

Pile Foundations

10-1 INTRODUCTION

Chapter 9 covered shallow foundations. Sometimes, however, the soil upon which a structure is to be built is of such poor quality that a shallow foundation would be subject to bearing capacity failure and/or excessive settlement. In such cases, *pile foundations* may be used to support the structure (i.e., to transmit the load of the structure to firmer soil, or rock, at a greater depth below the structure).

A pile foundation is a relatively long and slender member that is forced or driven into the soil, or it may be poured in place. If a pile is driven until it rests on a hard, impenetrable layer of soil or rock, the load of the structure is transmitted primarily axially through the pile to the impenetrable layer. This type of pile is called an *end-bearing pile*. With end-bearing piles, care must be exercised to ensure that the hard, impenetrable layer is adequate to support the load. If a pile cannot be driven to a hard stratum of soil or rock (e.g., if such a stratum is located too far below the ground surface), the load of the structure must be borne primarily by skin friction or adhesion between the surface of the pile and adjacent soil. Such a pile is known as a *friction pile*.

In addition to simply supporting the load of a structure, piles may perform other functions, such as densifying loose cohesionless soils, resisting horizontal loads, anchoring structures subject to uplift, and so on. The emphasis in this book, however, is on piles that support the load of a structure.

10-2 TYPES OF PILES

Piles may be classified according to the types of materials from which they are made. Virtually all piles are made of timber, concrete, or steel (or a combination of these). Each of these is discussed in general terms in this section.

Timber piles have been used for centuries and are still widely used. They are made relatively easily by delimiting tall, straight tree trunks. They generally make economical pile foundations. Timber piles have certain disadvantages, however. They have less capacity to carry a load than do concrete or steel piles. Also, the length of a timber pile is limited by the height of the tree available. Timber pile length is generally limited to around 60 ft (18 m), although longer timber piles are available in some locales. Timber piles may be damaged in the pile-driving process. In addition, they are subject to decay and attack by insects. This generally is not a problem if the pile is both in soil and always below the water table; if above the water table, timber piles can be treated chemically to increase their life.

Concrete piles can be either *precast* or *cast-in-place*. Precast concrete piles may be manufactured with circular, square, octagonal, or other cross-sectional shapes. They can be made of uniform cross section (with a pointed tip), or they may be tapered. Precast piles can be made of prestressed concrete. The main disadvantages of precast concrete piles have to do with problems of manufacturing and handling of the piles (space needed, time required for curing, heavy equipment necessary for handling and transporting, etc.).

Cast-in-place concrete piles may be *cased* or *uncased*. The cased type can be made by driving a shell containing a core into the soil, removing the core, and filling the shell with concrete. The uncased type can be made in a similar manner, except that the shell is withdrawn as concrete is poured. Cast-in-place concrete piles have several advantages over concrete piles that are precast. One is that, because the concrete is poured in place, damage due to pile driving is eliminated. Also, the length of the pile is known at the time the concrete is poured. (With a precast pile, the exact length of the pile to be cast must be known initially. If a given pile turns out to be too long or too short, extra cost is involved in cutting off the extra length of the pile or adding to it.)

Concrete piles generally have a somewhat larger capacity to carry load than do timber piles. They are usually not very susceptible to deterioration, except possibly by seawater and strong chemicals.

Steel piles are commonly either pipe-shaped or H-sections. Pipe-shaped steel piles may be filled with concrete after being driven. H-shaped steel piles are strong and capable of being driven to great depths through stiff layers. Steel piles are subject to damage by corrosion. They generally have a somewhat larger capacity to carry load than do timber piles or concrete piles. Table 10–1 gives some properties of H-pile sections used in the United States.

Table 10–2 gives some customary design loads for different types of piles.

10–3 LENGTH OF PILES

In the case of end-bearing piles, the required pile length can be found fairly accurately because it is the distance from the structure being supported by the pile to the hard, impenetrable layer of soil or rock on which the pile rests. This distance is established from soil boring tests.

With friction piles, the required pile length is determined indirectly. Friction piles must be driven to such a depth that adequate lateral surface area of the pile is in contact with soil in order that sufficient skin friction or adhesion can be developed.

Table 10-3 gives available lengths of various types of piles.

10-4 PILE CAPACITY

The capacity of a single pile may be evaluated by the structural strength of the pile and by the supporting strength of the soil.

Pile Capacity as Evaluated by the Structural Strength of the Pile

Obviously, a pile must be strong enough structurally to carry the load imposed upon it. A pile's structural strength depends on its size and shape, as well as the type of material from which it is made.

Allowable structural strengths of different types of piles are specified by a number of building codes. Table 10-4 shows allowable stress in various types of pile, according to one code.

Pile Capacity as Evaluated by the Supporting Strength of the Soil

In addition to the strength of the pile itself, pile capacity is limited by the soil's supporting strength. As mentioned previously, the load carried by a pile is ultimately borne by either or both of two ways. The load is transmitted to the soil surrounding the pile by friction or adhesion between the soil and the sides of the pile, and/or the load is transmitted directly to the soil just below the pile's tip. This can be expressed in equation form as follows:

$$Q_{\text{ultimate}} = Q_{\text{friction}} + Q_{\text{tip}} \quad (10-1)$$

where Q_{ultimate} = ultimate (at failure) bearing capacity of a single pile

Q_{friction} = bearing capacity furnished by friction or adhesion between the soil and the sides of the pile

Q_{tip} = bearing capacity furnished by the soil just below the pile's tip

The term Q_{friction} in Eq. (10-1) can be evaluated by multiplying the unit skin friction or adhesion between the soil and the sides of the pile (f) by the pile's surface (skin) area (A_{surface}). The term Q_{tip} can be evaluated by multiplying the ultimate bearing capacity of the soil at the tip of the pile (q) by the area of the tip (A_{tip}). Hence, Eq. (10-1) can be expressed as follows:

$$Q_{\text{ultimate}} = f \cdot A_{\text{surface}} + q \cdot A_{\text{tip}} \quad (10-2)$$

In the case of end-bearing piles, the term Q_{tip} of Eq. (10-1) or $q \cdot A_{\text{tip}}$ of Eq. (10-2) will be predominant, whereas with friction piles, the term Q_{friction} of Eq. (10-1) or $f \cdot A_{\text{surface}}$ of Eq. (10-2) will be predominant.

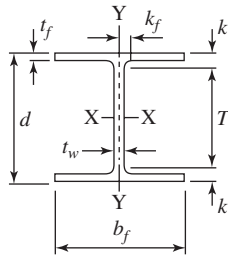
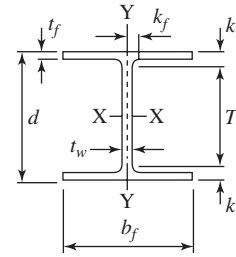


TABLE 10-1
HP-Shapes (Bearing
Piles) Dimensions
(American Institute of
Steel Construction, Inc.
(2001))

Shape	Area, A	Depth, d		Web			Flange		Distance					
				Thickness, t_w	$\frac{t_w}{2}$		Width, b_f	Thickness, t_f	k	k_1	T	Workable Gage		
	in. ²	in.		in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	
HP14 × 117	34.4	14.2	14 ¹ / ₄	0.805	¹³ / ₁₆	⁷ / ₁₆	14.9	14 ⁷ / ₈	0.805	¹³ / ₁₆	1 ¹ / ₂	1 ¹ / ₁₆	11 ¹ / ₄	5 ¹ / ₂
× 102	30.0	14.0	14	0.705	¹¹ / ₁₆	³ / ₈	14.8	14 ³ / ₄	0.705	¹¹ / ₁₆	1 ³ / ₈	1	↓	↓
× 89	26.1	13.8	13 ⁷ / ₈	0.615	⁵ / ₈	⁵ / ₁₆	14.7	14 ³ / ₄	0.615	⁵ / ₈	1 ³ / ₁₆	¹⁵ / ₁₆	↓	↓
× 73	21.4	13.6	13 ⁵ / ₈	0.505	¹ / ₂	¹ / ₄	14.6	14 ⁵ / ₈	0.505	¹ / ₂	1 ¹ / ₁₆	⁷ / ₈	↓	↓
HP12 × 84	24.6	12.3	12 ¹ / ₄	0.685	¹¹ / ₁₆	³ / ₈	12.3	12 ¹ / ₄	0.685	¹¹ / ₁₆	1 ³ / ₈	1	9 ¹ / ₂	5 ¹ / ₂
× 74	21.8	12.1	12 ¹ / ₈	0.605	⁵ / ₈	⁵ / ₁₆	12.2	12 ¹ / ₄	0.610	⁵ / ₈	1 ⁵ / ₁₆	¹⁵ / ₁₆	↓	↓
× 63	18.4	11.9	12	0.515	¹ / ₂	¹ / ₄	12.1	12 ¹ / ₈	0.515	¹ / ₂	1 ¹ / ₄	⁷ / ₈	↓	↓
× 53	15.5	11.8	11 ³ / ₄	0.435	⁷ / ₁₆	¹ / ₄	12.0	12	0.435	⁷ / ₁₆	1 ¹ / ₈	⁷ / ₈	↓	↓
HP10 × 57	16.8	9.99	10	0.565	⁹ / ₁₆	⁵ / ₁₆	10.2	10 ¹ / ₄	0.565	⁹ / ₁₆	1 ¹ / ₄	¹⁵ / ₁₆	7 ¹ / ₂	5 ¹ / ₂
× 42	12.4	9.7	9 ³ / ₄	0.415	⁷ / ₁₆	¹ / ₄	10.1	10 ¹ / ₈	0.420	⁷ / ₁₆	1 ¹ / ₈	¹³ / ₁₆	7 ¹ / ₂	5 ¹ / ₂
HP8 × 36	10.6	8.02	8	0.445	⁷ / ₁₆	¹ / ₄	8.15	8 ¹ / ₈	0.445	⁷ / ₁₆	1 ¹ / ₈	⁷ / ₈	5 ¹ / ₄	5 ¹ / ₂

Equations (10-1) and (10-2) are generalized and therefore applicable for all soils. The manner in which some of the terms of Eq. (10-2) are evaluated differs, however, depending on whether the pile is driven in sand or clay. It is convenient, therefore, to consider separately piles driven in sand and those driven in clay.

**TABLE 10-1 (Continued)
HP-Shapes (Bearing Piles)
Properties**



Nom- inal Wt. lb/ft	Compact Section Criteria			X_1 ksi	$X_2 \times 10^6$ (1/ksi) ²	Axis X-X				Axis Y-Y			
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	F_y'' ksi			I in. ⁴	S in. ³	r in.	Z in. ³	I in. ⁴	S in. ³	r in.	Z in. ³
117	9.25	14.2	-	3870	659	1220	172	5.96	194	443	59.5	3.59	91.4
102	10.5	16.2	-	3390	1090	1050	150	5.92	169	380	51.4	3.56	78.8
89	11.9	18.5	-	2960	1850	904	131	5.88	146	326	44.3	3.53	67.7
73	14.4	22.6	-	2450	3880	729	107	5.84	118	261	35.8	3.49	54.6
84	8.97	14.2	-	3860	670	650	106	5.14	120	213	34.6	2.94	53.2
74	10.0	16.1	-	3440	1050	569	93.8	5.11	105	186	30.4	2.92	46.6
63	11.8	18.9	-	2940	1940	472	79.1	5.06	88.3	153	25.3	2.88	38.7
53	13.8	22.3	-	2500	3640	393	66.7	5.03	74.0	127	21.1	2.86	32.2
57	9.05	13.9	-	3920	631	294	58.8	4.18	66.5	101	19.7	2.45	30.3
42	12.0	18.9	-	2930	1960	210	43.4	4.13	48.3	71.7	14.2	2.41	21.8
36	9.16	14.2	-	3840	685	119	29.8	3.36	33.6	40.3	9.88	1.95	15.2

Piles Driven in Sand. In the case of piles driven in sand, skin friction between the soil and the sides of the pile [$f \cdot A_{\text{surface}}$ in Eq. (10-2)] can be evaluated by multiplying the coefficient of friction between sand and pile surface ($\tan \delta$) by the total horizontal soil pressure acting on the pile. The coefficient of friction between sand and pile surface can be obtained from Table 10-5. The total

TABLE 10-2
Customary Design Loads for Piles (Terzaghi and Peck, 1967)

Type of Pile	Allowable Load (tons) ¹
Wood	15–30
Composite	20–30
Cast-in-place concrete	30–50
Precast reinforced concrete	30–50
Steel pipe, concrete-filled	40–60
Steel H-section	30–60

¹1 ton = 8.896 kN.

Source: K. Terzaghi and R. B. Peck, *Soil Mechanics in Engineering Practice*, 2nd ed. Copyright © 1967 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.

TABLE 10-3
Available Lengths of Various Pile Types

Pile Type	Comment, Available Maximum Length ¹
Timber	Depends on wood (tree) type. Lengths in the 50- to 60-ft range are usually available in most areas; lengths to about 75 ft are available but in limited quantity; lengths up to the 100-ft range are possible but very limited.
Steel H and pipe	Unlimited length; “short” sections are driven, and additional sections are field-welded to obtain a desired total length.
Steel shell, cast-in-place	Typically to between 100 and 125 ft, depending on shell type and manufacturer–contractor.
Precast concrete	Solid, small cross-section piles usually extend into the 50- to 60-ft length, depending on cross-sectional shape, dimensions, and manufacturer. Large-diameter cylinder piles can extend to about 200 ft long.
Drilled shaft, cast-in-place concrete	Usually in the 50- to 75-ft range, depending on contractor equipment.
Bulb-type, cast-in-place concrete	Up to about 100 ft.
Composite	Related to available lengths of material in the different sections. If steel and thin-shell cast-in-place concrete are used, the length can be unlimited; if timber and thin-shell cast-in-place concrete are used, lengths can be on the order of 150 ft.

¹1ft = 0.3048 m.

Source: D. F. McCarthy, *Essentials of Soil Mechanics and Foundations*, 4th ed., p. 463, fig. 14–24a, 1993. Reprinted by permission of Pearson Education, Upper Saddle River, NJ.

TABLE 10-4
Allowable Stress in Piles (North Carolina State Building Code, 1994)

(a) Allowable Unit Stresses for Treated Round Timber Piles, Normal Load Duration Values at Tip of Pile

<i>Species</i>	<i>Compression Parallel to Grain, psi⁴</i>	<i>Bending, psi⁴</i>	<i>Shear Horizontal, psi⁴</i>	<i>Compression Perpendicular to Grain, psi⁴</i>	<i>Modulus of Elasticity, psi⁴</i>
Pacific Coast					
Douglas Fir ¹	1250	2450	115	230	1,500,000
Southern Pine ^{1,2}	1200	2400	110	250	1,500,000
Red Oak ³	1200	2450	135	350	1,250,000
Red Pine ⁵	900	1900	85	155	1,280,000

(b) Steel Piles

The design load shall not cause a stress in the steel greater than 12,600 lb/in.² and a stress in any concrete used to fill piles, driven either open or closed end, greater than 25% of its ultimate 28-day compressive strength.

(c) Concrete Piles

Cast-in-place piles: The stress in concrete shall not exceed 25% of the ultimate 28-day strength of the concrete.

Prestressed concrete piles: The maximum allowable compressible stress in precast piles due to an externally applied load shall not exceed

$$f_c = 0.33f'_c - 0.27f_{pe}$$

where f'_c is the 28-day compression strength of concrete, and f_{pe} is the effective prestress stress on the gross section.

1psi = 6.8948 kPa

NOTES:

¹ The allowable unit stresses in compression parallel to grain for Pacific Coast, Douglas Fir, and Southern Pine may be increased 0.2% for each foot of length from the tip of the pile to the critical section. The increase shall not exceed 10% for any pile. The stress increase is cumulative with increase in section properties due to pile taper.

² Southern Pine values apply to Longleaf, Slash, Loblolly, and Shortleaf Pines.

³ Red Oak values apply to Northern and Southern Red Oak.

⁴ The working stresses in the above table have been adjusted to compensate for strength reductions due to conditioning prior to treatment. Where piles are air dried or kiln dried prior to pressure treatment, or where untreated piles are to be used, the above working stresses shall be increased by multiplying the tabulated values by the following factors:

- Pacific Coast Douglas Fir, Red Oak, Red Pine..... 1.11
- Southern Pine..... 1.18

⁵ Red Pine values apply to Red Pine grown in the United States.

Source: North Carolina State Building Code, Vol. 1, General Construction, 1978 ed.

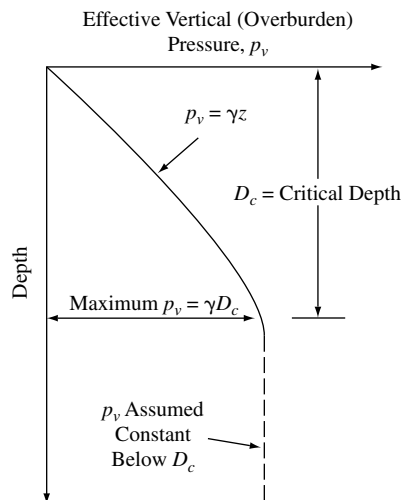
TABLE 10-5
Coefficient of Friction between Sand and Pile
Materials

Material	Tan δ
Concrete	0.45
Wood	0.4
Steel (smooth)	0.2
Steel (rough, rusted)	0.4
Steel (corrugated)	Use tan ϕ of sand

Source: D. F. McCarthy, *Essentials of Soil Mechanics and Foundations*, 4th ed., p. 512, fig. 15-14, 1993. Reprinted by permission of Pearson Education, Upper Saddle River, NJ.

horizontal soil pressure acting on the pile is a function of effective vertical (overburden) pressure of soil adjacent to the pile. Soil pressure normally increases as depth increases. In the special case of piles driven in sand, however, it has been determined that the effective vertical (overburden) pressure of soil adjacent to a pile does not increase without limit as depth increases. Instead, effective vertical pressure increases as depth increases until a certain depth of penetration is reached. Below this depth, which is called the *critical depth* and denoted D_c , effective vertical pressure remains more or less constant. The critical depth is dependent on the field condition of the sand and the pile's size. Tests indicate that critical depth ranges from about 10 pile diameters for loose sand to about 20 pile diameters for dense compact sand (McCarthy, 1977). Thus, effective vertical pressure of soil adjacent to a pile varies with depth as illustrated in Figure 10-1.

