

CHAPTER 11

DEEP FOUNDATIONS INCLUDING DRILLED PIERS

11-1. General. A deep foundation derives its support from competent strata at significant depths below the surface or, alternatively, has a depth to diameter ratio greater than 4. A deep foundation is used in lieu of a shallow foundation when adequate bearing capacity or tolerable settlements cannot be obtained with a shallow foundation. The term deep foundation includes piles, piers, or caissons, as well as footings or mats set into a deep excavation. This chapter discusses problems of placing footings and mats in deep excavations and design of drilled piers. Drilled piers (or caissons) are simply large-diameter piles, but the design process is somewhat different. An arbitrary distinction between a pile and pier is that the caisson is 30 inches or more in diameter.

11-2. Floating foundations. A foundation set into a deep excavation is said to be *compensated* or *floating* if the building load is significantly offset by the load of soil removed during excavation. The foundation is *fully compensated* if the structural load equals the load removed by excavation, *partially compensated* if the structural load is greater, and *overcompensated* if the structural load is less than the weight of the excavated soil. A compensated foundation requires a study of expected subsoil rebound and settlement, excavation support systems, means to maintain foundation subsoil or rock integrity during excavation, and allowable bearing pressures for the soil or rock.

11-3. Settlements of compensated foundations.

a. The sequence of subsoil heave during excavation and subsequent settlement of a deep foundation is illustrated in figure 11-1(a). If effective stresses do not change in the subsoils upon the initial excavation, i.e., the soil does not swell due to an increase in water content, and if no plastic flow occurs, then only immediate or elastic rebound from change in stress occurs. If the structural load is fully compensated, the measured settlement of the foundation would consist only of recompression of the elastic rebound, generally a small quantity, provided subsoils are not disturbed by excavation.

b. If the negative excess pore pressures set up during excavation "dissipate," i.e., approach static values, before sufficient structural load is applied, foundation swell occurs in addition to elastic rebound. (The original effective stresses will decrease.) The

foundation load recompresses the soil, and settlement of the foundation consists of elastic and consolidation components as shown in figure 11-1(b). Consolidation occurs along the recompression curve until the preconsolidation stress is reached, whereupon it proceeds along the virgin compression curve. Calculate the foundation heave and subsequent settlement using procedures outlined in chapter 5.

c. If the depth with respect to the type and shear strength of the soil is such that plastic flow occurs, loss of ground may develop around the outside of the excavation with possible settlement damage to structures, roads, and underground utilities.

d. The rate and amount of heave may be estimated from the results of one-dimensional consolidation tests; however, field evidence shows that the rate of heave is usually faster than predicted. A study of 43 building sites found that the field heave amounted to about one-third the computed heave. Where excavations are large and are open for substantial time before significant foundation loadings are applied, the actual heave may be close to the computed heave. Figure 11-2 is a plot of a series of field results of heave versus excavation depth, in which the heave increases sharply with the depth of excavation. An example of heave and subsequent settlement calculations for a compensated foundation is shown in figure 5-4.

e. The yielding of the excavation bottom can be caused by high artesian water pressures under the excavation or by a bearing capacity failure resulting from the overburden pressure on the soil outside the excavation at subgrade elevation. Artesian pressure can be relieved by cutoffs and dewatering of the underlying aquifer using deep wells. The pumped water may be put back in the aquifer using recharge wells outside the excavation perimeter to avoid perimeter settlements or to preserve the groundwater table for environmental reasons, but this operation is not simple and should be done only when necessary.

f. The likelihood of bearing capacity failure exists primarily in clayey soils and should be analyzed as shown in chapter 14. A factor of safety, $F_s \geq 2$, should ideally be obtained to minimize yielding and possible settlement problems. A large plastic flow may cause the bottom of the excavation to move upwards with re-

