (17.24)

$$f_s = \alpha c_u$$

From Fig. (17.15) $\alpha = 0.55$ for $c_u/p_a = 150/101 = 1.5$

where $p_a$ is the atmospheric pressure $= 101$ kPa

Therefore $f_s = 0.55 \times 150 = 82.5$ kN/m$^2$

The effective length of the shaft for computing the frictional load (Fig. 17.13 a) is

$L' = (L - (d + 1.5))$ m $= 25 - (1.5 + 1.5) = 22$ m

The effective surface area $A_s = \pi d L' = 3.14 \times 1.5 \times 22 = 103.62$ m$^2$

Therefore $Q_f = f_s A_s = 82.5 \times 103.62 = 8,549$ kN

The total ultimate load is

$Q_u = Q_f + Q_b = 8,549 + 2,384 = 10,933$ kN

The allowable load may be determined by applying an overall factor of safety to $Q_u$. Normally $F_s = 2.5$ is sufficient.

$$Q_a = \frac{10,933}{2.5} = 4,373$$ kN
Example 17.2
For the problem given in Ex. 17.1, determine the allowable load for a settlement of 10 mm (= $S_a$). All the other data remain the same.

Solution
Allowable skin load

\[
S_R = \frac{S_a}{d} = \frac{10}{1.5 \times 10^3} \times 100 = 0.67\%
\]

From Fig. 17.18 for $S_R = 0.67\%$, \( N_{fm} = \frac{Q_{fm}}{Q_f} = 0.95 \) by using the trend line.

\[Q_{fm} = 0.95 \times 8,549 = 8,122 \, \text{kN}.
\]

Allowable base load for $S_a = 10 \, \text{mm}$

From Fig. 17.19 for $S_R = 0.67 \%$, \( N_{bm} = \frac{Q_{bm}}{Q_f} = 0.4 \)

\[Q_{bm} = 0.4 \times Q_b = 0.4 \times 2,384 = 954 \, \text{kN}.
\]

Now the allowable load $Q_{as}$ based on settlement consideration is

\[Q_{as} = Q_{fm} + Q_{bm} = 8,122 + 954 = 9,076 \, \text{kN}.
\]

$Q_{as}$ based on settlement consideration is very much higher than $Q_a$ (Ex. 17.1) and as such $Q_a$ governs the criteria for design.

Example 17.3
Figure Ex. 17.3 depicts a drilled pier with a belled bottom. The details of the pile and the soil properties are given in the figure. Estimate (a) the ultimate load, and (b) the allowable load with $F_s = 2.5$.

Solution
Based load

Use Eq. (17.8) for computing $q_b$

\[q_b = 9c_u = 9 \times 200 = 1,800 \, \text{kN/m}^2
\]

Base load $Q_b = \frac{\pi d^2}{4} \times q_b = \frac{3.14 \times 3^2}{4} \times 1,800 = 12,717 \, \text{kN}$

Frictional load

The effective length of shaft $L' = 25 - (2.75 + 1.5) = 20.75 \, \text{m}$

From Eq. (17.24) \( f_s = \alpha c_u \)

For $\frac{c_u}{p_a} = \frac{100}{101} = 1.0$, \( \alpha = 0.55 \) from Fig. 17.15

Hence $f_s = 0.55 \times 100 = 55 \, \text{kN/m}^2$
Example 17.4
For the problem given in Ex. 17.3, determine the allowable load $Q_{at}$ for a settlement $S_a = 10$ mm.

Solution

Skin load $Q_{fm}$ (mobilized)

Settlement ratio $S_R = \frac{10}{1.5 \times 10^3} \times 100 = 0.67\%$

From Fig. 17.18 for $S_R = 0.67$, $N_{fm} = 0.95$ from the trend line.

Therefore $Q_{fm} = 0.95 \times 5.375 = 5.106$ kN

Base load $Q_{bm}$ (mobilized)

$S_R = \frac{10}{3 \times 10^3} \times 100 = 0.33\%$

From Fig. 17.19 for $S_R = 0.33\%$, $N_{bm} = 0.3$ from the trend line.
Hence $Q_{em} = 0.3 \times 12,717 = 3815 \text{ kN}$

$Q_{ar} = Q_{fm} + Q_{em} = 5,106 + 3,815 = 8,921 \text{ kN}$

The factor of safety with respect to $Q_a$ is (from Ex. 17.3)

$$F_s = \frac{18,092}{8,921} = 2.03$$

This is low as compared to the normally accepted value of $F_s = 2.5$. Hence $Q_a$ rules the design.