FIGURE 10-1 Variation of effective vertical (overburden) pressure of soil adjacent to a pile with depth (Meyerhof, G.G., 1976).



The term $f \cdot A_{\text{surface}}$ of Eq. (10-2) can now be determined for a pile by multiplying the pile's circumference by the area under the p_v versus depth curve (Figure 10-1) by the coefficient of lateral earth pressure (K) by the coefficient of friction between sand and pile surface ($\tan \delta$). The coefficient of lateral earth pressure is assumed to vary between 0.60 and 1.25, with lower values used for silty sands and higher values for other deposits (Bowles, 1977).

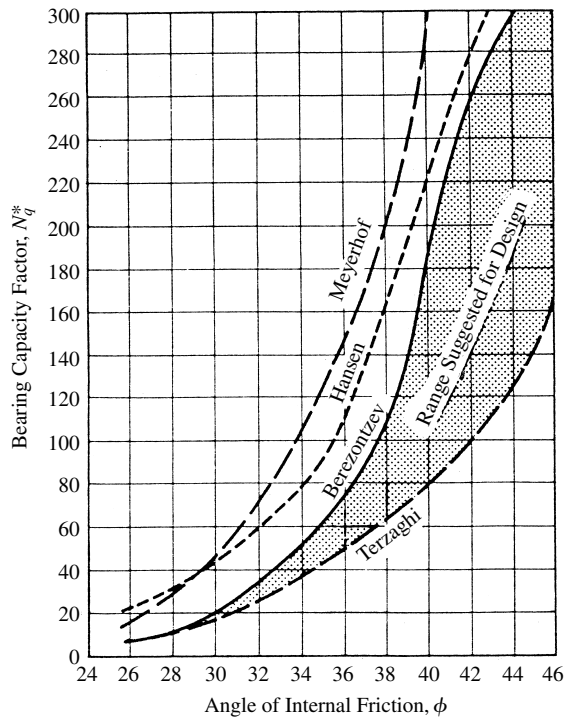
The bearing capacity at the pile tip [q in Eq. (10-2)] can be calculated by using the following equation:

$$q_{\text{tip}} = p_v N_q^* \tag{10-3}$$

where p_v = effective vertical pressure adjacent to pile's tip
 N_q^* = bearing capacity factor (see Figure 10-2)

The value of N_q^* is related to the angle of internal friction (ϕ) of the sand, and it should, of course, be based on the value of the angle of internal friction of the sand located in the general vicinity of where the pile tip will ultimately rest. The angle of internal friction of the sand at this location can be determined by laboratory tests on a sample taken from the specified location or by correlation with penetration resistance tests in a boring hole [i.e., corrected standard penetration

FIGURE 10-2 Bearing capacity factor, N_q^* , for piles penetrating into sand.
 Source: D. F. McCarthy, *Essentials of Soil Mechanics and Foundations*, 4th ed., 1993. Reprinted by permission of Pearson Education, Upper Saddle River, NJ.



test (SPT) N -value] (see Figure 9-9). Values of N_q^* can then be obtained from Figure 10-2.

To summarize the method described in this section for computing pile capacity for piles driven in sand, Eq. (10-2) is used, with the term $f \cdot A_{\text{surface}}$ evaluated by multiplying the pile's circumference by the area under the p_v versus depth curve (Figure 10-1) by the coefficient of lateral earth pressure (K) by the coefficient of friction between sand and pile surface ($\tan \delta$) and the term $q \cdot A_{\text{tip}}$ evaluated by multiplying the value of q_{tip} obtained from Eq. (10-3) by the area of the pile tip. Pile capacity thus determined represents the ultimate load that can be applied to the pile. In practice, it is common to apply a factor of safety of 2 to determine the (downward) design load for the pile (McCarthy, 2002).

Examples 10-1 and 10-2 illustrate the procedure for calculating pile capacity for piles driven in sand.

EXAMPLE 10-1

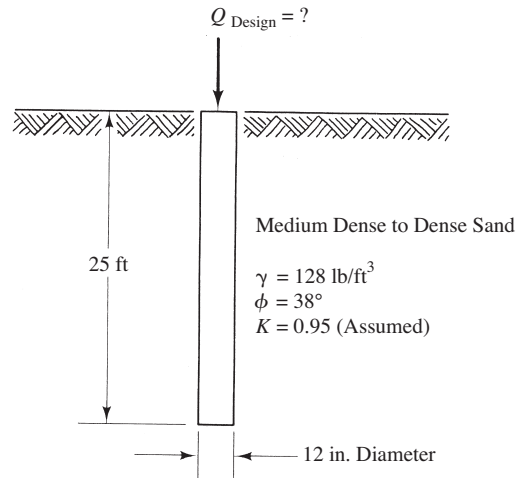
Given

1. A concrete pile is to be driven into a medium dense to dense sand.
2. The pile's diameter is 12 in., and its embedded length is 25 ft.
3. Soil conditions are shown in Figure 10-3.
4. No groundwater was encountered, and the groundwater table is not expected to rise during the life of the structure.

Required

The pile's axial capacity if the coefficient of lateral earth pressure (K) is assumed to be 0.95, and the factor of safety (F.S.) is 2.

FIGURE 10-3



Solution

For dense sand,

$$D_c = 20 \times \text{Pile's diameter} = 20 \times 1 \text{ ft} = 20 \text{ ft} \quad (\text{see Figure 10-4})$$

From Eq. (10-2),

$$Q_{\text{ultimate}} = f \cdot A_{\text{surface}} + q \cdot A_{\text{tip}} \quad (10-2)$$

$$f \cdot A_{\text{surface}} = (\text{Circumference of pile})(\text{Area of } p_v \text{ diagram})(K)(\tan \delta)$$

$$\text{Circumference of pile} = \pi d = (\pi)(1 \text{ ft}) = 3.14 \text{ ft}$$

$$\text{Area of } p_v \text{ diagram} = \left(\frac{1}{2}\right)(2560 \text{ lb/ft}^2)(20 \text{ ft}) + (2560 \text{ lb/ft}^2)(25 \text{ ft} - 20 \text{ ft})$$

$$= 38,400 \text{ lb/ft}$$

$$K = 0.95 \quad (\text{given})$$

$$\tan \delta = 0.45 \quad (\text{see Table 10-5 for concrete pile})$$

$$f \cdot A_{\text{surface}} = (3.14 \text{ ft})(38,400 \text{ lb/ft})(0.95)(0.45)$$

$$= 51,500 \text{ lb} = 51.5 \text{ kips}$$

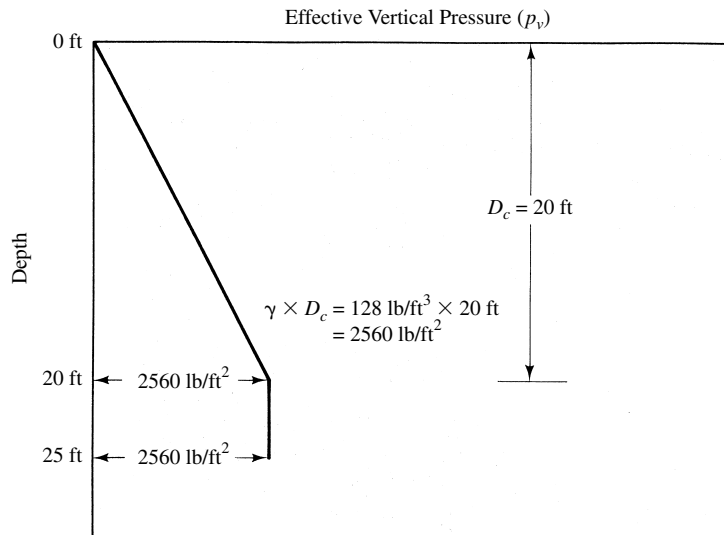
From Eq. (10-3),

$$q_{\text{tip}} = p_v N_q^* \quad (10-3)$$

$$p_v = 2560 \text{ lb/ft}^2 \quad (\text{see Figure 10-4})$$

$$N_q^* = 80 \quad (\text{from Figure 10-2 for } \phi = 38^\circ, \text{ using the mid-area of the "Range suggested for design"})$$

FIGURE 10-4



$$q_{\text{tip}} = (2560 \text{ lb/ft}^2)(80) = 204,800 \text{ lb/ft}^2$$

$$A_{\text{tip}} = \frac{\pi d^2}{4} = \left(\frac{\pi}{4}\right)(1 \text{ ft})^2 = 0.785 \text{ ft}^2$$

$$q \cdot A_{\text{tip}} = (204,800 \text{ lb/ft}^2)(0.785 \text{ ft}^2) = 160,800 \text{ lb} = 160.8 \text{ kips}$$

$$Q_{\text{ultimate}} = 51.5 \text{ kips} + 160.8 \text{ kips} = 212.3 \text{ kips}$$

$$Q_{\text{design}} = \frac{Q_{\text{ultimate}}}{\text{F. S.}} = \frac{212.3 \text{ kips}}{2} = 106.2 \text{ kips}$$

EXAMPLE 10-2*Given*

The same conditions as in Example 10-1, except that groundwater is located 10 ft below the ground surface (see Figure 10-5).

Required

The pile's axial capacity if K is 0.95, and a factor of safety of 2 is used.

Solution

$$D_c = 20 \times 1 \text{ ft} = 20 \text{ ft} \quad (\text{see Figure 10-6})$$

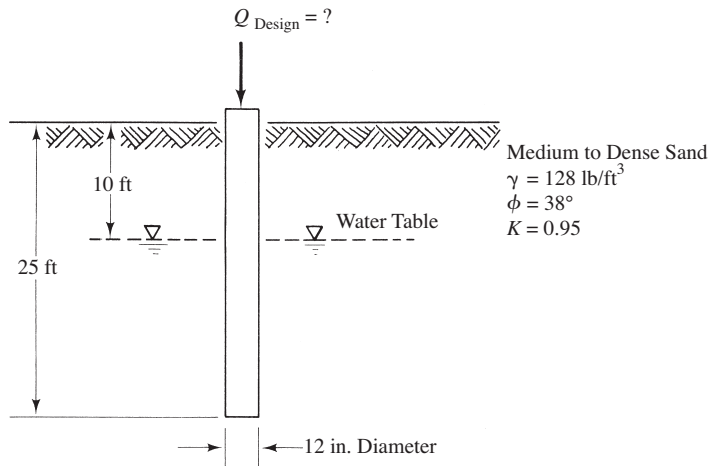
$$f \cdot A_{\text{surface}} = (\text{Circumference of pile})(\text{Area of } p_v \text{ diagram})(K)(\tan \delta)$$

$$\text{Circumference of pile} = \pi d = (\pi)(1 \text{ ft}) = 3.14 \text{ ft}$$

$$\begin{aligned} \text{Area of } p_v \text{ diagram} &= \left(\frac{1}{2}\right)(1280 \text{ lb/ft}^2)(10 \text{ ft}) + \left(\frac{1}{2}\right)(1280 \text{ lb/ft}^2 \\ &\quad + 1936 \text{ lb/ft}^2)(10 \text{ ft}) + (1936 \text{ lb/ft}^2)(5 \text{ ft}) \\ &= 32,200 \text{ lb/ft} \end{aligned}$$

$$K = 0.95$$

$$\tan \delta = 0.45$$

FIGURE 10-5

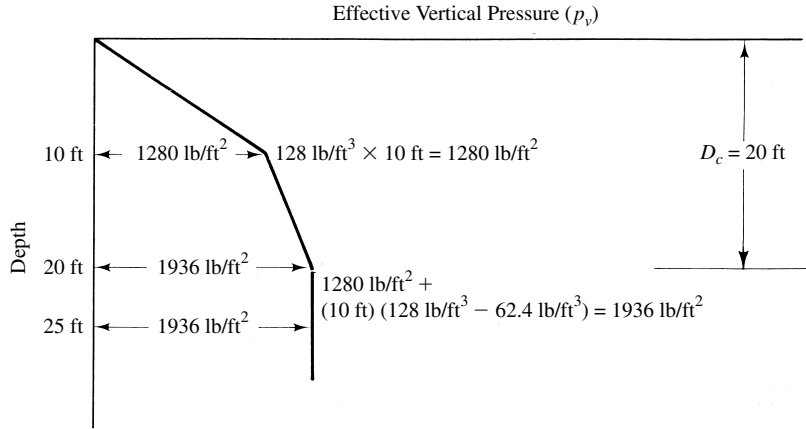


FIGURE 10-6

$$\begin{aligned}
 f \cdot A_{\text{surface}} &= (3.14 \text{ ft})(32,200 \text{ lb/ft})(0.95)(0.45) = 43,200 \text{ lb} = 43.2 \text{ kips} \\
 q_{\text{tip}} &= p_v N_q^* && (10-3) \\
 N_q^* &= 80 \\
 q_{\text{tip}} &= (1936 \text{ lb/ft}^2)(80) = 154,900 \text{ lb/ft}^2 \\
 A_{\text{tip}} &= 0.785 \text{ ft}^2 \\
 q \cdot A_{\text{tip}} &= (154,900 \text{ lb/ft}^2)(0.785 \text{ ft}^2) = 121,600 \text{ lb} = 121.6 \text{ kips} \\
 Q_{\text{ultimate}} &= 43.2 \text{ kips} + 121.6 \text{ kips} = 164.8 \text{ kips} \\
 Q_{\text{design}} &= \frac{164.8 \text{ kips}}{2} = 82.4 \text{ kips}
 \end{aligned}$$

Dennis and Olson (1983) studied the results of a number of load tests on piles carried to ultimate failure in sand. In their statistical analyses, they subdivided the data on the basis of the description of the sands and types of piles; established sets of values for the coefficient of lateral pressure (K), friction angle between sand and pile surface (δ), and bearing capacity factor (N_q); and set upper limiting values for skin friction and end-point resistance. They then substituted various values into the load equation [Eq. (10-2)] until they found the combinations that gave the best answers corresponding to the respective results of actual load tests. In other words, they found the combinations that gave mean ratios of computed bearing capacities to measured load capacities nearest to 1.0 for all tests and produced the least scatter.

Taking K equal to 0.8 and assigning reasonable values to δ and N_q in accordance with the standard penetration resistance values, Olson (1990) developed a table of soil properties for use in Eq. (10-2) that gives mean ratios of computed bearing capacities to measured load capacities nearest to 1.0 for all tests and

TABLE 10–6
Soil Properties Used in Olson's Final Analyses¹

Soil Type	Range in N-Values	δ (deg)	f_{lim} (kN/m ²)	N_q	q_{lim} (MN/m ²)
Gravel	0–4	(20)	(70)	(12)	(3)
	5–10	(25)	(85)	(20)	(5)
	11–30	(30)	(100)	(40)	(10)
	over 30	(35)	(120)	(60)	(12.5)
Sand/gravel	0–4	(20)	(70)	(12)	(3)
	5–10	(25)	(85)	(20)	(5)
	11–30	(30)	(100)	(40)	(10)
	over 30	(35)	(120)	(60)	(12.5)
Sand	0–4	(20)	(50)	(50)	(2)
	5–10	30	55	120	6
	11–30	35	95	120	9.5
	31–50	40	130	120	9.5
	51–100	40	165	130	10
	over 100	40	190	220	26.5
Sand/silt	0–4	10	(50)	(10)	(0.5)
	5–10	10	(50)	(20)	(2)
	11–30	15	(70)	50	5.5
	31–50	20	100	100	8
	51–100	(30)	(100)	(100)	(10)
Silt	over 100	(34)	(1000)	(100)	(10)
	0–4	(10)	(50)	(10)	(2)
	5–10	15	(50)	(10)	(2)
	11–30	20	(70)	(10)	(2)
	31–50	20	(70)	(12)	(3)
	over 50	(25)	(70)	(12)	(3)

¹Numbers in parentheses were not used in the analyses.

Source: K. Terzaghi, R. B. Peck, and G. Mesri, *Soil Mechanics in Engineering Practice*, 3rd ed., John Wiley & Sons, Inc., New York, 1996. Copyright © 1996, by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.

produces the least scatter (see Table 10–6 and Figure 10–7). Such a semiempirical approach, with load-test data correlating to a static equation such as Eq. (10–2), seems to be a logical way to improve practice. However, a large number of good-quality load-test data will be needed for further statistical studies (Terzaghi et al., 1996).*

If detailed information for computing pile bearing capacity is unavailable, rough estimates of unit skin friction (f_s) can be obtained from Figure 10–8 as a function of

*From K. Terzaghi, R. B. Peck, and G. Mesri, *Soil Mechanics in Engineering Practice*, 3rd ed., John Wiley & Sons, Inc., New York, 1996. Copyright © 1996, by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.

FIGURE 10-7 Comparison of measured and calculated axial load capacities of driven steel piles [after Olson (1990)].
 Source: From K. Terzaghi, R. B. Peck, and G. Mesri, *Soil Mechanics in Engineering Practice*, 3rd ed., John Wiley & Sons, Inc., New York, 1996. Copyright © 1996, by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.

