University of Anbar
Engineering College
Civil Engineering Department

CHAPTER SIX

PILE FOUNDATIONS

LECTURE
DR. AHMED H. ABDULKAREEM
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6.1. Introduction

Piles are structural members that are made of steel, concrete, or timber. They are used to build pile foundations, which are deep and which cost more than shallow foundations. Despite the cost, the use of piles often is necessary to ensure structural safety. The following list identifies some of the conditions that require pile foundations (Vesic, 1977):

1. When one or more upper soil layers are highly compressible and too weak to support the load transmitted by the superstructure, piles are used to transmit the load to underlying bedrock or a stronger soil layer, as shown in Figure 9.1a. When bedrock is not encountered at a reasonable depth below the ground surface, piles are used to transmit the structural load to the soil gradually. The resistance to the applied structural load is derived mainly from the frictional resistance developed at the soil–pile interface. (See Figure 9.1b.)

2. When subjected to horizontal forces (see Figure 9.1c), pile foundations resist by bending, while still supporting the vertical load transmitted by the superstructure. This type of situation is generally encountered in the design and construction of earth-retaining structures and foundations of tall structures that are subjected to high wind or to earthquake forces.

3. In many cases, expansive and collapsible soils may be present at the site of a proposed structure. These soils may extend to a great depth below the ground surface. Expansive soils swell and shrink as their moisture content increases and decreases, and the pressure of the swelling can be considerable. If shallow foundations are used in such circumstances, the structure may suffer considerable damage. However, pile foundations may be considered as an alternative when piles are extended beyond the active zone, which is where swelling and shrinking occur. (See Figure 9.1d.) Soils such as loess are collapsible in nature. When the moisture content of these soils increases, their structures may break down. A sudden decrease in the void ratio of soil induces large settlements of structures supported by shallow foundations. In such cases, pile foundations may be used in which the piles are extended into stable soil layers beyond the zone where moisture will change.

4. The foundations of some structures, such as transmission towers, offshore platforms, and basement mats below the water table, are subjected to uplifting forces. Piles are sometimes used for these foundations to resist the uplifting force. (See Figure 9.1e.)

5. Bridge abutments and piers are usually constructed over pile foundations to avoid the loss of bearing capacity that a shallow foundation might suffer because of soil erosion at the ground surface. (See Figure 9.1f.)

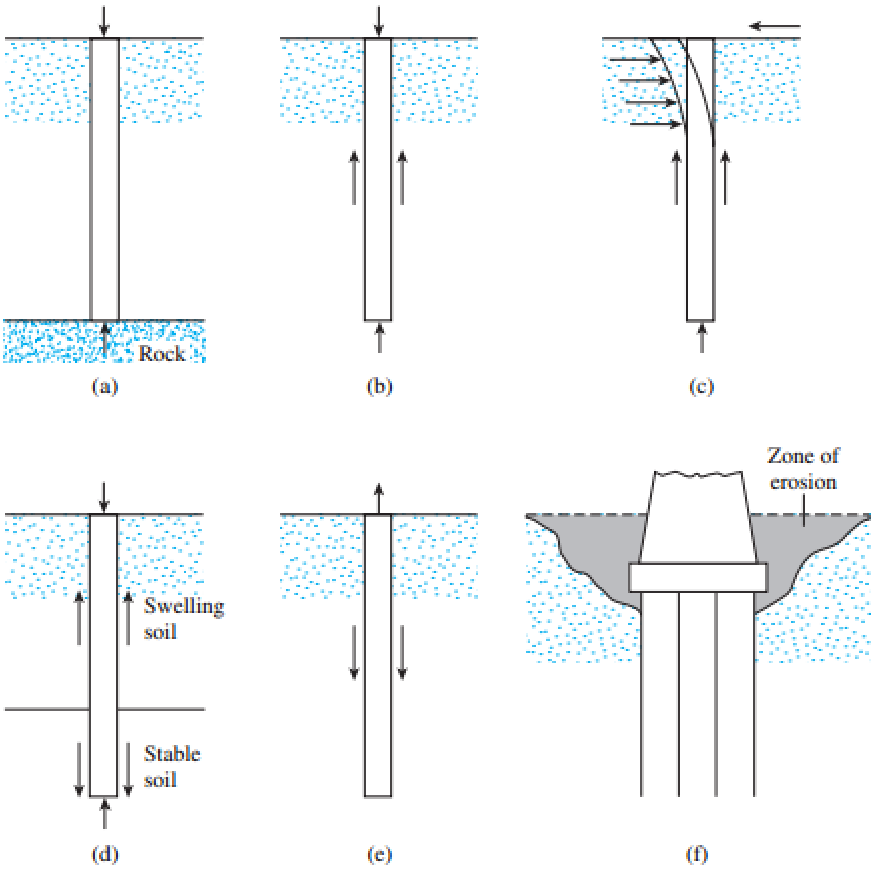


Figure 9.1 Conditions that require the use of pile foundations

9.2 Types of Piles and Their Structural Characteristics

- Different types of piles are used in construction work, depending on the type of load to be carried, the subsoil conditions, and the location of the water table.
- Piles can be divided into the following categories with the general descriptions for conventional steel, concrete, timber, and composite piles.

Steel Piles

- *Steel piles* generally are either *pipe piles* or *rolled steel* H-*section piles.*
- Pipe piles can be driven into the ground with their ends open or closed.
- Wide-flange and I-section steel beams can also be used as piles. However, H-section piles are usually preferred because their web and flange thicknesses are equal. (In wide-flange and I-section beams, the web thicknesses are smaller than the thicknesses of the flange.) Table 9.1 gives the dimensions of some standard H-section steel piles used in the United States.
- Table 9.2 shows selected pipe sections frequency used for piling purposes.
- In many cases, the pipe piles are filled with concrete after they have been driven.
- The allowable structural capacity for steel piles is

$$Q_{all} = A_s f_s \tag{9.1}$$

where

A_s = cross-sectional area of the steel
f_s = allowable stress of steel ($\approx 0.33 - 0.5\, f_y$)

When hard driving conditions are expected, such as driving through dense gravel, shale, or soft rock, steel piles can be fitted with driving points or shoes. Figures 9.2d and 9.2e are diagrams of two types of shoe used for pipe piles.