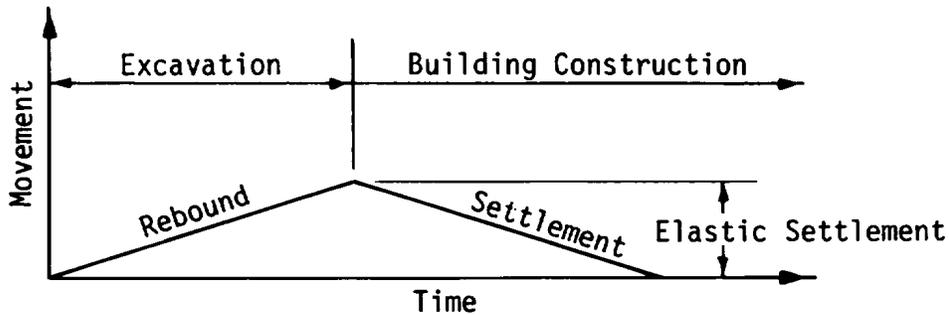
suiting loss of ground. To avoid this possibility, investigate-

- (1) Potential for plastic flow, i.e., relationship between shear stress and shear strength.
- (2) Sequence of placing wall bracing.
- (3) Depth of penetration of sheeting below base of excavation.

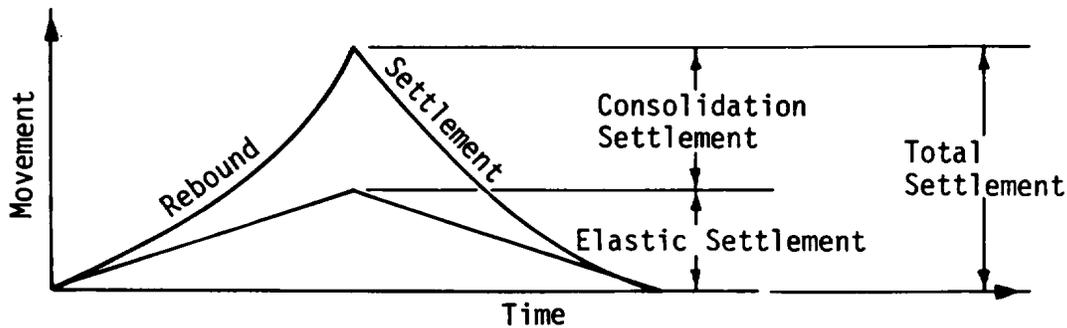
g. Two commonly used procedures to control bottom heave are dewatering and sequential excavation of the final 5 feet or more of soil. Groundwater lowering increases effective stresses and may reduce heave. Where subsoil permeabilities are not large, a deep and economical lowering of the groundwater to minimize

heave can sometimes be achieved by an educator-type wellpoint system. Permitting a controlled rise of the groundwater level as the building load is applied acts to reduce effective stresses and counteracts the effect of the added building load. Sequential excavation is accomplished by removing soil to final grade via a series of successive trenches. As each trench is opened, the foundation element is poured before any adjacent trench is opened. This procedure recognizes the fact that more heave occurs in the later excavation stages than in earlier stages and is frequently used in shales.

h. The tilting of a compensated foundation can occur if structural loads are not symmetrical or if soil



(a) No effective stress change during excavation



(b) Decrease in effective stress during excavation (causes subsoil rebound and leads to additional settlement)

Note: Figures (a) and (b) are illustrative only. Some rebound may continue after start of building construction until sufficient building load has been applied. Additional settlement exceeding that shown in figures occurs if building load exceeds weight of excavation.

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Figure 11-1. Effect of pore pressure dissipation during excavation and settlement response.

conditions are nonuniform. Tilting can be estimated from settlement calculations for different locations of the excavation. Control of tilt is not generally necessary but can be provided by piles or piers, if required. Bearing capacity is not usually important unless the building is partially compensated and founded on clay. The factor of safety against bearing failure is calculated (see chap 6 for quit) and compared with the final total soil stress using the building load, q_0 , less the excavation stress as follows:

