Pile foundation

Pile foundations are used in the following conditions:

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. (Figure 9.1f.)

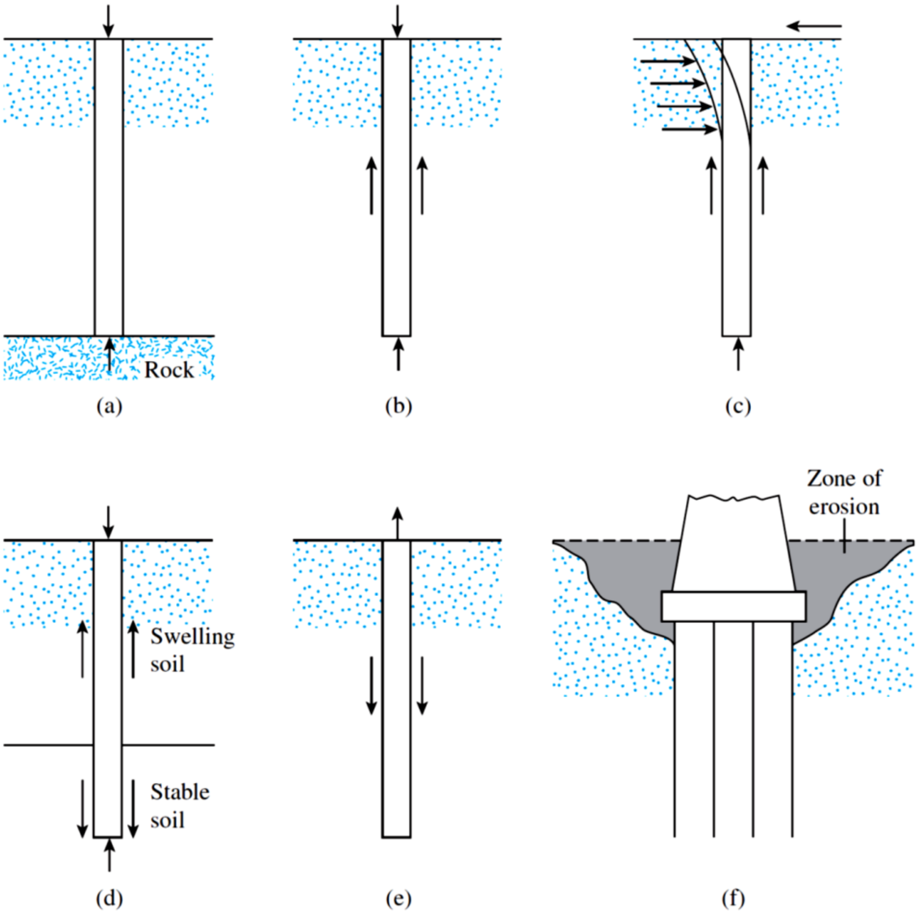


Figure 9.1 Conditions that require the use of pile foundations

Types of Piles Materials and Installation

Concrete piles

Several types of concrete piles are commonly used; these include cast-in-place concrete piles, precast concrete piles. Cast –in-place concrete piles are formed by driven a cylindrical steel shell into the ground to the desired length and then filling the cavity of the shell by fluid concrete. Various types of cast-in-place concrete piles are currently used in construction. 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. Figure 9.4d shows 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 Precast concrete piles usually have square or circular or octagonal cross section and are fabricated in construction yard from reinforced or prestressed concrete.

Advantages of concrete piles:

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

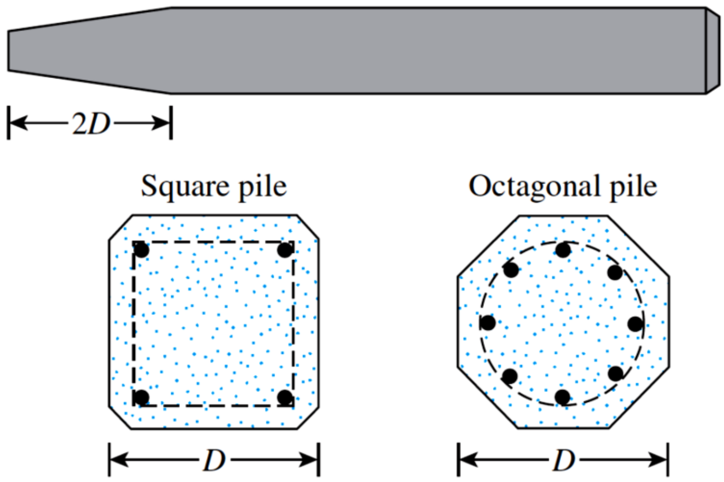


Figure 9.3 Precast piles with ordinary reinforcement

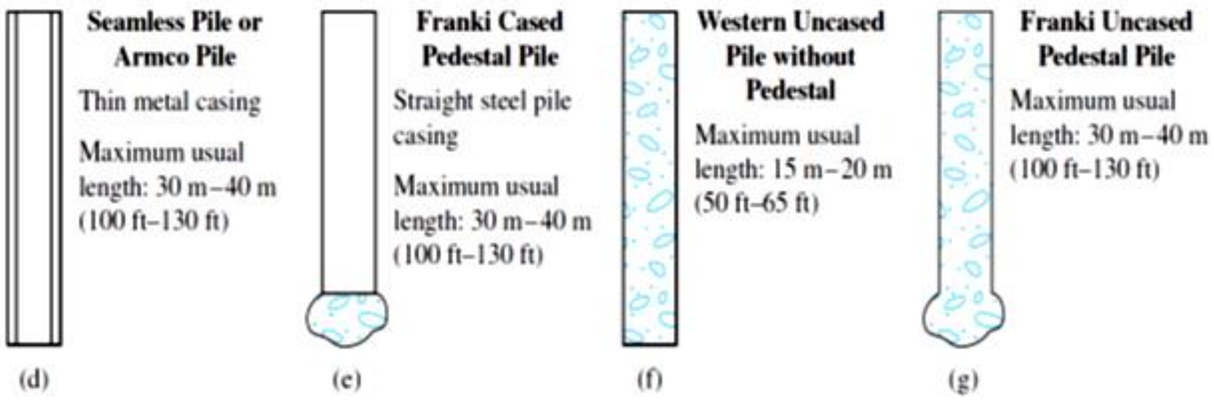


Figure 9.4 Cast-in-place concrete piles

Continuous Flight Auger (CFA) Piles

The continuous flight auger (CFA) piles are also referred to as auger-cast, auger-cast-in place, and auger-pressure grout piles. CFA piles are constructed by using continuous flight augers and by drilling to the final depth in one continuous process. When the drilling to the final depth is complete, the auger is gradually withdrawn as concrete or sand/cement grout is pumped into the hole through the hollow center of the auger pipe to the base of the auger. Reinforcement, if needed, can be placed in CFA piles immediately after the withdrawal of the auger. The reinforcement is usually confined to the top 10 to 15 m of the pile.