Here are some general facts about steel piles:

- Usual length: 15 m to 60 m (50 ft to 200 ft)
- Usual load: 300 kN to 1200 kN (67 kip to 265 kip)

- Advantages:
  a. Easy to handle with respect to cutoff and extension to the desired length
  b. Can stand high driving stresses
  c. Can penetrate hard layers such as dense gravel and soft rock
  d. High load-carrying capacity
- Disadvantages:
  a. Relatively costly
  b. High level of noise during pile driving
  c. Subject to corrosion
  d. H-piles may be damaged or deflected from the vertical during driving through hard layers or past major obstructions

Table 9.1a Common H-Pile Sections used in the United States (SI Units)

Designation, size (mm) × weight (kg/m)	Depth d_1 (mm)	Section area ($m^2 \times 10^{-3}$)	Flange and web thickness w (mm)	Flange width d_2 (mm)	Moment of inertia ($m^4 \times 10^{-6}$) I_{xx}	I_{yy}
HP 200 × 53	204	6.84	11.3	207	49.4	16.8
HP 250 × 85	254	10.8	14.4	260	123	42
× 62	246	8.0	10.6	256	87.5	24
HP 310 × 125	312	15.9	17.5	312	271	89
× 110	308	14.1	15.49	310	237	77.5
× 93	303	11.9	13.1	308	197	63.7
× 79	299	10.0	11.05	306	164	62.9
HP 330 × 149	334	19.0	19.45	335	370	123
× 129	329	16.5	16.9	333	314	104
× 109	324	13.9	14.5	330	263	86
× 89	319	11.3	11.7	328	210	69
HP 360 × 174	361	22.2	20.45	378	508	184
× 152	356	19.4	17.91	376	437	158
× 132	351	16.8	15.62	373	374	136
× 108	346	13.8	12.82	371	303	109

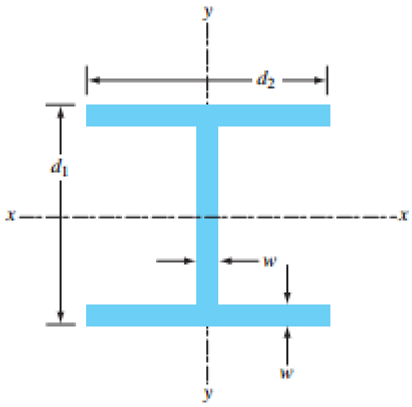


Table 9.2a Selected Pipe Pile Sections (SI Units)

Outside diameter (mm)	Wall thickness (mm)	Area of steel (cm^2)
219	3.17	21.5
	4.78	32.1
	5.56	37.3
	7.92	52.7
254	4.78	37.5
	5.56	43.6
	6.35	49.4
305	4.78	44.9
	5.56	52.3
	6.35	59.7
406	4.78	60.3
	5.56	70.1
	6.35	79.8
457	5.56	80
	6.35	90
	7.92	112
508	5.56	88
	6.35	100
	7.92	125
610	6.35	121
	7.92	150
	9.53	179
	12.70	238

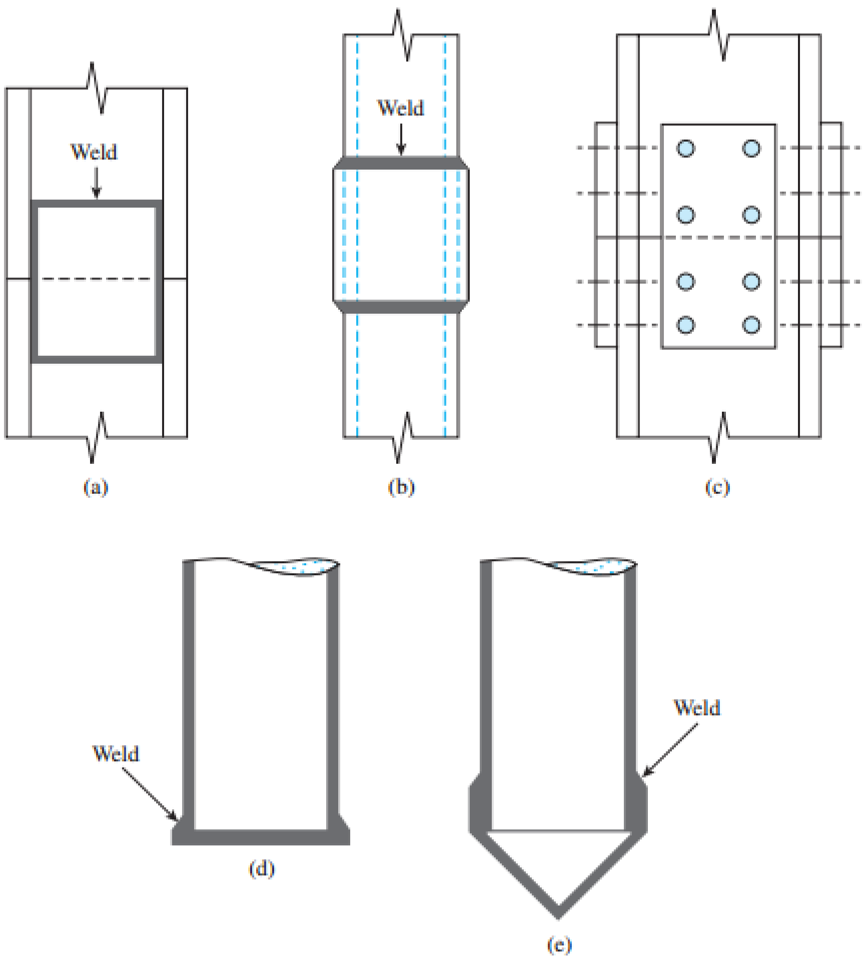


Figure 9.2 Steel piles: (a) splicing of H-pile by welding; (b) splicing of pipe pile by welding; (c) splicing of H-pile by rivets and bolts; (d) flat driving point of pipe pile; (e) conical driving point of pipe pile

Concrete Piles

Concrete piles may be divided into two basic categories:

(a) precast piles and

(b) cast-in-situ piles.

Precast piles can be prepared by using ordinary reinforcement, and they can be square or octagonal in cross section. (See Figure 9.3.) Reinforcement is provided to enable the pile to resist the bending moment developed during pickup and transportation, the vertical load, and the bending moment caused by a lateral load. The piles are cast to desired lengths and cured before being transported to the work sites.

Some general facts about concrete piles are as follows:

- Usual length: 10 m to 15 m (30 ft to 50 ft)
- Usual load: 300 kN to 3000 kN (67 kip to 675 kip)
- Advantages:
  a. Can be subjected to hard driving
  b. Corrosion resistant
  c. Can be easily combined with a concrete superstructure
- Disadvantages:
  a. Difficult to achieve proper cutoff
  b. Difficult to transport

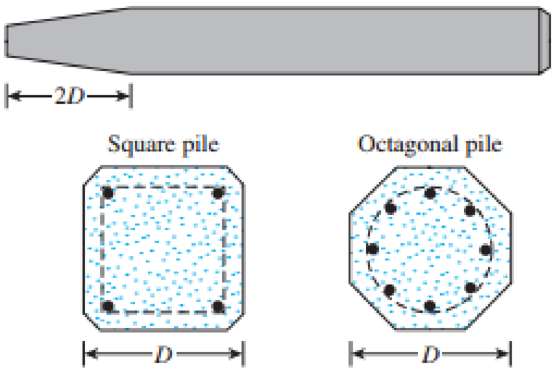


Figure 9.3 Precast piles with ordinary reinforcement

Precast piles can also be prestressed by the use of high-strength steel prestressing cables. The ultimate strength of these cables is about 1800 MN/m2. During casting of the piles, the cables are pretensioned to about

900 to 1300 MN/m2, and concrete is poured around them. After curing, the cables are cut, producing a compressive force on the pile section. Table 9.3 gives additional information about prestressed concrete piles with square and octagonal cross sections.

Some general facts about precast prestressed piles are as follows:

- Usual length: 10 m to 45 m (30 ft to 150 ft)
- Maximum length: 60 m (200 ft)
- Maximum load: 7500 kN to 8500 kN (1700 kip to 1900 kip)

The advantages and disadvantages are the same as those of precast piles.