$$FS = \frac{q_{ult}}{q_0 - \gamma D} \quad (11-1)$$

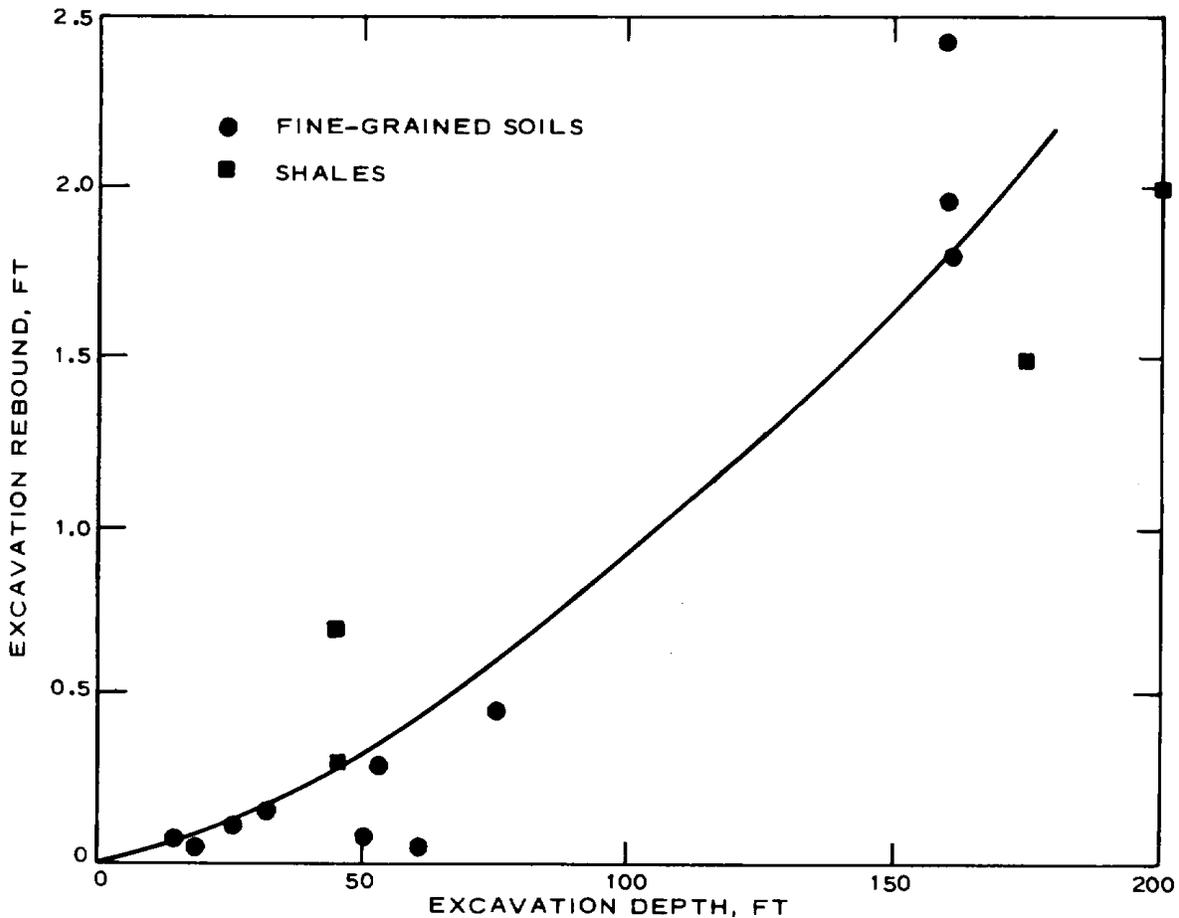
The factor of safety should be between 2.5 and 3.0 for dead load plus normal live load.

i. Settlement adjacent to excavations depends on the soil type and the excavation support system method employed (chap 14). With properly installed strutted or anchored excavations in

cohesionless soils, settlement will generally be less than 0.5 percent of excavation depth. Loss of ground due to uncontrolled seepage or densification of loose cohesionless soils will result in larger settlements. Surface settlements adjacent to open cuts in soft to firm clay will occur because of lateral yielding and movement of soil beneath the bottom of the cut. Figure 11-3 can be used to estimate the magnitude and extent of settlement.