In general, CFA piles are usually 0.3 to 0.9 m in diameter with a length up to about 30 m. In the United States, smaller diameter piles [i.e., 0.3 to 0.5 m] are generally used. However, piles with larger diameters [up to about 1.5 m] have been used. Typical center-to-center pile spacing is kept at 3 to 5 pile diameters. **Advantages of CFA piles are:**

a. Noise and vibration during construction are minimized.

b. Eliminates splicing and cutoff.

- **Disadvantages:**

a. Soil spoils need collection and disposal.

Steel Piles

Steel pile come in various shapes and sizes and include cylindrical seamless pipe, tapered and H –piles which is rolled steel sections, concrete-filled steel pile can be done by replacing the soil inside the tube by concrete to increase the load capacity.

Timber Piles

Timber piles have been used since ancient times with a common length about 12 meters.

Pile Installation

Piles can be installed in a predrilled hole (bored piles or drilled shafts) by drilling a hole and either inserting a pile into it or, more commonly, filling the cavity with concrete, which produces a pile upon hardening.

Alternatively, the piles can be driven into the ground (driven piles). Driving can be done by:

1. Driving with a steady succession of blows on the top of the pile using a pile hammer. This produces both considerable noise and local vibrations, which may be disallowed by local codes or environmental agencies and, of course, may damage adjacent property.
2. Driving using a vibratory device attached to the top of the pile. This method is usually relatively quiet, and driving vibrations may not be excessive. The Method is more applicable in deposits with little cohesion.
3. Jacking the pile. This technique is more applicable for short stiff members.

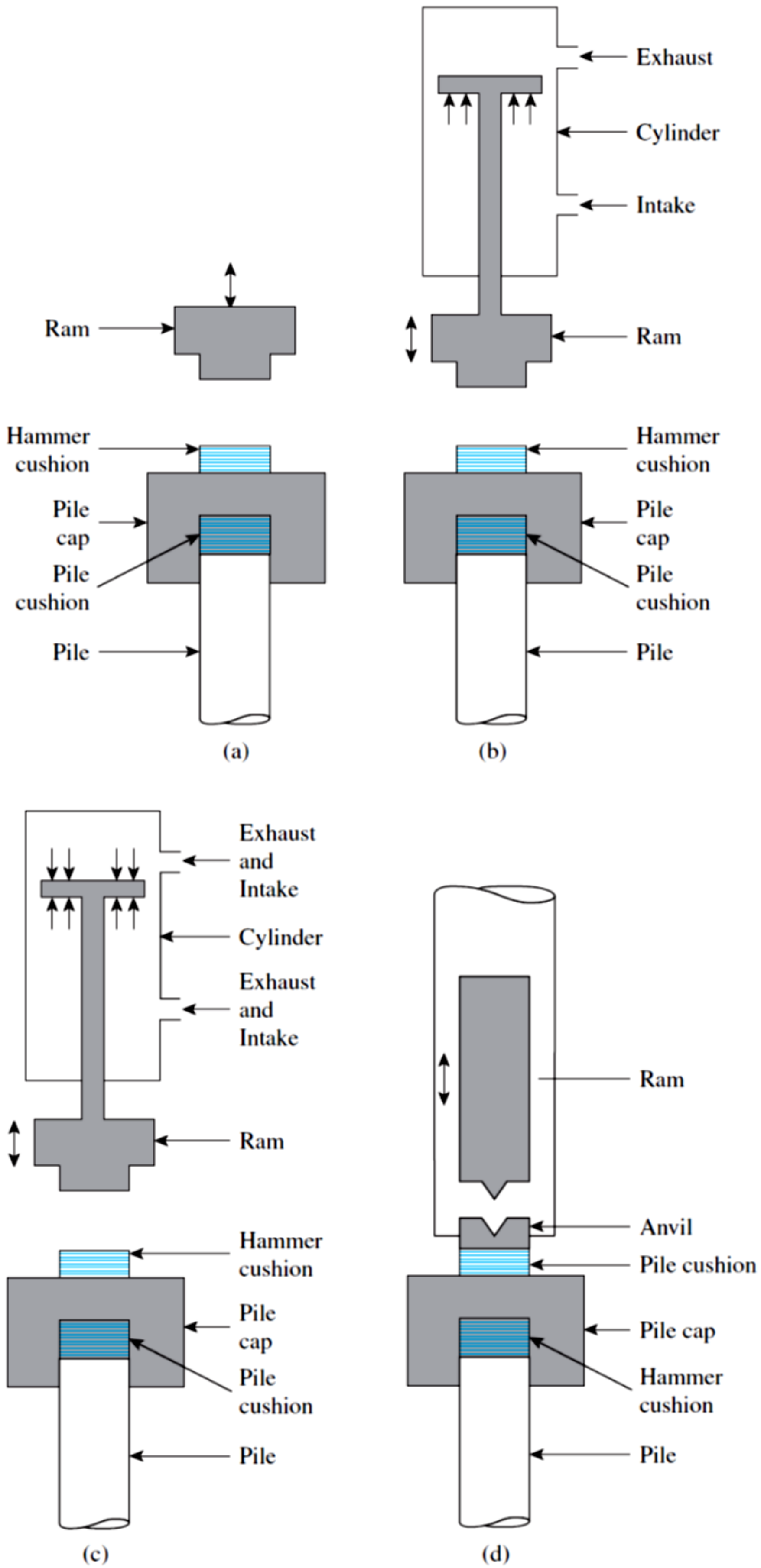


Figure 9.7 Pile-driving equipment: (a) drop hammer; (b) single-acting air or steam hammer; (c) double-acting and differential air or steam hammer; (d) diesel hammer