Cast-in-situ, or *cast-in-place, piles* are built by making a hole in the ground and then filling it with concrete. Various types of cast-in-place concrete piles are currently used in construction, and most of them have been patented by their manufacturers. These piles may be divided into two broad categories: (a) cased and (b) uncased. Both types may have a pedestal at the bottom.

Cased piles are made by driving a steel casing into the ground with the help of a mandrel placed inside the casing. When the pile reaches the proper depth the mandrel is withdrawn and the casing is filled with concrete. Figures 9.4a, 9.4b, 9.4c, and 9.4d show some examples of cased piles without a pedestal. Figure 9.4e shows a cased pile with a pedestal. The pedestal is an expanded concrete bulb that is formed by dropping a hammer on fresh concrete.

Some general facts about cased cast-in-place piles are as follows:

- Usual length: 5 m to 15 m (15 ft to 50 ft)
- Maximum length: 30 m to 40 m (100 ft to 130 ft)

- Usual load: 200 kN to 500 kN (45 kip to 115 kip)
- Approximate maximum load: 800 kN (180 kip)
- Advantages:
  a. Relatively cheap
  b. Allow for inspection before pouring concrete
  c. Easy to extend
- Disadvantages:
  a. Difficult to splice after concreting
  b. Thin casings may be damaged during driving
- Allowable load:

$$Q_{\text{all}} = A_s f_s + A_c f_c \tag{9.2}$$

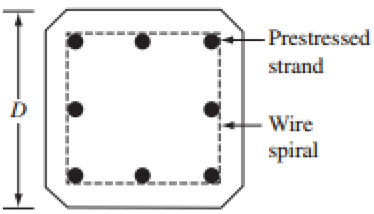
where

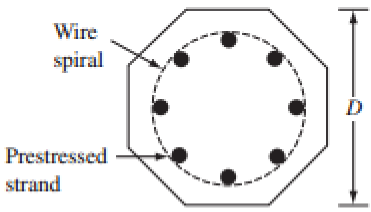
A_s = area of cross section of steel
A_c = area of cross section of concrete
f_s = allowable stress of steel
f_c = allowable stress of concrete

Table 9.3: Typical Prestressed Concrete Pile in Use (SI Units)

Pile shape[a]	*D* (mm)	Area of cross section (cm²)	Perimeter (mm)	Number of strands: 12.7-mm diameter	Number of strands: 11.1-mm diameter	Minimum effective prestress force (kN)	Section modulus ($m^3 \times 10^{-3}$)	Design bearing capacity (kN), Strength of concrete (MN/m²): 34.5	Design bearing capacity (kN), Strength of concrete (MN/m²): 41.4
S	254	645	1016	4	4	312	2.737	556	778
O	254	536	838	4	4	258	1.786	462	555
S	305	929	1219	5	6	449	4.719	801	962
O	305	768	1016	4	5	369	3.097	662	795
S	356	1265	1422	6	8	610	7.489	1091	1310
O	356	1045	1168	5	7	503	4.916	901	1082
S	406	1652	1626	8	11	796	11.192	1425	1710
O	406	1368	1346	7	9	658	7.341	1180	1416
S	457	2090	1829	10	13	1010	15.928	1803	2163
O	457	1729	1524	8	11	836	10.455	1491	1790
S	508	2581	2032	12	16	1245	21.844	2226	2672
O	508	2136	1677	10	14	1032	14.355	1842	2239
S	559	3123	2235	15	20	1508	29.087	2694	3232
O	559	2587	1854	12	16	1250	19.107	2231	2678
S	610	3658	2438	18	23	1793	37.756	3155	3786
O	610	3078	2032	15	19	1486	34.794	2655	3186

[a]S = square section; O = octagonal section





Figures 9.4f and 9.4g are two types of uncased pile, one with a pedestal and the other without. The uncased piles are made by first driving the casing to the desired depth and then filling it with fresh concrete. The casing is then gradually withdrawn. Following are some general facts about uncased cast-in-place concrete piles:

- Usual length: 5 m to 15 m (15 ft to 50 ft)
- Maximum length: 30 m to 40 m (100 ft to 130 ft)
- Usual load: 300 kN to 500 kN (67 kip to 115 kip)
- Approximate maximum load: 700 kN (160 kip)
- Advantages:
  a. Initially economical
  b. Can be finished at any elevation
- Disadvantages:
  a. Voids may be created if concrete is placed rapidly
  b. Difficult to splice after concreting
  c. In soft soils, the sides of the hole may cave in, squeezing the concrete
- Allowable load:

$$Q_{all} = A_c f_c \tag{9.3}$$

where

A_c = area of cross section of concrete
f_c = allowable stress of concrete

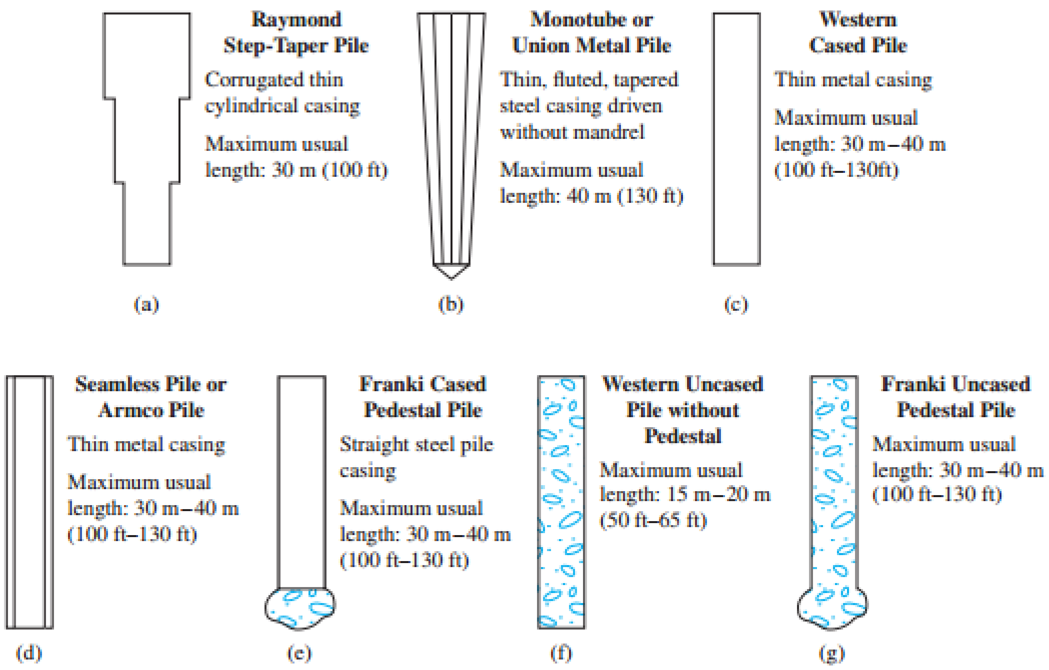


Figure 9.4 Cast-in-place concrete piles

Timber Piles

Timber piles are tree trunks that have had their branches and bark carefully trimmed off. The maximum length of most timber piles is 10 to 20 m (30 to 65 ft). To qualify for use as a pile, the timber should be straight, sound, and without any defects. The American Society of Civil Engineers' *Manual of Practice*, No. 17 (1959), divided timber piles into three classes:

1. *Class A piles* carry heavy loads. The minimum diameter of the butt should be 356 mm (14 in.).
2. *Class B piles* are used to carry medium loads. The minimum butt diameter should be 305 to 330 mm (12 to 13 in.).
3. *Class C piles* are used in temporary construction work. They can be used permanently for structures when the entire pile is below the water table. The minimum butt diameter should be 305 mm (12 in.).