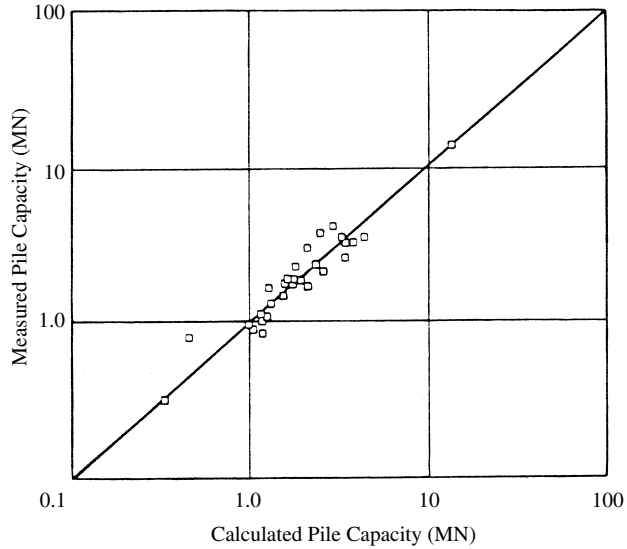


FIGURE 10-8 Ultimate unit side resistance, f_s , versus D/B —compression/tension ($1 \text{ tsf} = 95.76 \text{ kN/m}^2$).
 Source: H. M. Coyle and R. R. Castello, "New Design Correlations for Piles in Sand," *J. Geotech. Eng. Div. ASCE* 107 (GT7) 965-986 (1981). Reprinted by permission.

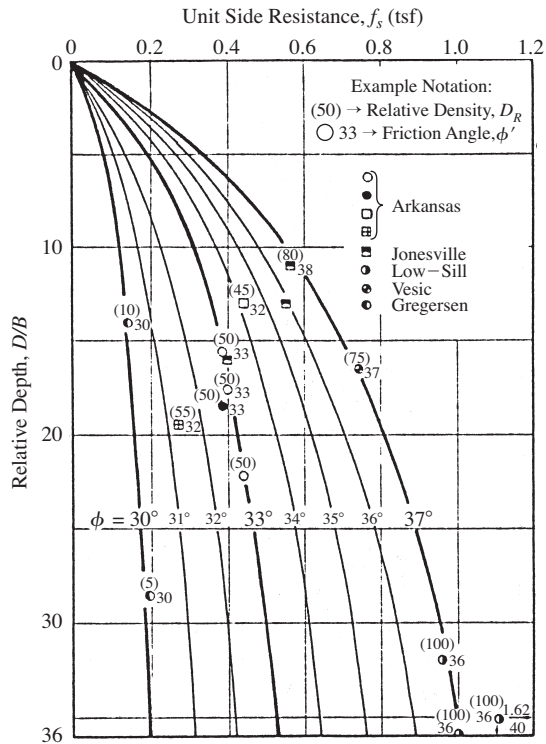
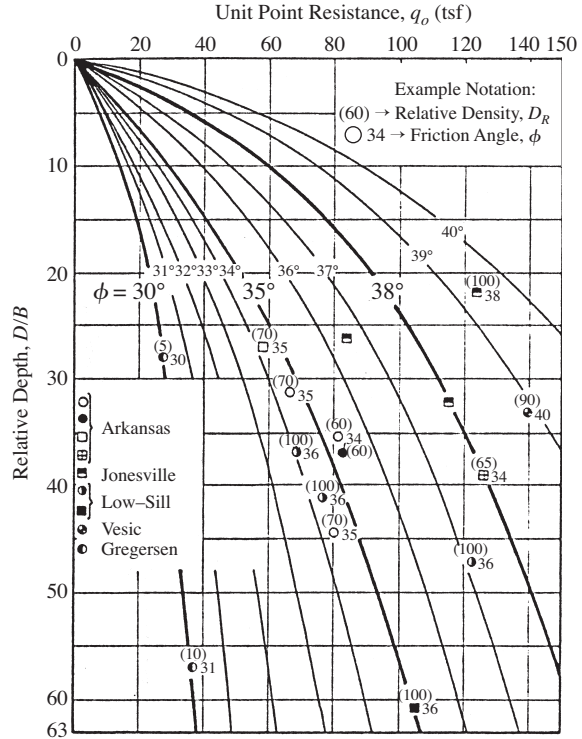


FIGURE 10-9 Ultimate unit point resistance, q , versus D/B —compression/tension ($1 \text{ tsf} = 95.76 \text{ kN/m}^2$).
 Source: H. M. Coyle and R. R. Castello, "New Design Correlations for Piles in Sand," *J. Geotech. Eng. Div. ASCE* 107 (GT7) 965-986 (1981). Reprinted by permission.



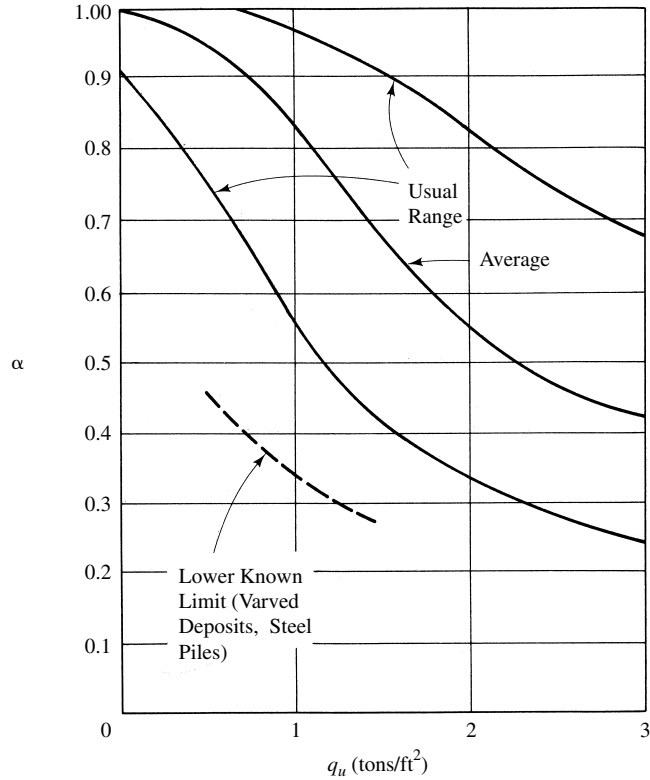
depth in pile diameters. This is an empirical relationship based on the fact that the skin friction developed between the sand and the pile is strongly influenced by the condition of the sand around the pile.

In a similar vein, Figure 10-9 gives unit tip resistance (q) as a function of depth in pile diameters. This is also an empirical relationship based on a number of pile load tests.

Piles Driven in Clay. Equation (10-2) also applies for piles driven in clay. Unit adhesion between the soil and the sides of the pile [f in Eq. (10-2)] can be evaluated by multiplying the cohesion of the clay (c) by the adhesion factor (α). The adhesion factor can be determined by using Figure 10-10. The term $f \cdot A_{\text{surface}}$ of Eq. (10-2) can thus be evaluated by multiplying the (undisturbed) cohesion of the clay (c) by the adhesion factor (α) by the surface (skin) area of the pile (A_{surface}). [That is, $f \cdot A_{\text{surface}} = \alpha c A_{\text{surface}}$.]

With soft clays, there is a tendency for the clay to come in close contact with the pile, in which case adhesion is assumed to be equal to cohesion (meaning $\alpha = 1.0$). In the case of stiff clays, pile driving disturbs surrounding soil and may cause a small open space to develop between the clay and the pile. Thus, adhesion is smaller than cohesion (meaning $\alpha < 1.0$). See Figure 10-10.

FIGURE 10-10 Relationship between adhesion factor, α , and unconfined compressive strength, q_u (1 ton/ft² = 95.76 kN/m²).
 Source: R. B. Peck, W. E. Hansen, and T. H. Thornburn, *Foundation Engineering*, 2nd ed., John Wiley & Sons, Inc., New York, 1974.
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The bearing capacity [q in Eq. (10-2)] at the pile tip can be calculated by using the following equation (McCarthy, 2002):

$$q_{\text{tip}} = cN_c \tag{10-4}$$

where q_{tip} = bearing capacity at pile tip

c = cohesion of the clay located in the general vicinity of where the pile tip will ultimately rest

N_c = bearing capacity factor and has a value of about 9 (McCarthy, 2002)

Thus, the term $q \cdot A_{\text{tip}}$ of Eq. (10-2) can be evaluated by multiplying the value of q_{tip} from Eq. (10-4) by the area of the pile tip. [That is, $q \cdot A_{\text{tip}} = cN_c A_{\text{tip}}$.]

To summarize the method described in this section for computing pile capacity for piles driven in clay, Eq. (10-2) is used, with the term $f \cdot A_{\text{surface}}$ evaluated by multiplying the cohesion of the clay (c) by the adhesion factor (α) by the surface (skin) area of the pile and the term $q \cdot A_{\text{tip}}$ evaluated by multiplying the value of q_{tip} obtained from Eq. (10-4) by the area of the pile tip. Hence, for piles driven in clay,

$$Q_{\text{ultimate}} = \alpha c A_{\text{surface}} + cN_c A_{\text{tip}} \tag{10-5}$$

Pile capacity thus determined represents the ultimate load that can be applied to the pile. In practice, it is common to apply a factor of safety of 2 to determine the (downward) design load for a pile (McCarthy, 2002).

Examples 10-3 through 10-5 illustrate the procedure for calculating pile capacity for piles driven in clay.

EXAMPLE 10-3

Given

1. A 12-in.-diameter concrete pile is driven at a site as shown in Figure 10-11.
2. The embedded length of the pile is 35 ft.

Required

Design capacity of the pile, using a factor of safety of 2.

Solution

From Eq. (10-5),

$$Q_{\text{ultimate}} = \alpha c A_{\text{surface}} + c N_c A_{\text{tip}} \quad (10-5)$$

$$q_u = 1400 \text{ lb/ft}^2 = 0.7 \text{ ton/ft}^2$$

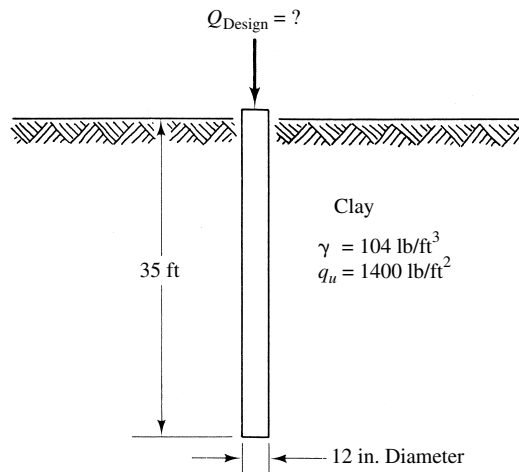
$$\alpha = 0.9 \quad (\text{see Figure 10-10 with } q_u = 0.7 \text{ ton/ft}^2)$$

$$c = \frac{q_u}{2} = \frac{1400 \text{ lb/ft}^2}{2} = 700 \text{ lb/ft}^2$$

$$A_{\text{surface}} = (\pi d)(L) = (\pi)(1 \text{ ft})(35 \text{ ft}) = 110 \text{ ft}^2$$

$$A_{\text{tip}} = \frac{\pi d^2}{4} = \frac{\pi}{4} (1 \text{ ft})^2 = 0.785 \text{ ft}^2$$

FIGURE 10-11



$$\begin{aligned}
 Q_{\text{ultimate}} &= (0.9)(700 \text{ lb/ft}^2)(110 \text{ ft}^2) + (700 \text{ lb/ft}^2)(9)(0.785 \text{ ft}^2) = 74,200 \text{ lb} \\
 &= 74.2 \text{ kips} \\
 Q_{\text{design}} &= \frac{74.2 \text{ kips}}{2} = 37.1 \text{ kips}
 \end{aligned}$$

EXAMPLE 10-4

Given

A 12-in.-diameter concrete pile is driven at a site as shown in Figure 10-12.

Required

Design capacity of the pile, using a factor of safety of 2.

Solution

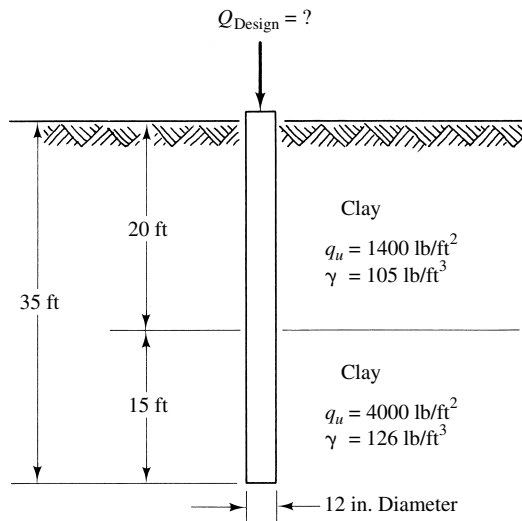
From Eq. (10-5),

$$\begin{aligned}
 Q_{\text{ultimate}} &= \alpha c A_{\text{surface}} + c N_c A_{\text{tip}} & (10-5) \\
 \alpha c A_{\text{surface}} &= \alpha_1 c_1 A_{\text{surface}_1} + \alpha_2 c_2 A_{\text{surface}_2}
 \end{aligned}$$

From Figure 10-10, with $q_{u_1} = 1400 \text{ lb/ft}^2 = 0.7 \text{ ton/ft}^2$, $\alpha_1 = 0.9$.

$$\begin{aligned}
 c_1 &= \frac{q_{u_1}}{2} = \frac{1400 \text{ lb/ft}^2}{2} = 700 \text{ lb/ft}^2 \\
 A_{\text{surface}_1} &= (\pi d)(L_1) = (\pi)(1 \text{ ft})(20 \text{ ft}) = 62.8 \text{ ft}^2
 \end{aligned}$$

FIGURE 10-12



From Figure 10–10, with $q_{u_2} = 4000 \text{ lb/ft}^2 = 2.0 \text{ tons/ft}^2$, $\alpha_2 = 0.56$.

$$c_2 = \frac{q_{u_2}}{2} = \frac{4000 \text{ lb/ft}^2}{2} = 2000 \text{ lb/ft}^2$$

$$A_{\text{surface}_2} = (\pi d)(L_2) = (\pi)(1 \text{ ft})(15 \text{ ft}) = 47.1 \text{ ft}^2$$

$$\begin{aligned} \alpha c A_{\text{surface}} &= (0.9)(700 \text{ lb/ft}^2)(62.8 \text{ ft}^2) + (0.56)(2000 \text{ lb/ft}^2)(47.1 \text{ ft}^2) = 92,300 \text{ lb} \\ &= 92.3 \text{ kips} \end{aligned}$$

$$q_{\text{tip}} = cN_c \quad (10-4)$$

$$q_{\text{tip}} = (2000 \text{ lb/ft}^2)(9) = 18,000 \text{ lb/ft}^2$$

$$A_{\text{tip}} = \frac{\pi}{4} d^2 = \frac{\pi}{4} (1 \text{ ft})^2 = 0.785 \text{ ft}^2$$

$$cN_c A_{\text{tip}} = (2000 \text{ lb/ft}^2)(9)(0.785 \text{ ft}^2) = 14,100 \text{ lb} = 14.1 \text{ kips}$$

$$Q_{\text{ultimate}} = 92.3 \text{ kips} + 14.1 \text{ kips} = 106.4 \text{ kips}$$

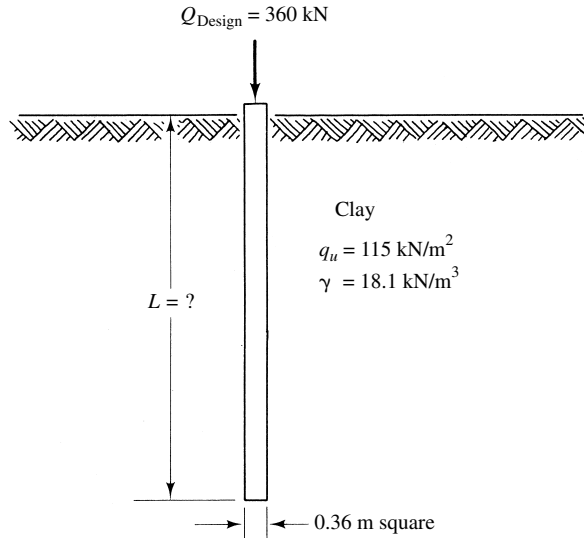
$$Q_{\text{design}} = \frac{106.4 \text{ kips}}{2} = 53.2 \text{ kips}$$

EXAMPLE 10–5

Given

1. A 0.36-m square prestressed concrete pile is to be driven in a clayey soil (see Figure 10–13).
2. The design capacity of the pile is 360 kN.

FIGURE 10–13



Required

The necessary length of the pile if the factor of safety is 2.

Solution

$$\begin{aligned}
 Q_{\text{design}} &= 360 \text{ kN} \\
 Q_{\text{ultimate}} &= \text{F.S.} \times Q_{\text{design}} = (2)(360 \text{ kN}) = 720 \text{ kN} \\
 c &= \frac{115 \text{ kN/m}^2}{2} = 57.5 \text{ kN/m}^2 \\
 q_{\text{tip}} &= cN_c = (57.5 \text{ kN/m}^2)(9) = 518 \text{ kN/m}^2 \\
 Q_{\text{tip}} &= (518 \text{ kN/m}^2)(0.36 \text{ m})(0.36 \text{ m}) = 67.1 \text{ kN}
 \end{aligned}$$

From Eq. (10-1),

$$\begin{aligned}
 Q_{\text{ultimate}} &= Q_{\text{friction}} + Q_{\text{tip}} & (10-1) \\
 Q_{\text{friction}} &= Q_{\text{ultimate}} - Q_{\text{tip}} \\
 Q_{\text{friction}} &= 720 \text{ kN} - 67.1 \text{ kN} = 652.9 \text{ kN} \\
 Q_{\text{friction}} &= \alpha c A_{\text{surface}}
 \end{aligned}$$

From Figure 10-10, with $q_u = 115 \text{ kN/m}^2$,

$$\alpha = 0.76$$

$$\begin{aligned}
 652.9 \text{ kN} &= (0.76)(57.5 \text{ kN/m}^2)(4 \times 0.36 \text{ m})(L) \\
 L &= 10.4 \text{ m}
 \end{aligned}$$

The required length of the 0.36-m square pile is 10.4 m.