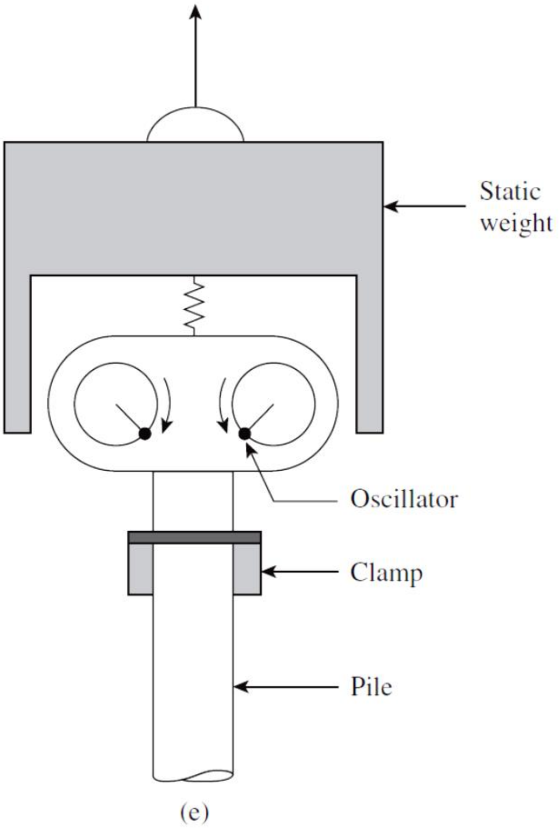


Figure 9.7 (*continued*) Pile-driving equipment: (e) vibratory pile driver; (f) photograph of a vibratory pile driver (*Courtesy of Reinforced Earth Company, Reston, Virginia*)

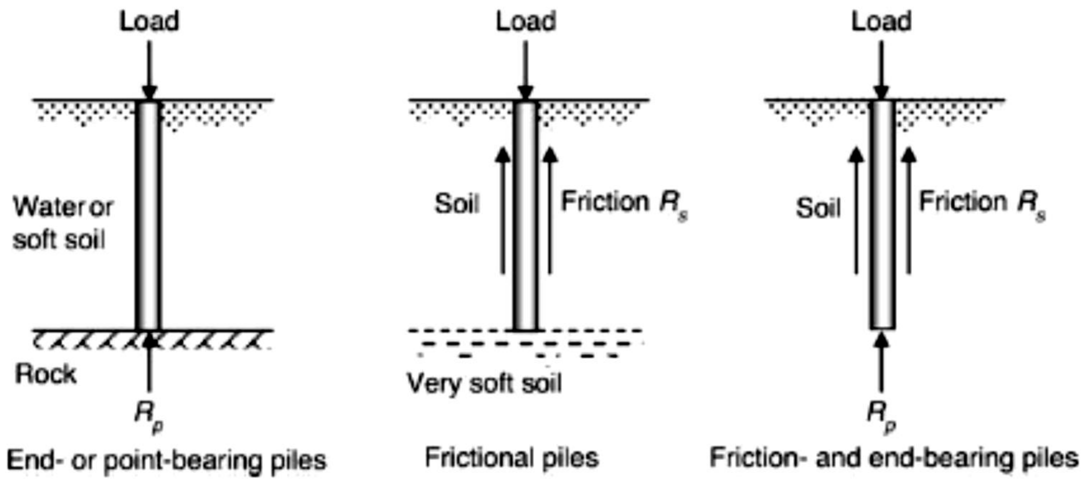
Axial Capacity of Piles in Compression

Axial capacity of piles primarily depends on how and where the applied loads are transferred into the ground. Based on the location of the load transfer in deep foundations, they can be classified as follows:

1. End- or point-bearing piles: The load is primarily distributed at the tip or base of the pile.

2. Frictional piles: The load is distributed primarily along the length of the pile through friction between the pile material and the surrounding soil.

3. Combination of friction and end bearing: The load is distributed both through friction along the length of the pile and at the tip or base of the pile.



$$P_{ult} = P_p + P_s$$

Pile in Cohesionless Soil

1. Point Capacity

If we incorporate the effect of shape and depth in determination of the N factors, the equation for bearing capacity of shallow foundations may be modified for deep foundations after neglecting the third part because of the small diameter or width of the piles as:

$$q_{ult} = c\, N_c^* + \bar{q} N_q^*$$

$$P_{pu} = (c\, N_c^* + \bar{q} N_q^*)\, A_p$$

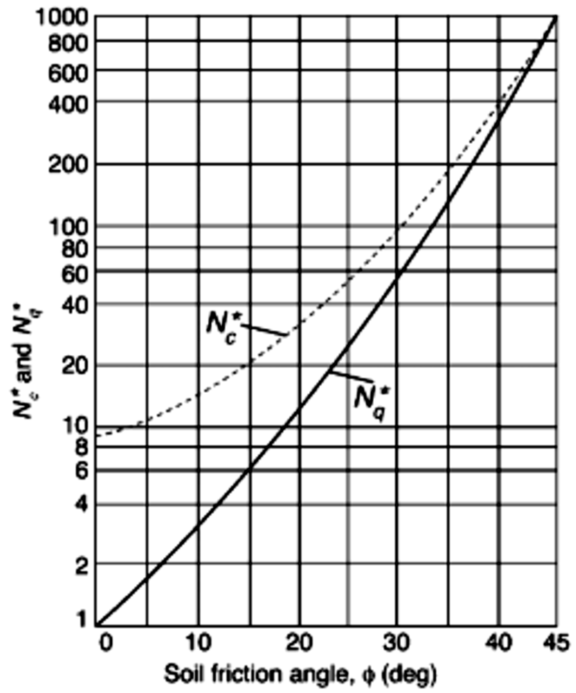


FIGURE 5.16 Meyerhof (1976) bearing capacity factors N_c^* and N_q^* (adapted from Das 1999).