In any case, a pile tip should not have a diameter less than 150 mm.

Timber piles cannot withstand hard driving stress; therefore, the pile capacity is generally limited. Steel shoes may be used to avoid damage at the pile tip (bottom). The tops of timber piles may also be damaged during the driving operation. The crushing of the wooden fibers caused by the impact of the hammer is referred to as *brooming*. To avoid damage to the top of the pile, a metal band or a cap may be used.

Composite Piles

The upper and lower portions of *composite piles* are made of different materials. For example, composite piles may be made of steel and concrete or timber and concrete. Steel-and-concrete piles consist of a lower portion of steel and an upper portion of cast-inplace concrete. This type of pile is used when the length of the pile required for adequate bearing exceeds the capacity of simple cast-in-place concrete piles. Timber-and-concrete piles usually consist of a lower portion of timber pile below the permanent water table and an upper portion of concrete. In any case, forming proper joints between two dissimilar materials is difficult, and for that reason, composite piles are not widely used.

6.4 Estimating Pile Length

Selecting the type of pile to be used and estimating its necessary length are fairly difficult tasks that require good judgment. In addition to being broken down into the classification given in Section 6.2, piles can be divided into three major categories, depending on their lengths and the mechanisms of load transfer to the soil:

(a) point bearing piles,

(b) friction piles, and

(c) compaction piles.

a. Point Bearing Piles

If soil-boring records establish the presence of bedrock or rocklike material at a site within a reasonable depth, piles can be extended to the rock surface. (See Figure 9.6a.) In this case, the ultimate capacity of the piles depends entirely on the load-bearing capacity of the underlying material; thus, the piles are called *point bearing piles*. In most of these cases, the necessary length of the pile can be fairly well established.

If, instead of bedrock, a fairly compact and hard stratum of soil is encountered at a reasonable depth, piles can be extended a few meters into the hard stratum. (See Figure 9.6b.) Piles with pedestals can be constructed on the bed of the hard stratum, and the ultimate pile load may be expressed as

$$Q_u = Q_p + Q_s \tag{9.5}$$

where

Q_p = load carried at the pile point
Q_s = load carried by skin friction developed at the side of the pile (caused by shearing resistance between the soil and the pile)

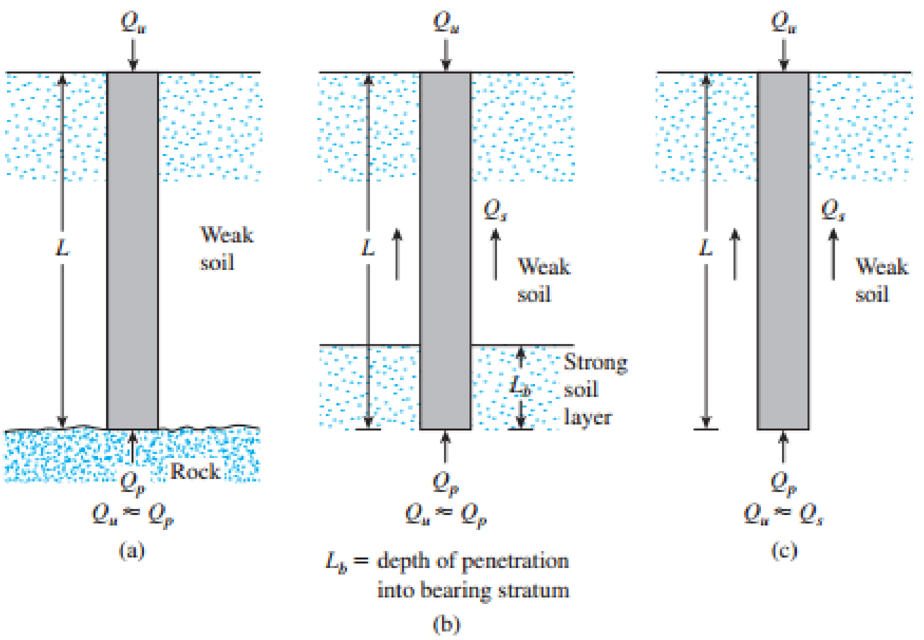


Figure 9.6 (a) and (b) Point bearing piles; (c) friction piles

If Q_s is very small,

$$Q_s \approx Q_p \tag{9.6}$$

In this case, the required pile length may be estimated accurately if proper subsoil exploration records are available.

b- Friction Piles

When no layer of rock or rocklike material is present at a reasonable depth at a site, point bearing piles become very long and uneconomical. In this type of subsoil, piles are driven through the softer material to specified depths. (See Figure 9.6c.) The ultimate load of the piles may be expressed by Eq. (9.5). However, if the value of Q_p is relatively small, then

$$Q_u \approx Q_s \qquad (9.7)$$

These piles are called *friction piles,* because most of their resistance is derived from skin friction. However, the term *friction pile,* although used often in the literature, is a misnomer: In clayey soils, the resistance to applied load is also caused by *adhesion.*

The lengths of friction piles depend on the shear strength of the soil, the applied load, and the pile size. To determine the necessary lengths of these piles, an engineer needs a good understanding of soil–pile interaction, good judgment, and experience.

c- Compaction Piles

Under certain circumstances, piles are driven in granular soils to achieve proper compaction of soil close to the ground surface. These piles are called *compaction piles.* The lengths of compaction piles depend on factors such as

(a) the relative density of the soil before compaction,

(b) the desired relative density of the soil after compaction, and

(c) the required depth of compaction.

These piles are generally short; however, some field tests are necessary to determine a reasonable length.

6.5 Installation of Piles

Most piles are driven into the ground by means of *hammers* or *vibratory drivers*. In special circumstances, piles can also be inserted by *jetting* or *partial augering*. The types of hammer used for pile driving include

(a) the drop hammer,

(b) the single-acting air or steam hammer,

(c) the double-acting and differential air or steam hammer, and

(d) the diesel hammer.

In the driving operation, a cap is attached to the top of the pile. A cushion may be used between the pile and the cap. The cushion has the effect of reducing the impact force and spreading it over a longer time; however, the use of the cushion is optional. A hammer cushion is placed on the pile cap. The hammer drops on the cushion.

Figure 9.7 illustrates various hammers.