Soft clays adjacent to piles may lose a large portion of their strength as a result of being disturbed by pile driving. Propitiously, the disturbed clay gains strength after driving stops. The original clay's full strength is usually regained within a month or so after pile driving has terminated. Ordinarily, this is not a problem because piles are not usually loaded immediately after driving; thus, the clay has time to regain its original strength prior to being loaded. In cases where piles are to be loaded immediately after driving, however, the effect of decreased strength must be taken into account by performing laboratory tests to determine the extent of strength reduction and rate of strength recovery.

Slender piles driven in soft clay have a tendency to buckle when loaded. The ultimate load for buckling of slender steel piles in soft clay can be estimated by using the following equation (U.S. Department of the Navy, 1971):

$$Q_{\text{ult}} = \lambda \sqrt{cEI} \quad (10-6)$$

where Q_{ult} = ultimate bearing capacity of a single slender pile for buckling in soft clay
 $\lambda = 8$ for very soft clay; 10 for soft clay
 c = cohesion of the soil
 E = modulus of elasticity of the steel
 I = moment of inertia of the cross section of the pile

Heavy steel, timber, and concrete piles do not tend to buckle if embedded in the soil for their entire lengths.

10-5 PILE-DRIVING FORMULAS

In theory, it seems possible to calculate pile capacity based on the amount of energy delivered to a pile by the hammer and resulting penetration of the pile. Intuitively, the greater the resistance required to drive a pile, the greater will be the capacity of the pile to carry load. Hence, many attempts have been made to develop *pile-driving formulas* by equating energy delivered by the hammer to work done by the pile as it penetrates a certain distance against a certain resistance, with an allowance made for energy losses.

Generally, no pile-driving formula has been developed that gives accurate results for pile capacity. Soil resistance does not remain constant during and after the pile-driving operation. In addition, pile-driving formulas give varying results. Although pile-driving formulas are not generally used to determine pile capacity, they may be used to determine when to stop driving a pile so that its bearing capacity will be the same as that of a test pile or of other piles driven in the same subsoil. To accomplish this, one should drive piles until the number of blows required to drive the last inch is the same as that of the test piles that furnished the information for evaluating the design load. However, piles driven in soft silt or clay should all be driven to the same depth rather than driven a certain number of blows (Terzaghi and Peck, 1967). Penetration resistance can also be used to prevent pile damage due to overdriving.

One simple and widely used pile-driving formula is known as the *Engineering-News formula*. It is given as follows (Karol, 1960):

$$Q_a = \frac{2W_r H}{S + C} \quad (10-7)$$

where Q_a = allowable pile capacity, lb
 W_r = weight of ram, lb
 H = height of fall of ram, ft
 S = amount of pile penetration per blow, in./blow
 $C = 1.0$ for drop hammer
 $C = 0.1$ for steam hammer

For use with SI units, Eq. (10-7) may be expressed as

$$Q_a = \frac{1000W_r H}{6(S + C)} \quad (10-8)$$

with Q_a computed in kN if W_r is in kN, H in m, S in mm/blow, and $C = 25$ for drop hammers and 2.5 for steam hammers. The Engineering-News formula has a built-in factor of safety of 6. Tests have shown that this formula is not reliable for computing pile loads, and it should be avoided except as a rough guide (McCarthy, 2002).

EXAMPLE 10-6

Given

The design capacity of a 0.3-m-diameter concrete pile is 160 kN. The pile is driven by a drop hammer with a manufacturer's hammer energy rating of 40 kN · m.

Required

Average penetration of the pile from the last few driving blows.

Solution

From Eq. (10-8),

$$Q_a = \frac{1000W_r H}{6(S + C)} \quad (10-8)$$

$$Q_a = 160 \text{ kN}$$

$$W_r H = 40 \text{ kN} \cdot \text{m}$$

$$C = 25 \text{ (for a drop hammer)}$$

Therefore,

$$160 \text{ kN} = \frac{(1000)(40 \text{ kN} \cdot \text{m})}{(6)(S + 25)}$$

$$S = 17 \text{ mm/blow}$$

Another pile-driving formula is known as the *Danish formula*. It is given as follows (McCarthy, 2002):

$$Q_{\text{ultimate}} = \frac{e_h(E_h)}{S + 1/2 S_0} \quad (10-9)$$

where Q_{ultimate} = ultimate capacity of the pile
 e_h = efficiency of pile hammer (see Table 10-7)
 E_h = manufacturer's hammer energy rating (see Table 10-8)

TABLE 10–7
Pile Hammer Efficiency

Type of Hammer	Efficiency, e_h
Drop hammer	0.75–1.00
Single-acting hammer	0.75–0.85
Double-acting hammer	0.85
Diesel hammer	0.85–1.00

Source: J. E. Bowles, *Engineering Properties of Soils and Their Measurement*, 2nd ed., McGraw-Hill Book Company, New York, 1978. Reprinted by permission.

S = average penetration of the pile from the last few driving blows

S_0 = elastic compression of the pile

$$S_0 = [(2e_h E_h L)/(AE)]^{1/2}$$

L = length of pile

A = cross-sectional area of pile

E = modulus of elasticity of pile material

Statistical studies indicate that a factor of safety of 3 should be used with the Danish formula.

Example 10–7 demonstrates how the Danish formula can be used as a field control during pile driving to indicate when the desired pile capacity has been obtained.

EXAMPLE 10–7

Given

1. The design capacity of a 12-in. steel-pipe pile is 100 kips.
2. The pile's modulus of elasticity is 29,000 kips/in.²
3. The pile's length is 40 ft.
4. The pile's cross-sectional area is 16 in.²
5. The hammer is a Vulcan 140C with a weight of pile hammer ram of 14,000 lb and manufacturer's hammer energy rating of 36,000 ft-lb.
6. Hammer efficiency is assumed to be 0.80.

Required

1. What should be the average penetration of the pile from the last few driving blows?
2. How many blows/ft for the last foot of penetration are required for the design capacity, using the Danish formula?

TABLE 10–8
Properties of Selected Impact Pile Hammers¹

Rated Energy (ft-lb)	Make	Model	Type ²	Blows per Minute ³	Stroke at Rated Energy (in.)	Weight Striking Parts (lb)
7260	Vulcan	2	S	70	29	300
8750	MKT ⁴	9B3	DB	145	17	1600
13,100	MKT	10B3	DB	105	19	3000
15,000	Vulcan	1	S	60	36	5000
15,100	Vulcan	50C	DF	120	15½	5000
16,000	MKT	DE-20	DE	48	96	2000
18,200	Link-Belt	440	DE	86–90	36⅞	4000
19,150	MKT	11B3	DB	96	19	5000
19,500	Raymond	65C	DF	100–110	16	6500
19,500	Vulcan	06	S	60	36	6500
22,400	MKT	DE-30	DE	48	96	2800
22,500	Delmag	D-12	DE	42–60		2750
24,375	Vulcan	0	S	50	39	7500
24,400	Kobe	K13	DE	45–60	102	2870
24,450	Vulcan	80C	DF	111	16	8000
26,000	Vulcan	08	S	50	39	8000
26,300	Link-Belt	520	DE	80–84	43⅜	5070
32,000	MKT	DE-40	DE	48	96	4000
32,500	MKT	S10	S	55	39	10,000
32,500	Vulcan	010	S	50	39	10,000
32,500	Raymond	00	S	50	39	10,000
36,000	Vulcan	140C	DF	103	15½	14,000
39,700	Delmag	D-22	DE	42–60		4850
40,600	Raymond	000	S	50	39	12,500
41,300	Kobe	K-22	DE	45–60	102	4850
42,000	Vulcan	014	S	60	36	14,000
48,750	Vulcan	016	S	60	36	16,250

¹ 1 ft-lb = 1.356 N · m; 1 in. = 25.4 mm; 1 lb = 4.448 N.

² S, single-acting steam; DB, double-acting steam; DF, differential-acting steam; DE, diesel.

³ After development of significant driving resistance.

⁴ For many years known as McKiernan-Terry.

Source: R. B. Peck, W. E. Hansen, T. H. Thornburn, *Foundation Engineering*, 2nd ed., John Wiley & Sons, Inc., New York, 1974. Copyright © 1974 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.

Solution

1. From Eq. (10–9),

$$Q_{\text{ultimate}} = \frac{e_h(E_h)}{S + \frac{1}{2}S_0} \tag{10-9}$$

$$S + \frac{1}{2}S_0 = \frac{e_h(E_h)}{Q_{\text{ultimate}}}$$

$$S = \frac{e_h(E_h)}{Q_{\text{ultimate}}} - \frac{1}{2}S_0$$

$$Q_{\text{design}} = \frac{Q_{\text{ultimate}}}{F.S.} = \frac{Q_{\text{ultimate}}}{3}$$

$$Q_{\text{ultimate}} = 3 \times Q_{\text{design}} = 3 \times 100 \text{ kips} = 300 \text{ kips}$$

$$S_0 = [(2e_h E_h L)/(AE)]^{1/2}$$

$$e_h = 0.80$$

$$E_h = 36,000 \text{ ft-lb} = 36 \text{ ft-kips}$$

$$L = 40 \text{ ft}$$

$$A = 16 \text{ in.}^2$$

$$E = 29,000 \text{ kips/in.}^2$$

$$S_0 = \left[\frac{(2)(0.80)(36 \text{ ft-kips})(40 \text{ ft})}{(16 \text{ in.}^2)(29,000 \text{ kips/in.}^2)} \right]^{1/2} = 0.070 \text{ ft} = 0.84 \text{ in.}$$

$$S = \frac{(0.80)(36 \text{ ft-kips})(12 \text{ in./ft})}{300 \text{ kips}} - (\frac{1}{2})(0.84 \text{ in.}) = 0.73 \text{ in./blow}$$

2. Number of blows required for last foot of penetration

$$= \frac{12 \text{ in./ft}}{0.73 \text{ in./blow}} = 16 \text{ blows/ft}$$

10-6 PILE LOAD TESTS

Load tests are performed on-site on test piles to determine or verify the design capacity of piles. Normally, piles are designed initially by analytic or other methods, based on estimated loads and soil characteristics. Pile load tests are performed on test piles during the design stage to check the design capacity. Should load test results indicate possible bearing failure or excessive settlement, the pile design should be revised accordingly. Also, data collected from pile load tests are used in the development of criteria for the foundation installation.

To carry out pile load tests, one must first drive test piles. They should be driven at a location where soil conditions are known (such as near a borehole) and where soil conditions are relatively poor. Both test piles and the method of driving them should be exactly the same as will be used in the construction project. A penetration record should be kept as each test pile is driven.

The next step is to load the test piles. For reasons explained previously in this chapter, test piles in clays should not be loaded until some time (at least several weeks) has passed after the piles are driven. Test piles in sands, however, may be loaded several days after they are driven. Test piles may be loaded by adding dead weight or by hydraulic jacking (against a fixed platform, for example). (Figure 10–14 illustrates schematically how test piles can be loaded by these methods.) The total load on test piles should be 200% of the proposed design load. The load should be applied to the pile in increments of 25% of the total test load. For specific details regarding loading, the reader is referred to the *ASTM Book of Standards*. In any event, a record of the load and corresponding settlement must be kept as each test pile is loaded and unloaded.

The next step is to plot a load versus settlement graph, as shown in Figure 10–15. From this graph, the relationship between the load and net settlement can be obtained. Ordinates along the loading curve of Figure 10–15 give gross settlement. Subtracting the final settlement upon unloading (point A in Figure 10–15) from ordinates along the unloading curve gives the rebound. Net settlement can then be determined by subtracting the rebound from the corresponding gross settlement.

The allowable pile load is generally determined based on criteria specified by applicable building codes. There are many building codes and therefore many criteria for determining allowable pile loads based on pile tests. It is, of course, the responsibility of engineers to follow criteria specified by the applicable building code. Examples 10–8 and 10–9, in addition to illustrating the determination of allowable pile loads, give two possible building code criteria for determining pile capacity by the pile load test.

EXAMPLE 10–8

Given

1. A 12-in.-diameter pipe pile with a length of 50 ft was subjected to a pile load test.
2. The test results were plotted and the load-settlement curve is shown in Figure 10–16.
3. The local building code states that the allowable pile load is taken as one-half of that load that produces a net settlement of not more than 0.01 in./ton but in no case more than 0.75 in.

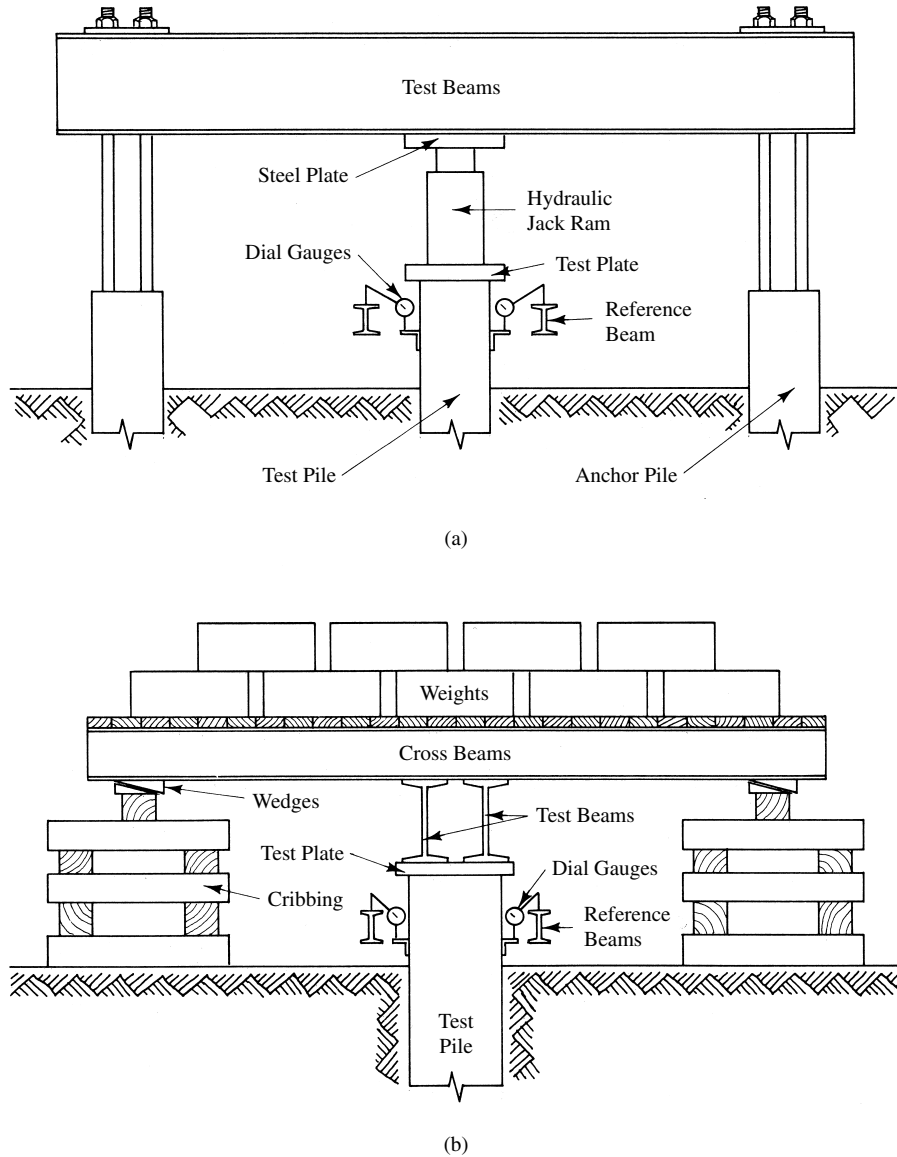


FIGURE 10-14 Schematic setup for test-pile loading: (a) using hydraulic jack acting against anchored reaction frame; (b) using weighted platform.

Source: *Annual Book of ASTM Standards*, ASTM, Philadelphia, 1989. Copyright American Society for Testing and Materials. Reprinted with permission.

Required

Allowable pile load.

Solution

$$\text{Net settlement} = \text{Gross settlement} - \text{Rebound}$$

Test Load (kips)	Test Load (tons)	Gross Settlement (in.)	Rebound (in.)	Net Settlement (in.)	Building Code Maximum Allowable Settlement (in.)
100	50	0.20	$2.39 - 2.20 = 0.19$	$0.20 - 0.19 = 0.01$	< 0.5
200	100	0.45	$2.54 - 2.20 = 0.34$	$0.45 - 0.34 = 0.11$	< 1.0 (use 0.75)
300	150	0.76	$2.64 - 2.20 = 0.44$	$0.76 - 0.44 = 0.32$	< 1.5 (use 0.75)
400	200	1.25	$2.73 - 2.20 = 0.53$	$1.25 - 0.53 = 0.72$	< 2.0 (use 0.75)
500	250	2.80	$2.80 - 2.20 = 0.60$	$2.80 - 0.60 = 2.20$	> 2.5 (use 0.75)

FIGURE 10-15 Typical load versus settlement graph.

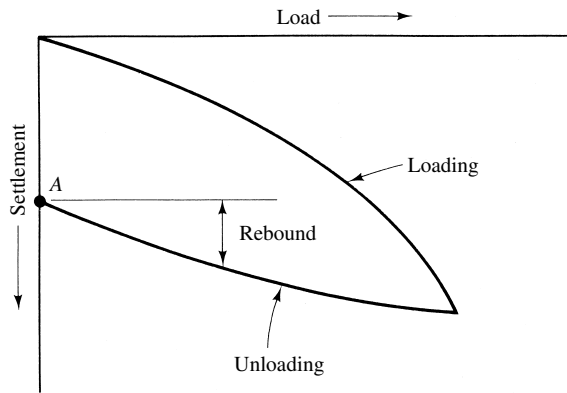
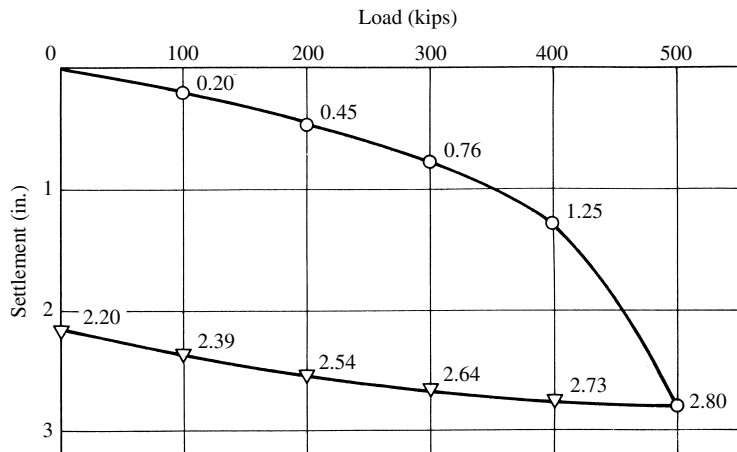


FIGURE 10-16



Because a test load of 200 tons produces a net settlement of 0.72 in. and the maximum allowable settlement is 0.75 in.,

$$\text{Allowable pile load} = \frac{200 \text{ tons}}{2} = 100 \text{ tons}$$

EXAMPLE 10–9

Given

The same conditions as in Example 10–8, except that another local building code is to be applied as follows: “The allowable pile load shall be not more than one-half of that test load that produces a net settlement per ton of test load of not more than 0.01 in., but in no case more than 0.5 inch.”

Required

Allowable pile load.

Solution

From Example 10–8,

Test Load (tons)	Net Settlement (in.)	Building Code Maximum Allowable Settlement (in.)
50	0.01	<0.5
100	0.11	<1.0(use 0.5)
150	0.32	<1.5(use 0.5)
200	0.72	>2.0(use 0.5)
250	2.20	>2.5(use 0.5)

Because a test load of 150 tons produces a net settlement of 0.32 in. and the maximum allowable settlement is 0.5 in.,

$$\text{Allowable pile load} = \frac{150 \text{ tons}}{2} = 75 \text{ tons}$$

Some building codes use a “breaking in the curve” or the point defined by tangents drawn on either side of a break of a load–settlement graph. One building code states that

the design load on piles may be determined by the designer based on an analysis of the results of pile load tests performed in accordance with ASTM D-1143. The allowable pile load shall be determined by the application of a safety factor of 2 to the ultimate pile capacity as determined by the intersection of the initial and final tangents to a curve fitted to the plotted results of the pile load test. The fitted curve shall not extend to any point at which the pile continued to move under the applied load. . . .

EXAMPLE 10–10

Given

The results of a pile load test are as follows:

Load (kN)	Settlement (mm)
250	2.7
500	5.8
750	9.3
1000	12.5
1250	16.2
1500	20.0
1750	44.0
2000	80.0

Required

Assuming that the building code given just prior to this example is applicable, find the allowable load on the pile.

Solution

Load-test data are shown plotted in Figure 10–17. Initial and final tangents to the plotted curve intersect at a load of 1600 kN. Hence, according to the code, the allowable load on the pile is $(1600 \text{ kN})/2$, or 800 kN.

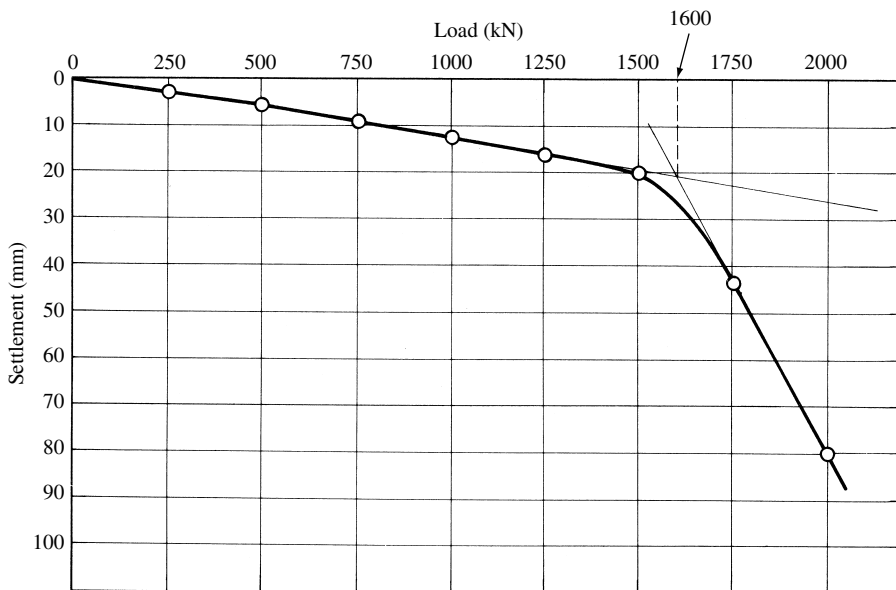


FIGURE 10–17 Plot of load-test data for Example 10–10.

10-7 NEGATIVE SKIN FRICTION (DOWN DRAG)

As related throughout this chapter, piles depend, in part at least, on skin friction for support. Under certain conditions, however, skin friction may develop that causes down drag on a pile rather than support. Skin friction that causes down drag is known as *negative skin friction*.

Negative skin friction may occur if soil adjacent to a pile settles more than the pile itself. This is most likely to happen when a pile is driven through compressible soil, such as soft to medium clay or soft silt. Subsequent consolidation of the soil (caused by newly placed fill, for example) can cause negative skin friction as soil adjacent to the pile moves downward while the pile, restrained at the tip, remains fixed. A similar phenomenon may occur as a result of lowering the water table at the site.

Negative skin friction is, of course, detrimental with regard to a pile's ability to carry load. Hence, if conditions at a particular site suggest that negative skin friction may occur, its magnitude should be determined and subtracted from the pile's load-carrying ability.

10-8 PILE GROUPS AND SPACING OF PILES

Heretofore in this chapter, discussion has pertained to a single pile. In reality, however, piles are almost always arranged in groups of three or more. Furthermore, the group of piles is commonly tied together by a pile cap, which is attached to the head of individual piles and causes the several piles to act together as a pile foundation. Figure 10-18 illustrates some typical pile grouping patterns.

If two piles are driven close together, soil stresses caused by the piles tend to overlap, and the bearing capacity of the pile group consisting of two piles is less than the sum of the individual capacities. If the two piles are moved farther apart so that individual stresses do not overlap, the bearing capacity of the pile group is not reduced significantly from the sum of the individual capacities. Thus, it would appear that piles should be spaced relatively far apart. This consideration is offset, however, by the unduly large pile caps that would be required for the wider spacing.

Minimum allowable pile spacing is often specified by applicable building codes. For example, a building code may state that "the minimum center-to-center spacing of piles not driven to rock shall be not less than twice the average diameter of a round pile, nor less than 1.75 times the diagonal dimension of a rectangular or rolled structural steel pile, nor less than 2 ft 6 in. (0.76 m). For piles driven to rock, the minimum center-to-center spacing of piles shall be not less than twice the average diameter of a round pile, nor less than 1.75 times the diagonal dimension of a rectangular or rolled structural steel pile, nor less than 2 ft 0 in. (0.61 m)."

10-9 EFFICIENCY OF PILE GROUPS

As related in the last section, the capacity of a pile group may be less than the sum of the individual capacities of the piles making up the group. Inasmuch as it would be convenient to estimate the capacity of a group of piles based on the capacity of a single pile, attempts have been made to determine the efficiency of pile groups.

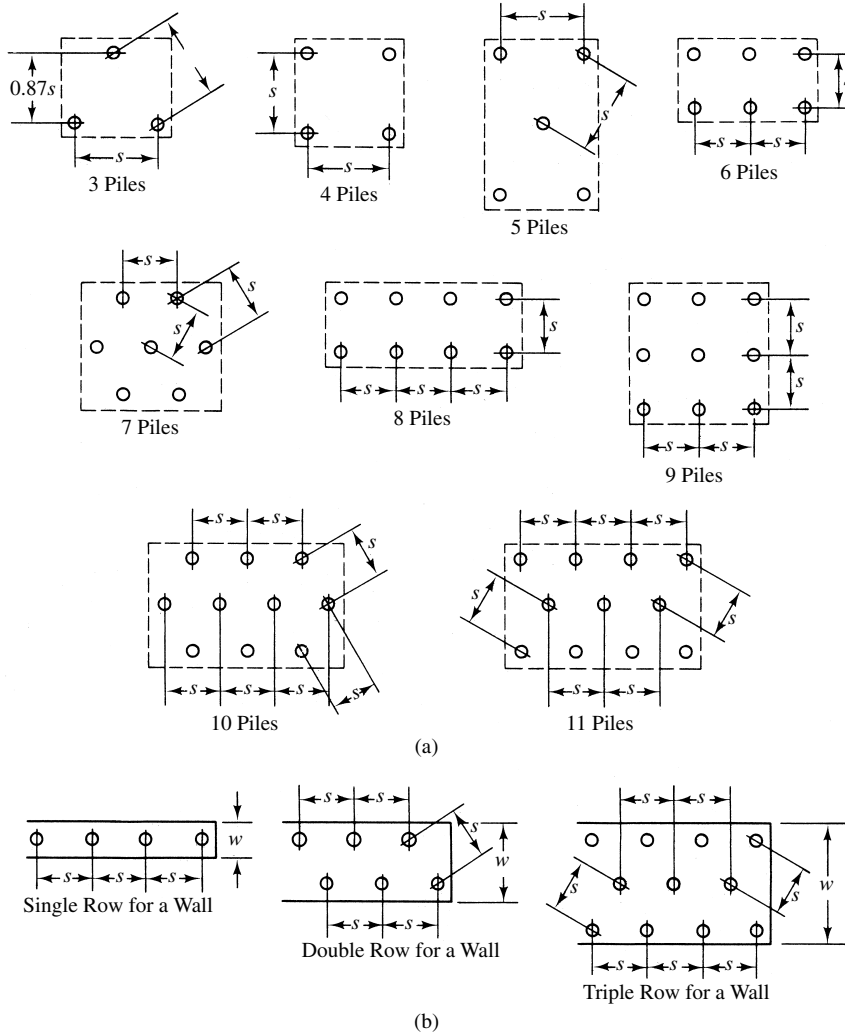


FIGURE 10-18 Typical pile grouping patterns for (a) single footings and (b) foundation walls. Source: J. E. Bowles, *Engineering Properties of Soils and Their Measurement*, 2nd ed., McGraw-Hill Book Company, New York, 1978. Reprinted by permission.

(Efficiency of a pile group is the capacity of a pile group divided by the sum of the individual capacities of the piles making up the group.)

In the case where a pile group is comprised of end-bearing piles resting on bedrock (or on a layer of dense sand and gravel overlying bedrock), an efficiency of 1.0 may be assumed (Jumikis, 1971). (In other words, the group of n piles will carry n times the capacity of a single pile.) An efficiency of 1.0 is also often assumed by designers for friction piles driven in cohesionless soil. For a pile group composed of

friction piles driven in cohesive soil, an efficiency of less than 1.0 is to be expected because stresses from individual piles build up and reduce the capacity of the pile group.

One equation that has been used to compute pile-group efficiency is known as the *Converse–Labarre equation* (Jumikis, 1971):

$$E_g = 1 - \theta \frac{(n - 1)m + (m - 1)n}{90mn} \quad (10-10)$$

where E_g = pile-group efficiency

θ = $\arctan d/s$, deg

n = number of piles in a row

m = number of rows of piles

d = diameter of piles

s = spacing of piles, center to center, in same units as pile diameter

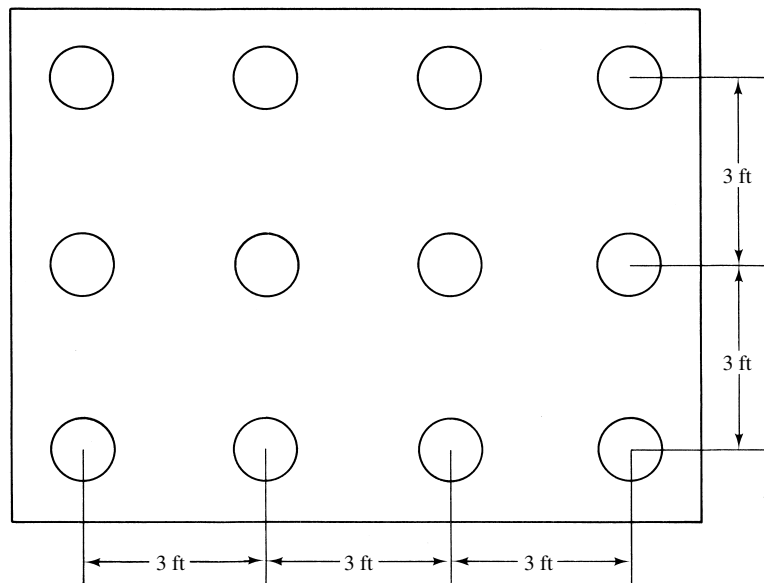
Example 10–11 illustrates the application of the Converse–Labarre equation.

EXAMPLE 10–11

Given

1. A pile group consists of 12 friction piles in cohesive soil (see Figure 10–19).
2. Each pile's diameter is 12 in., and center-to-center spacing is 3 ft.
3. By means of a load test, the ultimate load of a single pile was found to be 100 kips.

FIGURE 10–19



Required

Design capacity of the pile group, using the Converse–Labarre equation.

Solution

$$E_g = 1 - \theta \frac{(n-1)m + (m-1)n}{90mn} \quad (10-10)$$

$$\theta = \arctan \frac{d}{s} = \arctan \frac{1}{3} = 18.4^\circ$$

$$E_g = 1 - (18.4) \frac{(4-1)(3) + (3-1)(4)}{(90)(3)(4)} = 0.710$$

$$\text{Allowable bearing capacity of a single pile} = \frac{100 \text{ kips}}{2} = 50 \text{ kips}$$

$$\text{Design capacity of the pile group} = (0.710)(12)(50 \text{ kips}) = 426 \text{ kips}$$

For friction piles driven in cohesive soil, Coyle and Sulaiman suggested that pile-group efficiency may be assumed to vary linearly from a value of 0.7 at a pile spacing of three times the pile diameter to a value of 1.0 at a pile spacing of eight times the pile diameter (McClelland, 1972 and Coyle and Sulaiman, 1970). For pile spacings less than three times the pile diameter, group capacity may be considered as block capacity, and total capacity can be estimated by treating the group as a pier and applying the following equation (Terzaghi and Peck, 1967 and McClelland, 1972):

$$Q_g = 2D(W + L)f + 1.3 \times c \times N_c \times W \times L \quad (10-11)$$

where Q_g = ultimate bearing capacity of pile group
 D = depth of pile group
 W = width of pile group
 L = length of pile group
 f = unit adhesion developed between cohesive soil and pile surface (equal to αc)
 α = ratio of adhesion to cohesion (see Figure 10-10)
 c = cohesion
 N_c = bearing capacity factor for a shallow rectangular footing (see Figure 9-7)

A pile group can be considered safe against block failure if the total design load (i.e., “safe design load” per pile multiplied by the number of piles) does not exceed $Q_g/3$. If the total design load exceeds $Q_g/3$, the foundation design must be revised.

Figure 10-20 gives a summary of criteria for pile-group capacity.

FIGURE 10–20 Summary of criteria for pile-group capacity. (a) Individual pile failure in cohesionless soils:

$Q_g = n \times Q_u$; individual pile failure in cohesive soils; for $S \geq 3$ diameters, $Q_g = E_g \times n \times Q_u$, E_g varies linearly from 0.7 at $S = 3$ diameters to 1.0 at $S \geq 8$ diameters.

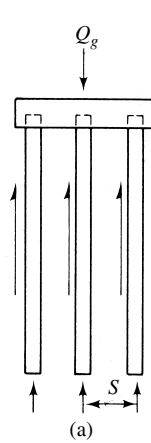
Source: B. McClelland, "Design and Performance of Deep Foundations." *Proc. Specialty Conf. Perform. Earth Earth-Supported Struct.*, ASCE, 2 (June 1972). Reprinted by permission.

(b) Block failure in cohesive soils: for $S < 3.0$ diameters,

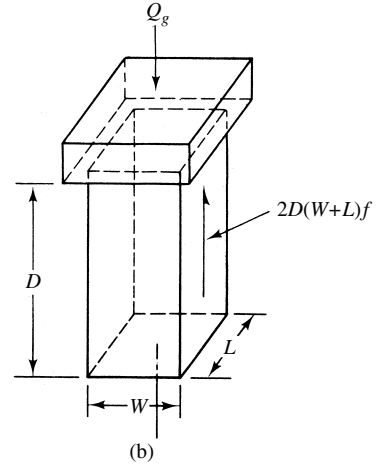
$$Q_g = 2D(W + L)f + 1.3$$

$$\times c \times N_c \times W \times L.$$

Source: H. M. Coyle and I. H. Sulaiman, *Bearing Capacity of Foundation Piles: State of the Art.*, Highway Research Board, Record N, 333, 1970.



(a)



(b)

In Cohesionless Soils:

$$Q_g = n \times Q_u$$

In Cohesive Soils:

For $S < 3.0$ Diameters,

$$Q_g = 2D(W+L)f + 1.3 \times c \times N_c \times W \times L$$

In Cohesive Soils:

For $S \geq 3$ Diameters,

$$Q_g = E_g \times n \times Q_u$$

E_g Varies Linearly from 0.7 at $S = 3$ Diameters to 1.0 at $S \geq 8$ Diameters

EXAMPLE 10–12

Given

1. A pile group consists of four friction piles in cohesive soil (see Figure 10–21).
2. Each pile's diameter is 12 in., and center-to-center spacing is 2.5 ft.

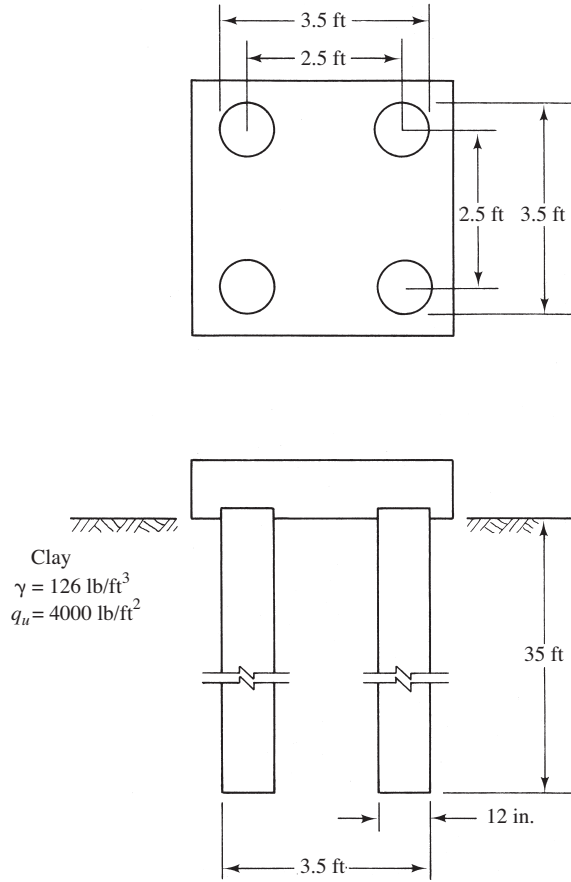
Required

1. Block capacity of the pile group. Use a factor of safety of 3.
2. Allowable group capacity based on individual pile failure. Use a factor of safety of 2, along with the Converse–Labarre equation for pile-group efficiency.
3. Design capacity of the pile group.

Solution

1. *Block capacity:* Because center-to-center spacing of the piles is 2.5 ft, which is less than 3 ft (i.e., 3 diameters), according to the criteria suggested by Coyle

FIGURE 10-21



and Sulaiman (Coyle and Sulaiman, 1970), the block capacity of the pile group can be estimated by Eq. (10-11):

$$Q_g = 2D(W + L)f + 1.3 \times c \times N_c \times W \times L \quad (10-11)$$

$$D = 35 \text{ ft}$$

$$W = 2.5 \text{ ft} + 0.5 \text{ ft} + 0.5 \text{ ft} = 3.5 \text{ ft}$$

$$L = 2.5 \text{ ft} + 0.5 \text{ ft} + 0.5 \text{ ft} = 3.5 \text{ ft}$$

$$f = \alpha c$$

$$q_u = 4000 \text{ lb/ft}^2 = 2.0 \text{ tons/ft}^2$$

$$c = \frac{4000 \text{ lb/ft}^2}{2} = 2000 \text{ lb/ft}^2 = 2 \text{ kips/ft}^2$$

From Figure 10-10, with $q_u = 2.0$ tons/ft²,

$$\alpha = 0.56$$

$$f = (0.56)(2000 \text{ lb/ft}^2) = 1120 \text{ lb/ft}^2 = 1.12 \text{ kips/ft}^2$$

$$N_c = 5.14 \quad (\text{from Figure 9-7 for } \phi = 0^\circ \text{ for clay})$$

$$Q_g = (2)(35 \text{ ft})(3.5 \text{ ft} + 3.5 \text{ ft})(1.12 \text{ kips/ft}^2) \\ + (1.3)(2 \text{ kips/ft}^2)(5.14)(3.5 \text{ ft})(3.5 \text{ ft}) = 713 \text{ kips}$$

$$\text{Allowable block capacity} = \frac{713 \text{ kips}}{3} = 238 \text{ kips}$$

2. *Group capacity based on individual pile:*

$$Q_{\text{ultimate}} = Q_{\text{friction}} + Q_{\text{tip}} \quad (10-1)$$

$$Q_{\text{friction}} = f \cdot A_{\text{surface}}$$

$$f = 1.12 \text{ kips/ft}^2 \quad [\text{from (1) above}]$$

$$A_{\text{surface}} = (\pi d)(L) = (\pi)(1 \text{ ft})(35 \text{ ft}) = 110.0 \text{ ft}^2$$

$$Q_{\text{friction}} = (1.12 \text{ kips/ft}^2)(110.0 \text{ ft}^2) = 123 \text{ kips}$$

$$Q_{\text{tip}} = cN_c A_{\text{tip}} = (2 \text{ kips/ft}^2)(9) \left(\frac{\pi}{4} \right) (1 \text{ ft})^2 = 14 \text{ kips}$$

$$Q_{\text{ultimate}} = 123 \text{ kips} + 14 \text{ kips} = 137 \text{ kips}$$

$$Q_a = \frac{137 \text{ kips}}{2}$$

$$= 68.5 \text{ kips} \quad (\text{allowable load for an individual pile})$$

$$E_g = 1 - \theta \frac{(n-1)m + (m-1)n}{90mn} \quad (10-10)$$

$$\theta = \arctan \frac{d}{s} = \arctan \frac{1}{2.5} = 21.8^\circ$$

$$n = 2$$

$$m = 2$$

$$E_g = 1 - (21.8) \frac{(2-1)(2) + (2-1)(2)}{(90)(2)(2)} = 0.758$$

$$\text{Allowable } Q = (68.5 \text{ kips})(4)(0.758)$$

$$= 208 \text{ kips} \quad (\text{allowable load for pile group})$$

3. *Design capacity of the pile group:* This is the smaller group capacity of (1) and (2), which is 208 kips.

10-10 DISTRIBUTION OF LOADS IN PILE GROUPS

The load on any particular pile within a pile group may be computed by using the elastic equation:

$$Q_m = \frac{Q}{n} \pm \frac{M_y x}{\Sigma(x^2)} \pm \frac{M_x y}{\Sigma(y^2)} \tag{10-12}$$

where Q_m = axial load on any pile m
 Q = total vertical load acting at the centroid of the pile group
 n = number of piles
 M_x, M_y = moment with respect to x and y axes, respectively
 x, y = distance from pile to y and x axes, respectively

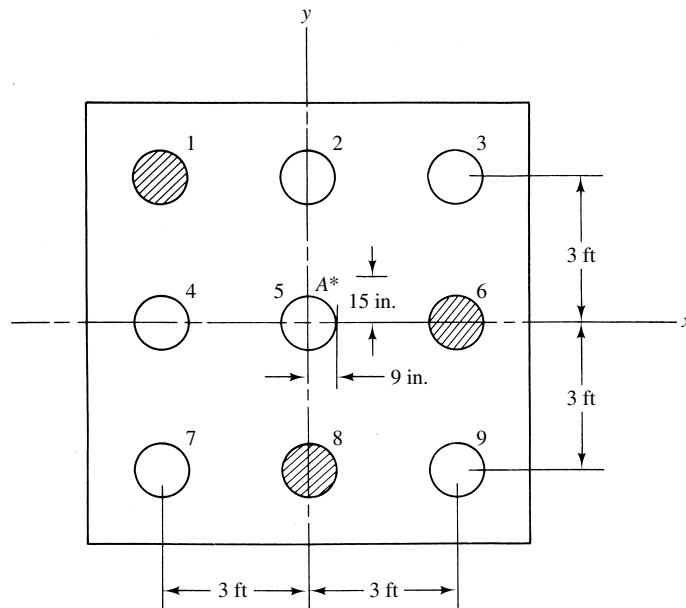
(Both x and y axes pass through the centroid of the pile group and are perpendicular to each other.) It should be noted that shears and bending moments can be determined for any section of pile cap by using elastic and static equations.

EXAMPLE 10-13

Given

1. A pile group consists of nine piles as shown in Figure 10-22.
2. A column load of 450 kips acts vertically on point A.

FIGURE 10-22



Required

Load on piles 1, 6, and 8.

Solution

From Eq. (10-12),

$$Q_m = \frac{Q}{n} \pm \frac{M_y x}{\Sigma(x^2)} \pm \frac{M_x y}{\Sigma(y^2)} \quad (10-12)$$

$$Q = 450 \text{ kips}$$

$$n = 9$$

$$\Sigma(x^2) = (6)(3 \text{ ft})^2 = 54 \text{ ft}^2$$

$$\Sigma(y^2) = (6)(3 \text{ ft})^2 = 54 \text{ ft}^2$$

$$M_x = (450 \text{ kips}) \left(\frac{15 \text{ in.}}{12 \text{ in./ft}} \right) = 562.5 \text{ kip-ft}$$

$$M_y = (450 \text{ kips}) \left(\frac{9 \text{ in.}}{12 \text{ in./ft}} \right) = 337.5 \text{ kip-ft}$$

Load on Pile No. 1

$$Q_1 = \frac{450 \text{ kips}}{9} + \frac{(337.5 \text{ kip-ft})(-3 \text{ ft})}{54 \text{ ft}^2} + \frac{(562.5 \text{ kip-ft})(+3 \text{ ft})}{54 \text{ ft}^2} = 62.5 \text{ kips}$$

Load on Pile No. 6

$$Q_6 = \frac{450 \text{ kips}}{9} + \frac{(337.5 \text{ kip-ft})(+3 \text{ ft})}{54 \text{ ft}^2} + \frac{(562.5 \text{ kip-ft})(0)}{54 \text{ ft}^2} = 68.8 \text{ kips}$$

Load on Pile No. 8

$$Q_8 = \frac{450 \text{ kips}}{9} + \frac{(337.5 \text{ kip-ft})(0)}{54 \text{ ft}^2} + \frac{(562.5 \text{ kip-ft})(-3 \text{ ft})}{54 \text{ ft}^2} = 18.8 \text{ kips}$$

EXAMPLE 10-14

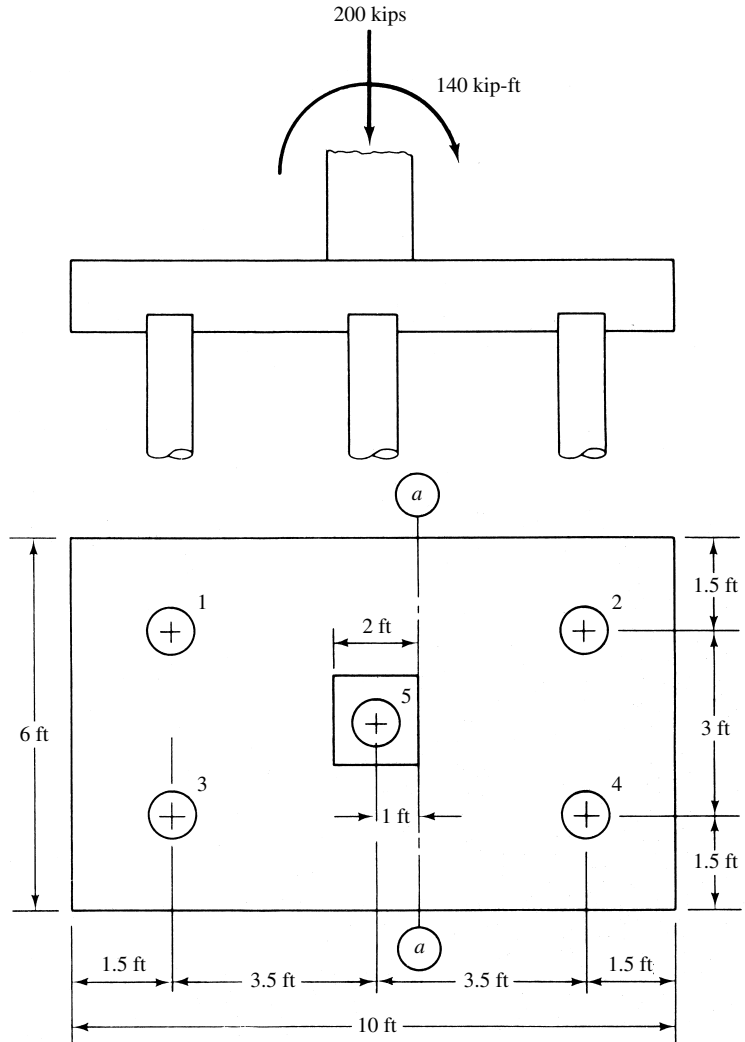
Given

1. Figure 10-23 shows a pile foundation consisting of five piles.
2. The pile foundation is subjected to a 200-kip vertical load and a moment with respect to the y axis of 140 kip-ft (Figure 10-23).

Required

Shear and bending moment on section $a-a$ due to the pile reacting under the pile cap.

FIGURE 10-23



Solution

From Eq. (10-12),

$$Q_m = \frac{Q}{n} \pm \frac{M_y x}{\Sigma(x^2)} \pm \frac{M_x y}{\Sigma(y^2)} \quad (10-12)$$

$$Q = 200 \text{ kips}$$

$$n = 5$$

$$M_y = 140 \text{ kip-ft}$$

$$M_x = 0$$

$$\Sigma(x^2) = (4)(3.5 \text{ ft})^2 = 49 \text{ ft}^2$$

$$Q_2 = Q_4 = \frac{200 \text{ kips}}{5} + \frac{(140 \text{ kip-ft})(3.5 \text{ ft})}{49 \text{ ft}^2} + \frac{(0)y}{\Sigma(y^2)} = 50 \text{ kips}$$

$$\text{Shear at section } a-a = (50 \text{ kips})(2) = 100 \text{ kips}$$

$$\text{Moment at section } a-a = (2)(50 \text{ kips})(3.5 \text{ ft} - 1 \text{ ft}) = 250 \text{ kip-ft}$$

10-11 SETTLEMENT OF PILE FOUNDATIONS

Like shallow foundations, pile foundations must be analyzed to predict their settlement to ensure that it is tolerable. Unfortunately, universally accepted methods for predicting pile settlements are not available today. The following give some possible methods for predicting pile settlement for end-bearing piles on bedrock, piles in sand, and piles in clay.

Settlement of End-Bearing Piles on Bedrock

A well-designed and constructed pile foundation on hard bedrock generally will not experience an objectionable amount of settlement. The amount of settlement of pile foundations on soft bedrock is very difficult to predict accurately and can be estimated only by judging from the characteristics of rock core samples. Local experience, if available, should be employed as guidance.

Settlement of Piles in Sand

Settlement of a pile group is substantially larger than that of a single test pile. In fact, group settlement can be two to 10 times that of a single pile or even greater. Also, the larger the pile group, the greater the settlement will generally be. For sandy soils, the settlement for a pile group can be estimated based on the settlement of a single test pile (from a field load test) using the equation (U.S. Department of the Navy, 1982)

$$S = S_0 [\bar{B}/B]^{1/2} \quad (10-13)$$

where S = group settlement
 S_0 = settlement of a single pile (from a field load test)
 \bar{B} = smallest dimension of the pile group
 B = diameter of the tested pile

All terms in Eq. (10-13) are in length units, but S and S_0 must be in the same units (ordinarily in. or cm) and \bar{B} and B must be in the same units (usually ft or m). For a 12-in.-diameter test pile, Eq. (10-13) shows that a 16-ft-wide pile group would settle about four times as much as that of the test pile.

Settlement of Piles in Clay

Prediction of pile settlements in deep clay requires first an estimate of load distribution in the soil, followed by settlement calculation in accordance with consolidation theory. One method of estimating load distribution is to assume that the load is

applied to an equivalent mat (i.e., an imaginary mat) at some selected level and then to compute the distribution of the load from that imaginary mat. For friction piles in deep clay, the equivalent (imaginary) mat may be assumed at a plane located at two-thirds the pile depth (Terzaghi and Peck, 1967) (see Figure 10–24a). Consolidation of soil below that plane is then computed as if the piles are no longer present. If piles pass through a layer of very soft clay to a firm bearing in a layer of stiff clay, an equivalent mat may be placed at the level of the pile tips, assuming eventual concentration of the load at that level (Figure 10–24b).

Settlement analysis is then performed, based on consolidation test results, to predict the expected, approximate settlement that would occur for an ordinary (unpiled) foundation as if the foundation were a mat of the same depth and dimensions at the same plane. In such cases, the method of settlement analysis of pile-supported foundations is the same as that used for shallow foundations. From Chapter 7, based on consolidation test results, the amount of settlement due to consolidation can be calculated for a layer of compressible soil by the following equation (Terzaghi and Peck, 1967):

$$S_c = \frac{e_0 - e}{1 + e_0} (H) \quad (7-15)$$

or

$$S_c = C_c \left(\frac{H}{1 + e_0} \right) \log \frac{p_0 + \Delta p}{p_0} \quad (7-19)$$

where S_c = consolidation settlement
 e_0 = initial void ratio (void ratio *in situ*)
 e = final void ratio
 H = thickness of layer of compressible soil
 C_c = compression index (slope of field e - $\log p$ curve)
 p_0 = effective overburden pressure (effective weight of soil above midheight of the consolidating layer)
 Δp = consolidation pressure (net additional pressure)

Example 10–15 illustrates computation of approximate total settlement of a pile foundation in deep clay.