Meyerhof Method Cohesionless soil

$$P_{pu} = \bar{q} N_q^* \ A_p \ < (50\ N_q^*\ tan\emptyset\ )\ A_p\ kN$$

2. Skin friction Capacity

Field studies have shown that the unit frictional resistance of piles embedded in **cohesionless soils** increases with depth. However, beyond a certain depth, the unit frictional resistance remains more or less constant, as illustrated; this depth, beyond which the unit frictional resistance does not increase, is called the critical depth and has been observed to vary between 15 to 20 times the pile diameter.

$$\boldsymbol{P_s = \sum A_s\, f_s}$$

Where:

$\boldsymbol{A_s}$ = effective pile surface area on which $\boldsymbol{f_s}$ acts

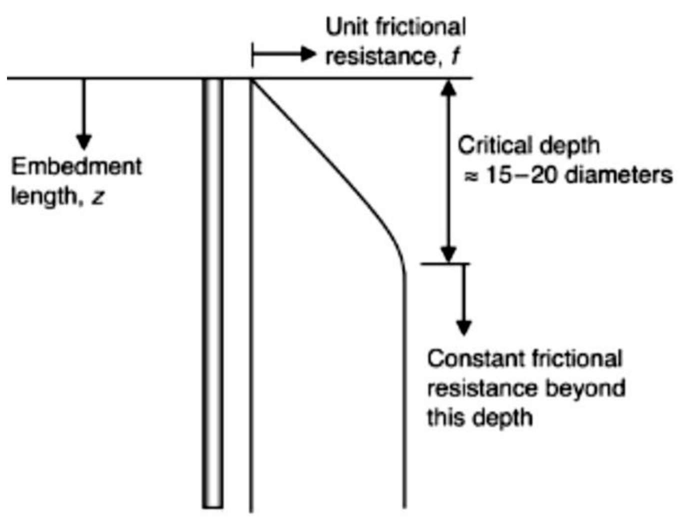
Skin resistance $\boldsymbol{f_s = K\, \sigma'_v\, tan\delta}$

$\boldsymbol{K = K_0}$ Bored or jetted piles

$\boldsymbol{K = 1.4\, K_0}$ Low-displacement driven piles

$\boldsymbol{K = 1.8 K_0}$ High-displacement driven piles

where $\boldsymbol{K_0 = 1 - sin\phi}$ for sands.



Example:
A concrete pile is 15 m (L) long and 0.4x0.4 m in cross section, the pile is fully embedded in sand for which γ = 15.5 kN/m^3, and ϕ=30°. Calculate;
1. The ultimate point load of the pile?
2. The frictional resistance force if K=1.3 and friction angle between pile and soil δ= 0.8ϕ?
3. The allowable pile load, FS=4?

Solution:

1.

Using Meyerhof Method

$$P_{pu} = \bar{q} N_q^* \ A_p$$

From figure, for =30° N_q^* = 55

P_{pu} = 15.5 x 15 x 55 (0.4x0.4) = 2046 kN

Check with max. limit $(50 \ N_q^* \ tan\emptyset \ ) \ A_p$ = 50 x 55 x 0.577 x 0.4x0.4 = 254 kN

Use P_{pu} 254 kN

2.

Fs)$_o$ = K $\boldsymbol{\sigma}'_v$ tanδ =0

Critical depth = 20x pie diameter =20x0.4 =8m

Fs)$_8$ = K $\boldsymbol{\sigma}'_v$ tanδ = 1.3x 15.5 x 8 x tan (0.8x30) = 71.7 kN/m^2

$$P_s = \frac{0 \ + 71.7}{2} (4 \times 0.4 \times 8) \ \ + 71.7 \times (1.6 \times 7) = 1262 \ kN$$

3.

P_{ult}= P_p +P_s = 254 + 1262 = 1516 kN P_{all}= P_{ult}/FS = 1516/4 = 379 kN

Pile in Cohesive Soil

1. Point Capacity

In clay $\boldsymbol{\phi}$**=0** $q_u = \ c \ N_c^*$

Bearing capacity factor $\boldsymbol{N_c^*}$ is commonly taken as **9**

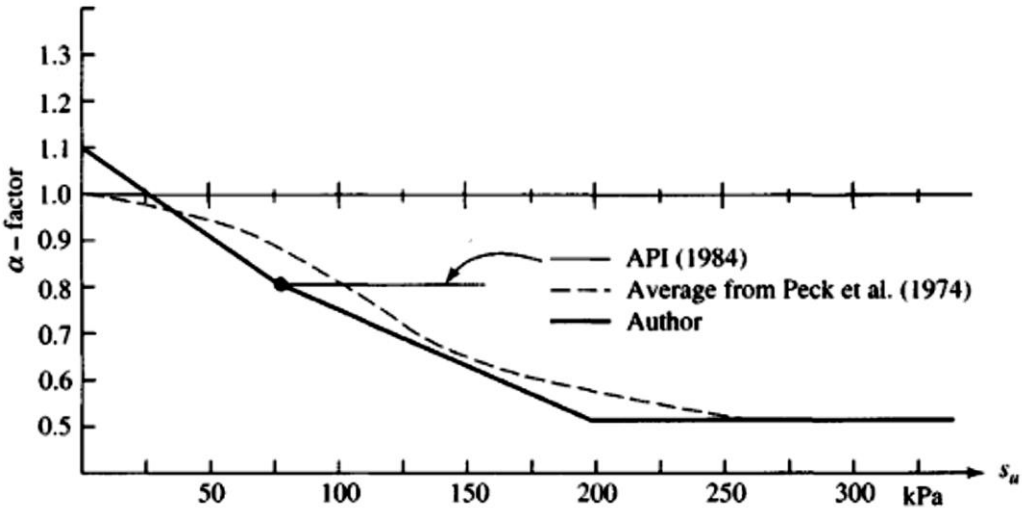
$$P_u = 9 \ c \ A_p$$

2. Skin friction Capacity

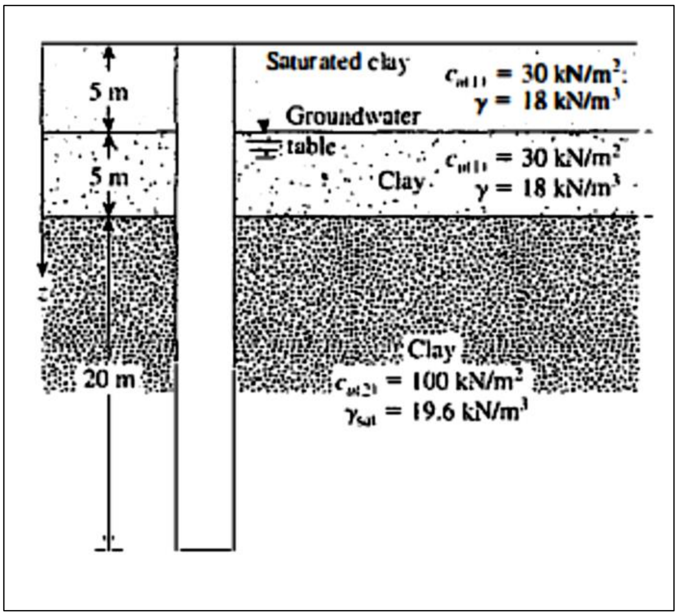
$$\boldsymbol{P_s \ = \ \sum A_s \, f_s}$$

$$\boldsymbol{f_s = \alpha \ c}$$

Where
α = coefficient from figure
c = average cohesion (or S_u) for the soil stratum of interest



Example:
A driven-pipe pile in clay is shown in figure.
The pipe has an outside diameter of 406 mm
 and a wall thickness of 6.35 mm.
a. Calculate the net point bearing capacity.
b. Calculate the skin resistance.
c. Estimate the net allowable pile capacity.
Use FS = 4.