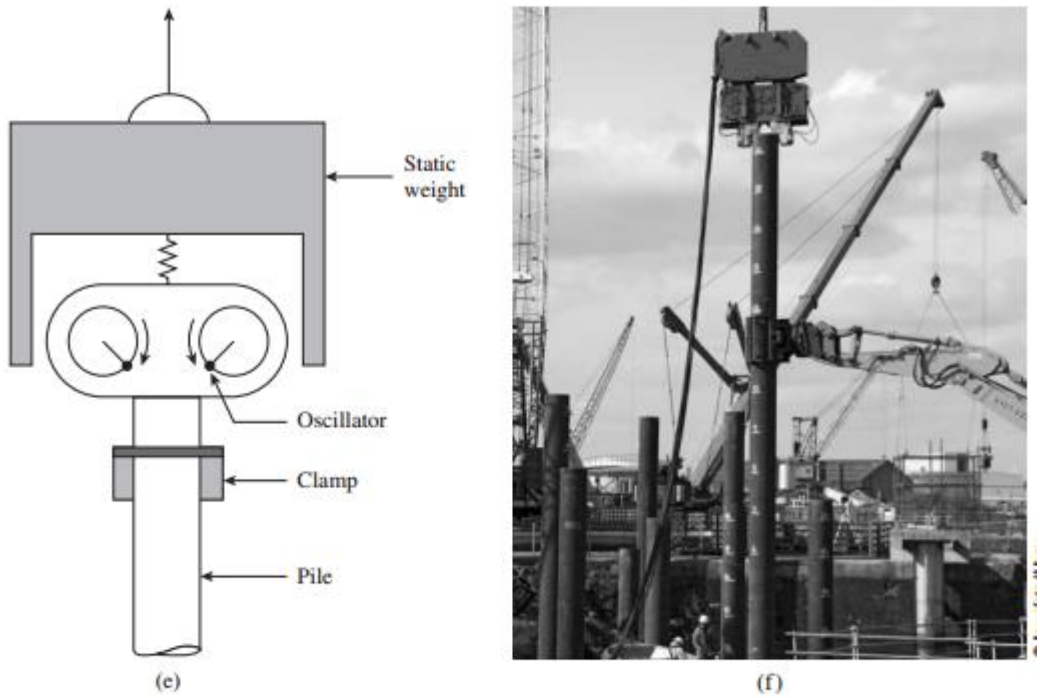


Figure 9.7 (*continued*) Pile-driving equipment: (e) vibratory pile driver; (f) photograph of a vibratory pile driver (*Courtesy of Reinforced Earth Company, Reston, Virginia*)

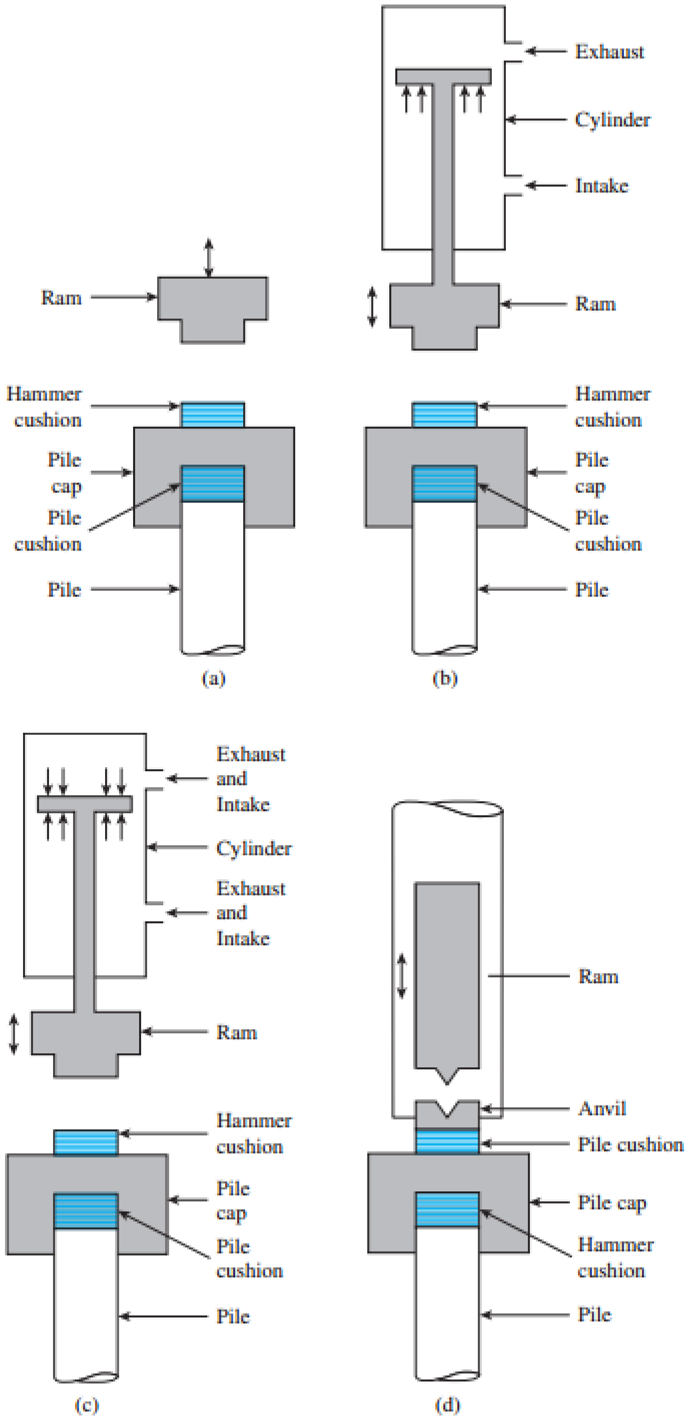


Figure 9.7 Pile-driving equipment: (a) drop hammer; (b) single-acting air or steam hammer; (c) double-acting and differential air or steam hammer; (d) diesel hammer

6.6 Load Transfer Mechanism

The load transfer mechanism from a pile to the soil is complicated. To understand it, consider a pile of length L, as shown in Figure 9.9a. The load on the pile is gradually increased from zero to $Q_{(z=0)}$ at the ground surface. Part of this load will be resisted by the side friction developed along the shaft, Q_1 , and part by the soil below the tip of the pile, Q_2 . Now, how are Q_1 and Q_2 related to the total load? If measurements are made to obtain the load carried by the pile shaft, $Q_{(z)}$, at any depth z, the nature of the variation found will be like that shown in curve 1 of Figure 9.9b. The *frictional resistance per unit area* at any depth z may be determined as

$$f_{(z)} = \frac{\Delta Q_{(z)}}{(p)(\Delta z)} \qquad (9.8)$$

Where

p = perimeter of the cross section of the pile. Figure 9.9c shows the variation of $f_{(z)}$ with depth.

If the load Q at the ground surface is gradually increased, maximum frictional resistance along the pile shaft will be fully mobilized when the relative displacement between the soil and the pile is about 5 to 10 mm ,irrespective of the pile size and length L. However, the maximum point resistance $Q_2 = Q_p$ will not be mobilized until the tip of the pile has moved about 10 to 25% of the pile width (or diameter). (The lower limit applies to driven piles and the upper limit to bored piles). At ultimate load (Figure 9.9d and curve 2 in Figure 9.9b), $Q_{(z=0)} = Q_u$. Thus,

$$Q_1 = Q_s$$

and

$$Q_2 = Q_p$$

The preceding explanation indicates that Q_s (or the unit skin friction, *f*, along the pile shaft) is developed at a *much smaller pile displacement compared with the point resistance*, Q_p .

At ultimate load, the failure surface in the soil at the pile tip (a bearing capacity failure caused by Q_p) is like that shown in Figure 9.9e. Note that pile foundations are deep foundations and that the soil fails mostly in a *punching mode*. That is, a *triangular zone,* I, is developed at the pile tip, which is pushed downward without producing any other visible slip surface. In dense sands and stiff clayey soils, a *radial shear zone,* II, may partially develop.