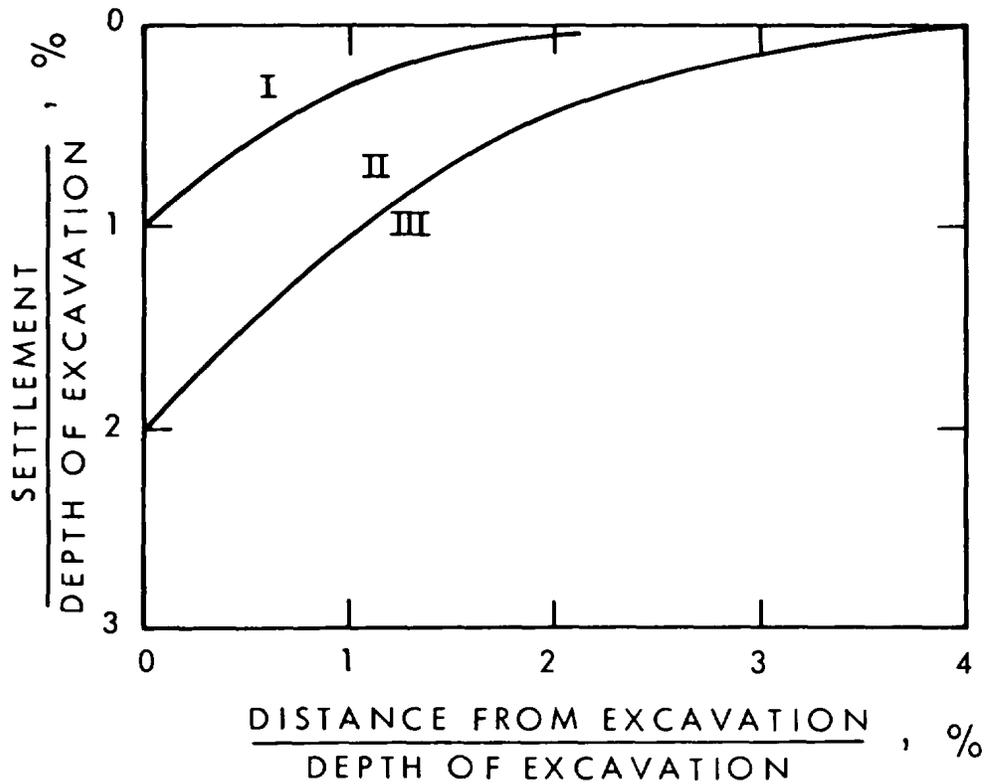
11-4. Underpinning.

a. Structures supported by shallow foundations or short piles may have to be underpinned if located near an excavation. Techniques for underpinning are depicted in figure 11-4. The most widely accepted methods are jacked down piles or piers, which have the advantage of forming positive contact with the building foundation since both can be prestressed. The use of drilled piers is of more recent vintage and is more economical where it can be used. In sandy soils, chemical



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Figure 11-2. Excavation rebound versus excavation depth.



ZONE I - SAND AND SOFT TO HARD CLAY, ($s_u > 500$ LB/SQ FT)

ZONE II - VERY SOFT TO SOFT CLAY ($s_u < 500$ LB/SQ FT)
 1) - LIMITED DEPTH OF CLAY BELOW BASE OF EXCAVATION
 2) - SIGNIFICANT DEPTH OF CLAY BELOW BASE OF EXCAVATION WHERE $\frac{\gamma H}{s_u} < 5$

ZONE III - VERY SOFT TO SOFT CLAY, ($s_u < 500$ LB/SQ FT)
 1) - SIGNIFICANT DEPTH OF CLAY BELOW BASE OF EXCAVATION AND WHERE $\frac{\gamma H}{s_u} \geq 5$

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Figure 11-3. Probable settlements adjacent to open cuts.

injection stabilization (chap 16) may be used to underpin structures by forming a zone of hardened soil to support the foundation.

b. In carrying out an underpinning operation, important points to observe include the following:

(1) Pits opened under the building must be as small as possible, and survey monitoring of the building must be carried out in the areas of each pit to determine if damaging movements are occurring.

(2) Care must be taken to prevent significant lifting of local areas of the building during jacking.

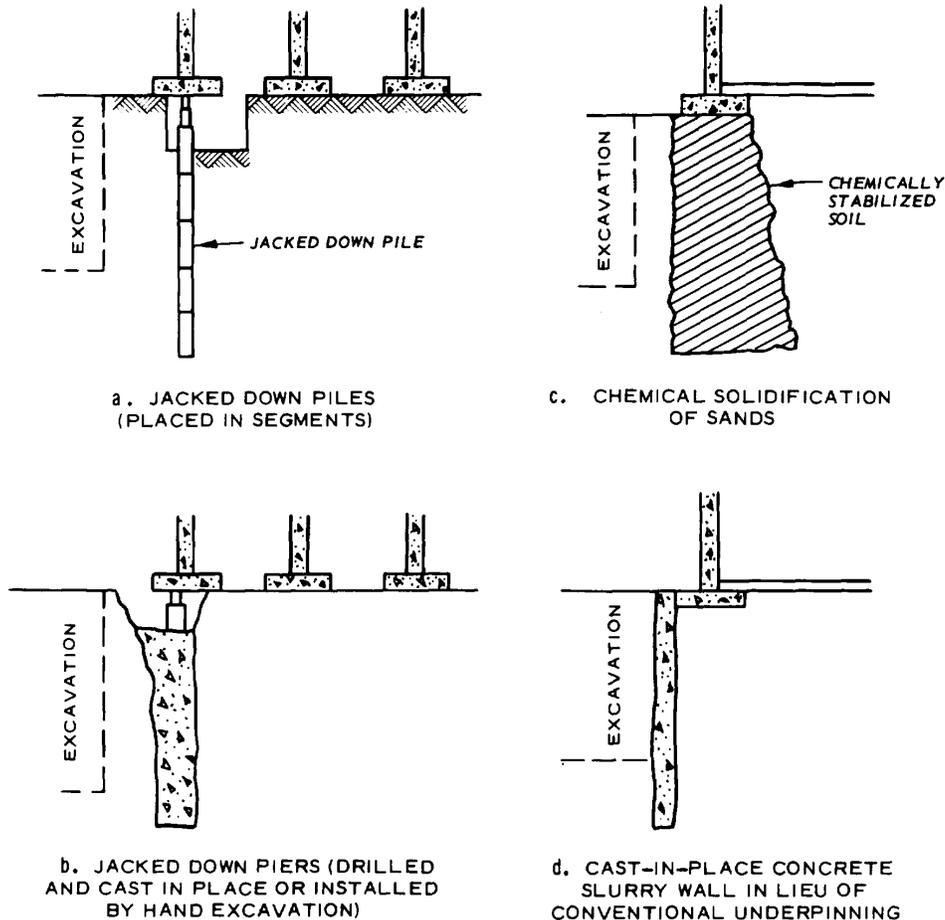
(3) Concrete in piers must be allowed to set before any loading is applied.

(4) Chemically stabilized sands must not be subject to creep under constant load.

c. The decision to underpin is a difficult one because it is hard to estimate how much settlement a

building can actually undergo before being damaged. The values given in tables 5-2 and 5-3 may be used as guidelines.

11-5. Excavation protection. During foundation construction, it is important that excavation subsoils be protected against deterioration as a result of exposure to the elements and heavy equipment. Difficulties can occur as a result of slaking, swelling, and piping of the excavation soils. Also, special classes of soils can collapse upon wetting (chap 3). Methods for protecting an excavation are described in detail in table 11-1 along with procedures for identifying problem soils. If these measures are not carried out, soils likely will be subject to a loss of integrity and subsequent foundation performance will be impaired.



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Figure 11-4. Methods of underpinning.

Table 11-1. Excavation Protection.

Soil Type	Identification	Problems and Mechanism	Preventative Measures
Overconsolidated clays near plastic limit	(1) Stiff plastic clays, natural water content heave at excavation (2) See figure 3-14 for swelling potential	(1) One dimensional heave, maximum around excavation center (2) Swelling dependent on plasticity (3) Usually fractured and fissured. Excavation opens these, causing softening and strength loss	(1) Rapid collection of surface water, or grading (2) Deep pressure relief to minimize rebound (3) Place 4" - 6" working mat of lean concrete immediately after exposing subgrade (mat may be placed over underseepage and pressure relief systems placed in sand blanket) (4) If sloped walls, use asphalt sealer on vertical walls, burlap with rubber sheeting or other membrane on flatter slopes
Chemically inert, uncemented claystone or shale	(1) Lab testing (2) Soil and geologic maps (3) Local experience	(1) Swelling, slaking and strength loss if water infiltration (2) Rebound if overconsolidated (generally the case) (3) Cracking if dry or evaporation allowed	(1) Protect from wetting and drying by limiting area open at subgrade (2) Concrete working mat (3) Paving, impervious materials to avoid water infiltration; place sealing coats immediately after exposure (4) Reduce evaporation and drying to prevent swelling on resaturation
(Continued)			

(Sheet 1 of 3)

Table 11-1. Excavation Protection-Continued

Soil Type	Identification	Problems and Mechanism	Preventative Measures
Limestone	(1) Geologic maps and local experience (2) Borings and soundings to determine cavity locations	Open cavities and caverns; soft infilling in joints, cavities. Dewatering can cause cavity collapse or settlements of infilling materials	(1) Avoid infiltration; collect surface runoff and convey it to a point where its infiltration will not affect excavation (2) Eliminate leaks from utility or industrial piping and provide for inspection to avoid infiltration continuing after correction (3) Avoid pumping which causes downward seepage or recharging
Collapsing soils (primarily loess and volcanic ash)	(1) See figure 3-16, Y_d vs W_L plot (2) $Y_d < 85$ PCF (3) Large open "structure" with temporary source of strength (4) Liquid limit $< 45\%$ plasticity index 0-25% (5) Natural water content well below 100% saturation (6) Optimum water content for collapse between 13-39%(3)	(1) When infiltrated with water, sudden decrease in bulk volume; may or may not require loading in conjunction with seepage (2) Water action reduces temporary source of strength, usually capillary tension or root structure Low erosion resistance	(1) Drainage and collection system to avoid water infiltration (2) Preload excavation area, and pond with water to cause collapse prior to excavation

(Continued)

(Sheet 2 of 3)

Table 11-1. Excavation Protection-Continued.

Soil Type	Identification	Problems and Mechanism	Preventative Measures
Soft, normally consolidated or sensitive clays	<ul style="list-style-type: none"> (1) Recently deposited, or no geologic loading and unloading (2) Leached marine clays (3) Natural water content near or above liquid limit 	<ul style="list-style-type: none"> (1) Primarily low strength, unable to support const. equipment (2) Remolded by construction activity, causing strength loss 	Provide support, and prevent remolding: place timber beams under heavy equipment; cover excavation bottom with 1' - 2' of sand and gravel fill and/or 6" - 8" of lean concrete

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11-6. Drilled piers. Drilled piers (also drilled shafts or drilled caissons) are often more economical than piles where equipment capable of rapid drilling is readily available, because of the large capacity of a pier as compared with a pile.

a. *Pier dimension and capacities.* Drilled piers can support large axial loads, up to 4,000 kips or more, although typical design loads are on the order of 600 to 1,000 kips. In addition, drilled piers are used under lightly loaded structures where subsoils might cause building heaving. Shaft diameters for high-capacity piers are available as follows:

From 2- 1/2 feet by 6-inch increments
 From 5 feet by 1-foot increments

Also available are 15- and 2-foot-diameter shafts. Commonly, the maximum diameter of drilled piers is under 10 feet with a 3- to 5-foot diameter very common. Drilled piers can be belled to a maximum bell size of three times the shaft diameter. The bells may be hemispherical or sloped. Drilled piers can be formed to a maximum depth of about 200 feet. Low capacity drilled piers may have shafts only 12 to 18 inches in diameter and may not be underreamed.

b. *Installation.* The drilled pier is constructed by drilling the hole to the desired depth, belling if increased bearing capacity or uplift resistance is required, placing necessary reinforcement, and filling the cavity with concrete as soon as possible after the hole is drilled. The quantity of concrete should be measured to ensure that the hole has been completely filled. Reinforcement may not be necessary for vertical loads; however, it will always be required if the pier carries lateral loads. A minimum number of dowels will be required for unreinforced piers to tie the superstructure to the pier. Reinforcement should be used only if necessary since it is a construction obstruction. Consideration should be given to an increased shaft diameter or higher strength concrete in lieu of reinforcement. In caving soils and depending on local experience, the shaft is advanced by:

(1) *Drilling a somewhat oversize hole and advancing the casing with shaft advance.* Casing may be used to prevent groundwater from entering the shaft. When drilling and underreaming is completed, the reinforcing steel is placed, and concrete is placed immediately. The casing may be left in place or withdrawn while simultaneously maintaining a head of concrete. If the casing is withdrawn, the potential exists for voids to be formed in the concrete, and special attention should be given to the volume of concrete poured.

(2) *Use of drilling mud to maintain the shaft cavity.* Drilling mud may be used also to prevent water from entering the shaft by maintaining a positive head differential in the shaft, since the drilling fluid has a

higher density than water. The reinforcing steel can be placed in the slurry-filled hole. Place concrete by tremie.

(3) *Use of drilling mud and casing.* The shaft is drilled using drilling mud, the casing is placed, and the drilling mud is bailed. Core barrels and other special drilling tools are available to socket the pier shaft into bedrock. With a good operator and a drill in good shape, it is possible to place 30- to 36-inch cores into solid rock at a rate of 2 to 3 feet per hour. Underreams are either hemispherical or 30- or 45-degree bell slopes. Underreaming is possible only in cohesive soils such that the underslope can stand without casing support, as no practical means currently exists to case the bell.

c. *Estimating the load capacity of a drilled pier.* Estimate the ultimate capacity, Q_u , of a drilled pier as follows:

$$Q_u = Q_{us(\text{skin resistance})} + Q_{up(\text{point})} \quad (11-2)$$

The design load based on an estimated 1-inch settlement is:

$$Q_d = Q_{us} + \frac{Q_{up}}{3} \quad (11-3)$$

(1) *Drilled piers in cohesive soil.* The skin resistance can be computed from the following:

$$Q_{us} = \alpha_{avg} \int_0^H C c_z dz$$

where

- α_{avg} = factor from table 11-2
- H = shaft length
- C = shaft circumference
- c_z = undrained shear strength at depth z

Use table 11-2 for the length of shaft to be considered in computing H and for limiting values of side shear. The base resistance can be computed from the following:

$$Q_{up} = N_c C_B A_B \quad (11-5)$$

where

- N_c = bearing capacity factor of 9 (table 11-2)
- C_B = undrained shear strength for distance of two diameters below tip
- A_B = base area

(2) *Drilled piers in sand.* Compute skin resistance from the following:

$$Q_{us} = \alpha_{avg} C \int_0^H P_z \tan \phi dz \quad (11-6)$$

where

- α_{avg} = 0.7, for shaft lengths less than 25 feet
- α_{avg} = 0.6, for shaft lengths between 25 and 40 feet
- α_{avg} = 0.5, for shaft lengths more than 40 feet

Table 11-2. Design Parameters for Drilled Piers in Clay

Design Category	Side Resistance		Tip Resistance N_c	Remarks
	a_{avg}	Limit on side shear - tsf		
A. Straight-sided shafts in either homogeneous or layered soil with no soil of exceptional stiffness below the base				
1. Shafts installed dry or by the slurry displacement method	0.6	2.0	9	
2. Shafts installed with drilling mud along some portion of the hole with possible mud entrapment	0.3 ^(a)	0.5 ^(a)	9	(a) a_{avg} may be increased to 0.6 and side shear increased to 2.0 tsf for segments drilled dry
B. Belled shafts in either homogeneous or layered clays with no soil of exceptional stiffness below the base				
1. Shafts installed dry or by the slurry displacement methods	0.3	0.5	9	
2. Shafts installed with drilling mud along some portion of the hole with possible mud entrapment	0.15 ^(b)	0.3 ^(b)	9	(b) a_{avg} may be increased to 0.3 and side shear increased to 0.5 tsf for segments drilled
C. Straight-sided shafts with basedry resting on soil significantly stiffer than soil around stem	0	0	9	
D. Belled shafts with base resting on soil significantly stiffer than soil around stem	0	0	9	

Note: In calculating load capacity, exclude (1) top 5 ft of drilled shaft, (2) periphery of bell, and (3) bottom 5 ft of straight shaft and bottom 5 ft of stem of shaft above bell.

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p_z = effective overburden pressure at depth

z

ϕ = effective angle of internal friction

Arching develops at the base of piers in sand similar to piers in clay; thus, the bottom 5 feet of shaft should not be included in the integration limits of the above equations. The base resistance for a settlement of about 1 inch can be computed from the following:

where

A_B = base area

B = base diameter

q_t = 0 for loose sand

q_t = 32,000 pounds per square foot for medium sand

q_t = 80,000 pounds per square foot for dense sand

$$Q_{up} = \frac{A_B}{0.6B} q_t = 1.31 B q_t \quad (11-7)$$