Solution:
a.
P_{pu} = 9 C A_p = 9x 100x (3.14x0.406^2/4)
= 116.5 kN
b.
Perimeter of the pile= 0.406x3.14=1.275 m
P_s=30x0.95x5x1.275 +30x0.95x5x1.275 +100x0.72x20x1.275 =2200 kN
c.
P_{ult}= P_p +P_s =116.5 +2200= 2316.5 kN
P_{all}= P_{ult}/FS = 2316.5/4 = 580 kN

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

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

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 ≈ 100 kN/m^2
Briaud et al. (1985) suggested the following correlation for q_p in granular soil with the standard penetration resistance N_{60}.

$q_p = 19.7p_a(N60)^{0.36}$ (9.38)

Meyerhof (1956) also suggested that

$q_p \approx q_c$ (9.39)

where q_c = cone penetration resistance.

Example 9.3

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),

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

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

Frictional Resistance (Q_s) in Sand

Correlation with Standard Penetration Test Results

Meyerhof (1976) indicated that the average unit frictional resistance, f_{av} , for high-displacement driven piles may be obtained from average standard penetration resistance values as

$$f_{av} = 0.02p_a(\bar{N}_{60}) \qquad (9.45)$$

where

$\bar{N}_{60}$ = average value of standard penetration resistance

p_a = atmospheric pressure ≈100 kN/m^2

For low-displacement driven piles

$$f_{av} = 0.01p_a(\bar{N}_{60}) \qquad (9.46)$$

Briaud et al. (1985) suggested that

$$f_{av} \approx 0.224\, p_a(\bar{N}_{60})^{0.29} \qquad (9.47)$$

Thus,

$$Q_s = p\, L\, f_{av}$$

Example 9.4

Refer to the pile described in Example 9.3. Estimate the magnitude of Q_s for the pile.

a. Use Eq. (9.45).

b. Use Eq. (9.47).

c. Considering the results in Example 9.3, determine the allowable load-carrying capacity of the pile based on Meyerhof's method and Briaud's method. Use a factor of safety, FS = 3.

Solution

The average N_{60} value for the sand for the top 12 m is

$$\overline{N}_{60} = \frac{8 + 10 + 9 + 12 + 14 + 18 + 11 + 17}{8} = 10.25 \approx 10$$

Part a

From Eq. (9.45),

$$f_{av} = 0.02p_a(\overline{N}_{60}) = (0.02)(100)(10) = 20 \text{ kN/m}^2$$

$$Q_s = pLf_{av} = (4 \times 0.305)(12)(20) = \mathbf{292.8\ kN}$$

Part b

From Eq. (9.47),

$$f_{av} = 0.224p_a(\overline{N}_{60})^{0.29} = (0.224)(100)(10)^{0.29} = 43.68 \text{ kN/m}^2$$

$$Q_s = pLf_{av} = (4 \times 0.305)(12)(43.68) = \mathbf{639.5\ kN}$$

Part c

$$\text{Meyerhof's method: } Q_{all} = \frac{Q_p + Q_s}{FS} = \frac{633 + 292.8}{3} = \mathbf{308.6\ kN}$$

$$\text{Briaud's method: } Q_{all} = \frac{Q_p + Q_s}{FS} = \frac{508.2 + 639.5}{3} = \mathbf{382.6\ kN}$$

So the allowable pile capacity may be taken to be about **345 kN**. ■

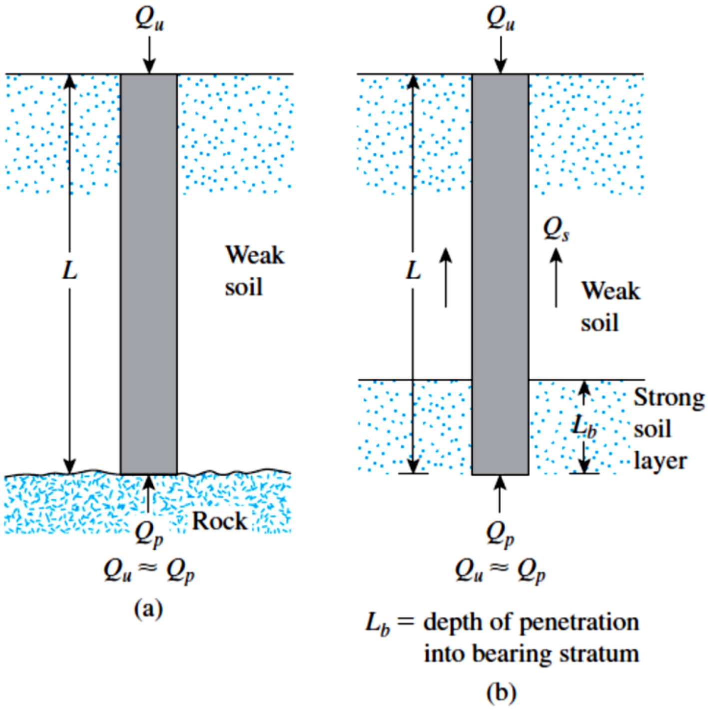


Figure 9.6 (a) and (b) Point bearing piles; (c) friction piles

Correlation with Cone Penetration Test Results

Nottingham and Schmertmann (1975) and Schmertmann (1978) provided correlations for estimating Q_s using the frictional resistance (f_c) obtained during cone penetration tests. According to this method

$$f = \alpha' f_c \tag{9.49}$$

The variations of α' with L/D for electric cone and mechanical cone penetrometers are shown in Figures 9.18 and 9.19, respectively. We have