9.7 Equations for Estimating Pile Capacity

The ultimate load-carrying capacity Q_u of a pile is given by the equation

$$Q_u = Q_p + Q_s \tag{9.9}$$

where

Q_p = load-carrying capacity of the pile point
Q_s = frictional resistance (skin friction) derived from the soil–pile interface (see Figure 9.11)

Numerous published studies cover the determination of the values of Q_p and Q_s . Excellent reviews of many of these investigations have been provided by Vesic (1977), Meyerhof (1976), and Coyle and Castello (1981). These studies afford an insight into the problem of determining the ultimate pile capacity.

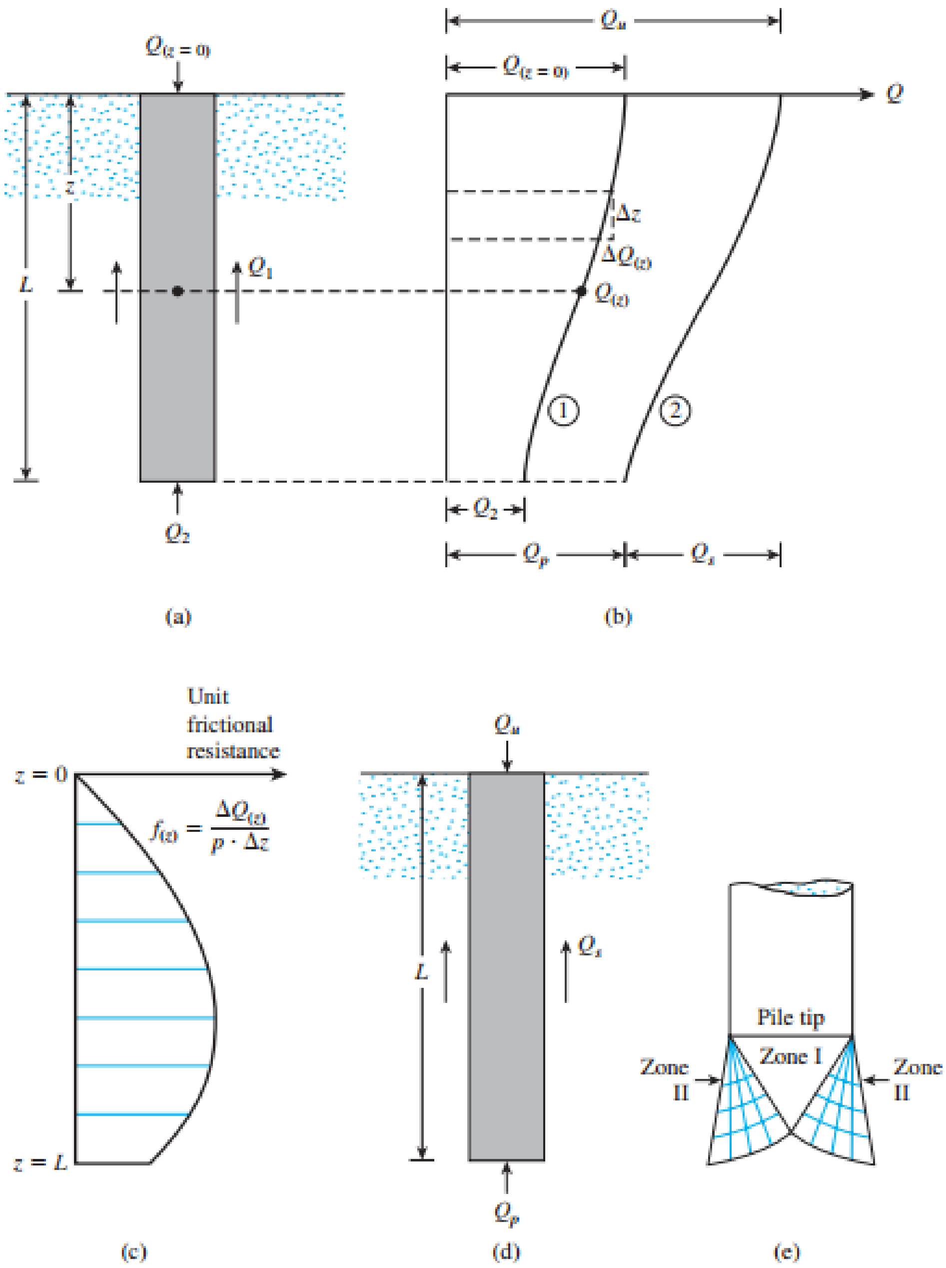


Figure 9.9 Load transfer mechanism for piles

Point Bearing Capacity, Q_p

The ultimate bearing capacity of shallow foundations was discussed in Chapter 3. According to Terzaghi's equations,

$$q_u = 1.3c'N_c + qN_q + 0.4\gamma BN_\gamma \qquad \text{(for shallow square foundations)}$$

and

$$q_u = 1.3c'N_c + qN_q + 0.3\gamma BN_\gamma \qquad \text{(for shallow circular foundations)}$$

Similarly, the general bearing capacity equation for shallow foundations was given in Chapter 4 (for vertical loading) as

$$q_u = c'N_cF_{cs}F_{cd} + qN_qF_{qs}F_{qd} + \tfrac{1}{2}\gamma BN_\gamma F_{\gamma s}F_{\gamma d}$$

Hence, in general, the ultimate load-bearing capacity may be expressed as

$$q_u = c'N_c^* + qN_q^* + \gamma BN_\gamma^* \tag{9.10}$$

where N_c^*, N_q^*, and N_γ^* are the bearing capacity factors that include the necessary shape and depth factors.

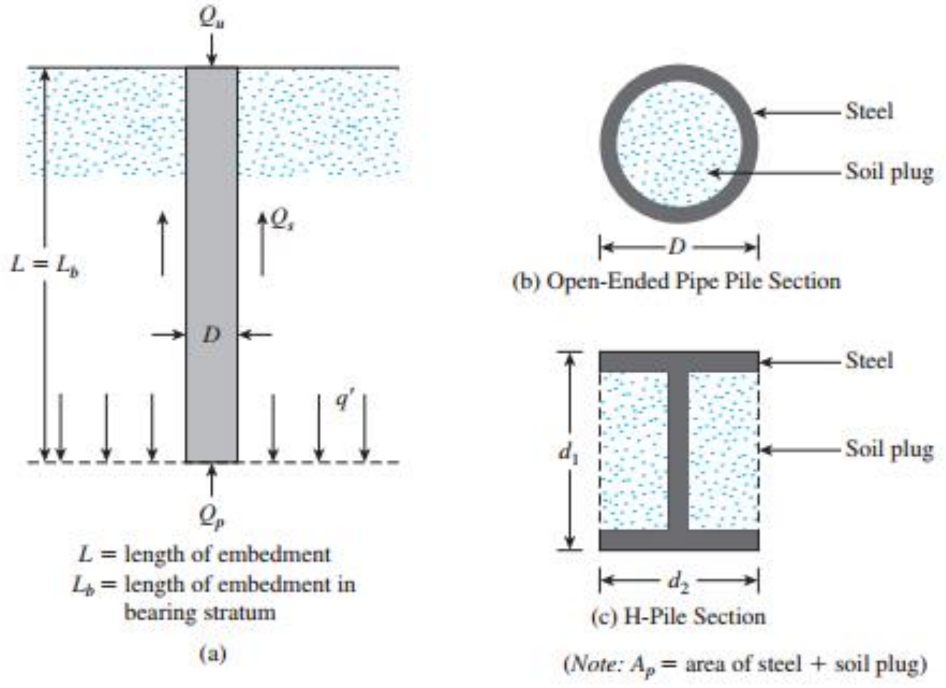


Figure 9.11 Ultimate load-carrying capacity of pile

Pile foundations are deep. However, the ultimate resistance per unit area developed at the pile tip, q_p, may be expressed by an equation similar in form to Eq. (9.10), although the values of N^*c, N^*q, and $N^*\gamma$ will change. The notation used in this chapter for the width of a pile is *D*. Hence, substituting *D* for *B* in Eq. (9.10) gives