EXAMPLE 10–15

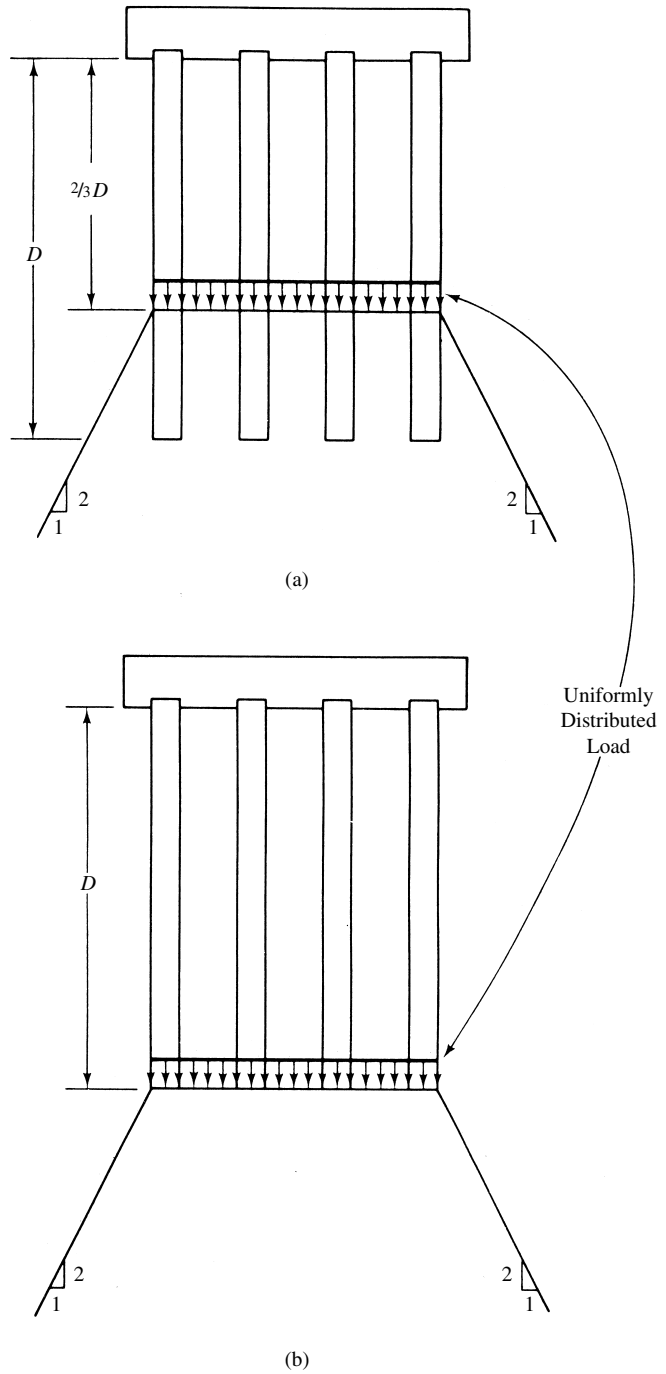
Given

1. A group of friction piles in deep clay is shown in Figure 10–25.
2. The total load on the piles reduced by the weight of soil displaced by the foundation is 300 kips.

Required

Approximate total settlement of the pile foundation.

FIGURE 10-24 Friction piles:
(a) in deep clay; (b) through
soft clay into stiff clay.
Source: B. McClelland, "Design
and Performance of Deep
Foundations," *Proc. Specialty Conf.
Perform. Earth Earth-Supported
Struct.*, ASCE, 2 (June 1972).



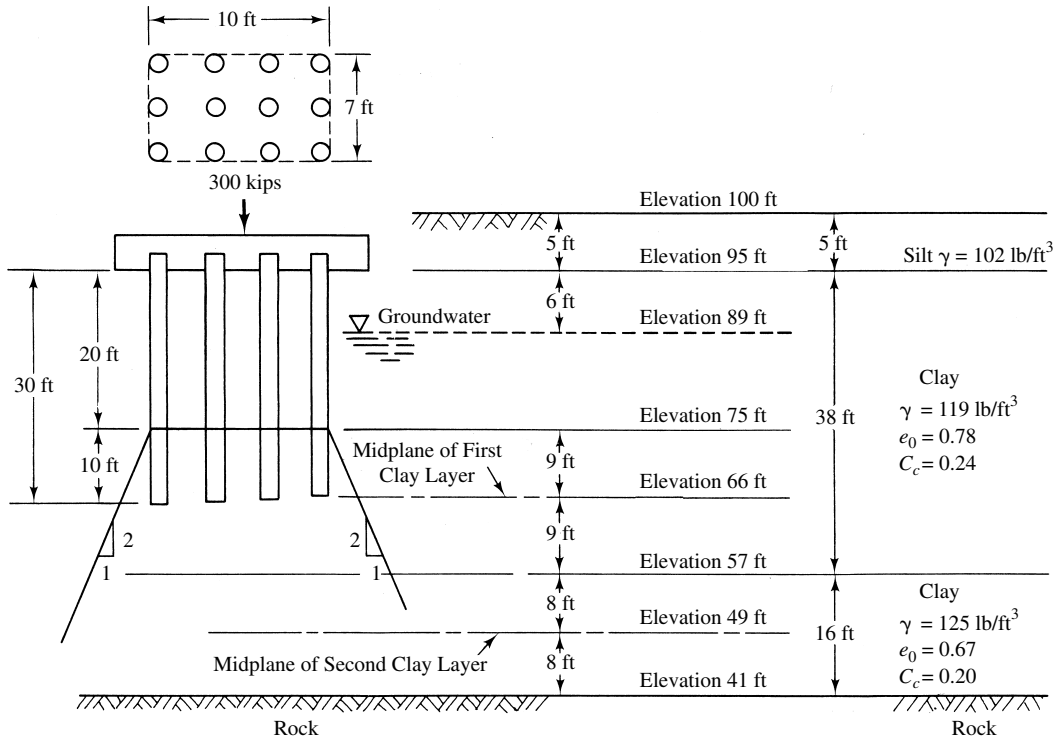


FIGURE 10-25

Solution

Computation of Effective Overburden Pressures (p_0)

$$\begin{aligned}
 p_0 \text{ at elev. } 66 \text{ ft} &= (100 \text{ ft} - 95 \text{ ft})(102 \text{ lb/ft}^3) + (95 \text{ ft} - 89 \text{ ft})(119 \text{ lb/ft}^3) \\
 &\quad + (89 \text{ ft} - 66 \text{ ft})(119 \text{ lb/ft}^3 - 62.4 \text{ lb/ft}^3) \\
 &= 2530 \text{ lb/ft}^2, \text{ or } 2.53 \text{ kips/ft}^2
 \end{aligned}$$

$$\begin{aligned}
 p_0 \text{ at elev. } 49 \text{ ft} &= (100 \text{ ft} - 95 \text{ ft})(102 \text{ lb/ft}^3) + (95 \text{ ft} - 89 \text{ ft})(119 \text{ lb/ft}^3) \\
 &\quad + (89 \text{ ft} - 57 \text{ ft})(119 \text{ lb/ft}^3 - 62.4 \text{ lb/ft}^3) \\
 &\quad + (57 \text{ ft} - 49 \text{ ft})(125 \text{ lb/ft}^3 - 62.4 \text{ lb/ft}^3) \\
 &= 3540 \text{ lb/ft}^2, \text{ or } 3.54 \text{ kips/ft}^2
 \end{aligned}$$

Computation of Δp

$$\begin{aligned}
 \text{Area at elev. } 66 \text{ ft} &= [10 \text{ ft} + (2)(75 \text{ ft} - 66 \text{ ft})(\frac{1}{2})] \cdot [7 \text{ ft} + (2)(75 \text{ ft} - 66 \text{ ft})(\frac{1}{2})] \\
 &= 304 \text{ ft}^2
 \end{aligned}$$

$$\Delta p \text{ at elev. } 66 \text{ ft} = \frac{300 \text{ kips}}{304 \text{ ft}^2} = 0.99 \text{ kip/ft}^2$$

$$\begin{aligned} \text{Area at elev. 49 ft} &= [10 \text{ ft} + (2)(75 \text{ ft} - 49 \text{ ft})(\frac{1}{2})] \cdot [7 \text{ ft} + (2)(75 \text{ ft} - 49 \text{ ft})(\frac{1}{2})] \\ &= 1188 \text{ ft}^2 \end{aligned}$$

$$\Delta p \text{ at elev. 49 ft} = \frac{300 \text{ kips}}{1188 \text{ ft}^2} = 0.25 \text{ kip/ft}^2$$

Settlement Computations

From Eq. (7-19),

$$S_c = C_c \left(\frac{H}{1 + e_0} \right) \log \frac{p_0 + \Delta p}{p_0} \quad (7-19)$$

Elev. 75 to 57 ft:

$$S_c = (0.24) \left(\frac{18 \text{ ft}}{1 + 0.78} \right) \log \frac{2.53 \text{ kips/ft}^2 + 0.99 \text{ kip/ft}^2}{2.53 \text{ kips/ft}^2} = 0.35 \text{ ft}$$

Elev. 57 to 41 ft:

$$S_c = (0.20) \left(\frac{16 \text{ ft}}{1 + 0.67} \right) \log \frac{3.54 \text{ kips/ft}^2 + 0.25 \text{ kip/ft}^2}{3.54 \text{ kips/ft}^2} = 0.06 \text{ ft}$$

$$\text{Approximate total settlement} = 0.35 \text{ ft} + 0.06 \text{ ft} = 0.41 \text{ ft} = 4.9 \text{ in.}$$

10-12 CONSTRUCTION OF PILE FOUNDATIONS

Construction of pile foundations consists of installing the piles (see Figure 10-26) (usually by driving) and constructing pile caps. Pile caps are often made of concrete, and their construction is usually a relatively simple structural problem.

With regard to pile installation, most piles are driven by a device called a *pile hammer*. Simply speaking, a pile hammer is a weight that is alternately raised and dropped onto the top of a pile to drive the pile into the soil. Hammer weights vary considerably. As a general rule, a hammer's weight should be at least half the weight of the pile being driven, and the driving energy should be at least 1 ft-lb for each pound of pile weight. The hammer itself is contained within a larger device, with the hammer operated between a pair of parallel steel members known as *leads*.

Several types of pile hammers are available. *Drop hammers* consist of a heavy ram that is raised by a cable and hoisting drum and dropped onto the pile. For *single-acting hammers*, the ram is raised by steam or compressed air and dropped onto the pile. With *double-acting hammers*, the ram is both raised and accelerated downward by steam or air. *Differential-acting hammers* are similar to double-acting hammers. *Diesel hammers* use gasoline for fuel, which causes an explosion that advances the pile and lifts the ram. The total driving energy delivered to the pile includes both the impact of the ram and the energy delivered by the explosion. Table 10-8 (in Section 10-5) gives more specific information on various pile hammers.

Selection of a pile hammer for a specific job depends on a number of factors. Table 10-9 gives data for selection of pile hammers for various conditions.

FIGURE 10–26 Pile installation.
Source: Courtesy of Associated Pile & Fitting Corporation of New Jersey.



Repeated striking of a pile by a pile hammer's heavy ram can damage the pile. A wood pile's fibers at its head (top) may be crushed by the ram (an action known as *brooming*), causing the pile to split near its top end. Brooming and splitting can be minimized by putting a heavy steel ring over the pile's head while it is being driven into the soil. Any damaged part of the pile must be cut off and removed prior to loading the pile. (Hence, a somewhat longer wood pile than is ultimately needed should be used at the beginning to allow for the length of pile that must be cut off.) Precast concrete piles may be protected by placing a metal cap over the pile's head with laminated layers of wood beneath the cap (i.e., between the cap and the pile's head) and a block of hardwood above the cap—all of this to help protect the pile as it is being driven by cushioning the ram's blow.

The other end of a pile—the tip—also needs protection—particularly if the pile is being driven through very hard soil or boulders. Such protection is provided by *driving points* (sometimes referred to as *driving shoes*). Figure 10–27 illustrates some commercially available driving points (or shoes) for various types of piles. If hard driving is anticipated for precast concrete piles, driving points (or shoes) may be cast at the tips of the piles (see Figure 10–28).

TABLE 10-9
Data for Selection of Pile Hammers for Driving Concrete, Timber, and Steel Sheetpiling under Average and Heavy Driving Conditions^{1,2}

Pile Length (ft)	Penetration Depth (%)	Sheet Pile ³			Timber Pile			Concrete Pile	
		Light (ft-lb per blow)	Medium (ft-lb per blow)	Heavy	Light (ft-lb per blow)	Heavy	Light (ft-lb per blow)	Heavy	
25	50	1000-1800	1000-1800	1800-2500	3600-4200	3600-7250	7250-8750	8750-15,000	
	100	1000-3600	1800-3600	1800-3600	3600-7250	3600-8750	7250-8750	13,000-15,000	
	50	1800-3600	1800-3600	3600-4200	3600-8750	7250-8750	8750-15,000	13,000-25,000	
75	100	3600-4200	3600-4200	3600-7500	7250-8750	7250-15,000	13,000-15,000	15,000-25,000	
	50	3600-7500	3600-7500	3600-8750	13,000-15,000	13,000-15,000	19,000-36,000	19,000-36,000	
25	50	<i>Driving through earth, sand, loose gravel—normal frictional resistance</i>							8750-15,000
		1800-2500	1800-2500	1800-4200	7250-8750	7250-8750	7250-8750		
		1800-3600	1800-3600	1800-4200	7250-8750	7250-8750	7250-15,000		
50	50	<i>Driving through stiff clay, compacted gravel—very resistant</i>							13,000-25,000
		1800-4200	3600-4200	3600-8750	7250-15,000	7250-15,000	13,000-15,000		
		3600-8750	3600-13,000	13,000-15,000	13,000-15,000	13,000-15,000	19,000-36,000		
75	50	3600-8750	3600-8750	7500-19,000	15,000-25,000	15,000-25,000	19,000-36,000	19,000-36,000	
		20	30	40	60	150	400		
Pile size (approx. inches)		15	15	15	13 (diam)	18 (diam)	12	20	

¹ Tennessee Valley Authority.

² 1 ft-lb = 1.356 N·m; 1 in. = 25.4 mm; 1 lb = 4.448 N.

³ Energy required in driving a single sheet pile. Double these when driving two piles at a time.

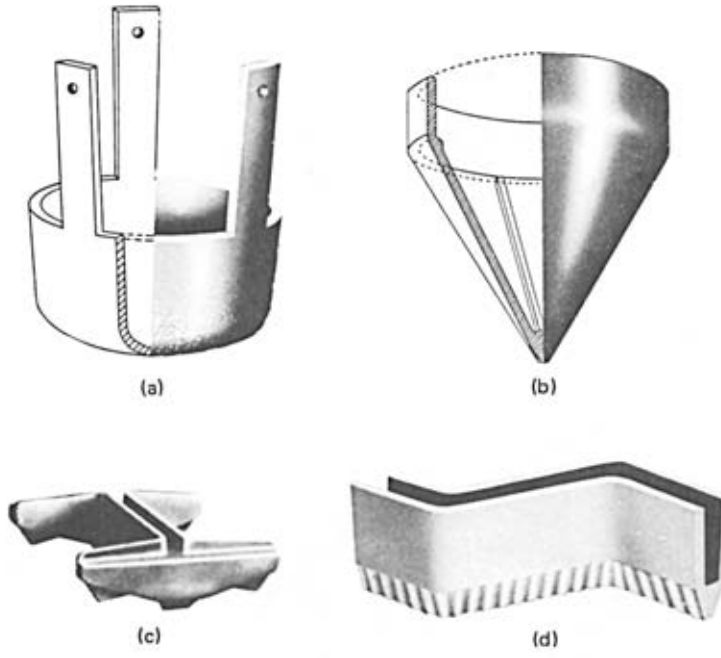
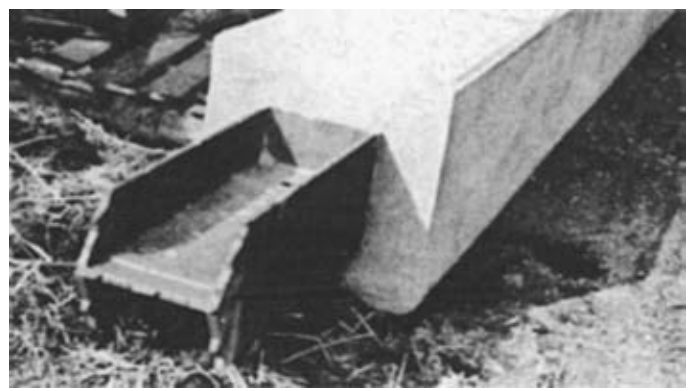
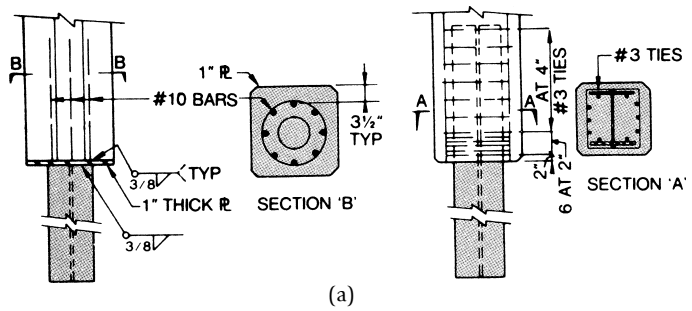


FIGURE 10-27 Driving points (or shoes): (a) timber pile shoes; (b) pipe pile point; (c) H-pile point; (d) sheet pile protector.
 Source: Courtesy of Associated Pile & Fitting Corporation of New Jersey.

FIGURE 10-28 Driving points (or shoes) cast at the tips of the piles.
 (a) The Prestressed Concrete Institute Standard for 10- to 36-in. piles has these details. HARD-BITE™ or Pluyn Points will protect the vulnerable corners of the H and assure penetration into dense and boulder-filled soils. The H and points prevent damage to the tip of the precast concrete pile.
 (b) H extends 4 ft into concrete; plate in web of H adds to bond strength.
 PILE-TIPS July–August 1983.
 Source: Courtesy of Associated Pile & Fitting Corporation of New Jersey.



(b)

10–13 PROBLEMS

- 10–1. A 12-in. square concrete pile is driven into loose sand to a depth of 30 ft. Soil conditions are shown in Figure 10–29. Find the pile's axial capacity if K is assumed to be 0.7 and the factor of safety is 2.
- 10–2. Rework Problem 10–1, assuming that the groundwater table is located 5 ft below the ground surface.
- 10–3. A 0.5-m-diameter steel pile is driven into dense sand. The pile is driven with the tip closed by a flat plate. The closed-end, steel-pipe pile is filled with concrete after driving. The embedded length of the pile is 20 m. Soil conditions are as shown in Figure 10–30. Determine the design capacity of the pile, using a factor of safety of 2.

FIGURE 10–29

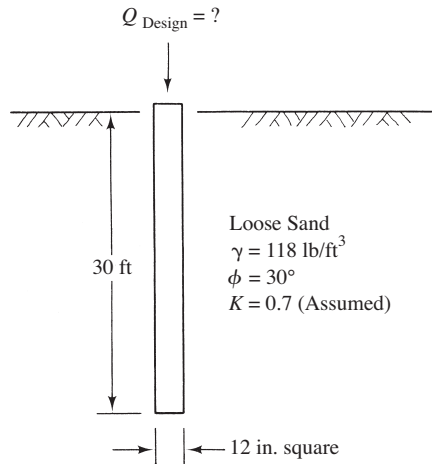
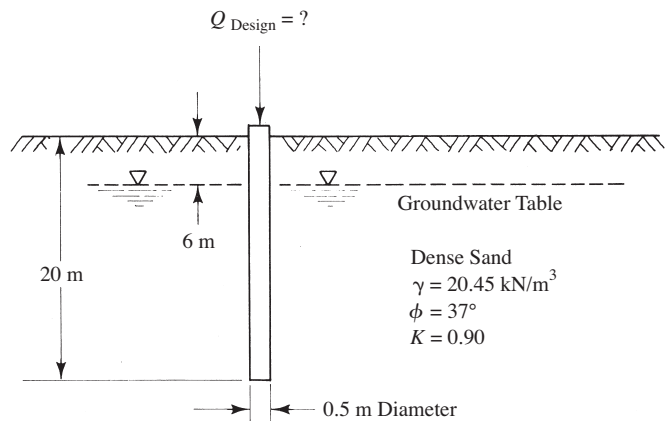


FIGURE 10–30



- 10-4. A 14-in. square concrete pile is driven at a site as shown in Figure 10-31. The embedded length of the pile is 40 ft. Determine the pile's design capacity, using a factor of safety of 2.
- 10-5. A 12-in.-diameter concrete pile is driven at a site as shown in Figure 10-32. What is the pile's design capacity if the factor of safety is 2?

FIGURE 10-31

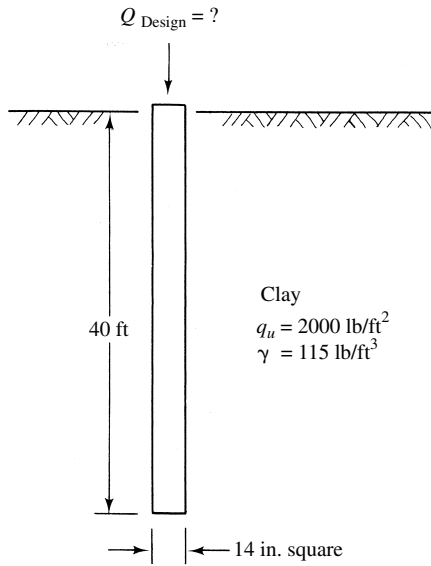


FIGURE 10-32

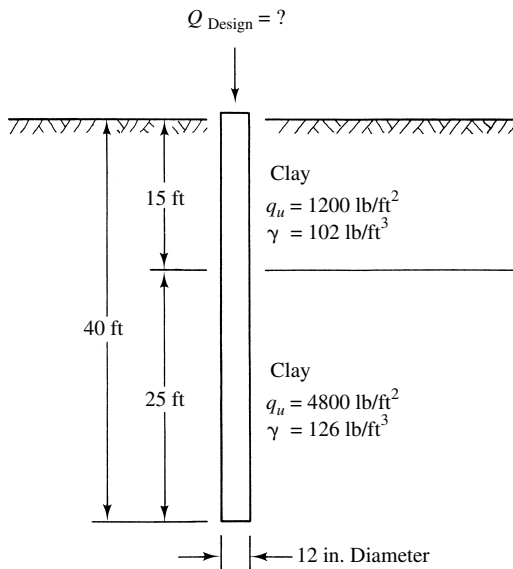
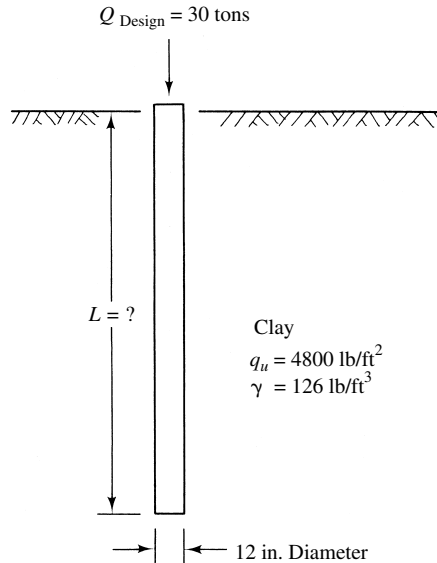
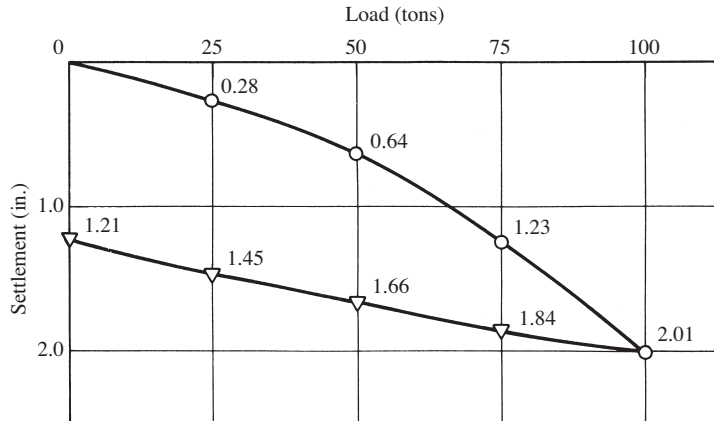


FIGURE 10–33



- 10–6. A 0.5-m-diameter steel pile is driven into a varved clay deposit. The pile is driven with the tip closed by a flat plate. The closed-end, steel-pipe pile is filled with concrete after driving. The embedded length of the pile is 15 m. The clay deposit has a unit weight of 17.92 kN/m^3 and an unconfined compressive strength of 120 kN/m^2 . Determine the design capacity of the pile, using a factor of safety of 2.
- 10–7. Rework Problem 10–6 if the embedded length of the pile is 20 m and the clay deposit's unit weight and unconfined compressive strength are 17.29 kN/m^3 and 96 kN/m^2 , respectively.
- 10–8. A 12-in.-diameter concrete pile is to be driven into a clay soil as shown in Figure 10–33. The pile's design capacity is 30 tons. Determine the pile's required length if the factor of safety is 2.
- 10–9. The design capacity of a steel pile is 250 kN. The pile is driven by a steam hammer with a manufacturer's hammer energy rating of $36 \text{ kN}\cdot\text{m}$. Determine the average penetration of the pile from the last few driving blows. Use the Engineering-News formula.
- 10–10. A steel-pipe pile is to be driven to an allowable load (design load) of 35-tons capacity by an MKT-11B3 double-acting steam hammer. The steel pipe has a net cross-sectional area of 17.12 in.^2 and a length of 45 ft. The Danish pile-driving formula is to be used to control field installation of the piles. How many blows per foot are required for the last foot of penetration?
- 10–11. Rework Problem 10–10 using the Engineering-News formula.
- 10–12. A pile load test produced the settlement and rebound curves given in Figure 10–34. The pile has a 12-in. diameter and is 25 ft long. Determine the allowable load for this pile using a local building code that states the

FIGURE 10–34



following: “The allowable load shall not be more than one-half of that test load that produces a net settlement per ton of test load of not more than 0.01 in., but in no case more than 0.75 in.”

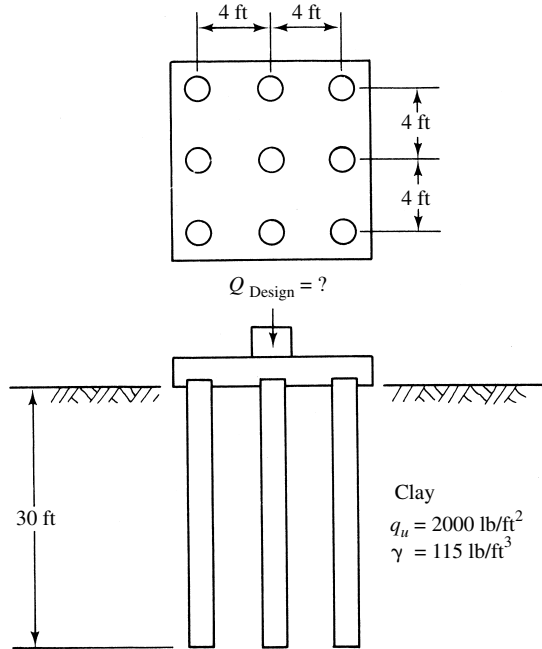
- 10–13. Rework Problem 10–12, except that the local building code is changed to read as follows: “The allowable pile load is taken as one-half of that load that produces a net settlement of not more than 0.01 in./ton of test load, but in no case more than 0.5 in.”
- 10–14. A pile group consists of nine friction piles in clay soil (see Figure 10–35). The diameter of each pile is 16 in., and the embedded length is 30 ft each. Center-to-center pile spacing is 4 ft. Soil conditions are shown in Figure 10–35. Find the pile group’s design capacity if the factor of safety is 2. Use the Converse–Labarre equation.
- 10–15. A concrete pile with a diameter of 0.3 m and length of 20 m was subjected to a pile load test, with the following results:

Load (kN)	Settlement (mm)
250	5.0
500	9.1
750	12.6
1000	16.2
1250	20.0
1500	32.0
1750	48.0
2000	67.1

Determine the allowable load for this pile using the building code cited on page 358.

- 10–16. A nine-pile group consists of 12-in.-diameter friction concrete piles 30 ft long. The piles are driven into clay, the unconfined compressive strength of

FIGURE 10–35



which is 6000 lb/ft^2 and the unit weight of which is 125 lb/ft^3 . Pile spacing is $2\frac{1}{2}$ diameters. Find (a) the block capacity of the pile group, using a factor of safety of 3; (b) the allowable group capacity based on individual pile failure, using a factor of safety of 2 along with the Converse-Labarre equation for pile-group efficiency; and (c) the design capacity of the pile group.

- 10–17. A pile group consists of 12 piles as shown in Figure 10–36. A vertical load of 480 kips acts vertically on point A. Determine the load on piles 2, 4, 7, and 9.
- 10–18. A pile group consists of four friction piles in cohesive soil. Each pile's diameter is 0.4 m, and center-to-center spacing is 1.5 m. The ultimate capacity of each pile is 453 kN. Estimate the design capacity of the pile group, using a factor of safety of 2 and the criteria suggested by Coyle and Sulaiman (Figure 10–20).
- 10–19. A pile group consists of nine friction piles in cohesive soil. Each pile's diameter is 0.3 m, and center-to-center spacing is 1.2 m. The ultimate capacity of each pile is 300 kN. Estimate the design capacity of the pile group, using a factor of safety of 2 and the criteria suggested by Coyle and Sulaiman (Figure 10–20).
- 10–20. The tower shown in Figure 10–37 is subjected to a wind pressure of 25 lb/ft^2 on its projected area. The tower and foundation weigh 320 kips. Determine the maximum and minimum pile reactions for the layout shown.

FIGURE 10-36

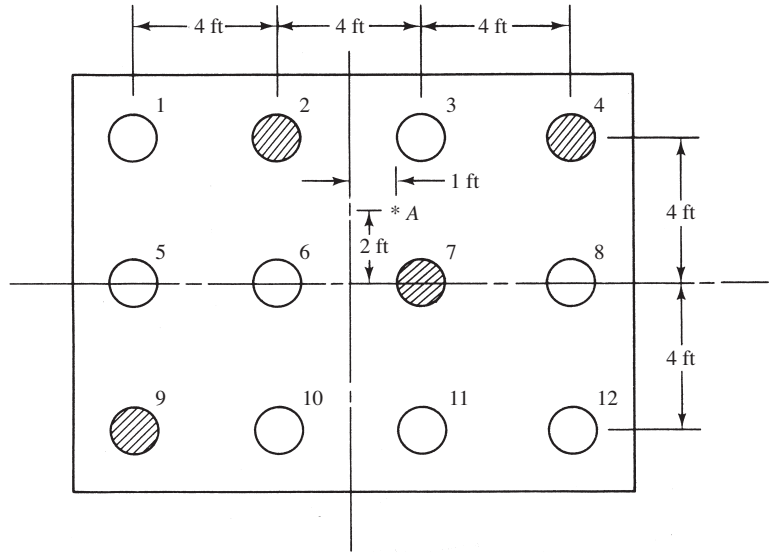
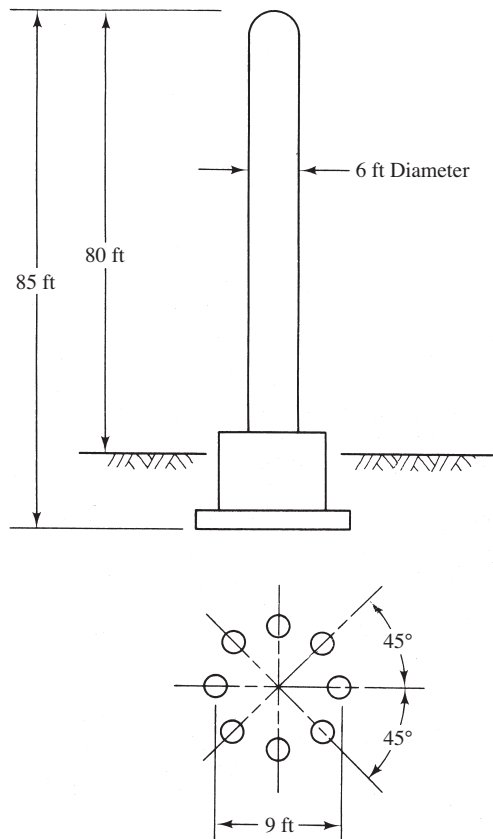


FIGURE 10-37



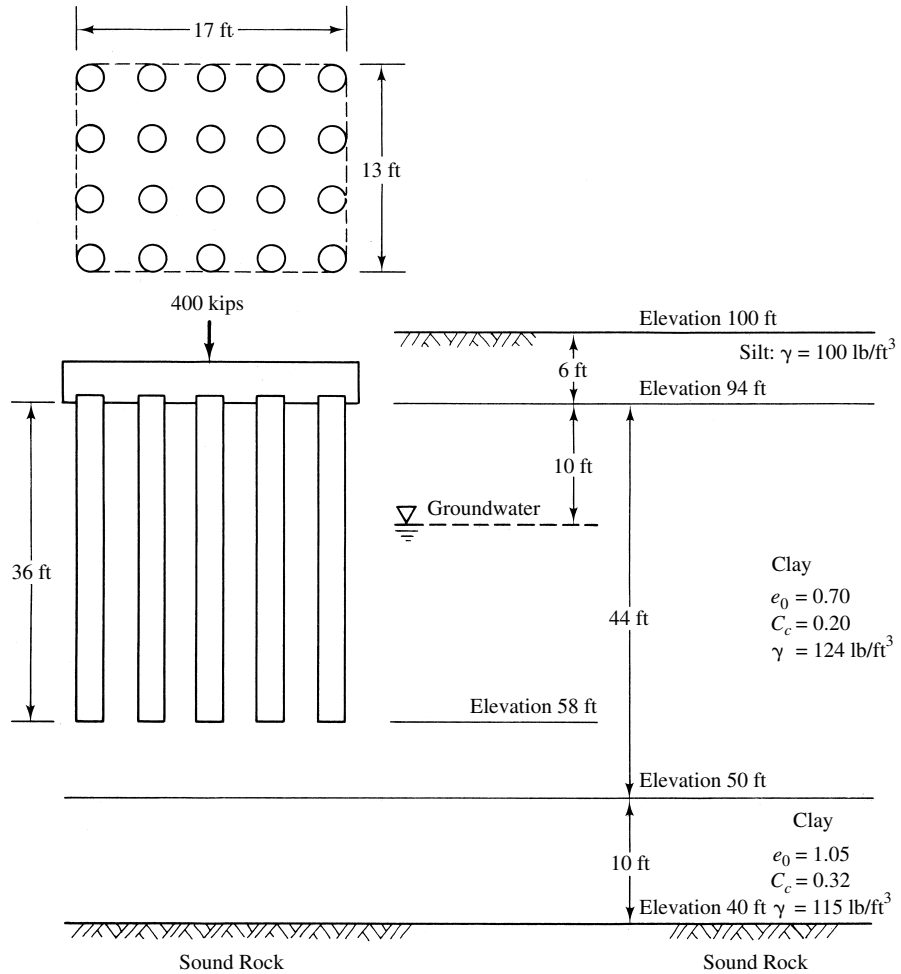


FIGURE 10-38

- 10-21. A group of friction piles in deep clay is shown in Figure 10-38. The total load on the piles reduced by the weight of soil displaced by the foundation is 400 kips. Find the expected total settlement of the pile foundation.