$$Q_s = \Sigma p(\Delta L) f = \Sigma p(\Delta L) \alpha' f_c \tag{9.50}$$

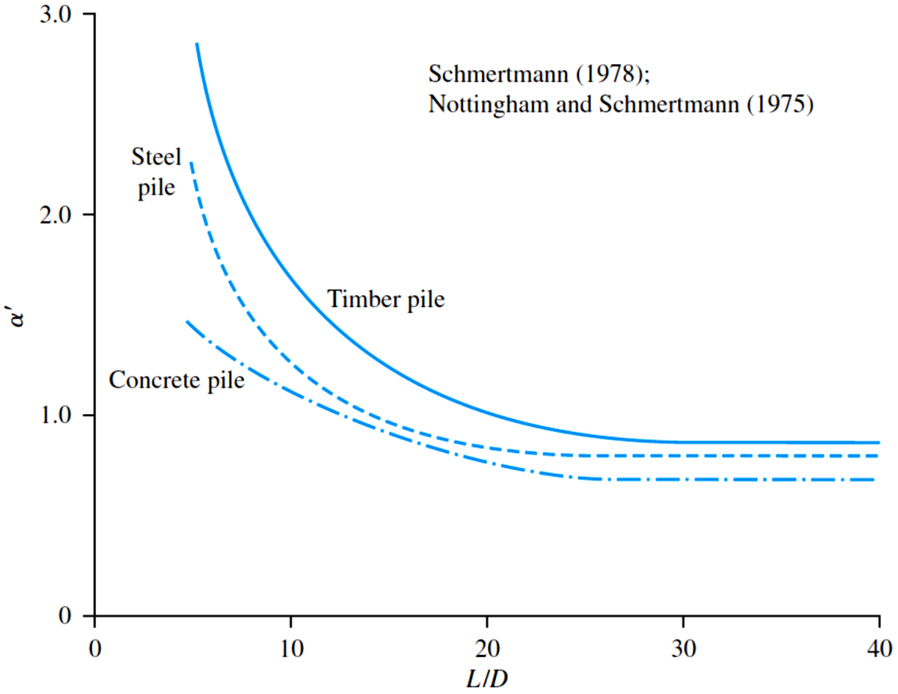


Figure 9.18 Variation of α' with embedment ratio for pile in sand: electric cone penetrometer

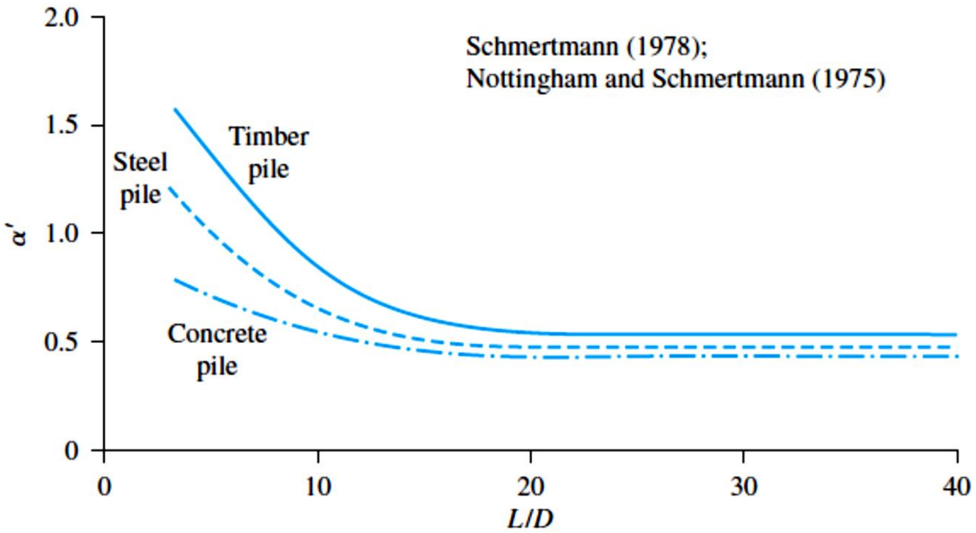


Figure 9.19 Variation of α' with embedment ratio for piles in sand: mechanical cone penetrometer

Example 9.6

Consider an 18-m-long concrete pile (cross section: 0.305 m × 0.305 m) fully embedded in a sand layer. For the sand layer, the following is an approximation of the cone penetration resistance q_c (mechanical cone) and the frictional resistance f_c with depth. Estimate the allowable load that the pile can carry. Use FS = 3.

Depth from ground surface (m)	q_c (kN/m²)	f_c (kN/m²)
0–5	3040	73
5–15	4560	102
15–25	9500	226

Solution

$$Q_u = Q_p + Q_s$$

From Eq. (9.39),

$$q_p \approx q_c$$

At the pile tip (i.e., at a depth of 18 m), $q_c \approx 9500$ kN/m². Thus,

$$Q_p = A_p q_c = (0.305 \times 0.305)(9500) = 883.7 \text{ kN}$$

To determine Q_s, the following table can be prepared. (*Note:* $L/D = 18/0.305 = 59$.)

Depth from ground surface (m)	ΔL (m)	f_c (kN/m²)	α' (Figure 9.19)	$p\Delta L\alpha' f_c$ (kN)
0–5	5	73	0.44	195.9
5–15	10	102	0.44	547.5
15–18	3	226	0.44	363.95

$$Q_s = 1107.35 \text{ kN}$$

Hence,

$$Q_u = Q_p + Q_s = 883.7 + 1107.35 = 1991.05 \text{ kN}$$

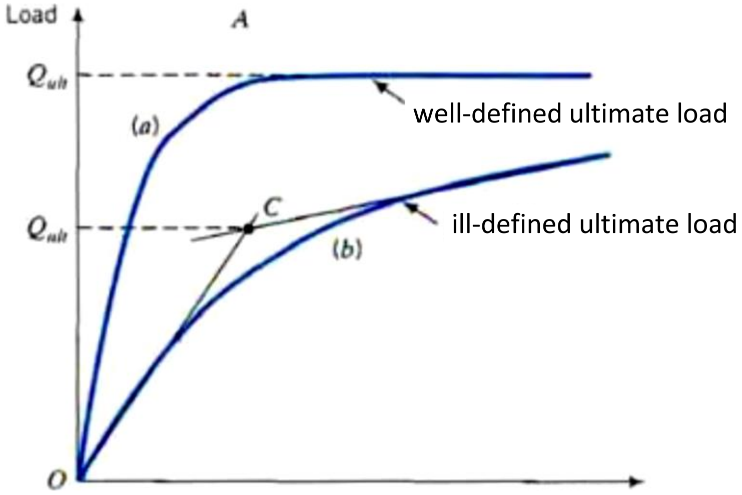
$$Q_{all} = \frac{Q_u}{FS} = \frac{1991.05}{3} = 663.68 \approx \mathbf{664\ kN}$$

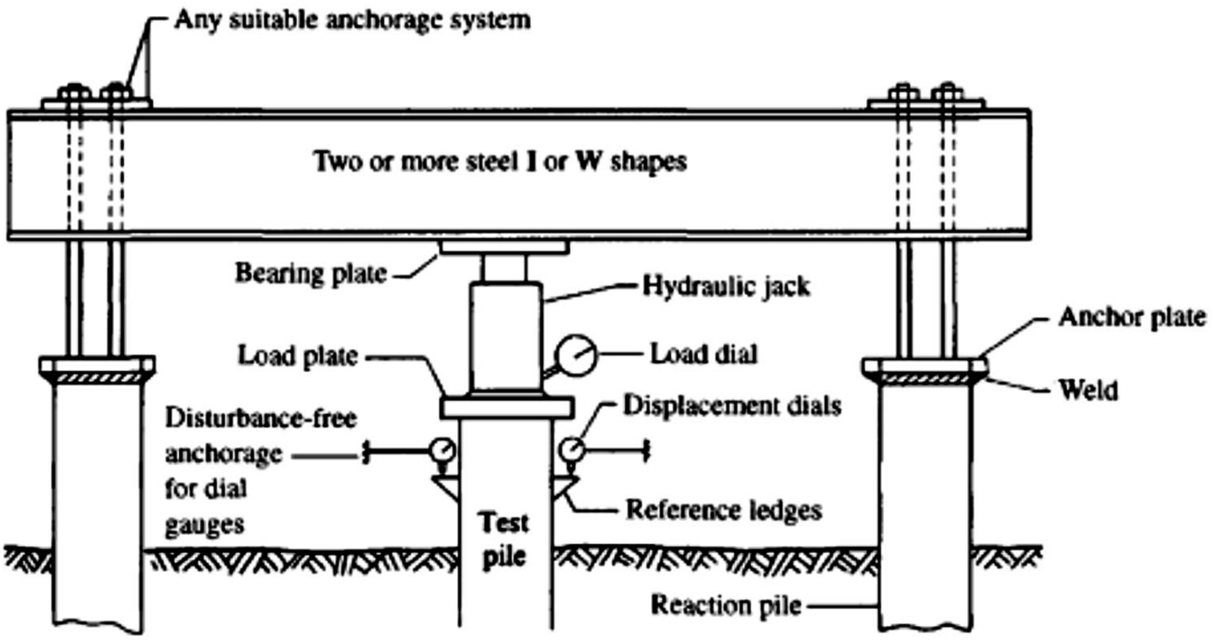
■

Pile Load Test

The purposes of a pile load test are:
• To determine the axial load capacity of a single pile.
• To determine the settlement of a single pile at working load.
• To verify the estimated axial load capacity.
• To obtain information on load transfer in skin friction and end bearing.
The allowable bearing capacity is found by dividing the ultimate load, found from the load settlement curve, by a factor of safety, usually 2. An alternative criterion is to determine the allowable pile load capacity for a desired serviceability limit state, for example, a settlement of 10% of the pile diameter. Also pile settlement under double working load should not be more than 25 mm.

Load- settlement curve of pile





(c) Typical pile load test setup using adjacent piles in group for reaction.

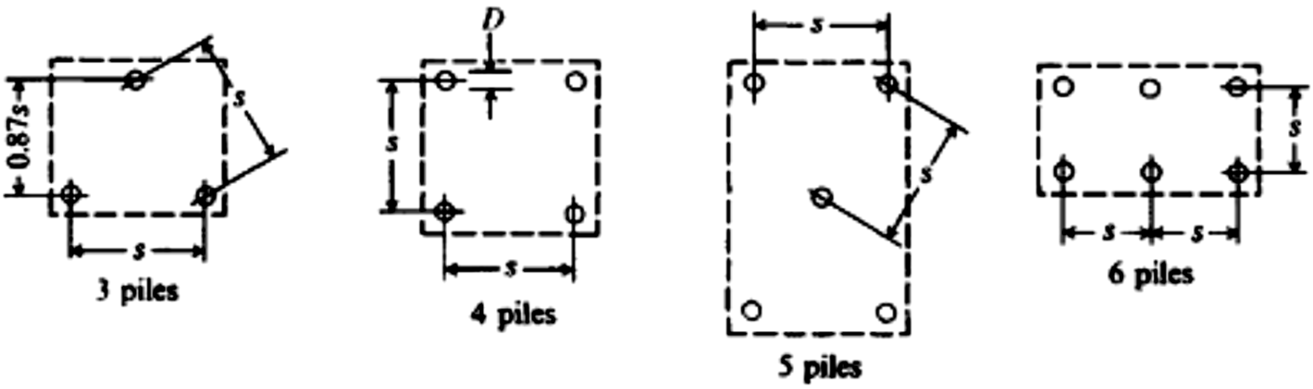
EFFICIENCY OF PILE GROUPS

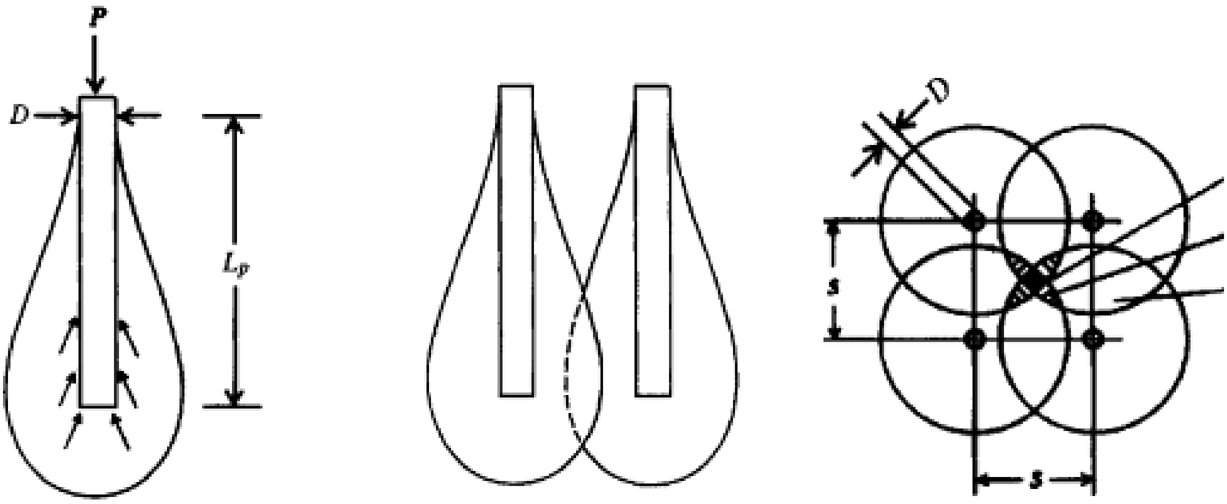
When several pile butts are attached to a common structural element termed a pile cap the result is a pile group. A question of some concern is whether the pile group capacity is the sum of the individual pile capacities or something different—either more or less. If the capacity is the sum of the several individual pile contributions, the group efficiency Eg = 1.0.

Optimum spacing s seems to be on the order of 2.5 to 3.5D or 2 to 3H for vertical loads where D = pile diameter; H = diagonal of rectangular shape or HP pile. Group efficiency can be estimated using

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

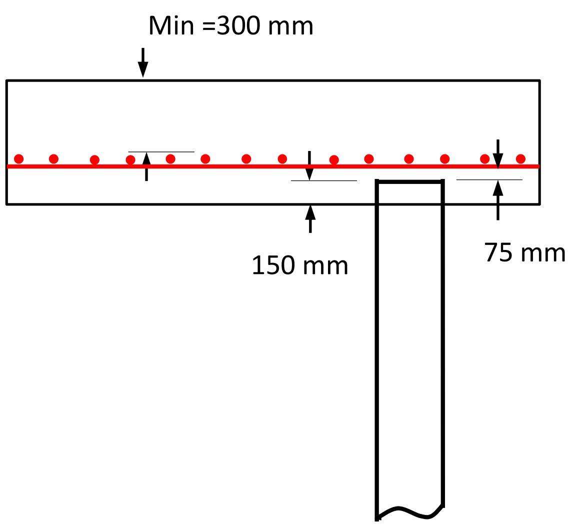
Where ***m*** and ***n*** are no. of columns and rows of piles $\boldsymbol{\theta}$**= tan^{-1} D/s** in degrees.





Function of Pile Cap

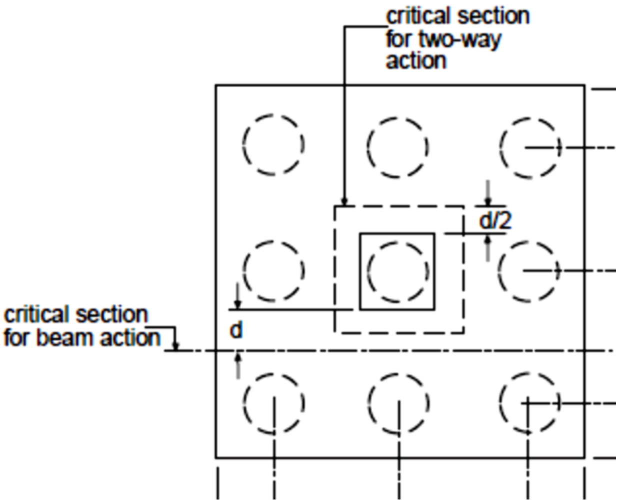
1. Transfer column load to pile bed.
2. To substitute the ill effect of one pile to others
3. To take any deviation in the location of piles



Minimum Total Thickness of Pile Cap
150 mm pile penetration in cap
75 mm concrete cover for cap steel above pile

Twice bar diameter
300 mm minimum concrete thickness above reinforcement

Example:
Estimate the pile group efficiency shown if the load per pile is as follows
P_D = 90 kN, P_L = 45 kN
What should be the minimum required allowable pile capacity of each Individual pile, then design the pile cap for the case shown
Footing size = 2.6 × 2.6 m.
Column size = 0.4× 0.4 m.
Pile diameter = 0.3 m.
c/c spacing between piles= 0.9 m
fc′ = 30 MPa



Solution*:*

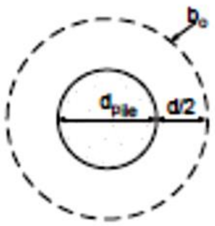
$$E_g = 1 - \theta \frac{(n-1)m + (m-1)n}{90mn}$$

$$\theta = tan^{-1}\frac{0.3}{0.9} = 18.43^{o}$$

$$E_g = 1 - 18.43\frac{(3-1)3+(3-1)3}{90\times3\times3} = 0.72$$

Total working load on each pile = 90 + 45 = 135 kN
Required allowable individual pile capacity = 135/0.72 = 187.5 kN

Design of pile cap:
Ultimate Pile Load
P_u = 1.2 x 90 +1.6 x 45 = 180 kN
Find Depth of Footing Using Shear Strength
1. Wide Beam Shear – Section at d from column face
V_u = 3 x 180 = 540 kN
ϕV_c = 0.75 x 0.17 $\sqrt{f_c'}$ b_w d = 0.75 x 0.17 x $\sqrt{30}$ x 2.6 x d x1000 = 1815 d
d = 540/1815 = **0.3 m**
2. Two- Way Shear – Section at d/2 from column face
V_u = 8 x 180 = 1440 kN
ϕV_c = 0.75 x 0.33$\sqrt{f_c'}$ $b_o d$ = 0.75 x 0.33 x $\sqrt{30}$ x 4(0.4 + d) d x1000 = 1356 (1.6d +4d^2)
1440 = 1356 (1.6d +4d^2)
d = 0.35 m
3. Check Punching Shear Strength at Corner pile.
P_u = 180 kN
ϕV_c = 0.75 x 0.33$\sqrt{f_c'}$ $b_o d$
= 0.75 x 0.33 x $\sqrt{30}$ x 3.14 (0.3 + d)x d x1000 = 4257 (0.3d + d^2)
d = 0.1 m
Use d = 350 mm



Total thickness of pile cap = 150 + 75 + 25+ 350 = 600 mm