$$q_u = q_p = c'N_c^* + qN_q^* + \gamma DN_\gamma^* \tag{9.11}$$

Because the width D of a pile is relatively small, the term γDN_γ^* may be dropped from the right side of the preceding equation without introducing a serious error; thus, we have

$$q_p = c'N_c^* + q'N_q^* \tag{9.12}$$

Note that the term q has been replaced by q' in Eq. (9.12), to signify effective vertical stress. Thus, the point bearing of piles is

$$Q_p = A_p q_p = A_p(c'N_c^* + q'N_q^*) \tag{9.13}$$

where

A_p = area of pile tip
c' = cohesion of the soil supporting the pile tip
q_p = unit point resistance
q' = effective vertical stress at the level of the pile tip
N_c^*, N_q^* = the bearing capacity factors

Frictional Resistance, Q_s

The frictional, or skin, resistance of a pile may be written as

$$Q_s = \Sigma\, p\, \Delta L f \tag{9.14}$$

where

p = perimeter of the pile section
ΔL = incremental pile length over which p and f are taken to be constant
f = unit friction resistance at any depth z

The various methods for estimating Q_p and Q_s are discussed in the next several sections. It needs to be reemphasized that, in the field, for full mobilization of the point resistance (Q_p), the pile tip must go through a displacement of 10 to 25% of the pile width (or diameter).

Allowable Load, Q_{all}

After the total ultimate load-carrying capacity of a pile has been determined by summing the point bearing capacity and the frictional (or skin) resistance, a reasonable factor of safety should be used to obtain the total allowable load for each pile, or

$$Q_{all} = \frac{Q_u}{FS}$$

where

Q_{all} = allowable load-carrying capacity for each pile
FS = factor of safety

The factor of safety generally used ranges from 2.5 to 4, depending on the uncertainties surrounding the calculation of ultimate load.

6.8 Meyerhof's Method for Estimating Q_p

Sand

The point bearing capacity, q_p , of a pile in sand generally increases with the depth of embedment in the bearing stratum and reaches a maximum value at an embedment ratio of $L_b/D = (L_b/D)_{cr}$. Note that in a homogeneous soil L_b is equal to the actual embedment length of the pile, L. However, where a pile has penetrated into a bearing stratum, $L_b < L$. Beyond the critical embedment ratio, $(L_b/D)_{cr}$, the value of q_p remains constant $(q_p = q_l)$. That is, as shown in Figure 9.12 for the case of a homogeneous soil, $L = L_b$.

For piles in sand, $c' = 0$, and Eq. (9.13) simplifies to

$$Q_p = A_p q_p = A_p q' N_q^* \qquad (9.15)$$

The variation of N^*q with soil friction angle ϕ' is shown in Figure 9.13. The interpolated values of N^*q for various friction angles are also given in Table 9.5. However, Q_p should not exceed the limiting value $A_p q_l$; that is,

$$Q_p = A_p q' N_q^* \leq A_p q_l \qquad (9.16)$$

Figure 9.13 Variation of the maximum values of N_q^* with soil friction angle ϕ' (Based on Meyerhof, G. G. (1976). "Bearing Capacity and Settlement of Pile Foundations," *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, Vol. 102, No. GT3, pp. 197–228.)

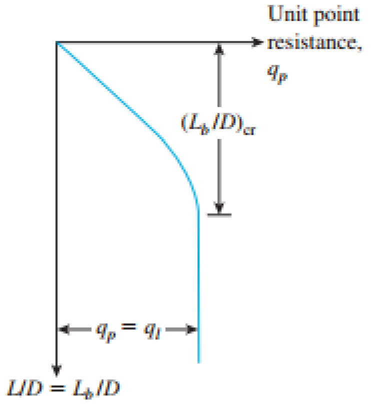


Figure 9.12 Nature of variation of unit point resistance in a homogeneous sand

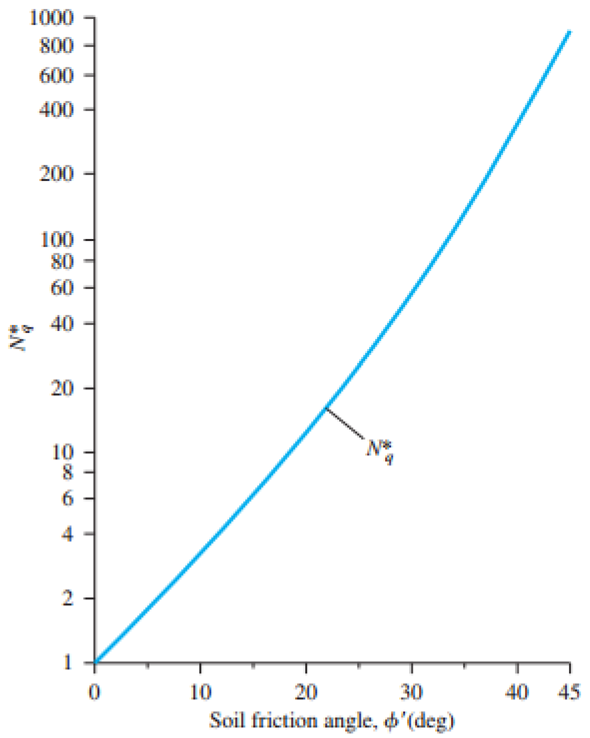


Table 9.5 Interpolated Values of N_q^* Based on Meyerhof's Theory

Soil friction angle, ϕ (deg)	N_q^*
20	12.4
21	13.8
22	15.5
23	17.9
24	21.4
25	26.0
26	29.5
27	34.0
28	39.7
29	46.5
30	56.7
31	68.2
32	81.0
33	96.0
34	115.0
35	143.0
36	168.0
37	194.0
38	231.0
39	276.0
40	346.0
41	420.0
42	525.0
43	650.0
44	780.0
45	930.0

The limiting point resistance is

$$q_l = 0.5\, p_a N_q^* \tan \phi' \quad (9.17)$$

where

p_a = atmospheric pressure (=100 kN/m^2 or 2000 lb/ft^2)
ϕ' = effective soil friction angle of the bearing stratum

Clay ($\phi = 0$)

For piles in *saturated clays* under undrained conditions ($\phi = 0$), the net ultimate load can be given as

$$Q_p \approx N_c^* c_u A_p = 9 c_u A_p \quad (9.18)$$

where c_u = undrained cohesion of the soil below the tip of the pile.