**Example 17.5**

Figure Ex. 17.5 shows a straight shaft drilled pier constructed in homogeneous loose to medium dense sand. The pile and soil properties are:

$L = 25 \text{ m}, \ d = 1.5 \text{ m}, \ c = 0, \ \phi = 36^\circ \text{ and } \gamma = 17.5 \text{ kN/m}^3$

Estimate (a) the ultimate load capacity, and (b) the allowable load with $F_s = 2.5$. The average SPT value $N_{cor} = 30$ for $\phi = 36^\circ$.

Use (i) Vesic’s method, and (ii) the O’Neill and Reese method.

**Solution**

(i) **Vesic’s method**

From Eq. (17.10) for $c = 0$

$$q_o = (N_q - 1)q_o's_q'd_q' C_q$$

$$q_o' = 25 \times 17.5 = 437.5 \text{ kN/m}^2$$

From Eq. (17.16) $\phi_{rel} = \frac{\phi' - 25^\circ}{45^\circ - 25^\circ} = \frac{36 - 25}{20} = 0.55$

From Eq. (17.15) $\Delta = \frac{0.005(1-0.55) \times 437.5}{101} \approx 0.01$

From Eq. (17.14) $\mu_d = 0.1 + 0.3\phi_{rel} = 0.1 + 0.3 \times 0.55 = 0.265$

From Eq. (17.17) $E_d = 200 \rho_d = 200 \times 101 = 20,200 \text{ kN/m}^2$

From Eq. (17.13) $I_r = \frac{E_d}{2(1+\mu_d)q_o' \tan \phi} = \frac{20,200}{2(1+0.265) \times 437.5 \tan 36^\circ} = 25$

From Eq. (17.12) $I_m = \frac{I_r}{1+\Delta I_r} = \frac{25}{1+0.01 \times 25} = 20$

From Eq. (17.11b)

$$C_q = \exp \left[-3.8 \tan 36^\circ\right] + \frac{3.07 \sin 36^\circ \log_{10} 2 \times 20}{1 + \sin 36^\circ} \approx \exp (-0.9399) = 0.391$$

From Fig. 17.14, $N_q = 30$ for $\phi = 36^\circ$
From Table (17.2) \( s_q = 1 + \tan 36^\circ = 1.73 \)

\[
d_q = 1 + 2 \tan 36^\circ (1 - \sin 36^\circ)^2 = 1.73
\]

Substituting in Eq. (a)

\[
q_b = (30 - 1) \times 437.5 \times 1.73 \times 1.373 \times 0.391 = 11,783 \text{ kN/m}^2 > 11,000 \text{ kN/m}^2
\]

As per Tomlinson (1986) the computed \( q_b \) should be less than 11,000 kN/m².

Hence \( Q_b = \frac{3.14}{4} \times (1.5)^2 \times 11,000 = 19,429 \text{ kN} \)

**Skin load** \( Q_f \)

From Eqs (17.31) and (17.32)

\[
f_s = \beta q'_o, \quad \beta = 1.5 - 0.245 z^{0.5}, \text{ where } z = \frac{L}{2} = \frac{25}{2} = 12.5 \text{ m}
\]

Substituting

\[
\beta = 1.5 - 0.245 \times (12.5)^{0.5} = 0.63
\]

Hence \( f_s = 0.63 \times 437.5 = 275.62 \text{ kN/m}^2 \)

Per Tomlinson (1986) \( f_s \) should be limited to 110 kN/m². Hence \( f_s = 110 \text{ kN/m}^2 \)

Therefore \( Q_f = \pi d l f_s = 3.14 \times 1.5 \times 25 \times 110 = 12,953 \text{ kN} \)
Ultimate load \( Q_u = 19,429 + 12,953 = 32,382 \text{ kN} \)

\[ Q_u = \frac{32,382}{2.5} = 12,953 \text{ kN} \]

**O’Neill and Reese method**

This method relates \( q_b \) to the SPT \( N \) value as per Eq. (17.19a)

\[ q_b = 57.5 N \text{ kN/m}^2 = 57.5 \times 30 = 1,725 \text{ kN/m}^2 \]

\[ Q_b = A_q q_b = 1.766 \times 1,725 = 3,046 \text{ kN} \]

The method for computing \( Q_f \) remains the same as above.

Now \( Q_u = 3,406 + 12,953 = 15,999 \)

\[ Q_u = \frac{15,999}{2.5} = 6,400 \text{ kN} \]

---

**Example 17.6**

Compute \( Q_u \) and \( Q_a \) for the pier given in Ex. 17.5 by the following methods.

1. Use the SPT value [Eq. (15.48)] for bored piles
2. Use the Tomlinson method of estimating \( Q_b \) and Table 15.2 for estimating \( Q_f \). Compare the results of the various methods.

**Solution**

**Use of the SPT value [Meyerhof Eq. (15.48)]**

\[ q_b = 133 N_{cor} = 133 \times 30 = 3,990 \text{ kN/m}^2 \]

\[ Q_b = \frac{3.14 \times (1.5)^2}{4} \times 3,990 = 7,047 \text{ kN} \]

\[ f_s = 0.67 N_{cor} = 0.67 \times 30 = 20 \text{ kN/m}^2 \]

\[ Q_f = 3.14 \times 1.5 \times 25 \times 20 = 2,355 \text{ kN} \]

\[ Q_u = 7,047 + 2,355 = 9,402 \text{ kN} \]

\[ Q_u = \frac{9,402}{2.5} = 3,760 \text{ kN} \]

**Tomlinson Method for \( Q_b \)**

For a driven pile

From Fig. 15.9 \( N_q = 65 \) for \( \phi = 36^\circ \) and \( \frac{L}{d} = \frac{25}{15} = 17 \)

Hence \( q_b = q'_\phi N_q = 437.5 \times 65 = 28,438 \text{ kN/m}^2 \)
For bored pile

\[ q_b = \frac{1}{3} q_p \text{ (driven pile)} = \frac{1}{3} \times 28.438 = 9.479 \text{ kN/m}^2 \]

\[ Q_b = A_b q_b = 1.766 \times 9.479 = 16.740 \text{ kN} \]

**\( Q_f \) from Table 15.2**

For \( \phi = 36^\circ, \delta = 0.75 \times 36 = 27, \text{ and } \bar{K}_s = 1.5 \) (for medium dense sand).

\[ f_s = q_p' \bar{K}_s \tan \delta = \frac{437.5}{2} \times 1.5 \tan 27^\circ = 167 \text{ kN/m}^2 \]

As per Tomlinson (1986) \( f_s \) is limited to 110 kN/m². Use \( f_s = 110 \text{ kN/m}^2 \).

Therefore \( Q_f = 3.14 \times 1.5 \times 25 \times 110 = 12,953 \text{ kN} \)

\[ Q_u = Q_b + Q_f = 16,740 + 12,953 = 29,693 \text{ kN} \]

\[ Q_a = \frac{29,693}{2.5} = 11,877 \text{ kN} \]

**Comparison of estimated results (\( F = 2.5 \))**

<table>
<thead>
<tr>
<th>Example No</th>
<th>Name of method</th>
<th>( Q_b ) (kN)</th>
<th>( Q_f ) (kN)</th>
<th>( Q_u ) (kN)</th>
<th>( Q_a ) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>17.5</td>
<td>Vesic</td>
<td>19,429</td>
<td>12,953</td>
<td>32,382</td>
<td>12,953</td>
</tr>
<tr>
<td>17.5</td>
<td>O’Neill and Reese, for ( Q_b ) and Vesic for ( Q_f )</td>
<td>3,046</td>
<td>12,953</td>
<td>15,999</td>
<td>6,400</td>
</tr>
<tr>
<td>17.6</td>
<td>Meyerhof Eq. (15.49)</td>
<td>7,047</td>
<td>2,355</td>
<td>9,402</td>
<td>3,760</td>
</tr>
<tr>
<td>17.6</td>
<td>Tomlinson for ( Q_b ) (Fig. 15.9) Table 15.2 for ( Q_f )</td>
<td>16,740</td>
<td>12,953</td>
<td>29,693</td>
<td>11,877</td>
</tr>
</tbody>
</table>

**Which method to use**

The variation in the values of \( Q_b \) and \( Q_f \) are very large between the methods. Since the soils encountered in the field are generally heterogeneous in character no theory holds well for all the soil conditions. Designers have to be practical and pragmatic in the selection of any one or combination of the theoretical approaches discussed earlier.

**Example 17.7**

For the problem given in Example 17.5 determine the allowable load for a settlement of 10 mm. All the other data remain the same. Use (a) the values of \( Q_f \) and \( Q_b \) obtained by Vesic’s method, and (b) \( Q_b \) from the O’Neill and Reese method.

**Solution**

(a) Vesic’s values \( Q_f \) and \( Q_b \)

Settlement ratio for \( S_a = 10 \text{ mm} \) is

\[ S_R = \frac{S_a}{d} = \frac{10 \times 10^2}{1.5 \times 10^3} = 0.67\% \]

From Fig. 17.20 for \( S_R = 0.67\% \) \( N_{fm} = 0.96 \) (approx.) using the trend line.
\[ Q_{fa} = 0.96 \times Q_f = 0.96 \times 12,953 = 12,435 \text{ kN} \]

From Fig. 17.21 for \( S_h = 0.67\% \)

\[ N_{bm} = 0.20, \text{ or } Q_{bm} = 0.20 \times 19,429 = 3,886 \text{ kN} \]

\[ Q_{as} = 12,435 + 3,886 = 16,321 \text{ kN} \]

Shear failure theory gives \( Q_a = 12,953 \text{ kN} \) which is much lower than \( Q_{as} \). As such, \( Q_a \) determines the criteria for design.

(b) O’Neill and Reese \( Q_b = 3,046 \text{ kN} \)

As above, \( Q_{bm} = 0.20 \times 3,046 = 609 \text{ kN} \)

Using \( Q_{fa} \) in (a) above,

\[ Q_{as} = 609 + 12,435 = 13,044 \text{ kN} \]

The value of \( Q_{as} \) is closer to \( Q_a \) (Vesic) but much higher than \( Q_a \) calculated by all the other methods.

**Example 17.8**

Figure Ex. 17.8 shows a drilled pier penetrating an IGM: clay-shale to a depth of 8 m. Joints exist within the IGM stratum. The following data are available:

- \( L_s = 8 \text{ m } (= z_c) \)
- \( d = 1.5 \text{ m} \)
- \( q_u \) (rock) = \( 3 \times 10^3 \) kN/m\(^2\)
- \( E_f \) (rock) = \( 600 \times 10^3 \) kN/m\(^2\)
- Concrete slump = 175 mm, unit weight of concrete \( \gamma_c = 24 \) kN/m\(^3\)
- \( E_c \) (concrete) = \( 435 \times 10^6 \) kN/m\(^2\)
- RQD = 70 percent, \( q_u \) (concrete) = \( 435 \times 10^6 \) kN/m\(^2\).

Determine the ultimate frictional load \( Q_f \) (max).
Solution

(a) Determine $\alpha$ in Eq. (17.26)

$$f_a = \alpha q_u$$

where $q_u = 3$ MPa for rock

For the depth of socket $L_s = 8$ m, and slump = 175 mm

$M = 0.76$ from Fig. 17.17.

From Eq. (17.27)

$$\sigma_n = Myz_c = 0.76 \times 24 \times 8 = 146 \text{ kN/m}^2$$

$$p_a = 101 \text{ kN/m}^2, \quad \sigma_n/p_a = 146/101 = 1.45$$

From Fig. 17.16 for $q_u = 3$ MPa and $\sigma_n/p_a = 1.45$, we have $\alpha = 0.11$

(b) Determination of $f_a$

$$f_a = 0.11 \times 3 = 0.33 \text{ MPa}$$

(c) Determination $f_{aa}$ in Eq. (17.28)

For RQD = 70%, $E_m/E_i = 0.1$ from Table 17.5 for open joints, and $f_{aa}/f_a (= R_a) = 0.55$ from Table 17.6

$$f_{\text{max}} = f_{aa} = 0.55 \times 0.33 = 0.182 \text{ MPa} = 182 \text{ kN/m}^2$$

(d) Ultimate friction load $Q_f$

$$Q_f = PLf_{aa} = 3.14 \times 1.5 \times 8 \times 182 = 6,858 \text{ kN}$$

Example 17.9

For the pier given in Ex. 17.8, determine the ultimate bearing capacity of the base. Neglect the frictional resistance. All the other data remain the same.

Solution

For RQD between 70 and 100 percent

from Eq. (17.22)

$$q_b (= q_{\text{max}}) = 4.83(q_u)^{0.5} \text{ MPa} = 4.83 \times (3)^{0.5} = 8.37 \text{ MPa}$$

$$Q_b(\text{max}) = \frac{3.14}{4} \times 1.5^2 \times 8.37 = 14.78 \text{ MN = 14,780 kN}$$

17.16 UPLIFT CAPACITY OF DRILLED PIERS

Structures subjected to large overturning moments can produce uplift loads on drilled piers if they are used for the foundation. The design equation for uplift is similar to that of compression. Figure 17.22 shows the forces acting on the pier under uplift-load $Q_{ul}$. The equation for $Q_{ul}$ may be expressed as

$$Q_{ul} = Q_{fr} + W_p = A_s f_r + W_p$$  \hspace{1cm} (17.40)
where, \( Q_{fr} \) = total side resistance for uplift
\( W_p \) = effective weight of the drilled pier
\( A_s \) = surface area of the pier
\( f_r \) = frictional resistance to uplift

**Uplift Capacity of Single Pier (straight edge)**

For a drilled pier in cohesive soil, the frictional resistance may expressed as (Chen and Kulhawy, 1994)

\[
Q_{fr} = W_p + f_r + A_s f_r
\]

\[
W_p = \frac{1}{2} A_s \rho g h
\]

\[
f_r = \alpha c_u
\]

where, \( \alpha = \) adhesion factor
\( c_u = \) undrained shear strength of cohesive soil
\( P_o = \) atmospheric pressure (101 kPa)

Chen and Kulhawy (1994) used the relationship between \( c_u \) and \( \alpha \) as given in Fig. 17.23. The curves in the figure are based on pull out test data collected by Sowa (1970).

**Uplift Resistance of Piers in Sand**

There are no confirmatory methods available for evaluating uplift capacity of piers embedded in cohesionless soils. Poulos and Davis (1980) suggest that the skin frictional resistance for pull out may be taken as equal to two-thirds of the shaft resistance for downward loading.

**Uplift Resistance of Piers in Rock**

According to Carter and Kulhawy (1988), the frictional resistance offered by the surface of the pier under uplift loading is almost equal to that for downward loading if the drilled pier is rigid relative.
to the rock. The effective rigidity is defined as \((E/E_m)(d/D)^2\), in which \(E\) and \(E_m\) are the Young's modulus of the drilled pier and rock mass respectively, \(d\) is the socket diameter and \(D\) is the depth of the socket. A socket is rigid when \((E/E_m)(d/D)^2 \geq 4\). When the effective rigidity is less than 4, the frictional resistance \(f_r\) for upward loading may be taken as equal to 0.7 times the value for downward loading.

**Example 17.10**

Determine the uplift capacity of the drilled pier given in Fig. Ex. 17.10. Neglect the weight of the pier.

**Solution**

From Eq. (17.40)

\[ Q_{ul} = A_s f_r \]

From Eq. (17.41a) \( f_r = \alpha c_u \)

From Eq. (17.41b) \( \alpha = 0.31 + 0.17 \frac{c_u}{P_d} \)

Given: \( L = 25\) m, \( d = 1.5\) m, \( c_u = 150\) kN/m²

Hence \( \alpha = 0.31 + 0.17 \times \frac{150}{101} = 0.56 \)

\( f_r = 0.56 \times 150 = 84\) kN/m²

\( Q_{ul} = 3.14 \times 1.5 \times 25 \times 84 = 9,891\) kN

It may be noted here that \( f_r = 82.5\) kN/m² for downward loading and \( f_r = 84\) kN/m² for uplift. The two values are very close to each other.

**17.17 LATERAL BEARING CAPACITY OF DRILLED PIERS**

It is quite common that drilled piers constructed for bridge foundations and other similar structures are also subjected to lateral loads and overturning moments. The methods applicable to piles are applicable to piers also. Chapter 16 deals with such problems. This chapter deals with one more method as recommended by O’Neill and Reese (1999). This method is called **Characteristic load method** and is described below.

**Characteristic Load Method (Duncan et al., 1994)**

The characteristic load method proceeds by defining a characteristic or normalizing shear load \( P_c \) and a characteristic or normalizing bending moment \( M_c \) as given below.

For clay

\[ P_c = 7.34d^2(ER_1) \left( \frac{c_u}{ER_1} \right)^{0.68} \]  

\[ M_c = 3.86d^3(ER_1) \left( \frac{c_u}{ER_1} \right)^{0.46} \]
For sand

\[ P_c = 1.57d^3(ER_f) \frac{\gamma'd \phi' K_p}{ER_f} \]  \hspace{1cm} (17.44)

\[ M_c = 1.33d^3(ER_f) \frac{\gamma'd \phi' K_p}{ER_f} \]  \hspace{1cm} (17.45)

where:

- \( d \) = shaft diameter
- \( E \) = Young's modulus of the shaft material
- \( R_f \) = ratio of moment of inertia of drilled shaft to moment of inertia of solid section (= 1 for a normal uncracked drilled shaft without central voids)
- \( c_u \) = average value of undrained shear strength of the clay in the top 8\( d \) below the ground surface
- \( \gamma' \) = average effective unit weight of the sand (total unit weight above the water table, buoyant unit weight below the water table) in the top 8\( d \) below the ground surface
- \( \phi' \) = average effective stress friction angle for the sand in the top 8\( d \) below ground surface
- \( K_p \) = Rankine's passive earth pressure coefficient \( = \tan^2 (45° + \phi/2) \)

In the design method, the moments and shears are resolved into groundline values, \( P_t \) and \( M_t \), and then divided by the appropriate characteristic load values [Equations (17.42) through (17.45)]. The lateral deflections at the shaft head, \( y_t \), are determined from Figures 17.23 and 17.24, considering the conditions of pile-head fixity. The value of the maximum moment in a free- or fixed-headed drilled shaft can be determined through the use of figure 17.25 if the only load that is applied is ground line shear. If both a moment and a shear are applied, one must compute \( y_t \) (combined), and then solve Eq. (17.46) for the “characteristic length” \( T \) (relative stiffness factor).

\[ y_t \text{ (combined)} = 2.43 \frac{PT^3}{EI} + 1.62 \frac{MT^2}{EI} \]  \hspace{1cm} (17.46)

where \( I \) is the moment of inertia of the cross-section of the drilled shaft.

Figure 17.23  Groundline shear-deflection curves for (a) clay and (b) sand (Duncan et al., 1994)
The principle of superposition is made use of for computing ground line deflections of piers (or piles) subjected to groundline shears and moments at the pier head. The explanation given here applies to a free-head pier. The same principle applies for a fixed head pile also.

Consider a pier shown in Fig. 17.26(a) subjected to a shear load \( P_t \) and moment \( M_t \) at the pile head at ground level. The total deflection \( y_t \) caused by the combined shear and moment may be written as

\[
y_t = y_p + y_m \tag{17.47}
\]

where \( y_p = \) deflection due to shear load \( P_t \) alone with \( M_t = 0 \)

\( y_m = \) deflection due to moment \( M_t \) alone with \( P_t = 0 \)

Again consider Fig. 17.26(b). The shear load \( P_t \) acting alone at the pile head causes a deflection \( y_p \) (as above) which is equal to deflection \( y_{pm} \) caused by an equivalent moment \( M_p \) acting alone.
In the same way Fig. 17.26(c) shows a deflection \( y_m \) caused by moment \( M_t \) at the pile head. An equivalent shear load \( P_m \) causes the same deflection \( y_m \) which is designated here as \( y_{mp} \). Based on the principles explained above, groundline deflection at the pile head due to a combined shear load and moment may be explained as follows.

1. Use Figs 17.23 and 17.24 to compute groundline deflections \( y_p \) and \( y_m \) due to shear load and moment respectively.
2. Determine the groundline moment \( M_p \) that will produce the same deflection as by a shear load \( P_t \) (Fig. 17.26(b)). In the same way, determine a groundline shear load \( P_m \) that will produce the same deflection as that by the groundline moment \( M_t \) (Fig. 17.26(c)).
3. Now the deflections caused by the shear loads \( P_t + P_m \) and that caused by the moments \( M_t + M_p \) may be written as follows:

\[
y_t = y_p + y_m
\]
Figure 17.27 Parameters $A_m$ and $B_m$ (Matlock and Reese, 1961)

\[ y_{ip} = y_p + y_{mp} \]  \hspace{1cm} (17.48)

\[ y_{im} = y_m + y_{pm} \]  \hspace{1cm} (17.49)

Theoretically $y_{ip} = y_{im}$

4. Lastly the total deflection $y_t$ is obtained as

\[ y_t = \frac{y_{ip} + y_{im}}{2} = \frac{(y_p + y_{mp}) + (y_m + y_{pm})}{2} \]  \hspace{1cm} (17.50)

The distribution of moment along a pier may be determined using Eq. (16.11) and Table 16.2 or Fig. 17.27.

**Direct Method by Making Use of $n_h$**

The direct method developed by Murthy and Subba Rao (1995) for long laterally loaded piles has been explained in Chapter 16. The application of this method for long drilled piers will be explained with a case study.

**Example 17.11 (O’Neill and Reese, 1999)**

Refer to Fig. Ex. 17.11. Determine for a free-head pier (a) the groundline deflection, and (b) the maximum bending moment. Use the Duncan et al. (1994) method. Assume $R_f = 1$ in the Eqs (17.44) and (17.45).

**Solution**

Substituting in Eqs (17.42) and (17.43)

\[ P_c = 7.34 \times (0.80)^2 \times [25 \times 10^3 \times (1)] \times \frac{0.06}{25 \times 10^3} = 0.68 \]  \hspace{1cm} 17.72 MN
\[ M_c = 3.86(0.80)^3[25 \times 10^3 \times (1)] \frac{0.06}{25 \times 10^3} = 128.5 \text{ MN} \cdot \text{m} \]

Now \( \frac{P_p}{P_c} = \frac{0.080}{17.72} = 0.0045 \), \( M_p/M_c = 0.4/128.5 = 0.0031 \)

Step 1

From Fig. 17.23a for \( \frac{P_p}{P_c} = 0.0045 \)

\[ \frac{y_p}{d} = 0.003 \text{ or } y_p = 0.003 \times 0.8 \times 10^3 = 2.4 \text{ mm} \]

From Fig. 17.24a, \( M_p/M_c = 0.0031 \)

\[ \frac{y_m}{d} = 0.006 \text{ or } y_m = 0.006d = 0.006 \times 0.8 \times 10^3 = 4.8 \text{ mm} \]

Step 2

From Fig. 17.23(a) for \( y_m/d = 0.006 \), \( P_p/P_c = 0.0055 \)

From Fig. 17.24(a), for \( y_p/d = 0.003 \), \( M_p/M_c = 0.0015 \).

\[ \text{Figure Ex. 17.11} \]
Step 3
The shear loads $P_t$ and $P_m$ applied at ground level, may be expressed as

$$\frac{P_t + P_m}{P_c} = 0.0045 + 0.0055 = 0.01$$

From Fig. 17.23,

$$\frac{y_{tp}}{d} = 0.013 \text{ for } \frac{P_t + P_m}{P_c} = 0.01$$

or $y_{tp} = 0.013 \times (0.80) \times 10^3 = 10.4 \text{ mm}$

Step 4
In the same way as in Step 3

$$\frac{M_t + M_p}{M_c} = 0.0031 + 0.0015 = 0.0046$$

From Fig. 17.24a

$$\frac{y_{m}}{d} = \frac{y_{m} + y_{pm}}{d} = 0.011$$

Hence $y_{tm} = 0.011 \times 0.8 \times 10^3 = 8.8 \text{ mm}$

Step 5
From Eq. (17.50)

$$y_t = \frac{y_{tp} + y_{tm}}{2} = \frac{10.4 + 8.8}{2} = 9.6 \text{ mm}$$

Step 6
The maximum moment for the combined shear load and moment at the pier head may be calculated in the same way as explained in Chapter 16. $M_{(\text{max})}$ as obtained is

$$M_{(\text{max})} = 470.5 \text{ kN-m}$$

This occurs at a depth of 1.3 m below ground level.

Example 17.12
Solve the problem in Ex. 17.11 by the direct method.

Given: $EI = 52.6 \times 10^4 \text{kN-m}^2$, $d = 80 \text{ cm}$, $y = 17.5 \text{kN/m}^3$, $e = 5 \text{ m}$, $L = 9 \text{ m}$, $c = 60 \text{kN/m}^2$ and $P_c = 80 \text{kN}$.

Solution

groundline deflection

From Eq. (16.30) for piers in clay

$$n_h = \frac{125e^{1.5}}{d} \sqrt{\frac{EIy}{d}}$$

$$1 + \frac{e}{d} \times \frac{P_c}{d}$$
Substituting and simplifying

\[ n_h = \frac{125 \times 60^{1.5} \sqrt{52.6 \times 10^4 \times 17.5 \times 0.8}}{1 + \frac{5}{0.8} \times P_e^{1.5}} = \frac{808 \times 10^4}{P_e^{1.5}} \text{ kN/m}^3 \]  

(a)

**Step 1**

Assume \( P_e = P_t = 80 \) kN,

From Eq. (a), \( n_h = 11,285 \) kN/m³ and

\[ T = \frac{EI}{n_h^{0.2}} = \frac{52.6 \times 10^4}{11,285^{0.2}} = 2.16 \text{ m} \]

**Step 2**

From Eq. (16.22) \( P_e = P_t \times 1 + 0.67 \frac{e}{T} = 80 \times 1 + 0.67 \times \frac{5}{2.16} = 204 \) kN

From Eq. (a), \( n_h = 2772 \) kN/m³, hence \( T = 2.86 \) m

**Step 3**

Continue the above process till convergence is reached. The final values are

\( P_e = 177 \) kN, \( n_h = 3410 \) kN/m³ and \( T = 2.74 \) m

For \( P_e = 190 \) kN, we have \( n_h = 8,309 \) kN/m³ and \( T = 2.29 \) m

**Step 4**

From Eq. (17.46)

\[ y_t = \frac{2.43 \times 177 \times (2.74)^3}{52.6 \times 10^4} \times 1000 = 16.8 \text{ mm} \]

By Duncan et al, method \( y_t = 9.6 \) mm

Maximum moment from Eq. (16.12)

\[ M = [P_T]A_m + [M_1]B_m = [80 \times 2.74]A_m + [400]B_m = 219.2A_m + 400B_m \]

<table>
<thead>
<tr>
<th>Depth x/T = Z</th>
<th>( A_m )</th>
<th>( B_m )</th>
<th>( M_1 )</th>
<th>( M_2 )</th>
<th>( M ) (kN-m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>400</td>
<td>400</td>
</tr>
<tr>
<td>0.4</td>
<td>0.379</td>
<td>0.99</td>
<td>83</td>
<td>396</td>
<td>479</td>
</tr>
<tr>
<td>0.5</td>
<td>0.46</td>
<td>0.98</td>
<td>101</td>
<td>392</td>
<td>493</td>
</tr>
<tr>
<td>0.6</td>
<td>0.53</td>
<td>0.96</td>
<td>116</td>
<td>384</td>
<td>500</td>
</tr>
<tr>
<td>0.7</td>
<td>0.60</td>
<td>0.94</td>
<td>132</td>
<td>376</td>
<td><strong>508</strong> (max)</td>
</tr>
<tr>
<td>0.8</td>
<td>0.65</td>
<td>0.91</td>
<td>142</td>
<td>364</td>
<td>506</td>
</tr>
</tbody>
</table>

The maximum bending moment occurs at \( x/T = 0.7 \) or \( x = 0.7 \times 2.74 = 1.91 \) m (6.26 ft). As per Duncan et al., method \( M(\text{max}) = 470.5 \) kN-m. This occurs at a depth of 1.3 m.
17.18 CASE STUDY OF A DRILLED PIER SUBJECTED TO LATERAL LOADS

Lateral load test was performed on a circular drilled pier by Davisson and Salley (1969). Steel casing pipe was provided for the concrete pier. The details of the pier and the soil properties are given in Fig. 17.28. The pier was instrumented and subjected to cyclic lateral loads. The load deflection curve as obtained by Davison et al., is shown in the same figure.

Direct method (Murthy and Subba Rao, 1995) has been used here to predict the load displacement relationship for a continuous load increase by making use of Eq. (16.29). The predicted curve is also shown in Fig. 17.28. There is an excellent agreement between the predicted and the observed values.

17.19 PROBLEMS

17.1 Fig. Prob. 17.1 shows a drilled pier of diameter 1.25 m constructed for the foundation of a bridge. The soil investigation at the site revealed soft to medium stiff clay extending to a great depth. The other details of the pier and the soil are given in the figure. Determine (a) the ultimate load capacity, and (b) the allowable load for \( F_s = 2.5 \). Use Vesic's method for base load and \( \alpha \)-method for the skin load.

17.2 Refer to Prob. 17.1. Given \( d = 3 \text{ ft}, L = 30 \text{ ft}, \) and \( c_u = 1050 \text{ lb/ft}^2 \). Determine the ultimate (a) base load capacity by Vesic's method, and (b) the frictional load capacity by the \( \alpha \)-method.
17.3 Fig. Prob. 17.3 shows a drilled pier with a belled bottom constructed for the foundation of a multistory building. The pier passes through two layers of soil. The details of the pier and the properties of the soil are given in the figure. Determine the allowable load $Q_a$ for $F_s = 2.5$. Use (a) Vesic’s method for the base load, and (b) the O’Neill and Reese method for skin load.

![Figure Prob. 17.1](image1)

**Figure Prob. 17.1**

17.4 For the drilled pier given in Fig. Prob. 17.1, determine the working load for a settlement of 10 mm. All the other data remain the same. Compare the working load with the allowable load $Q_{a_f}$.

17.5 For the drilled pier given in Prob 17.2, compute the working load for a settlement of 0.5 in. and compare this with the allowable load $Q_{a_f}$.

17.6 If the drilled pier given in Fig. Prob. 17.6 is to carry a safe load of 2500 kN, determine the length of the pier for $F_s = 2.5$. All the other data are given in the figure.

17.7 Determine the settlement of the pier given in Prob. 17.6 by the O’Neill and Reese method. All the other data remain the same.

17.8 Fig. Prob. 17.8 depicts a drilled pier with a belled bottom constructed in homogeneous clay extending to a great depth. Determine the
17.9 Determine the settlement of the pier in Prob 17.8 for a working load of 3000 kN. All the other data remain the same. Use the length $L$ computed.

17.10 Fig. Prob 17.10 shows a drilled pier. The pier is constructed in homogeneous loose to medium dense sand. The pier details and the properties of the soil are given in the figure. Estimate by Vesic’s method the ultimate load bearing capacity of the pier.

17.11 For Problem 17.10 determine the ultimate base capacity by the O’Neill and Reese method. Compare this value with the one computed in Prob. 17.10.

17.12 Compute the allowable load for the drilled pier given in Fig. 17.10 based on the SPT value. Use Meyerhof’s method.

17.13 Compute the ultimate base load of the pier in Fig. Prob. 17.10 by Tomlinson’s method.

17.14 A pier is installed in a rocky stratum. Fig. Prob. 17.14 gives the details of the pier and the properties of the rock materials. Determine the ultimate frictional load $Q_f^{(max)}$.

17.15 Determine the ultimate base resistance of the drilled pier in Prob. 17.14. All the other data remain the same. What is the allowable load with $F_s = 4$ by taking into account the frictional load $Q_f^{(max)}$ computed in Prob. 17.14?
17.16 Determine the ultimate point bearing capacity of the pier given in Prob. 17.14 if the base rests on sound rock with RQD = 100%.

17.17 Determine the uplift capacity of the drilled pier given in Prob. 17.1. Given: $L = 15$ m, $d = 1.25$ m, and $c_u = 25$ kN/m$^2$. Neglect the weight of the pile.

17.18 The drilled pier given in Fig. Prob. 17.18 is subjected to a lateral load of 120 kips. The soil is homogeneous clay. Given: $d = 60$ in., $EI = 93 \times 10^{10}$ lb-in.$^2$, $L = 38$ ft, $e = 12$ in., $c = 2000$ lb/ft$^2$, and $\gamma_b = 60$ lb/ft$^3$. Determine by the Duncan et al., method the groundline deflection.

![Figure Prob. 17.18](image-url)