6.9 Coyle and Castello's Method for Estimating *Qp* in Sand

Coyle and Castello (1981) analyzed 24 large-scale field load tests of driven piles in sand. On the basis of the test results, they suggested that, in sand,

$$Q_p = q' N_q^* A_p \quad (9.36)$$

where

q' = effective vertical stress at the pile tip
N_q^* = bearing capacity factor

Figure 9.15 shows the variation of N_q^* with L/D and the soil friction angle ϕ'.

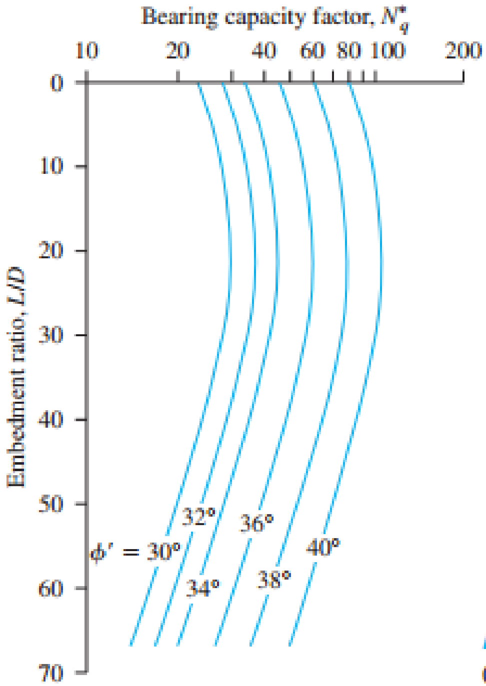


Figure 9.15 Variation of N_q^* with L/D (Based on Coyle and Costello, 1981)

Example 6.1

Consider a 20-m-long concrete pile with a cross section of 0.407 m × 0.407 m fully embedded in sand. For the sand, given: unit weight, $\gamma = 18$ kN/m^3; and soil friction angle, $\phi' = 35°$. Estimate the ultimate point Q_p with each of the following:

a. Meyerhof's method
c. The method of Coyle and Castello
d. Based on the results of parts a, b, and c, adopt a value for Q_p

Solution

Part a

From Eqs. (9.16) and (9.17),

$$Q_p = A_p q' N_q^* \leq A_p(0.5 p_a N_q^* \tan \phi')$$

For $\phi' = 35°$, the value of $N_q^* \approx 143$ (Table 9.5). Also, $q' = \gamma L = (18)(20) = 360$ kN/m^2. Thus,

$$A_p q' N_q^* = (0.407 \times 0.407)(360)(143) \approx 8528 \text{ kN}$$

Again,

$$A_p(0.5 p_a N_q^* \tan \phi') = (0.407 \times 0.407)[(0.5)(100)(143)(\tan 35)] \approx 829 \text{ kN}$$

Hence, $Q_p =$ **829 kN**.

Part c

From Eq. (9.36),

$$Q_p = q'N_q^* A_p$$

$$\frac{L}{D} = \frac{20}{0.407} = 49.1$$

For $\phi' = 35°$ and $L/D = 49.1$, the value of N_q^* is about 34 (Figure 9.15). Thus,

$$Q_p = q'N_q^* A_p = (20 \times 18)(34)(0.407 \times 0.407) \approx \mathbf{2028\ kN}$$

Part d

It appears that Q_p obtained from the method of Coyle and Castello is too large. Thus, the average of the results from parts a and b is

$$\frac{829 + 1731}{2} = 1280 \text{ kN}$$

$$\text{Use } Q_p = \mathbf{1280\ kN}.$$

Example 6.2

Consider a pipe pile (flat driving point—see Figure 9.2d) having an outside diameter of 457 mm. The embedded length of the pile in layered saturated clay is 20 m. The following are the details of the subsoil:

Depth from ground surface (m)	Saturated unit weight, γ(kN/m^3)	c_u(kN/m^2)
0–3	16	25
3–10	17	40
10–30	18	90

The groundwater table is located at a depth of 3 m from the ground surface. Estimate Q_p by using

a. Meyerhof's method

Solution

Part a

From Eq. (9.18),

$$Q_p = 9c_u A_p$$

The tip of the pile is resting on a clay with $c_u = 90$ kN/m^2. So,

$$Q_p = (9)(90)\left[\left(\frac{\pi}{4}\right)\left(\frac{457}{1000}\right)^2\right] = \mathbf{132.9\ kN}$$

6.11 Correlations for Calculating Q_p with SPT and CPT Results in Granular Soil

On the basis of field observations, Meyerhof (1976) also suggested that the ultimate point resistance q_p in a homogeneous granular soil ($L = L_b$) may be obtained from standard penetration numbers as

$$q_p = 0.4 p_a N_{60} \frac{L}{D} \leq 4 p_a N_{60} \quad (9.37)$$

where

N_{60} = the average value of the standard penetration number near the pile point (about $10D$ above and $4D$ below the pile point)

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

Briaud et al. (1985) suggested the following correlation for q_p in granular soil with the standard penetration resistance N_{60}.

$$q_p = 19.7 p_a (N_{60})^{0.36} \quad (9.38)$$

Meyerhof (1956) also suggested that

$$q_p \approx q_c \quad (9.39)$$

where q_c = cone penetration resistance.

Example 6.3

Consider a concrete pile that is 0.305 m × 0.305 m in cross section in sand. The pile is 12 m long. The following are the variations of N_{60} with depth.

Depth below ground surface (m)	N_{60}
1.5	8
3.0	10
4.5	9
6.0	12
7.5	14
9.0	18
10.5	11
12.0	17
13.5	20
15.0	28
16.5	29
18.0	32
19.5	30
21.0	27

a. Estimate Q_p using Eq. (9.37).
b. Estimate Q_p using Eq. (9.38).

Solution

Part a

The tip of the pile is 12 m below the ground surface. For the pile, $D = 0.305$ m. The average of N_{60} $10D$ above and about $5D$ below the pile tip is

$$N_{60} = \frac{18 + 11 + 17 + 20}{4} = 16.5 \approx 17$$

From Eq. (9.37)

$$Q_p = A_p(q_p) = A_p\left[0.4p_aN_{60}\left(\frac{L}{D}\right)\right] \le A_p(4p_aN_{60})$$

$$A_p\left[0.4p_aN_{60}\left(\frac{L}{D}\right)\right] = (0.305 \times 0.305)\left[(0.4)(100)(17)\left(\frac{12}{0.305}\right)\right] = 2488.8 \text{ kN}$$

$$A_p(4p_aN_{60}) = (0.305 \times 0.305)[(4)(100)(17)] = 632.6 \text{ kN} \approx 633 \text{ kN}$$

Thus, Q_p = **633 kN**

Part b

From Eq. (9.38),

$$Q_p = A_pq_p = A_p[19.7p_a(N_{60})^{0.36}] = (0.305 \times 0.305)[(19.7)(100)(17)^{0.36}]$$

$$= \mathbf{508.2 \text{ kN}}$$

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Figure 3.5 A small enclosure with steel sheet piles for an excavation work (*Courtesy of N. Sivakugan, James Cook University*, *Australia*)

Table 3.1 Properties of Some Sheet-Pile Sections Production by Bethlehem Steel Corporation

Example 3.4: