25

Pile Foundations

25.1. INTRODUCTION

When the soil at or near the ground surface is not capable of supporting a structure, deep foundations are required to transfer the loads to deeper strata. Deep foundations are, therefore, used when surface soil is unsuitable for shallow foundation, and a firm stratum is so deep that it cannot be reached economically by shallow foundations. The most common types of deep foundations are piles, piers and caissons. The mechanism of transfer of the load to the soil is essentially the same in all types of deep foundations.

A deep foundation is generally much more expensive than a shallow foundation. It should be adopted only when a shallow foundation is not feasible. In certain situations, a fully compensated floating raft may be more economical than a deep foundation. In some cases, the soil is improved by various methods to make it suitable for a shallow foundation.

A pile is a slender structural member made of steel, concrete or wood. A pile is either driven into the soil or formed in-situ by excavating a hole and filling it with concrete. A pier is a vertical column of relatively larger cross-section than a pile. A pier is installed in a dry area by excavating a cylindrical hole of large diameter to the desired depth and then backfilling it with concrete. The distinction between a cast in-situ pile and a pier is rather arbitrary. A cast in-situ pile greater than 0.6 m diameter is generally termed as a pier. A caisson is a hollow, watertight box or chamber, which is sunk through the ground for laying foundation under water. The caisson subsequently becomes an integral part of the foundation. A pier and a caisson differ basically only in the method of construction.

Pile foundations are discussed in this chapter. Piers and caissons are dealt with in chapter 26. Well foundations, which are special type of caissons, are discussed in chapter 27.

25.2. NECESSITY OF PILE FOUNDATIONS

Pile foundations are used in the following conditions:

(1) When the strata at or just below the ground surface is highly compressible and very weak to support the load transmitted by the structure.

(2) When the plan of the structure is irregular relative to its outline and load distribution. It would cause non-uniform settlement if a shallow foundation is constructed. A pile foundation is required to reduce differential settlement.

(3) Pile foundations are required for the transmission of structural loads through deep water to a firm stratum.

(4) Pile foundations are used to resist horizontal forces in addition to support the vertical loads in earth-retaining structures and tall structures that are subjected to horizontal forces due to wind and earthquake.

(5) Piles are required when the soil conditions are such that a wash out, erosion or scour of soil may occur from underneath a shallow foundation.

(6) Piles are used for the foundations of some structures, such as transmission towers, off-shore plateforms, which are subjected to uplift.

(7) In case of expansive soils, such as black cotton soil, which swell or shrink as the water content changes, piles are used to transfer the load below the active zone.

(8) Collapsible soils, such as loess, have a breakdown of structure accompanied by a sudden decrease in void ratio when there is an increase in water content. Piles are used to transfer the load beyond the zone of possible moisture changes in such soils.

25.3. CLASSIFICATION OF PILES

Piles can be classified according to (1) the material used (2) the mode of transfer of load, (3) the method of construction, (4) the use, or (5) the displacement of soil, as described below.

(1) Classification according to material used

There are four types of piles according to materials used.

(*i*) **Steel Piles.** Steel piles are generally either in the form of thick pipes or rolled steel H-sections. Pipe steel piles are driven into the ground with their ends open or closed. Piles are provided with a driving point or shoe at the lower end.

Epoxy coatings are applied in the factory during manufacture of pipes to reduce corrosion of the steel piles. Sometimes, concrete encasement at site is done as a protection against corrosion. To take into account the corrosion, an additional thickness of the steel section is usually recommended.

(*ii*) **Concrete Piles.** Cement concrete is used in the construction of concrete piles. Concrete piles are either precast or cast-in situ. Precast concrete piles are prepared in a factory or a casting yard. The reinforcement is provided to resist handling and driving stresses. Precast piles can also be prestressed using high strength steel pretensioned cables.

A cast-in situ pile is constructed by making a hole in the ground and then filling it with concrete. A cast-in situ pile may be cased or uncased. A cased pile is constructed by driving a steel casing into the ground and filling it with concrete. An uncased pile is constructed by driving the casing to the desired depth and gradually withdrawing casing when fresh concrete is filled. An uncased pile may have a pedestal.

(*iii*) **Timber Piles.** Timber piles are made from tree trunks after proper trimming. The timber used should be straight, sound and free from defects.

Steel shoes are provided to prevent damage during driving. To avoid damage to the top of the pile, a metal band or a cap is provided. Splicing of timber piles is done using a pipe sleeve or metal straps and bolts. The length of the pipe sleeve should be at least five times the diameter of the pile.

Timber piles below the water table have generally long life. However, above the water table, these are attacked by insects. The life of the timber piles can be increased by preservatives such as creosote oils. Timber piles should not be used in marine environment where these are attacked by various organisms.

(*iv*) **Composite piles.** A composite pile is made of two materials. A composite pile may consist of the lower portion of steel and the upper portion of cast-in situ concrete. A composite pile may also have the lower portion of timber below the permanent water table and the upper portion of concrete. As it is difficult to provide a proper joint between two dissimilar materials, composite piles are rarely used in practice.

2) Classification Based on Mode of Transfer of Loads

Based on the mode of transfer of loads, the piles can be classified into 3 categories:

(*i*) **End-bearing piles.** End-bearing piles transmit the loads through their bottom tips. Such piles act as columns and transmit the load through a weak material to a firm stratum below. If bed rock is located within a reasonable depth, piles can be extended to the rock. The ultimate capacity of the pile depends upon the bearing capacity of the rock. If instead of bed rock, a fairly compact and hard stratum of soil exists at a reasonable depth, piles can be extended a few metres into the hard stratum. End-bearing piles are also known as *point- bearing piles.*

The ultimate load carried by the pile (Q_u) is equal to the load carried by the point or bottom end (Q_p).

(ii) **Friction piles.** Friction piles do not reach the hard stratum. These piles transfer the load through skin friction between the embedded surface of the pile and the surrounding soil. Friction piles are used when a hard stratum does not exist at a reasonable depth. The ultimate load (Q_u) carried by the pile is equal to the load transferred by skin friction (Q_s).

[**Note:** The term friction pile is actually a misnomer, as in the clayey soils, the load is transferred by adhesion and not friction between the pile surface and the soil].

The friction piles are also known as *floating piles*, as these do not reach the hard stratum.

(iii) **Combined end bearing and friction piles.** These piles transfer loads by a combination of end bearing at the bottom of the pile and friction along the surface of the pile shaft. The ultimate load carried by the pile is equal to the sum of the load carried by the pile point (Q_P), and the load carried by the skin friction (Q_s).

(3) Classification based on method of installation

Based on the method of construction, the piles may be classified into the following 5 categories:

(i) **Driven piles.** These piles are driven into the soil by applying blows of a heavy hammer on their tops.

(ii) **Driven and Cast-in-situ piles.** These piles are formed by driving a casing with a closed bottom end into the soil. The casing is later filled with concrete. The casing may or may not be withdrawn.

(iii) **Bored and Cast-in-situ piles.** These piles are formed by excavating a hole into the ground and then filling it with concrete.

(iv) **Screw piles.** These piles are screwed into the soil.

(v) **Jacked piles.** These piles are jacked into the soil by applying a downward force with the help of a hydraulic jack.

(4) Classification based on use

The piles can be classified into the following 6 categories, depending upon their use.

(i) **Load bearing piles.** These piles are used to transfer the load of the structure to a suitable stratum by end bearing, by friction or by both. These are the piles mainly discussed in this chapter.

(ii) **Compaction piles.** These piles are driven into loose granular soils to increase the relative density. The bearing capacity of the soil is increased due to densification caused by vibrations.

(iii) **Tension piles.** These piles are in tension. These piles are used to anchor down structures subjected to hydrostatic uplift forces or overturning forces.

(iv) **Sheet piles.** Sheet piles form a continuous wall or bulkhead which is used for retaining earth or water (see Chapter 20).

(v) **Fender piles.** Fender piles are sheet piles which are used to protect water-front structures from impact of ships and vessels.

(vi) **Anchor piles.** These piles are used to provide anchorage for anchored sheet piles. These piles provide resistance against horizontal pull for a sheet pile wall (see Chapter 20).

(5) Classification based on displacement of soil

Based on the volume of the soil displaced during installation, the piles can be classified into 2 categories:

(i) **Displacement piles.** All driven piles are displacement piles as the soil is displaced laterally when the pile is installed. The soil gets densified. The installation may cause heaving of the surrounding ground. Precast concrete pile and closed-end pipe piles are high displacement piles. Steel H-piles are low displacement piles.

(ii) **Non-displacement piles.** Bored piles are non-displacement piles. As the soil is removed when the hole is bored, there is no displacement of the soil during installation. The installation of these piles causes very little change in the stresses in the surrounding soil.

25.4. PILE DRIVING

Piles are driven into the ground by means of hammers or by using a vibratory driver. Such piles are called driven piles. In some special cases, piles are installed by jetting or partial augering.

The following methods are commonly used.

(1) **Hammer Driving.** Fig. 25.1 shows a pile driving rig. It consists of a hoist mechanism, a guiding frame and a hammer device. The hammers used for pile driving are of the following types:

(i) **Drop hammer.** A drop hammer is raised by a winch and allowed to drop on the top of the pile under gravity from a certain height. During the driving operation, a cap is fixed to the top of the pile and a cushion is generally provided between the pile and the cap. Another cushion, known as hammer cushion, is placed on the pile cap on which the hammer causes the impact. The drop hammer is the oldest type of hammer used for pile driving. It is rarely used these days because of very slow rate of hammer blows.

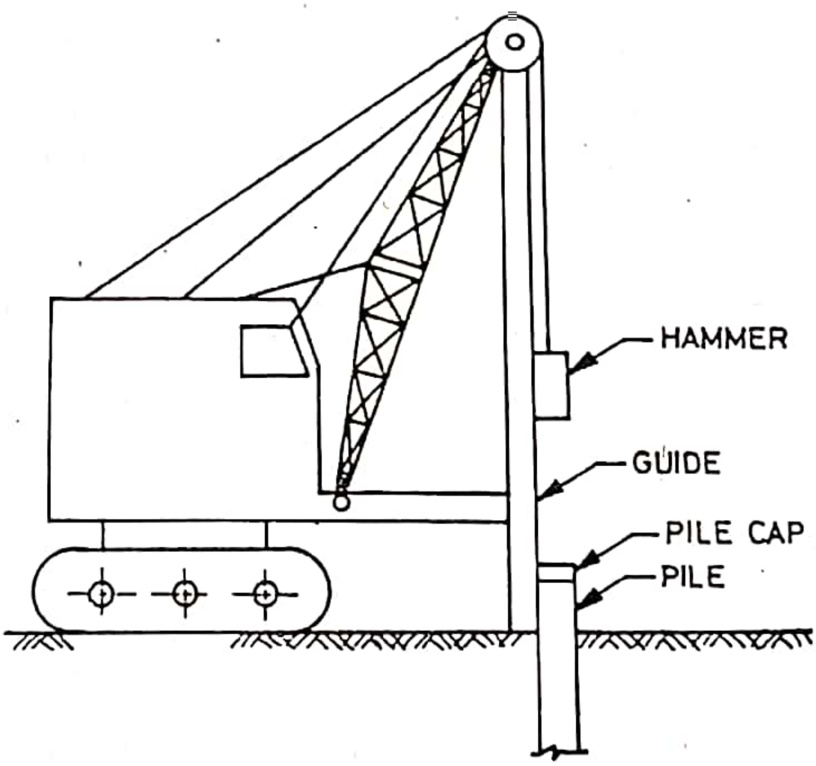


Fig. 25.1. Pile Driving Rig.

(ii) **Single-acting hammer.** In a single-acting hammer, the ram is raised by air (or steam) pressure to the required height. It is then allowed to drop under gravity on the pile cap provided with a hammer cushion.

(iii) **Double-acting hammer.** In a double-acting hammer, air (or steam) pressure is used to raise the hammer. When the hammer has been raised to the required height, air (or steam) pressure is applied to the other side of the piston and the hammer is pushed downward under pressure. This increases the impact energy of the hammer.

(iv) **Diesel hammer.** A diesel hammer consists of a ram and a fuel injection system. It is also provided with an anvil block at its lower end. The ram is first raised manually and the fuel is injected near the anvil. As soon as the hammer is released, it drops on the anvil and compresses the air-fuel mixture and ignition takes place. The pressure so developed pushes the pile downward and raises the ram. The fuel is again injected and the process is repeated.

The ram lifts automatically. It has to be manually raised only once at the beginning.

Diesel hammers are not suitable for driving piles in soft soils. In such soils, the downward movement of pile is excessive and the upward movement of the ram after impact is small. The height achieved after upward movement of the hammer may not be sufficient to ignite the air-fuel mixture.

Diesel hammers are self-contained and self-activated.

(2) **Vibratory Pile Driver.** A vibratory pile driver consists of two weights, called *exciters,* which rotate opposite directions. The horizontal components of the centrifugal force generated by exciters cancel each other but the vertical components add. Thus a sinosoidal dynamic vertical force is applied to the pile, which forces the pile downward. The frequency of vibration is kept equal to the natural frequency of pile-soil system for better results.

A vibratory pile driver is useful only for sandy and gravelly soils. The speed of penetration is good. The method is used where vibrations and noise of conventional driving methods cannot be permitted.

(3) **Jetting Techniques.** When the pile is to penetrate a thin hard layer of sand or gravel overlying a softer soil layer, the pile can be driven through the hard layer by jetting techniques. Water under pressure is discharged at the pile bottom point by means of a pipe to wash and loosen the hard layer.

(4) **Partial Augering Method.** Batter piles (inclined piles) are usually advanced by partial augering. In this method, a power auger is used to drill the hole for a part of the depth. The pile is then inserted in the hole and driven with hammers to the required depth.

.5. CONSTRUCTION OF BORED PILES

(a) Drilling of holes.

Bored piles are constructed after making a hole in the ground and filling it with concrete.

The following methods are used for drilling of the hole.

(1) **Hand auger.** A hand auger can be used for boring without casing in soils which are self-supporting, such as firm to stiff clays and silts and clayey sands and gravels above the water table. The depth of the hole is generally limited to about 4.5 m. The diameter of the hole is usually not more than 350 mm.

(2) **Mechanical auger.** For piles of diameter more than 350 mm or depth greater than 4.5 m, a hand auger becomes uneconomical. In such a case, a mechanical auger is used. A mechanical auger can be of rotary type or bucket type. It is power driven. The soil in this case must be self-supporting, with or without bentonite slurry. The soil should be free from tree roots, cobbles and boulders.

A continuous flight auger is also used to drill the bore hole.

(3) **Boring rig.** A boring rig is used to sink the hole in ground where hand or mechanical augering is not possible, such as water- bearing sand or gravels, very soft clays and silts and the soils having cobbles and boulders.

A specially designed boring rig, known as grab-type bored piling rig, is sometimes used. In this type of rig the casing is given a continuous semi-rotary motion which causes its sinking as the bore hole is advanced by percussion drilling.

(4) **Belling Bucket.** Underreamed piles are large diameter bored piles with enlarged bases. Excavation for the underreamed piles is done by a special type belling bucket.

(b) Concreting

Before concrete is placed, the bored hole is bailed dry of water. Any loose or softened soil is cleaned out and the bottom of the hole is rammed. A layer of dry concrete is placed and rammed if the bottom of the hole is wet. Then the concrete with a readily workable mix (7.5 to 10 cm slump), not leaner than 300 kg cement/m^3 of concrete, is poured into a hopper placed at the mouth of the hole.

If the hole cannot be bailed or pumped dry before placing the concrete, the hole is lined with a casing throughout its depth. A mass of concrete is then deposited at the base of the hole by a *tremie pipe*. As soon as the concrete has hardened and formed a plug, the hole is pumped free of water. The casing is then gently turned and lifted slightly to break the joint with the plug. The hole is pumped dry. The remainder of the concreting is done by placing it dry upto the ground surface. The casing is then lifted entirely from the bore hole.

If the ground water is under a high pressure, there will be inflow of water between the concrete plug and the inside of the casing. The inflow should be stemmed by caulking. The casing is cut by oxy-acetylene just above the plug. The shaft is then concreted and the casing raised. The cut portion of the casing around the plug is left permanently in place.

Instead of plugging the base of the pile and concreting, an alternative method is to concrete the entire shaft under water using a tremie pipe. Concrete should be easily workable (slump 12.5 to 17.5 cm) and the cement content should be at least 400 kg/m^3. A retarder is added to the concrete if there is a risk of the concrete setting before the casing is lifted out. However, the quality of concreting done under water is not good. This method should be avoided as far as possible.

25.6. DRIVEN CAST-IN-SITU CONCRETE PILES

A driven cast-in-situ concrete pile is formed in the ground by driving a casing with a plug or shoe at its bottom. If the casing is removed after concrete has been placed, it is known as uncased or shell-less pile. On the other hand, if the casing is left in the ground after concreting, it is called a cased pile. In uncased piles, the concrete comes in direct contact with the soil. The concrete may be rammed or vibrated after its deposition. A pedestal may be formed at the lower end of the shell-less pile if required.

Cast-in-situ driven concrete piles can be broadly classified into three types: (*i*) cased pile, (*ii*) uncased pile, and (*iii*) pedestal type. Different types of piles with patent rights are available. The main difference between different patents is in the method of construction, as described below.

(1) The *Franki pile* is a type of driven and cast-in-situ displacement pile. A heavy steel pile is first pitched in a shallow foundation. A plug of lean concrete is then placed in the bottom of the pipe and compacted with a heavy steel rammer. The plug is then rammed and with it the pipe also goes down. This driving operation is continued until the bearing stratum is reached. The concrete is hammered to form a pedestal. A reinforcement cage is then placed in the pipe and the pile shaft is concreted. The pipe is withdrawn as the concrete is rammed.

(2) In *uncased-Western pile*, a heavy steel drive pipe of 35 cm diameter with a steel core is driven. The concrete is deposited in the pipe after removing the core. The concrete is rammed as the pipe is withdrawn. The pedestal is formed after the drive pipe has been lifted to some height.

(3) In *cased-Western pile*, the hole is made using a heavy steel drive pipe as for the uncased-Western type. A shell of 30 cm diameter is lowered inside the drive pipe. After the shell has been filled with concrete, the drive pipe is withdrawn. A pedestal can be formed by placing some concrete before lowering the shell and ramming.

(4) A *Western button-bottom pile* is formed by driving a steel pipe with a 43 cm diameter precast concrete point at its bottom. After reaching the required depth, a shell is lowered into the pipe and locked into the point at its bottom. The shell is then filled with concrete and the drive pipe is withdrawn.

(5) The *Raymond Taper or Step-Taper piles* are steel shell piles driven with a tapered steel mandrel. The mandrel and shells are driven to the required depth. The mandrel is then contracted and withdrawn, and the shell is concreted with or without a reinforcing cage.

(6) A *Simplex pile* is formed by driving a steel tube with a detachable cast iron shoe. After the required depth has been reached, reinforcing cage is lowered. The tube is extracted by wire ropes connected to a winch. At the same time, the concrete is placed and rammed by a falling rammer working inside the cage.

(7) *Alpha piles* are formed by driving a steel tube closed with a detachable cast iron shoe. A concrete-filled mandrel is driven inside the tube. The mandrel is gradually raised and some concrete is allowed to slump down in the tube. The concrete is refilled in the mandrel and it is driven down as the tube is raised. Thus a pedestal is formed. After the formation of the pedestal, the mandrel is raised and refilled with concrete in stages. In each stage, the concrete in the pile shaft is pressed against the soil by the dead weight of the hammer on the mandrel.

25.7. LOAD-CARRYING CAPACITY OF PILES

Like a shallow foundation, a pile foundation should be safe against shear failure and also the settlement should be within the permissible limits. The methods for estimating the load-carrying capacity of a pile foundation can be grouped into the following 4 categories.

(1) **Static Methods.** The static methods give the ultimate capacity of an individual pile, depending upon the characteristics of the soil. The ultimate load capacity is given by

$$Q_u = Q_p + Q_s \qquad \ldots(25.1)$$

where Q_u = ultimate failure load, Q_p = point (or base or tip) resistance of the pile (Fig. 25.2), Q_s = shaft resistance developed by friction (or adhesion) between the soil and the pile shaft.

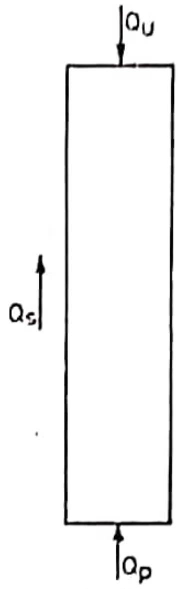


Fig. 25.2.

The methods for the determination of Q_p and Q_s are discussed in Sects. 25.8 and 25.9, respectively, for sand and clay.

The static formulas give a reasonable estimate of the pile capacity if judiciously applied.

(2) **Dynamic Formulas.** The ultimate capacity of piles driven in certain types of soils is related to the resistance against penetration developed during driving operation. The ultimate load capacity formulas are based on the principle that the resistance of a pile to further penetration by driving depends upon the energy imparted to the pile by the hammer. It is tacitly assumed that the load-carrying capacity of the pile is equal to the dynamic resistance during driving.

The dynamic formulas are not much reliable.

(3) **In-situ Penetration Tests.** The pile capacity can be determined from the results of in-situ standard penetration test. Empirical formulas are used to determine the point resistance and the shaft resistance from the standard penetration number (N). Alternatively, the static formulas can be used after determining the N-value, as this value is related to the angle of shearing resistance (ϕ).

Cone penetration tests are also used to estimate the pile capacity.

(4) **Pile Load Tests.** The most reliable method of estimating the pile capacity is to conduct the pile load test. The test pile is driven and loaded to failure. The pile capacity is related to the ultimate load or the load at which the settlements do not exceed the permissible limits.

All the above methods are discussed in detail in the following sections.

25.8 STATIC METHODS FOR DRIVEN PILES IN SAND

The ultimate capacity of a single pile driven into sand is obtained using Eq. 25.1, $Q_u = Q_p + Q_s$

where $$Q_p = q_p A_p \qquad \ldots(25.2)$$

and $$Q_s = f_s A_s \qquad \ldots(25.3)$$

In above equations, q_p is the ultimate bearing capacity of the soil at the pile tip and and A_p is the area of the pile tip; f_s is the average unit skin friction between the sand and the pile surface, and A_s is the effective surface area of the pile in contact with the soil.

(*a*) **Methods for determination of Q_p.** The ultimate bearing capacity (q_p) of the soil at the pile tip can be computed from the bearing capacity equation similar to that for a shallow foundation, as discussed chapter 23. For sandy soils,

$$q_p = \bar{q} N_q + 0.4 \gamma B N_\gamma \qquad ...(25.$$

where $\bar{q}$ = effective vertical pressure at the pile tip, B = pile tip width (or diameter),
γ = unit weight of the soil in the zone of the pile tip.
N_q and N_γ = bearing capacity factors for deep foundations.

In driven piles, the second term of Eq. 25.4 is generally small and is, therefore, neglected. Thus

$$q_p = \bar{q} N_q \qquad ...(25.$$

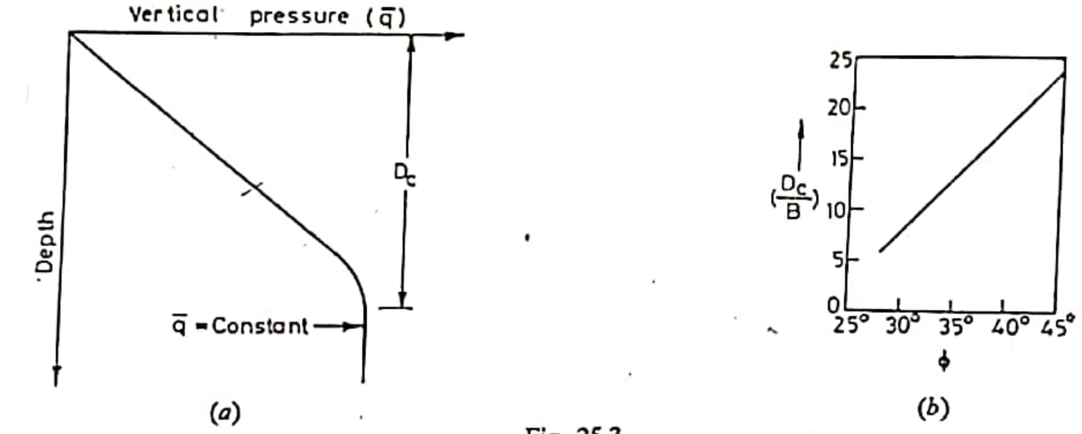


Fig. 25.3.

In case of driven piles, it has been established that the effective vertical pressure ($\bar{q}$) at the pile t increases with depth only until a certain depth of penetration, known as the critical depth (D_c). Below th critical depth, the effective vertical pressure remains essentially constant [Fig. 25.3 (a)]. The critical dep

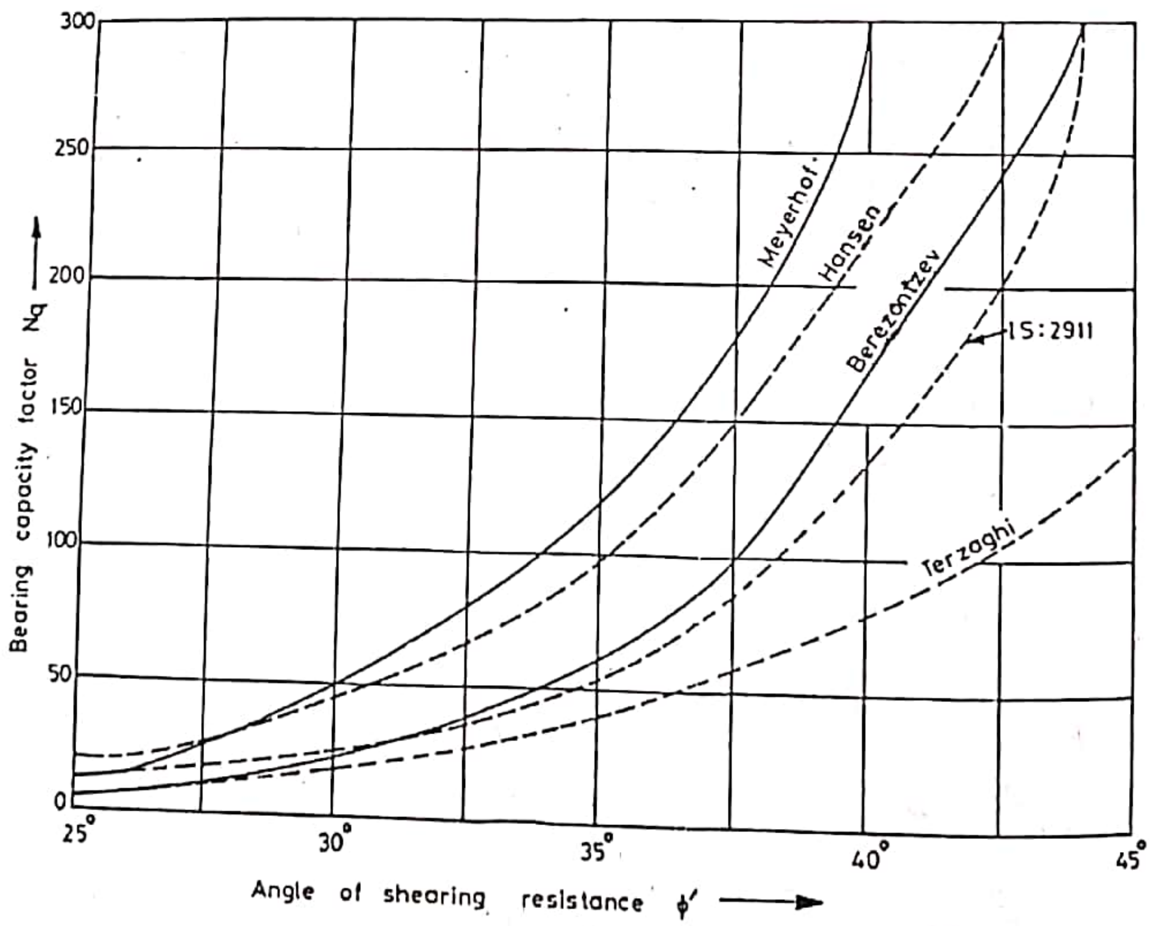


Fig. 25.4.

depends upon the angle of shearing resistance (ϕ') of the soil and the width (or diameter) of the pile [Fig. 25.3 (b)]. Its value can be roughly taken as 10 B for loose sands and 20 B for dense sands.

The bearing capacity factor N_q depends upon the angle of shearing resistance (ϕ'). Various investigators gave the expressions for N_q based on theoretical analysis. These values vary over a wide range because of different assumptions made in defining the shear zone near the pile tip. Fig. 25.4 shows the values of N_q given by various investigators and that given by IS : 2911. The values given by Berezontzev are quite dependable, and are generally used.

In the derivation of the value of N_q, it has been assumed that the soil above the pile tip is similar to the soil below the pile tip. If the pile penetrates a compact stratum only slightly and the soil above the tip is loose, it would be more appropriate to use the value of N_q for a shallow foundation given in chapter 23.

If the pile is of relatively large diameter, the second term in Eq. 25.4 becomes significant. The value of N_γ can be conservatively taken as the N_γ value used for shallow foundations, given in chapter 23.

Meyerhof's method for q_p. The point bearing capacity (q_p) of a pile generally increases with the depth of embedment (D_b) in the bearing stratum. It reaches a maximum value at an embedment ratio of $(D_b/B)_{cr}$. For a homogeneous soil, D_b is equal to the actual depth D of the pile, but for a pile which has penetrated into a bearing stratum for a small length, D_b is less than D. Beyond the critical value of $(D_b/B)_{cr}$, the value of q_p remains constant, equal to the limiting q_l. The critical ratio $(D_b/B)_{cr}$ depends upon the soil friction angle (ϕ) (Fig. 25.5).

Once the value of $(D_b/B)_{cr}$ has been determined, the following procedure is used to estimate q_p.

(1) Determine actual (D_b/B) ratio for the pile,

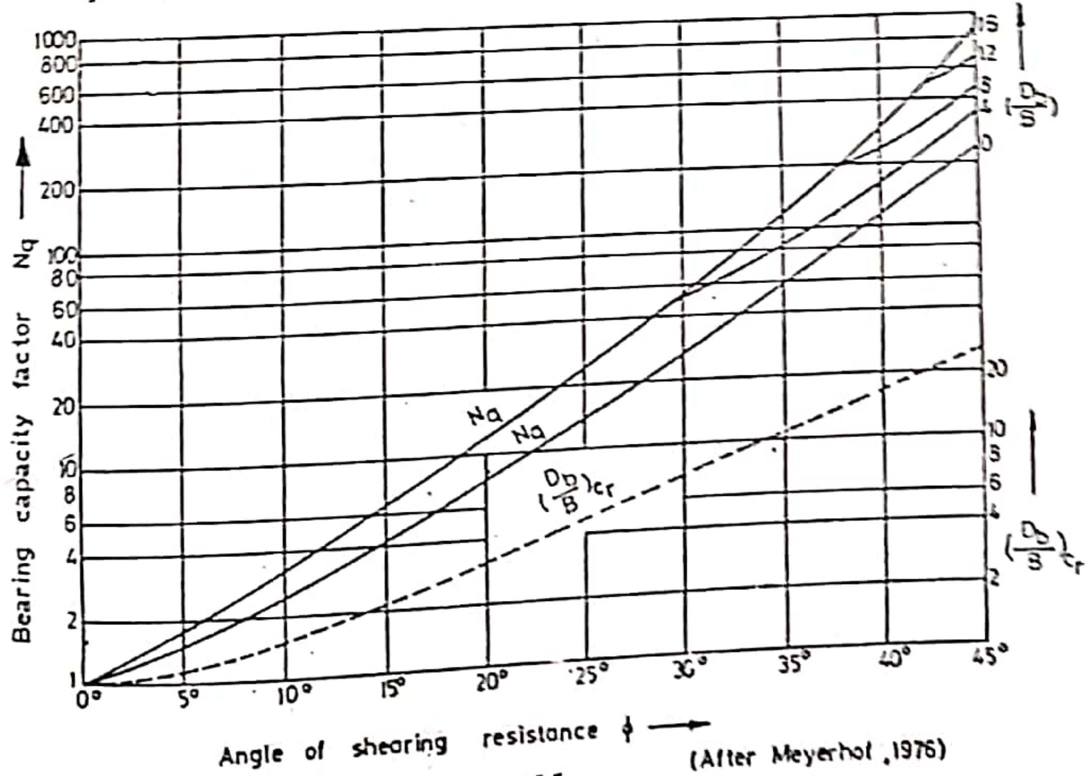
(2) Determine N_q for (D_b/B) ratio from Fig. 25.5.

The value of N_q increases linearly with (D_b/B) ratio and reaches a maximum value at $D_b/B = \frac{1}{2}(D_b/B)_{cr}$.

(3) Determine the point resistance Q_p as

$$Q_p = A_p \bar{q}\, N_q \leq A_p q_l \qquad \text{...(25.6)}$$

where $q_l = 50\, N_q \tan\phi$, $\bar{q}$ = vertical pressure at the pile tip (kN/m^2), A_p = area of the pile tip.



(After Meyerhof, 1976)

Fig. 25.5.

If the pile initially penetrates a loose sand layer and then a dense layer for a depth less than 10 *B*, the point resistance is given by

$$q_p = q_{l(1)} + \frac{[q_{l(2)} - q_{l(1)}]D_b}{10\,B} \leq q_{l(2)} \qquad \ldots(25.7)$$

where $q_{l(1)}$ = limiting unit point resistance of loose sand (= 50 N_{q1} tan ϕ_1)

$q_{l(2)}$ = limiting unit point resistance of dense sand (= 50 N_{q2} tan ϕ_2)

D_b = depth of penetration in dense sand.

It may be mentioned that the ultimate tip resistance given by Eq. 25.2 is the gross ultimate point resistance. The net tip load is given by

$$Q_p\,(\text{net}) = Q_p - (\bar{q}\,A_p)$$

However, in practice, the deduction of $\bar{q}\,A_p$ is not usually made and Q_p (net) is taken equal to Q_p.

In case of H-piles and open-ended pipe piles, the enclosed soil plug should be considered as the part of the pile for computing the area of the point (A_p).

(*b*) **Methods of determination of Q_s.** The frictional resistance Q_s is obtained from Eq. 25.3 after estimating the unit skin friction (f_s). The unit skin friction for a straight-sided pile depends upon the soil pressure acting normal to the pile surface and the coefficient of friction between the soil and the pile material (Fig. 25.6).

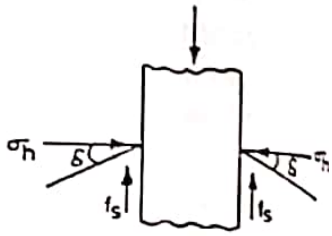


Fig. 25.6.

The soil pressure normal to the vertical pile surface is horizontal pressure (σ_h) and is related to the effective vertical soil pressure as

$$\sigma_h = K\bar{\sigma}_v$$

where K = earth pressure coefficient, $\bar{\sigma}_v$ = effective vertical pressure at that depth.

Thus unit skin friction (f_s) acting at any depth can be written as

$$f_s = \sigma_h \tan\delta \quad \text{or} \quad f_s = K\bar{\sigma}_v \tan\delta \qquad \ldots(25.8)$$

where tan δ = coefficient of friction between sand and the pile material.

Selection of suitable values of δ and K requires good engineering judgment. Tomilson (1975) gave the values of δ and K, as given in Table 25.1, based on the studies carried by Broms (1966).

Table 25.1. Values of δ and *K*.

Pile Material	δ	*K* (*loose sand*)	*K* (*dense sand*)
Steel	20°	0.50	1.0
Concrete	0.75 φ	1.0	2.0
Timber	0.67 φ	1.5	4.0

In general, the value of δ generally varies between 0.5 φ and 0.8 φ. In most cases, the value of K varies between 0.6 and 1.25. Meyerhof (1956) recommends that the value of K can be taken as 0.5 for loose sand (φ = 30°) and as 1.0 for dense sand (φ = 45°). According to IS : 2911—1979, the value of δ may be taken equal to φ. For driven piles in loose to medium sands, the recommended value of K is between 1 and 3.

Whether the sand should be considered as loose or dense depends upon not only on the initial relative density, but also on the method of installation. The larger the volume of soil displacement, the higher the value of the resulting friction. For high displacement driven piles, the soil is considered dense. For driven and cast-in place piles, the soil is considered as medium dense if the casing is left in place or if the concrete is compacted as the casing is withdrawn. The sand is considered to be loose, if the concrete is not compacted. Tapered piles develop greater unit friction than the straight piles. Further, the value of K is greater if the pile is driven into undisturbed soil than the one for installed in a predrilled hole.

As stated earlier, the effective vertical pressure ($\bar{\sigma}_v$) increases with depth only upto the critical depth. Below the critical depth, the value of $\bar{\sigma}_v$ remains constant.

The frictional resistance (Q_s) can be expressed as

$$Q_s = \sum_{i=1}^{n} K\,(\bar{\sigma}_v)_i \tan\delta \quad (A_s)_i \qquad \text{...[25.9(a)]}$$

where n = number of layers in which the pile is installed,

$(\bar{\sigma}_v)_i$ = effective normal stress in ith layer,

$(A_s)_i$ = surface area of the pile in ith layer,

Assuming linear variation of σ_v,

$$Q_s = K \tan\delta\ A_p \times (q_0 D) \qquad \text{...[25.9(b)]}$$

where A_p = perimeter of pile; q_0 = average effective pressure = $\left(\dfrac{\gamma' D}{2}\right) = \dfrac{\bar{q}}{2}$

and D = depth of pile.

Eq. 25.9 (*a*) can be written as

$$Q_s = \sum_{i=1}^{n} K \tan\delta \quad \text{(area of } \bar{\sigma}_v \text{ diagram)} \times \text{pile perimeter} \qquad \text{...[25.9(c)]}$$

Eq. 25.9 (*b*) is useful when variation of $\bar{\sigma}_v$ with depth has been plotted as $\bar{\sigma}_v$-diagram.

The ultimate load for the pile (Eq. 25.1) can be written as

$$Q_u = Q_p + Q_s$$

or

$$Q_u = \bar{q}\, N_q A_p + \sum_{i=1}^{n} K\,(\bar{\sigma}_v)_i \tan\delta \ (A_s)_i \qquad \text{...(25.10)}$$

$$Q_u = \bar{q}\, N_q A_p + K \tan\delta\, (\text{pile perimeter}) \left(\frac{\bar{q}}{2}\right) D \qquad \text{...[25.10 (a)]}$$

25.9. STATIC METHOD FOR DRIVEN PILES IN SATURATED CLAY

Eq. 25.1 can be used for the determination of the load-carrying capacity of driven piles in saturated clay. The point resistance (Q_p) can be expressed as (Eq. 25.2),

$$Q_p = q_p A_p$$

where q_p is the unit point resistance, equal to the ultimate bearing capacity (q_u) of the soil.

For cohesive soils ($\phi = 0$), the ultimate bearing capacity is found from the following equation, which is similar to that for a shallow foundation.

$$q_u = cN_c + q N_q$$

As $N_q = 1.0$ for $\phi = 0$, the above equation becomes

$$q_u = cN_c + q$$

Therefore,

$$Q_p \text{ (gross)} = (cN_c + q)\, A_p$$

or

$$Q_p\ (net) = cN_c A_p \qquad \text{...(25.11)}$$

In above equations, c is the cohesion of the clay in the zone surrounding the pile tip, and N_c is the bearing capacity factor for the deep foundation.

The value of N_c depends upon the *D/B* ratio and it varies from 6 to 9.0. A value of $N_c = 9.0$ is generally used for the piles. In the case of short piles ($D/B \le 5.0$), the value of N_c is reduced to the values proposed by Skempton (see chapter 23).

The skin resistance (Q_s) of the pile can be expressed as (Eq. 25.3),

$$Q_s = c_a A_s \qquad \ldots(25.12)$$

where c_a = unit adhesion (or skin friction) developed between clay and pile shaft.

The unit adhesion (c_a) is related to the unit cohesion by the relation

$$c_a = \alpha \bar{c} \qquad \ldots(25.13)$$

where α is the adhesion factor and $\bar{c}$ is the average cohesion along the shaft length.

The value of α depends upon the consistency of the clay. For normally consolidated clays, the value of α is taken as unity. According to IS : 2911—1979, the value of α can be taken as unity for soils having soft to very soft consistency. Fig. 25.7 shows the variation of α with the undrained cohesion c. It may be noted that for normally consolidated clays, with c less than about 50 kN/m^2, the value of α is equal to unity.

As c increases, the value of α decreases. For over-consolidated stiff to hard clays, its value is usually taken as 0.3. For tapered piles, the value of α is generally 20% greater than that for a straight pile.

For very long piles ($D \geq 25$ m), the above method for estimating the skin friction is very conservative. For such soils, the unit skin friction also depends upon the effective overburden pressure. According to Vijayvergiya and Focht (1972), the average unit skin friction can be expressed as

$$f_s = \lambda (\bar{\sigma}_v + 2c) \qquad \ldots(25.14)$$

where λ = friction capacity factor, $\bar{\sigma}_v$ = mean effective vertical stress for the embedment length,
c = undrained cohesion.

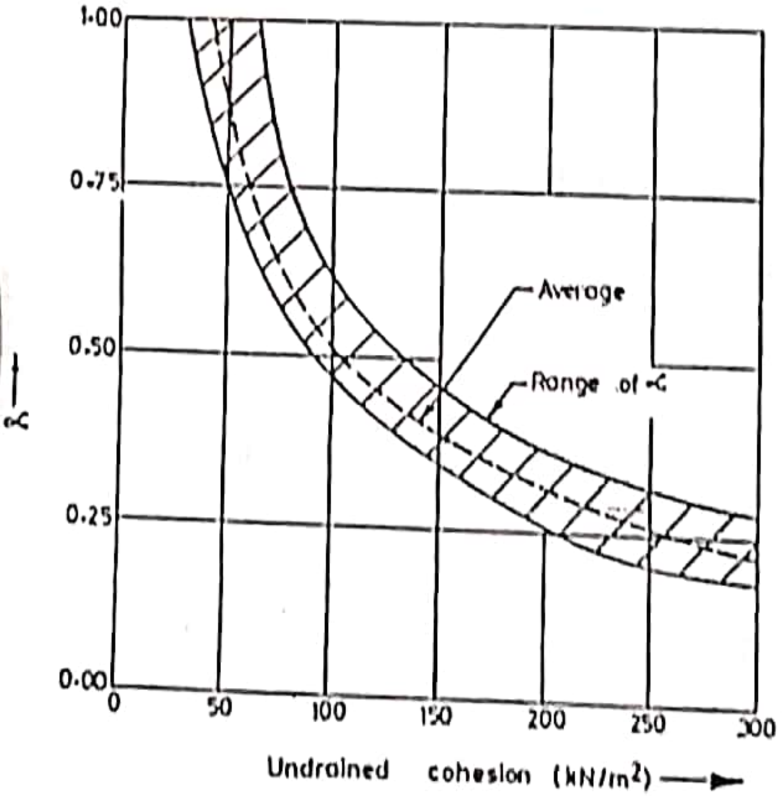


Fig. 25.7.

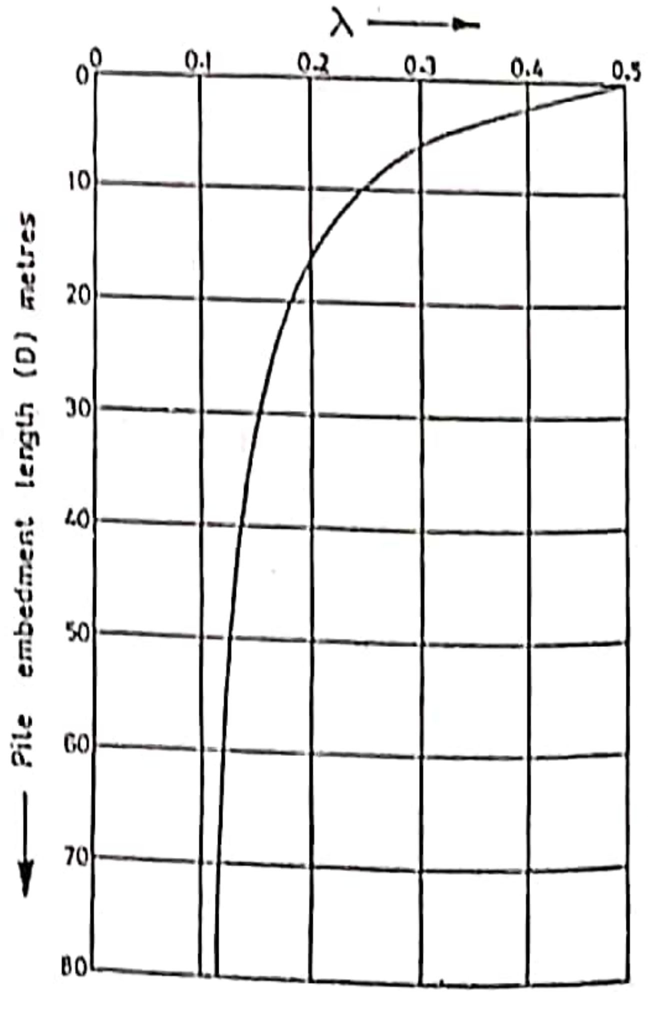


Fig. 25.8.

The value of λ can be obtained from Fig. 25.8 (McClelland, 1974).

Once the unit skin friction has been estimated, the shaft resistance is determined from Eq. 25.3.

For cohesive soils, the ultimate load can be determined by adding the point resistance and the shaft resistance (Eq. 25.1).

Thus $$Q_u = c\,N_c\,A_p + \alpha\,\bar{c}\,A_s \qquad ...(25.15)$$

As the clay gets remoulded when the pile is driven, this factor must be taken into account when estimating the load carrying capacity. The remoulded strength is always less than the undisturbed strength, but because of thixotropy, the strength improves with time. The rate of gain of strength depends upon the consolidation characteristics of the soil and the rate of dissipation of excess pore water pressure. When using Eq. 25.15, the value of c and $\bar{c}$ should be judiciously evaluated.

25.10. STATIC METHOD FOR BORED PILES

Bored piles are constructed by drilling a hole into the ground and filing it with concrete. The pile can be straight-sided for its full depth or may be constructed with a bell (or predestal) at its base. The piles with a pedestal are also known as *under-reamed* piles.

The load-carrying capacity of the bored piles can be determined using the procedure similar to that adopted for the driven piles. However, the values of the soil parameters are different, as described below.

(*a*) **Bored Piles in Sand.** Eq. 25.10 can be used to determine the ultimate load. The equation can be written as

$$Q_u = (\bar{q}\,N_q)A_p + \sum_{i=1}^{n} (K\,\bar{\sigma}_v\,\tan\delta)\,(A_s)_i \qquad ...(25.16)$$

where $\bar{\sigma}_v$ = effective vertical pressure, limited to a maximum value given by the critical depth.

K = lateral earth pressure coefficient for bored foundation.

$\tan\delta$ = coefficient of friction between sand and concrete.

The sand in bored piles is loosened as a result of the boring operation, even though it may initially be in a dense or medium dense state. The value of ϕ to be used to obtain N_q should be for the loose condition.

An approximate value of K can be obtained from the following equation.

$$K = 1 - \sin\phi$$

The value of K generally varies between 0.3 and 0.75. An average value of 0.5 is usually adopted.

The value of $\tan\delta$ can be taken equal to $\tan\phi$ for bored piles excavated in dry soil. If a slurry has been used during excavation, the value of $\tan\delta$ should be reduced.

In general, for a given initial value of ϕ, bored piles have a unit point resistance of $\frac{1}{2}$ to $\frac{2}{3}$ of that of corresponding driven piles. In driven piles, there is densification. Cast-in-place piles with a pedestal show about 50 to 100% greater unit point resistance compared with those without a pedestal. The impact energy of the hammer compacts the soil during the formation of the pedestal.

(*b*) **Bored piles in Clay.** Eq 25.15 can be used to estimate the ultimate load. The equation can be written as

$$Q_u = c\,N_c\,A_p + \alpha\,\bar{c}\,A_s \qquad ...(25.17)$$

where A_s = area of shaft that is effective in developing skin friction.

The value of α depends upon the pile type and the method of drilling. For straight shafts excavated dry, α is taken equal to 0.5 and that when drilled with slurry is 0.3. For belled shafts, the corresponding values are 0.3 and 0.15.

For calculating the area of shaft that is effective in developing skin friction, the lower 1.5 m (or 2 B) of the straight shaft and the bell section (if provided) are neglected, because of disturbance caused. For the same reason, the top 1.5 m is also neglected.

If a bored pile is installed in stiff, fissured clay, the value of cohesion (c) should be reduced to 75% of the value obtained from the triaxial test.

(c) **Underreamed Piles in Clay.** The base area of an underreamed pile is increased by underreaming and providing a bulb [Fig. 25.9 (*a*)]. The ultimate load is given by

$$Q_u = c N_c A_b + \alpha \bar{c} A_s' \quad \text{...[25.17 (a)]}$$

where A_b is the area of the enlarged base.

The value of N_c is taken as 9.0. The adhesion factor α is taken as 0.40. When the bulb is slightly above the tip, A_b is taken equal to the area of the diameter of the bulb and the projected stem below the bulb is ignored. The average value of c at the bulb is taken. However, if the bulb is quite long, and there is considerable difference in the value of c at the bulb level and the level of the bottom tip of the pile, the ultimate load is given by

$$Q_u = \frac{\pi}{4}(B^2) \times (9\,c) + \frac{\pi}{4}(B_1^2 - B^2) \times 9\,c' + \alpha \bar{c} A_s \quad \text{...[25.17(b)]}$$

where B = diameter of the pile shaft, B_1 is the diameter of the bulb, c is the unit cohesion at the tip, and c' is the unit cohesion at bulb level. $\bar{c}$ is the average cohesion on the shaft.

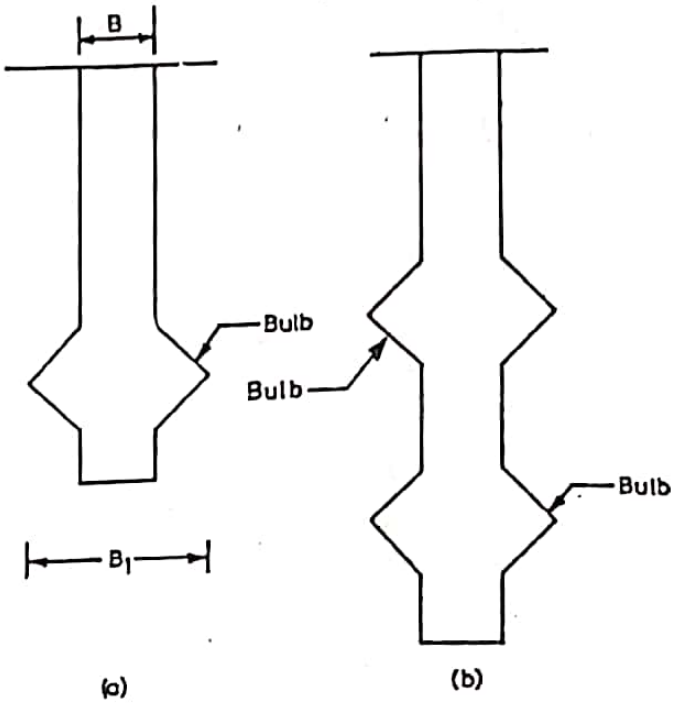


Fig. 25.9.

While calculating the surface area A_s, the length of the shaft equal to 2 B above the bulb is usually neglected. As the pile settles, there is a possibility of formation of a small gap between the top of the bulb and the overlying soil over a length of 2 B, and therefore, this length of the shaft is neglected. The little portion of the shaft projecting below the shaft is also neglected while computing A_s'.

When two or more bulbs are provided, the ultimate load is given by

$$Q_u = \frac{\pi}{4}(B^2) \times (9c) + \frac{\pi}{4}(B_1^2 - B^2) \times (9\,c') + \alpha\,\bar{c} A_s + c_a' A_{sb} \quad \text{...[25.17(c)]}$$

where A_s = surface area of shaft above the top bulb (ignoring 2 B length), A_{sb} = surface of the cylinder circumscribing the bulbs between top and bottom bulbs, c_a = average cohesion on A_s and c_a' = average cohesions on A_{sb}. For more details, see chapter 34.

25.11. ALLOWABLE LOAD

The allowable load (Q_{all}) is obtained from the ultimate load (Q_u) from the relation

$$Q_{all} = Q_u / FS \quad \text{...(25.18)}$$

where FS is the factor of safety. FS generally varies between 2.5 and 4.0, depending upon the uncertainties involved in the computation of the ultimate load. According to IS : 2911—1979, the minimum factor of safety on static formula shall be 2.5. The final selection of the value of the factor of safety should take into account the load settlement characteristics of the structure as a whole.

25.12. NEGATIVE SKIN FRICTION

When the soil layer surrounding a portion of the pile shaft settles more than the pile, a downward drag occurs on the pile. The drag is known as negative skin friction.

Negative skin friction develops when a soft or loose soil surrounding the pile settles after the pile has been installed. The negative skin friction occurs in the soil zone which moves downward relative to the pile. The negative friction imposes an extra downward load on the pile. The magnitude of the negative skin friction is computed using the same method as discussed in the preceding sections for the (positive) frictional resistance. However, the direction is downwards.

The net ultimate load-carrying capacity of the pile is given by the equation (Fig. 25.10).

$$Q_u' = Q_u - Q_{nsf} \quad \ldots(25.19)$$

where Q_{nsf} = negative skin friction,

Q_u' = net ultimate load.

Where it is anticipated that negative skin friction would impose undesirable, large downward drag on a pile, it can be eliminated by providing a protective sleeve or a coating for the section which is surrounded by the settling soil.

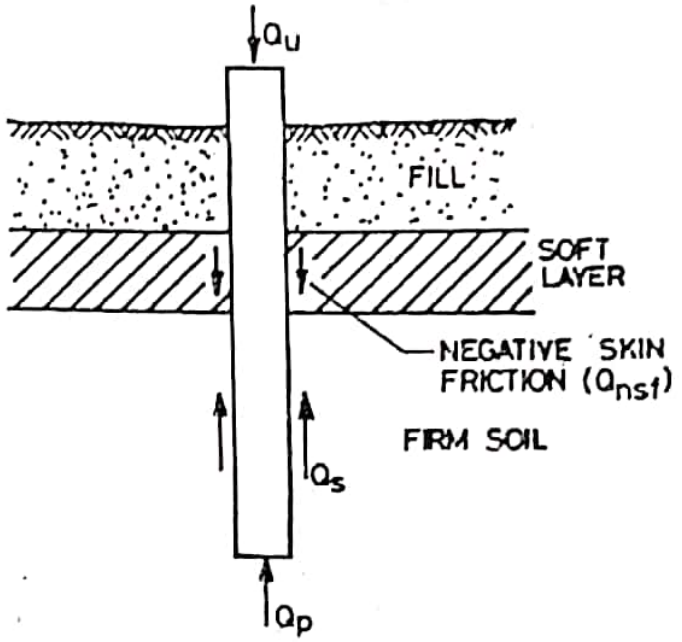


Fig. 25.10.

25.13. DYNAMIC FORMULAE

The load-carrying capacity of a driven pile can be estimated from the resistance against penetration developed during driving operation. The methods give fairly good results only in the case of free-draining sands and hard clays in which high pore water pressures does not develop during the driving of piles. In saturated fine-grained soils, high pore water pressure develops during the driving operation and the strength of the soil is considerably changed and the methods do not give reliable results. The methods cannot be used for submerged, uniform fine sands which may be loose enough to become quick temporarily and show a much less resistance.

The dynamic formulae are based on the assumption that the kinetic energy delivered by the hammer during driving operation is equal to the work done on the pile. Thus

$$W h \eta_h = R \times S \quad \ldots(25.20)$$

where W = weight of hammer (kN), h = height of ram drop (cm), η_h = efficiency of pile hammer, R = pile resistance (kN), taken equal to Q_u, and S = pile penetration per blow (cm).

In Eq. 25.20, no allowance has been made for the loss of energy during driving operation, loss caused by elastic contraction of the pile, soil, pile cap, cushion and due to the inertia of the pile. Some energy is also lost due to generation of heat. Various formulae have been proposed, which basically differ only in the methods for accounting of the energy losses, as described below.

(1) Engineering News Record Formula. According to Engineering News Record (ENR) formula (1888), the ultimate load is given by

$$Q_u = \frac{W h \eta_h}{S + C} \quad \ldots(25.21)$$

where S = penetration of pile per hammer blow. It is generally based on the average penetration obtained from the last few blows (cm), C = constant (For drop hammer, C = 2.54 cm and for steam hammer, C = 0.254 cm)

In Eq. 25.21, the product $W \times h$ can be replaced by the rated energy of hammer (E_n) in kN-cm. Thus

$$Q_u = \frac{E_n \eta_h}{S + C} \quad \ldots(25.22)$$

The efficiency η_h of the drop hammer is generally between 0.7 and 0.9, and that for a single-acting and a double-acting hammer is between 0.75 and 0.85. For diesel hammer, it usually lies between 0.80 and 0.90.

A factor of safety of 6 is usually recommended. However, the pile load tests reveal that the actual factor of safety varies between 2/3 and 30. The formula is, therefore, not dependable.

Modified Formula. The Engineering News Record formula has been modified recently. In the modified formula, the energy losses in the hammer system and that due to impact are considered. According to this formula,

$$Q_u = \frac{W h \eta_h}{S + C} \cdot \left(\frac{W + e^2 P}{W + P} \right) \qquad \text{...[25.21(a)]}$$

where P = weight of pile; e = coefficient of restitution, and η_h = hammer efficiency.

The hammer efficiency (η_h) depends upon various factors, such as pile driving equipment, driving procedure, type of pile and the ground conditions. For drop hammers, it is usually taken between 0.75 and 1.0; for single acting hammers between 0.75 and 0.85; for double-acting or differential hammer, η_h = 0.85 and for diesel hammer, η_h = 0.85 to 1.00.

The representative values of the coefficient of restitution (e) are as under.

Broomed timber pile	= 0.0
Good timber pile	= 0.25
Driving cap with timber dolly on steel pile	= 0.3
Driving cap with plastic dolly on steel pile	= 0.5
Helmet with composite plastic dolly and packing on R.C.C. pile	= 0.4

(2) Hiley Formula. Hiley (1925, 1930) gave a formula which takes into account various losses.

$$Q_u = \frac{W h \eta_b \eta_h}{(S + C/2)} \qquad \text{...(25.23)}$$

where η_h = efficiency of hammer blow, h = height of free fall of the ram or hammer (cm), S = final set or penetration per blow (cm), C = sum of temporary elastic compression of the pile, dolly, packings and ground (= $C_1 + C_2 + C_3$), C_1. temporary compression of dolly and packing (= 1.77 R/A, when the driving is without dolly, = 9.05 R/A, when the driving is with short dolly), C_2 = temporary compression of pile (= 0.657 RD/A), C_3 = temporary compression of ground (= 3.55 R/A), D = length of the pile, A = cross-sectional area of pile, R = pile resistance (tonnes).

The efficiency of hammer blow (η_b) depends upon the weight of hammer (W), weight of pile, anvil and helmet follower (P) and the coefficient of resistution (e).

(*a*) *For* $W > eP$,

$$\eta_b = \frac{W + e^2 P}{W + P} \qquad \text{...(25.24)}$$

(*b*) *For* $W < eP$,

$$\eta_b = \frac{W + e^2 P}{W + P} - \left(\frac{W - eP}{W + P} \right)^2 \qquad \text{...(25.25)}$$

The coefficient of resistution (e) varies from zero for a deteriorated condition of the head of pile to 0.5 for a steel ram of double-acting hammer striking on steel anvil and driving a reinforced concrete pile. For a C.I. ram of a single-acting or drop hammer striking on the head of R.C.C. pile, e = 0.4 and that striking on a well-conditioned driving cap and helmet with hard wood on R.C.C. pile, e = 0.25 (IS : 2911—1979).

(3) Danish Formula. According to Danish formula (1929),

$$Q_u = \frac{W \times h \times \eta_h}{S + 1/2 S_o} \qquad \text{...(25.26)}$$

where

$$S_o = \left[\frac{2 \eta_h (W h D)}{AE} \right]^{1/2} \qquad \text{...(25.27)}$$

in which S_o = elastic compression of pile, D = length of pile, A = cross-sectional area, E = modulus of elasticity of pile material.

The allowable load is found by taking a factor of safety of 3 to 4.

Eq. 25.27 can also be used to determine the final set (S) per blow.

Taking $Q_u = 3\,Q_a$,

$$S = \left(\frac{W h \eta_h}{3Q_a}\right) - \frac{1}{2} S_o \qquad ...(25.28)$$

where Q_a = allowable load.

25.14. WAVE EQUATION ANALYSIS

As the hammer strikes the top of a pile, a stress wave is transmitted through the length of the pile. The wave transmission theory can be used to determine the load carrying capacity of the pile and the maximum stresses that can occur within the pile during driving operation.

In the wave equation analysis (Smith, 1962), the pile is represented by a series of individual spring-connected weights and spring damping resistance (Fig. 25.11). The weight W_1 represents the weight of the ram, and W_2 represents the weight of the pile cap. Weights W_3 to W_{10} correspond to the weights of incremental sections of the pile. The spring constant K_1 represents the elasticity of the cap block; the constants K_2 to K_{11} are for the elasticity of the pile sections. The damping springs R_3 to R_{11} represent the frictional resistance of the soil surrounding the shaft; R_{12} reprèsents the soil resistance at the pile tip.

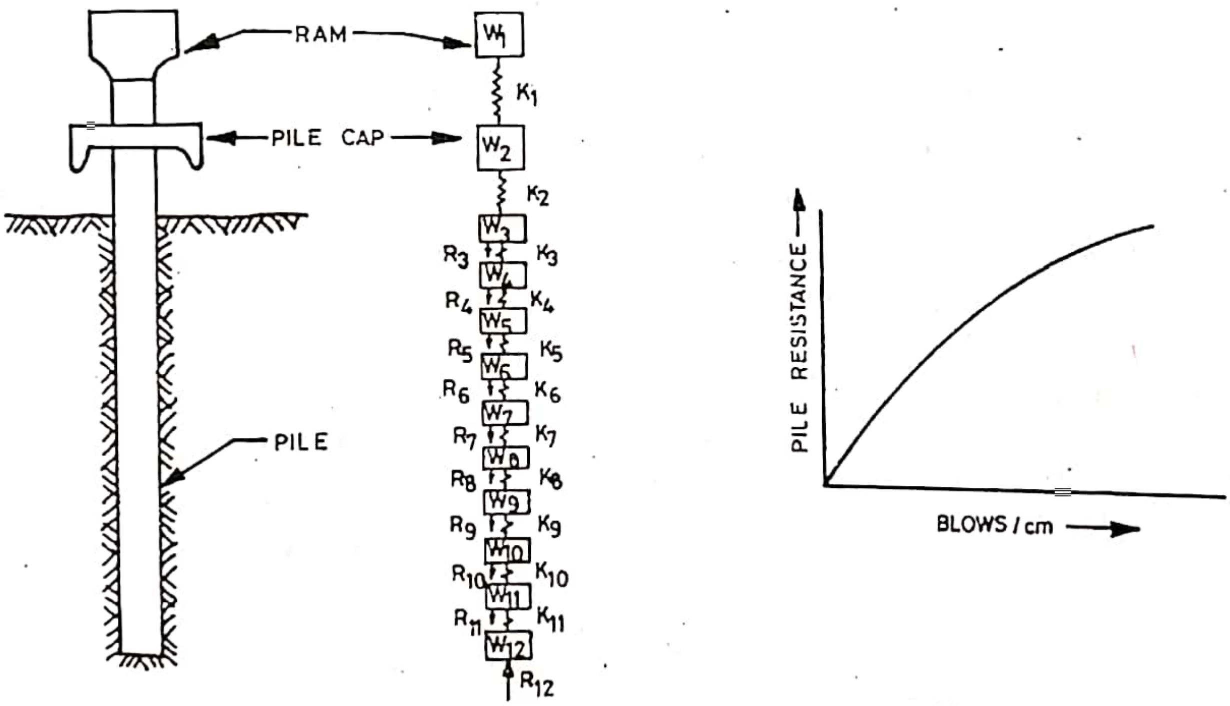


Fig. 25.11.

The propagation of the elastic wave through the pile is analogous to that caused by an impact on a long rod. A partial differential equation is written to describe the pile model shown in Fig. 25.11 (*a*). The equation is solved with the aid of a digital computer, and the pile capacity is determined. The pile capacity is expressed as a function of penetration per blow or blows per cm [Fig. 25.11 (*b*)].

The major drawback of the wave equation analysis for determination of the dynamic resistance is its dependence on a computer. Moreover, the field tests are required to estimate the equivalent spring constant and soil-damping values for the pile under study. Further, the results obtained are valid only for a particular pile driven by a specified pile hammer.

Despite the above shortcomings, the wave equation analysis is a useful tool for determining the pile capacity. The results can also be used for the selection of appropriate pile-driving equipment.

25.15. IN-SITU PENETRATION TESTS FOR PILE CAPACITY

(a) Standard penetration test. The load-carrying capacity of a pile can be estimated from the standard penetration test value (N).

(i) For driven piles in sand, the unit tip resistance (q_p) is related to the *uncorrected* blow count (N) near the pile point (Meyerhof 1976).

$$q_p = 40\,N\,(D/B) \le 400\,N \quad \text{...(25.29)}$$

where q_p = point resistance (kN/m^2), D = length of pile, B = width (diameter) of pile.

The value of q_p is usually limited to 400 N.

The average unit frictional resistance (f_s) is related to the average value of the blow count ($\overline{N}$).

$$\text{For high displacement piles, } f_s = 2.0\,\overline{N}\ \text{kN/m}^2 \quad \text{...[25.30 (a)]}$$

$$\text{For low displacement piles, } f_s = 1.0\,\overline{N}\ \text{kN/m}^2 \quad \text{...[25.30 (b)]}$$

where $\overline{N}$ is average of uncorrected N-values along the length of the pile.

(ii) For bored piles in sand, $q_p = 14\,N\,(D_b/B)$ kN/m^2 ...[25.31)]

where D_b = actual penetration into the granular soil.

For bored piles in sand, the unit frictional resistance (f_s) is given by

$$f_s = 0.67\,\overline{N}\ \text{kN/m}^2 \quad \text{...(25.32)}$$

(b) Dutch cone test. Meyerhof (1965) relates the unit point resistance (q_p) and the unit skin traction (f_s) of driven piles to the cone point resistance (q_c).

Point resistance, $q_p = \dfrac{q_c}{10}(D_b/B)$ ...(25.33)

Unit skin friction (a) f_s (dense sand) $= q_c/200$ ...(25.34)

(b) f_s (loose sand) $= q_c/400$ ...(25.35)

(c) f_c (silt) $= q_c/150$ ...(25.36)

25.16. PILE LOAD TEST

The most reliable method for determining the load carrying capacity of a pile is the pile load test. The set-up generally consists of two anchor piles provided with an anchor girder or a reaction girder at their top (Fig. 25.12). The test pile is installed between the anchor piles in the manner in which the foundation piles are to be installed. The test pile should be at least 3 B or 2.5 m clear from the anchor piles.

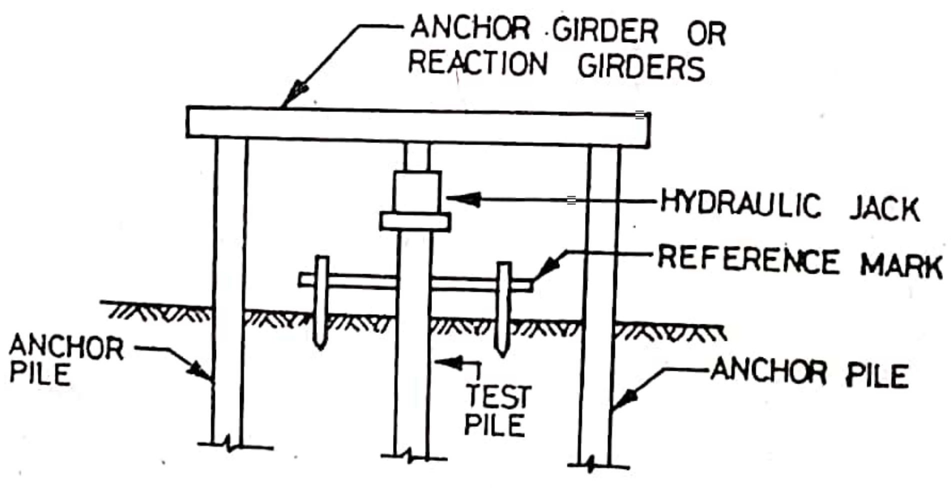


Fig. 25.12. Pile Load Test.

The load is applied through a hydraulic jack resting on the reaction girder. The measurements of pile movement are taken with respect to a fixed reference mark. The test is conducted after a rest period of 3 days after the installation in sandy soils and a period of one month in silts and soft clays. The load is applied in equal increment of about 20% of the allowable load. Settlements should be recorded with three dial gauges. Each stage of the loading is maintained till the rate of movement of the pile top is not more than 0.1 mm per hour in sandy soils and 0.02 mm per hour in case of clayey soils or a maximum of two hours (IS : 2911-1979). Under each load increment, settlements are observed at 0.5, 1, 2, 4, 8, 12, 16, 20, 60 minutes. The loading should be continued upto twice the safe load or the load at which the total settlement reaches a specified value. The load is removed in the same decrements at 1 hour interval and the final rebound is recorded 24 hours after the entire load has been removed.

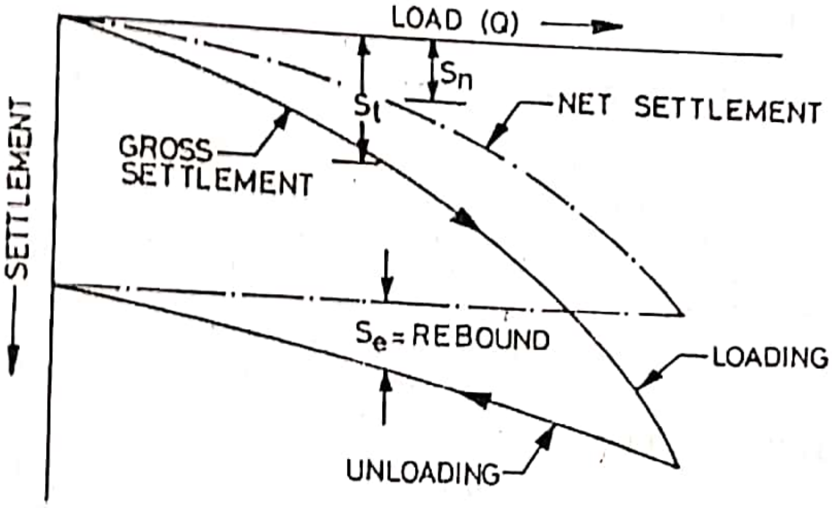


Fig. 25.13. Load Settlement Curve.

Fig. 25.13 shows a typical load-settlement curve (firm line) for loading as well as unloading obtained from a pile load test. For any given load, the net pile settlement (s_n) is given by

$$s_n = s_t - s_e \qquad ...(25.37)$$

where s_t = total settlement (gross settlement), s_e = elastic settlement (rebound).

Fig. 25.13 also shows the net settlement (chain dotted line).

Fig. 25.14 shows two load-net settlement curves obtained from a pile load tests on two different soils. At the ultimate load (Q_u), the load-net settlement curve becomes either linear as curve (2) or there is a sharp break as the curve (1), as shown in the figure. The safe load is usually taken as one-half of the ultimate load.

According to IS : 2911, the safe load is taken as one-half of the load at which the total settlement is equal to 10 per cent of the pile diameter (7.5 per cent in case of under-reamed piles) or two-thirds of the final load at which the total settlement is 12 mm, whichever is less. According to another cirterion, the safe load is taken as one-half to two-thirds of the load which gives a net settlement of 6 mm.

The limiting settlement criteria are also sometimes specified. Under the load twice the safe load, the net settlement should not be more than 20 mm or the gross settlement should not be more than ... mm.

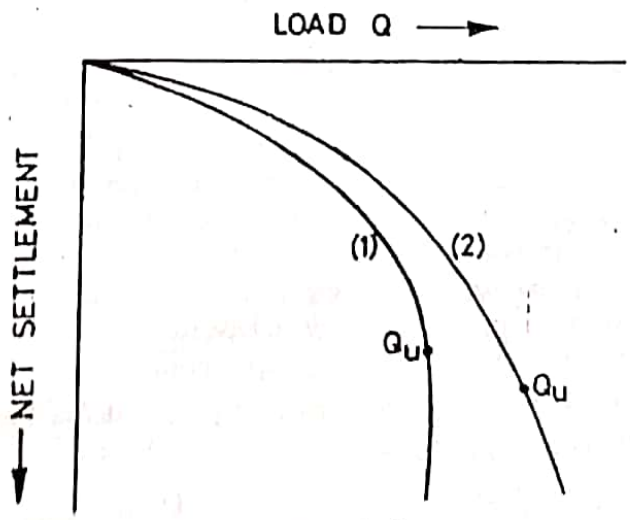


Fig. 25.14.

The test described above is known as *initial test.* It is carried out on a test pile to determine the ultimate load capacity and hence the safe load. The pile load test described in this section is a type of load-controlled test, in which the load is applied in steps. The test is also known as *slow maintained test.*

25.17. OTHER TYPES OF PILE LOAD TESTS

(1) Constant rate of penetration test. In a constant-rate of penetration test, the load on the pile is continuously increased to maintain a constant rate of penetration (from 0.25 to 5 mm per minute). The force required to achieve that rate of penetration is recorded, and a load-settlement curve is drawn. The ultimate load can be determined from the curve.

The test is considerably faster than a load-controlled test.

(2) Routine Load test. This test is carried out on a working pile with a view to determine the settlement corresponding to the allowable load. As the working pile would ultimately form a part of the foundation, the maximum load is limited to one and a half times the safe load or upto the load which gives a total settlement of 12 mm.

(3) Cyclic Load test. The test is carried out for separation of skin friction and point resistance of a pile. In the test, an incremental load is repeatedly applied and removed.

(4) Lateral Load test. The test is conducted to determine the safe lateral load on a pile. A hydraulic jack is generally introduced between two piles to apply a lateral load. The reaction may also be suitably obtained from some other support. The test may also be carried out by applying a lateral pull by a suitable set-up.

(5) Pull out test. The test is carried out to determine the safe tension for a pile. In the set-up, the hydraulic jack rests against a frame attached to the top of the test pile such that the pile gets pulled up.

25.18. GROUP ACTION OF PILES

A pile is not used singularly beneath a column or a wall, because it is extremely difficult to drive the pile absolutely vertical and to place the foundation exactly over its centre line. If eccentric loading results, the connection between the pile and the column may break or the pile may fail structurally because of bending stresses. In actual practice, structural loads are supported by several piles acting as a group. For columns, a minimum of three piles in a triangular pattern are used. For walls, piles are installed in a staggered arrangement on both sides of its centre line. The loads are usually transferred to the pile group through a reinforced concrete slab, structurally tied to the pile tops such that the piles act as one unit. The slab is known as a *pile cap.* The load acts on the pile cap which distributes the load to the piles (Fig. 25.15).

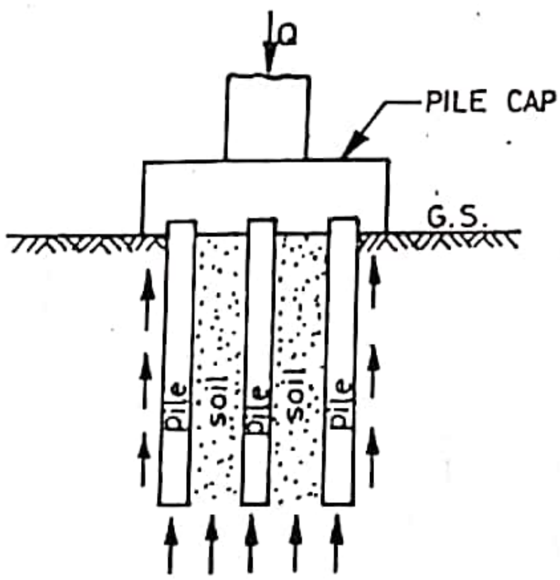


Fig. 25.15.

The load carrying capacity of a pile group is not necessarily equal to the sum of the capacity of the individual piles. Estimation of the load-carrying capacity of a pile group is a complicated problem. When the piles are spaced a sufficient distance apart, the group capacity may approach the sum of the individual capacities. On the other hand, if the piles are closely spaced, the stresses transmitted by the piles to the soil may overlap, and this may reduce the load-carrying capacity of the piles (Fig. 25.16). For such a case, the capacity is limited by the group action.

The efficiency (η_g) of a group of piles is defined as the ratio of the ultimate load of the group to the sum of individual ultimate loads.

Thus
$$\eta_g = \frac{Q_{g(u)}}{NQ_u} \times 100 \qquad \ldots(25.38)$$

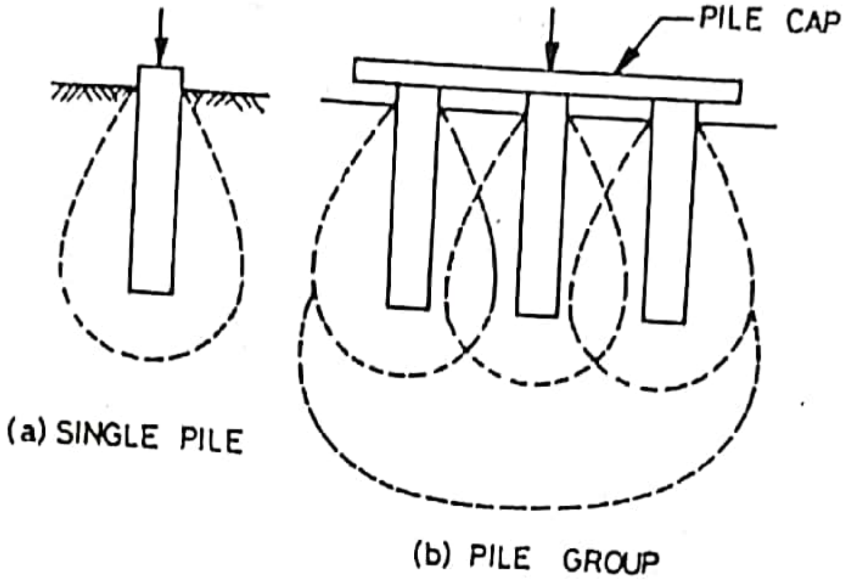


Fig. 25.16.

or
$$\eta_g = \frac{Q_{g(u)}/N}{Q_u} \times 100$$

where $Q_{g(u)}$ = ultimate load of the group, Q_u = ultimate load of the individual pile,

N = Number of piles in the group.

Thus the groups efficiency is equal to the ratio of the average load per pile in the group at which the failure occurs to the ultimate load of a comparable single pile,

The group efficiency depends upon the spacing of the piles. Ideally, the spacing should be such that the efficiency is 100%. Generally, the centre to centre spacing is kept between 2.5 B and 3.5 B, where B is the diameter of the pile.

The methods for the determination of the ultimate load of the individual piles have been discussed earlier. The methods for the estimation of the ultimate load of the group are explained in the following sections.

25.19. PILE GROUPS IN SAND AND GRAVEL

For piles driven in loose and medium dense cohesionless soils, the group efficiency is high. The soil around and between the piles is compacted due to vibration caused during the driving operation. For better results, it is essential to start driving the piles at the centre and then work outward.

The piles and the soil between them move together as a unit when subjected to loads. The group acts as pier foundation having a base equal to the gross plan area contained between the piles.

(*a*) **End-bearing piles.** For driven piles bearing on dense, compact sand with a spacing equal to or greater than 3 B, the group capacity is generally taken equal to the sum of individual capacity. Thus

$$Q_g = N\,Q_u \qquad \text{...(25.39)}$$

In this case, the load taken by the group is much greater (η_g > 100%) than the sum of the individual capacities, and the piles fail as individual piles.

For spacing less than 3 B, the group capacity is found for the block of piles group.

(*b*) **Friction piles.** The group efficiency of friction piles in sand is obtained from the following expression:

$$\eta_g = \frac{Q_{g(u)}}{NQ_u} \times 100 = \frac{f_s\,(P_g\,D)}{Nf_s\,(pD)} \times 100 \qquad \text{...(25.40)}$$

where P_g = perimeter of the block, p = perimeter of the individual pile, D = length of pile,

f_s = unit friction resistance.

If the centre-to-centre spacing is large, the group efficiency (η_g) may be more than 100%. The piles will behave as individual piles, and the group capacity is obtained from Eq. 25.39.

If η_g is less than 100%,

$$Q_g = \eta_g \frac{(NQ_u)}{100} \qquad ...(25.41)$$

The group efficiency can also be obtained from the Converse- Lebarre equation given below.

$$\eta_g = 1 - \left[\frac{(n-1)\,m + (m-1)\,n}{m\,n}\right] \cdot \frac{\theta}{90} \qquad ...(25.42)$$

where m = number of rows of piles, n = number of piles in a row, $\theta = \tan^{-1}$ (B/s), B = diameter of pile, s = spacing of pile, centre- to-centre, η_g = group efficiency (expressed as a ratio).

Bored piles. For bored piles in sand at conventional spacing of 3 B, the group capacity is taken as 2/3 end-bearing and the friction piles. Thus to 3/4 times the sum of individual capacities for both the end-bearing and the friction piles. Thus

$$Q_g\,(u) = (2/3 \text{ to } 3/4)\,(N\,Q_u) \qquad ...(25.43)$$

In bored piles, there is limited densification of the sand surrounding the pile group. Consequently, the efficiency is lower.

25.20. PILE GROUPS IN CLAY

As the pile group acts as a block, its ultimate capacity is determined by adding the base resistance and the shaft resistance of the block. The capacity of the block having closely spaced piles ($s \le 3B$) is often limited by the behaviour of the group acting as a block. The group capacity of the block is given by

or

$$Q_g\,(u) = q_p\,(A_g) + \alpha\,c\,(P_g\,D) \qquad ...(25.44)$$

where q_p = unit point resistance (N_c = 9.0), A_g = base area of the block, P_g = perimeter of the block, D = depth of the block, α = adhesion factor (= 1.0 for soft clays), c = undrained cohesion.

As discussed earlier, the individual pile capacity is given by Eq. 25.15,

$$Q_u = q_p\,A_p + \alpha\,c\,(p \times D) \qquad ...(25.45)$$

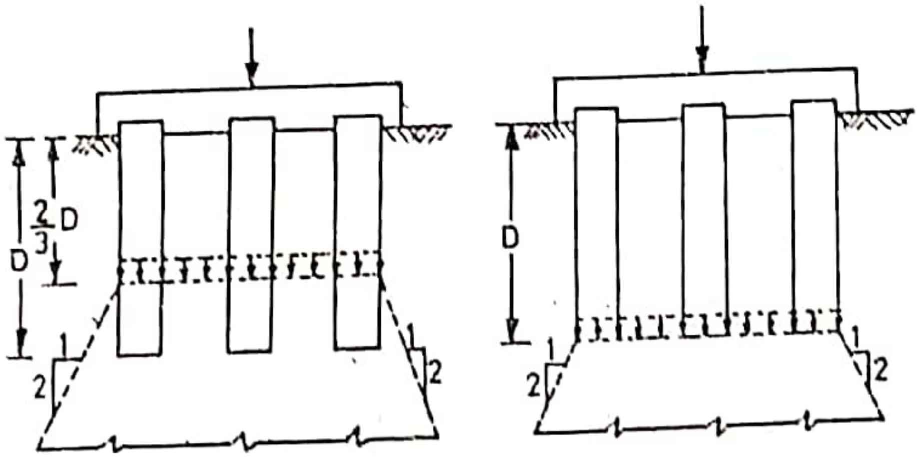
The group capacity considering the piles as individual piles is given by

$$Q_g\,(u) = N\,Q_u \qquad ...(25.46)$$

The lower of the two values, given by Eqs. 25.44 and Eq. 25.46, is the actual capacity.

25.21. SETTLEMENT OF PILE GROUPS

The settlement of a pile group is due to elastic shortening of piles and due to the settlement of the soil supporting the piles. It is assumed that the pile group acts as a *single large deep foundation*, such as a pier



(a) FRICTION PILES (b) END BEARING PILES

Fig. 25.17.

or a mat. The total load is assumed to act at a depth equal to two-thirds the pile length in the case of frictional piles [Fig. 25.17 (*a*)]. In the case of end-bearing piles, the total load is assumed to act at the pile tips [Fig. 25.17 (*b*)]. In the case of combined action, the frictional component is assumed to act at 2/3 D and the bearing component at the tip.

For determination of the settlements, the compression characteristics of the soil are required. For clayey soils, the characteristics are determined from laboratory tests on undisturbed samples. For cohesionless soils, the characteristics are obtained from empirical correlations developed from in-situ penetration tests.

(*a*) Cohesionless soils

(*i*) **Skempton method.** The settlement of the pile group is estimated from the settlement of a single pile, as determined in a pile-load test. The settlement of the group is generally very large because the pressure bulb for the group is much deeper than that of a single pile.

Skempton et al (1953) published curves (Fig. 25.18) relating the settlement of the pile group (s_g) of a given total foundation width to that of a single pile (s_o). The curves can be used for both driven and bored piles.

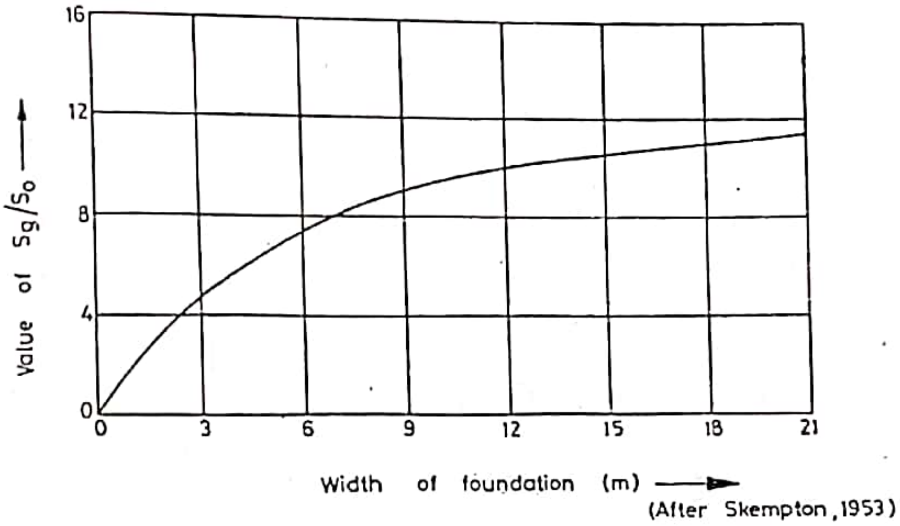


Fig. 25.18.

(*ii*) **Meyerhof method.** Meyerhof (1976) suggests the following empirical relation for the elastic settlement of a pile group in sands and gravels.

$$s_g = \frac{9.4\, q\, \sqrt{B_g}\, I}{N} \qquad \ldots(25.47)$$

where s_g = settlement of group (mm), q = load intensity (= Q_g/A_g), B_g = width of the group, I = influence factor [= $1 - D/(8\,B_g) \geq 0.5$], D = length of pile, N = corrected standard penetration number within the seat of settlement (approximately equal to B_g below the tip).

If static cone results are available, the settlement of the group can be obtained from the relation,

$$s_g = \frac{q\, B_g\, I}{2\, q_c} \qquad \ldots(25.48)$$

where q_c = average cone penetration resistance within the seat of settlement.

(*b*) Clayey soils

The consolidation settlement of a pile group in clay can be determined using the procedure discussed in chapter 12. Generally, a 2 : 1 load distribution is assumed from the level at which the load acts. Sometimes, the load is assumed to spread outwards from the edge of the block at an angle of 30° to the vertical. For 2 : 1 distribution, the stress increase at the middle of each layer is calculated as (see chapter 11),

$$q_i = \frac{Q_g}{(B_g + Z_i)(L_z + Z_i)} \quad ...(25.49)$$

where Z_i is the distance from the level of the application of the load to the middle of clay layer i.

The settlement of each layer caused by the increased stress is given by (see chapter 12).

$$\Delta s\,(i) = \frac{\Delta e\,(i)}{1 + e_o\,(i)}\, H_i \quad ...(25.50)$$

where $\Delta e\,(i)$ = change of void ratio caused by the stress increase, $e_o\,(i)$ = initial void ratio of layer i, H_i = thickness of layer i.

Alternatively,

$$\Delta s\,(i) = C_c \frac{H_i}{1 + e_o\,(i)} \log \left(\frac{\bar{\sigma}_o + \Delta \sigma_i}{\bar{\sigma}_o} \right) \quad ...(25.51)$$

The total consolidation settlement is equal to the sum of the settlement of all layers.

$$s_g = \Sigma \Delta s\,(i) \quad ...(25.52)$$

25.22. SHARING OF LOADS IN A PILE GROUP

All the piles in a group share equal load if the load is central.

$$Q = \frac{Q_g}{N} \quad ...(25.53)$$

However, if the load is eccentric or if the central load is accompanied by a moment, the sharing of load is computed assuming the pile cap as rigid. As the pressure distribution is planar, the pile reactions also vary linearly with the distance from the centroid of the cap (Fig. 25.19). The axial load in any pile m at a distance x from the centroid is given by

$$Q_m = \frac{Q_g}{N} \pm \frac{(Q_g \cdot e_x)\, x}{\Sigma x^2} \quad ...(25.54)$$

where e_x = eccentricity of load about Y-Y-axis,

If the load is eccentric about both the axes.

$$Q_m = \frac{Q_g}{N} \pm \frac{(Q_g \cdot e_x)\, x}{\Sigma x^2} \pm \frac{(Q_z\, e_y)\, y}{\Sigma y^2} \quad ...(25.55)$$

where e_y = eccentricity of load about X-X axis,

In the above equations, the positive sign is taken for the piles on the same side as the eccentricity.

If the load on any pile is negative, it indicates that the pile is in tension. If the pile is not designed for tension, the load in that pile is taken as zero, and the load between other piles is redistributed. This would cause extra compression in other piles.

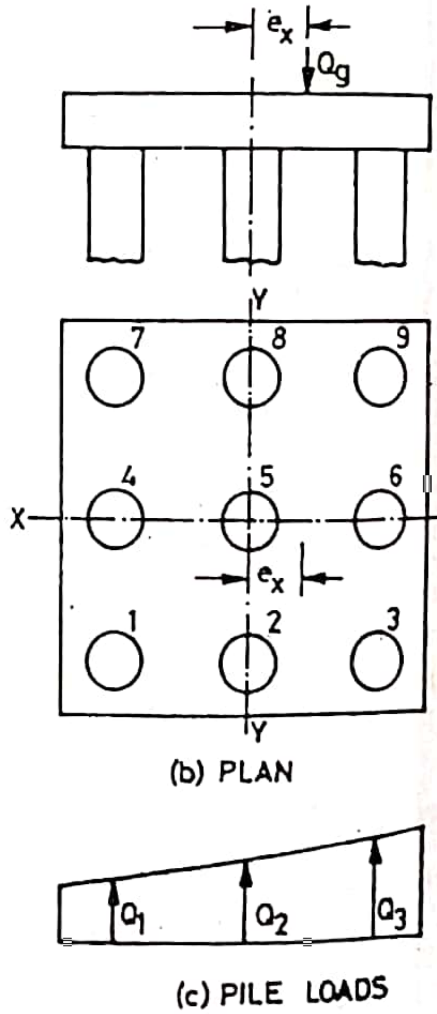


Fig. 25.19.

25.23. TENSION PILES

Piles supporting high structures, such as tall chimneys, transmission towers, water towers, are required to resist uplift forces due to wind. Some of the piles in these structures are required to resist tensile forces and are known as tension piles.

Resistance to uplift forces is provided by the friction between the pile and the surrounding soil. The uplift resistance of a straight-shaft pile can be computed in the same manner as the frictional resistance in frictional piles. However, the unit skin friction (f_s) and adhesion (c_a) for the uplift resistance are considerably less than those for the compressive loads. It is usual practice to reduce these values to one-half of the normal values if the piles are short. For large structures, it is essential to carry out pull out tests on piles to determine the safe value of the unit skin friction or adhesion for uplift forces.

The uplift resistance of piles can be considerably increased in the case of bored piles by under-reaming or belling out the bottom. A bulb can also be formed in the case of driven and cast- in place piles to increase the uplift resistance.

Mayerhof and Adams (1968) gave the following equations for the pull-out resistance (P_u).

(*a*) Shallow piles Fig. [25.20 (*a*)]. Pull-out resistance,

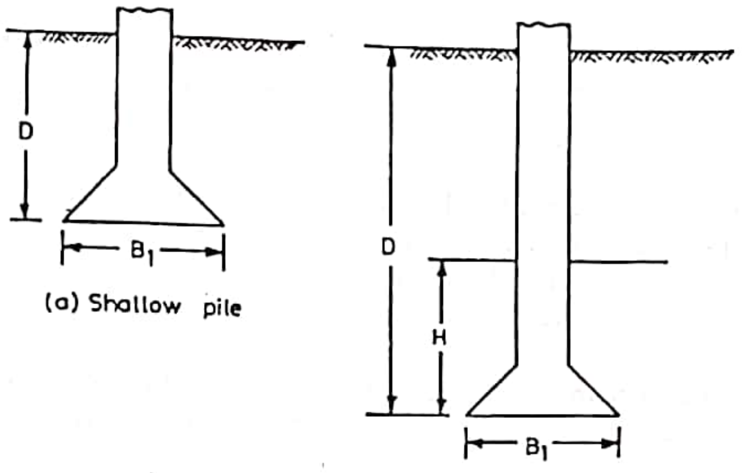


Fig. 25.20.

P_u = cohesive resistance + frictional resistance

or $$P_u = \pi B_1 c_u D + s_f \gamma \left(\frac{\pi}{2} B_1\right) D^2 K_u \tan \phi + W \quad ...(25.56)$$

where B_1 = diameter of enlarged base, c_u = undrained cohesion, D = length of pile, ϕ = angle of shearing resistance, s_f = shape factor (see Table 25.3), K_u = coefficient of lateral earth pressure (= $K_p \tan \frac{2}{3} \phi$), K_p = coefficient of passive earth pressure $\left(= \frac{1 + \sin \phi}{1 - \sin \phi} \right)$, γ = bulk unit weight, and W = weight of soil and pile in a cylinder of diameter B_1 and height D.

(*b*) Deep piles Fig. [25.20 (*b*)], Pull-out resistance,

P_u = cohesive resistance + frictional resistance

or $$P_u = \pi B_1 c_u H + s_f \gamma \left(\frac{\pi}{2} B_1\right) (2D - H) H K_u \tan \phi + W \quad ...(25.57)$$

where H = maximum height of rupture surface (see Table 25.3) (For deep piles $H \le D$)

Table 25.3. Values of H/B_1, m and s_f

ϕ	20°	25°	30°	35°	40°	45°	50°
H/B_1	2.5	3.0	4.0	5.0	7.0	9.0	11.0
m	0.05	0.10	0.15	0.25	0.35	0.50	0.60
s_f	1.12	1.30	1.60	2.25	3.45	5.50	7.60

All other notations are the same as before.

For purely cohesive soils, as $\phi = 0$, the second term in Eqs. 25.56 and 25.57 is zero. For cohesionless soils, as $c_u = 0$, the first term is zero. The shape factor (s_f) is equal to $1 + mD/B_1$ for short piles, and equal to $1 + mH/B_1$ for deep piles, where m is a coefficient depending on ϕ.

25.24. LATERALLY LOADED PILES

Piles are sometimes subjected to lateral loads due to wind pressure, water pressure, earth pressure, earthquakes, etc. When the horizontal component of the load is small in comparison with the vertical load (say, less than 20%), it is generally assumed to be carried by vertical piles and no special provision for lateral load is made.

If the horizontal load is large, inclined piles, known as *raking piles or batter piles*, are provided to take the horizontal load. These piles have a high resistance to lateral loads, as a large portion of the horizontal component of the load is carried axially by the pile. Batter piles, along with vertical piles, are provided in situations where the horizontal loads are significant, such as wharves, jetties, bridge piers, trestles, retaining wall and tall chimneys.

Batter piles are driven at a batter ranging from 1 : 12 to 1 : 25. However, driving of batter piles is more expensive than that of vertical piles. The resistance to failure of vertical piles subjected to horizontal loads is provided by the passive resistance of a wedge of soil in front of the piles. In case of batter piles, additional resistance is provided by the skin friction and the end bearing. Therefore, batter piles are more effective than vertical piles in resisting horizontal loads.

It is generally assumed that batter piles can take the axial load equal to that in the corresponding vertical pile. As the axis of the batter pile is inclined, it can resist the horizontal load equal to $Q \cos \theta$, where Q is the axial load capacity and θ is the angle which the pile makes with the horizontal. When piles are oriented in two or three directions, Culmann's method, as described below, is used.

Steps : (1) Group the piles according to their slopes. [In Fig. 25.21 (*a*), the piles are grouped in 3 directions].

(2) Draw the geometry of the pile group to some scale, and mark the directions of the inclined load Q_g and the centre line of each pile group (R_1, R_2 and R_3).

(3) Determine the location of point A which is at the intersection of R_1 and Q_g.

(4) Join A to the point B which is at the intersection of R_2 and R_3.

(5) Draw the force triangle [Fig. 25.21 (*b*)]. Select the line ab parallel to AB. From b draw a line bc parallel to Q_g to some scale. Draw a vertical at c to determine ca which is equal to R_1.

From b draw a line parallel to R_3, and from a, line parallel to R_2, to complete the triangle abd.

(6) Determine forces in piles as follows.

The magnitudes of R_2 and R_3 are, respectively, given by ad and bd. However, R_2 is compressive and R_3 is tensile.

The magnitude of R_1 is given by ca which is compressive.

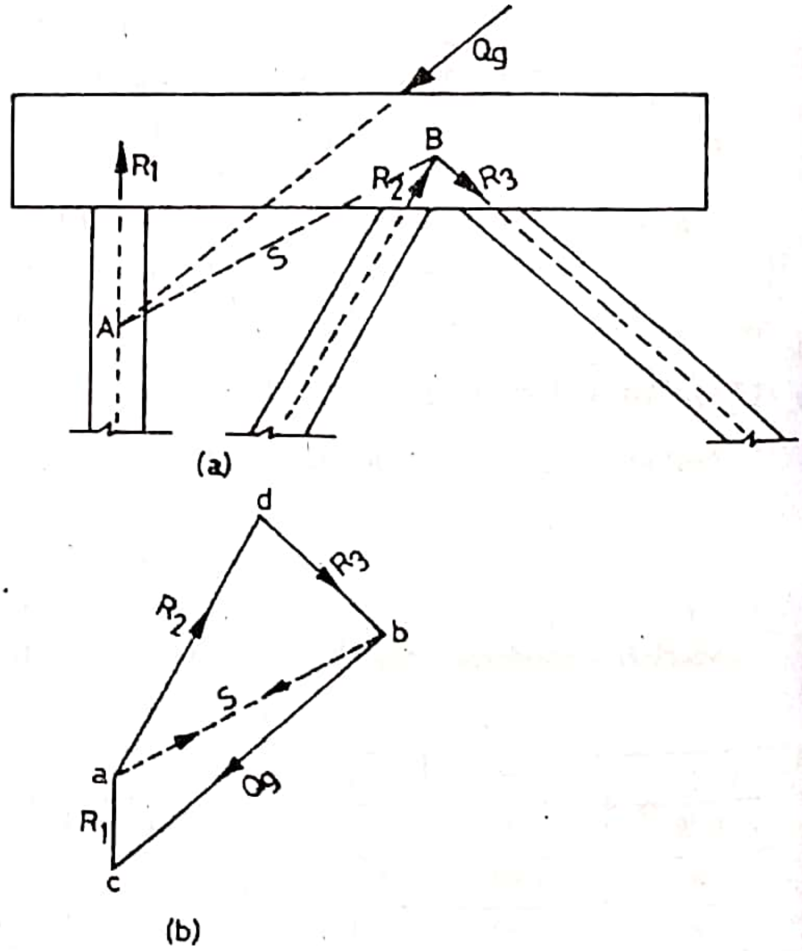


Fig. 25.21.

Illustrative Example 25.1. *A concrete pile, 30 cm diameter, is driven into a medium dense sand (ϕ = 35°, γ = 21 kN/m², K = 1.0, tan δ = 0.70) for a depth of 8 m. Estimate the safe load, taking a factor of safety of 2.50.*

Solution. From Eq. 25.10, $Q_u = \bar{q} N_q A_p + \sum_{i=1}^{n} K(\bar{\sigma}_v)_i \tan\delta\, (A_s)_i$

For ϕ = 35°, D_c/B = 12.0, from Fig. 25.3.

$$D_c = 12 \times 0.3 = 3.6 \text{ m}$$

$$\text{Maximum value of } \bar{\sigma}_v = 3.6 \times 21 = 75.6 \text{ kN/m}^2$$

The value of N_q is taken from Berezontzev's curve (Fig. 25.4). N_q = 60.

Therefore, $Q_u = 75.6 \times 60 \times \pi/4 \times (0.3)^2 + K \tan\delta$ (area of $\bar{\sigma}_v$ diagram) × piles perimeter

$$= 320.5 + 1.0 \times 0.70 \times \left(\frac{1}{2} \times 75.6 \times 3.6 + 75.6 \times 4.4\right) \times \pi \times 0.3$$

or $$Q_u = 320.5 + 309.2 = 629.7 \text{ kN}$$

$$\text{Safe load, } Q_a = \frac{Q_u}{2.5} = \frac{629.7}{2.5} = 251.9 \text{ kN}$$

Illustrative Example 25.2. *Determine the safe load for the pile in Illustrative Example 25.1, if the water table rises to 2 m below the ground surface. Take γ_w = 10 kN/m³.*

Solution. Vertical pressure at the critical depth,

$$\bar{\sigma}_v = 2 \times 21 + 1.6 \times (21 - 10) = 59.6 \text{ kN/m}^2$$

Therefore, $$Q_u = 59.6 \times 60 \times \pi/4 \times (0.3)^2 + \left(\frac{1}{2} \times 59.6 \times 3.6 + 59.6 \times 4.4\right) \times 0.7 \times \pi \times 0.3$$

or $$Q = 252.6 + 243.7 = 496.3 \text{ kN}$$

$$Q_a = \frac{496.3}{2.5} = 198.5 \text{ kN}$$

Illustrative Example 25.3. *A 30 cm diameter concrete pile is driven into a homogeneous consolidated clay deposit (c_u = 40 kN/m², α = 0.7). If the embedded length is 10 m, estimate the safe load (F.S. = 2.5).*

Solution. From Eq. 25.15, $Q_u = c N_c A_p + \alpha \bar{c} A_s$

Taking N_c = 9.0,

$$Q_u = (40 \times 9.0)\, \pi/4 \times (0.3)^2 + 0.7 \times 40\, (\pi \times 0.3) \times 10 = 289.2 \text{ kN}$$

$$Q_a = \frac{Q_u}{2.5} = \frac{289.2}{2.5} = 115.7 \text{ kN}$$

Illustrative Example 25.4. *A square concrete pile (30 cm side) 10 m long is driven into coarse sand (γ = 18.5 kN/m³, N = 20). Determine the allowable load (F.S. = 3.0).*

Solution From Eq. 25.29, $q_p = 40\, N\, (D/B) \le 400\, N$

In this case, $$40\, N\, (D/B) = 40 \times 20\, (10/0.3) = 26666.7 \text{ kN/m}^2$$

$$400\, N = 400 \times 10 = 8000 \text{ kN/m}^2$$

Adopt the lower value of 8000 kN/m²

From Eq. 25.30 (*a*), $f_s = 2.0 \times \bar{N} = 2.0 \times 20 = 40 \text{ kN/m}^2$

Therefore, $Q_u = q_p A_p + f_s A_s$

$$= 8000\,(0.3 \times 0.3) + 40 \times (4 \times 0.3 \times 10) = 1200 \text{ kN}$$

$$Q_a = \frac{Q_u}{3} = \frac{1200}{3} = 400 \text{ kN}$$

Illustrative Example 25.5. *A square concrete pile (35 cm × 35 cm) is driven into a homogeneous sand layer ($\phi = 30°$, $\gamma = 17$ kN/m^3) for a depth of 10 m. Calculate the ultimate load. Use Meyerhof's method. Take $K = 1.3$ and $\delta = 18°$.*

Solution. From Fig. 25.5, $(D_b/B)_{cr} = 7.0$

or $\quad D_c = 7 \times 0.35 = 2.45$

Also $\quad D_b/B = 10/0.35 = 28.57$

$$\bar{q} = 2.45 \times 17 = 41.7 \text{ kN/m}^2$$

From Fig. 25.5, $\quad N_q = 55.0$

From Eq. 25.6, $\quad Q_p = A_p \bar{q} N_q \le A_p q_l$

In this case, $\quad A_p \bar{q} N_q = (0.35 \times 0.35)(41.7 \times 55) = 280.9 \text{ kN}$

$$A_p q_l = (0.35 \times 0.35)(50 \times 55 \tan 30°) = 194.5 \text{ kN}$$

Adopting the lower value, $\quad Q_p = 194.5 \text{ kN}$

From Eq. 25.8, $\quad f_s = K \bar{\sigma}_v \tan\delta$

Therefore, $\quad Q_s = K \tan\delta$ (area of $\bar{\sigma}_v$ diagram) perimeter

$$= 1.3 \tan 18° \times (41.7/2 \times 2.45 + 41.7 \times 7.55) \times 4 \times 0.35 = 216.5 \text{ kN}$$

Thus $\quad Q_u = 194.5 + 216.5 = 411 \text{ kN}$

Illustrative Example 25.6. *A concrete pile, 40 cm diameter, is driven 25 m into a soft clay ($c_u = 25.0$ kN/m^2, $\gamma = 19$ kN/m^3). Determine the allowable load using Vijayvergia and Focht method (F.S. = 2.5). The water table is at the ground surface.*

Solution. Taking $N_c = 9.0$,

$$Q_p = c_u N_c A_p = 25 \times 9 \times \pi/4 \times (0.4)^2 = 28.3 \text{ kN}$$

From Eq. 25.14, $\quad f_s = \lambda\,(\bar{\sigma}_v + 2c)$

From Fig. 25.8, for D = 25 m, $\quad \lambda = 0.16$

Therefore, $\quad f_s = 0.16\left[\frac{1}{2} \times 25 \times (19 - 10) + 2 \times 25\right] = 26 \text{ kN/m}^2$

Thus $\quad Q_s = 26 \times (\pi \times 0.4) \times 25 = 816.4$

$$Q_u = 28.3 + 814.4 = 844.7 \text{ kN}$$

$$Q_u = 844.7/2.5 = 337.9 \text{ kN}$$

Illustrative Example 25.7. *A 25 m deep bored pile has a shaft of 1 m diameter and enlarged base of 2.5 m diameter in the lower 1.5 m depth. The undrained cohesion of the soil varies from 100 kN/m^2 at the top to 150 kN/m^2 at the base. Determine the safe load (F.S. = 2.5). Take $\alpha = 0.45$.*

Solution. Total depth of the shaft = 25 – 1.5 = 23.5 m

Assuming no adhesion for a distance 2 B above the bell, the effective depth is 21.5 m.

$$c_u \text{ at that depth} = 100 + \frac{(150 - 100)}{25} \times 21.5 = 143 \text{ kN/m}^2$$

Therefore, $\quad Q_u = (150 \times 9)(\pi/4) \times (2.5)^2 + 0.45 \times (100 + 143)/2 \times \pi \times 1 \times 21.5$

$$= 10319.5 \text{ kN}$$

$$Q_a = \frac{10319.5}{2.5} = 412.8 \text{ kN}$$

Illustrative Example 25.8. *A precast concrete pile (35 cm × 35 cm) is driven by a single-acting steam hammer. Estimate the allowable load using (a) Engineering News Record Formula (F.S. = 6), (b) Hiley Formula (F.S. = 4) and (c) Danish Formula (F.S. = 4).*

Use the following data.

(*i*)	Maximum rated energy	= 3500 kN-cm
(*ii*)	Weight of hammer	= 35 kN
(*iii*)	Length of pile	= 15 m
(*iv*)	Efficiency of hammer	= 0.8
(*v*)	Coefficient of resistitution	= 0.5
(*vi*)	Weight of pile cap	= 3 kN
(*vii*)	No. of blows for last 25.4 mm	= 6
(*viii*)	Modulus of elasticity of concrete	= 2×10^7 kN/m^2

Assume any other data, if required. Take the weight of pile as 73.5 kN.

Solution. (*a*) From Eq. 25.22,

$$Q_u = \frac{E_n \eta_h}{S + C} = \frac{3500 \times 0.80}{2.54/6 + 0.254} = 4133.9 \text{ kN}$$

Allowable load, $Q_a = 4133.9/6 = \mathbf{689\ kN}$

(*b*) Weight of pile = 73.5 kN

$$P = 73.5 + 3.00 = 76.5 \text{ kN}$$

$$eP = 0.5 \times 76.5 = 38.2 \text{ kN}$$

As $W = 35$ kN, $W < eP$

From Eq. 25.25,

$$\eta_b = \frac{W + e^2 P}{W + P} - \left(\frac{W - eP}{W + P}\right)^2$$

$$= \frac{35 + (0.5)^2 \times 76.5}{35 + 76.5} - \left(\frac{35 - 0.5 \times 76.5}{35 + 76.5}\right)^2$$

or $\eta_b = 0.484$

From Eq. 25.23,

$$Q_u = \frac{(Wh)\,\eta_b\,\eta_h}{(S + C/2)}$$

or

$$Q_u = \frac{3500 \times 0.484 \times 0.8}{2.54/6 + C/2} = \frac{1358}{2.54/6 + C/2} \qquad \ldots(a)$$

Assuming driving is with dolly,

$$C = (9.05 + 0.657\,D + 3.55)\,R/A$$

$$C = \frac{(9.05 + 0.657 \times 15 + 3.55)\,R}{35 \times 35}$$

or $C = 0.018\ R = 0.018\ Q_u$

where Q_u is in tonnes. For Q_u in kN,

$$C = 0.0018\ Q_u$$

Therefore, Eq. (*a*) gives

$$Q_u = \frac{1358}{0.423 + 0.0009\,Q_u}$$

Solving, $Q_u = 1016$ kN

Allowable load, $Q_a = \dfrac{1016}{4} = \mathbf{254\ kN}$

(c) From Eq. 25.26,
$$Q_u = \frac{(Wh) \times \eta_h}{S + S_u/2}$$
$$= \frac{3500 \times 0.8}{2.54/6 + 0.5\, S_u} = \frac{2800}{0.423 + 0.5\, S_u} \quad ...(b)$$

From Eq. 25.27,
$$S_u = \sqrt{\frac{2\, \eta_h\, (Wh)\, D}{AE}}$$
$$= \sqrt{\frac{2 \times 0.8 \times 3500 \times 1500}{35 \times 35 \times 2 \times 10^7 \times 10^{-4}}} = 1.85 \text{ cm}$$

Therefore, Eq. (b) gives
$$Q_u = \frac{2800}{0.423 + 0.5 \times 1.85} = 2077.2 \text{ kN}$$

Allowable load,
$$Q_a = \frac{2077.2}{4} = 519.3 \text{ kN}$$

Illustrative Example 25.9. *A 30 cm diameter pile of length 12 m was subjected to a pile load test and the following results were obtained.*

Load (kN)	0	500	1000	1500	2000	2500
Settlement during loading (cm)	0	0.85	1.65	2.55	3.8	6.0
Settlement during unloading (cm)	4.0	4.6	5.2	5.5	5.8	6.0

Determine the allowable load.

Solution. Fig. E-25.9 shows the load settlement curve, and the rebound curve. The net settlements are calculated below. The figure also shows the net settlements.

Load (kN)	0	500	1000	1500	2000	2500
Gross settlement (cm)	0	0.85	1.65	2.55	3.80	6.00
Rebound (cm)	0	0.60	1.20	1.50	1.80	2.00
Net settlement (cm)	0	0.25	0.45	1.05	2.00	4.00

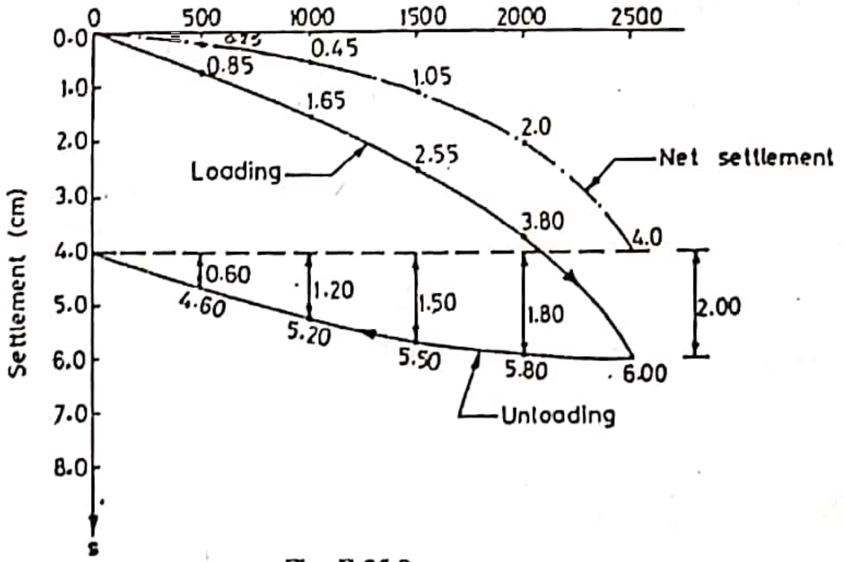


Fig. E-25.9

The allowable load is determined using the following criteria.

(*i*) $\frac{2}{3}$ of load corresponding to a gross settlement of 12 mm,

$$Q_a = \frac{2}{3} \times 750 = 500 \text{ kN}$$

(*ii*) $\frac{2}{3}$ of load corresponding to a net settlement of 6 mm.

$$Q_a = \frac{2}{3} \times 1125 = 750 \text{ kN}$$

(*iii*) $\frac{1}{2}$ of load corresponding to a gross settlement of $B/10$ (= 3 cm),

$$Q_a = \frac{1}{2} \times 1700 = 850 \text{kN}$$

The allowable load is the least of the three values.

$$Q_a = 500 \text{ kN}$$

Illustrative Example 25.10. *A pile group consisting of 12 piles (Fig. E-25.10) is subjected to a total load of 4 MN, with eccentricity $e_x = 0.3$ m, $e_y = 0.4$ m. Determine the maximum load in an individual pile.*

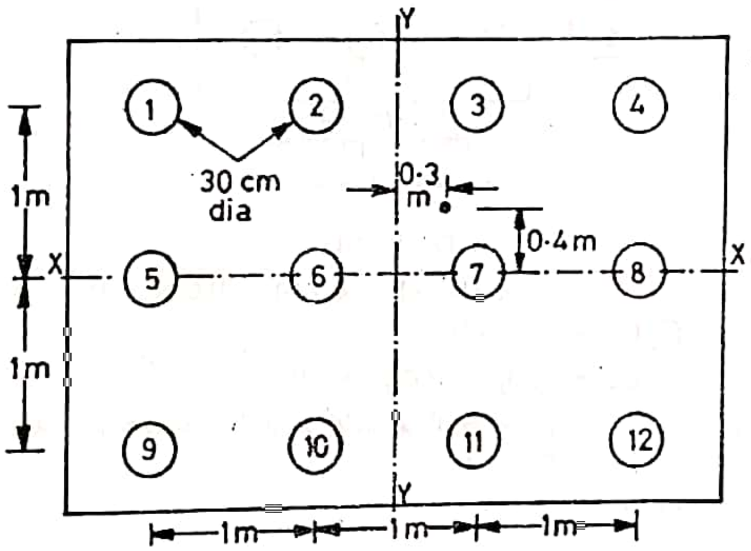


Fig. E-25.10

Solution. From Eq. 25.55,
$$Q_m = \frac{Q_g}{N} \pm \frac{(Q_g \cdot e_x)\, x}{\Sigma x^2} \pm \frac{(Q_g \cdot e_y)\, y}{\Sigma y^2}$$

In this case,
$$\Sigma x^2 = 6 \times (0.5)^2 + 6 \times (1.5)^2 = 15.0$$

$$\Sigma y^2 = 4 \times (1.0)^2 + 4 \times (1.0)^4 = 8.0$$

The maximum load occurs in pile 4.

$$Q_u = \frac{4.0}{12} + \frac{(4.0 \times 0.3)}{15} \times 1.5 + \frac{(4.0 \times 0.4)}{8} \times 1.0$$

$$= 0.6533 \text{ MN} = 653.3 \text{ kN}$$

Illustrative Example 25.11. *A pile group consists of 9 friction piles of 30 cm diameter and 10 m length driven in clay (c_u = 100 kN/m^2, γ = 20 kN/m^3), as shown in Fig. E-25.11. Determine the safe load for the group (FS = 3, α = 0.6).*

Solution. From Eq. 25.44, $Q_g(u) = q_p A_g + \alpha c (P_g D)$.

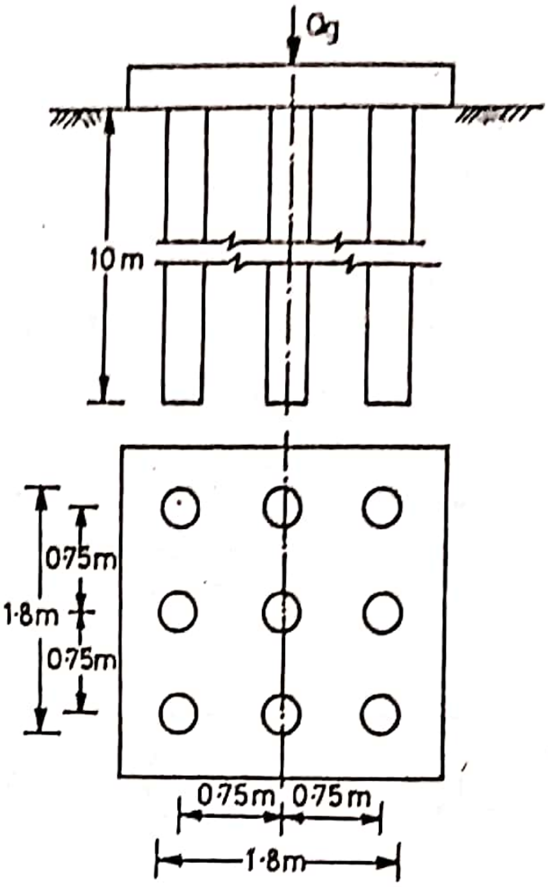


Fig. E-25.11.

$$= (9 \times 100)(1.8 \times 1.8) + 0.6 \times 100 \times (4 \times 1.8 \times 10)$$

or $$Q_g(u) = 7236 \text{ kN}$$

From Eq. 25.45, $$Q_u = q_p A_p + \alpha c (p \times D)$$

$$= (9 \times 100) \times \pi/4 \times (0.3)^2 + 0.6 \times 100 (\pi \times 0.3) \times 10$$

or $$Q_u = 628.8 \text{ kN}$$

From Eq. 25.46, $$Q_g(u) = NQ_u$$

$$= 9 \times 628.8 = 5659.2 \text{ kN}$$

As the ultimate load for individual pile failure is less than the pile group load, the safe load is given by

$$Q_a = \frac{5659.2}{3} = 1886.4 \text{ kN}$$

Illustrative Example 25.12. *A 40 cm diameter pile, 11 m long, has a bell of 2 m diameter and 1 m height. If the soil has $\phi = 25°$, $c_u = 20$ kN/m^2 and $\gamma = 19$ kN/m^3, estimate the allowable pull out resistance (FS = 3).*

Solution. From Table 25.3, $H/B_1 = 3.0$,

Therefore, $H = 3 \times 2.0 = 6$ m

As $D > H$, the pile is deep.

From Eq. 25.57,

$$P_u = \pi B_1 c_u H + s_f \gamma (\pi/2 \times B_1)(2D - H) H K_u \tan\phi + W$$

where $$W = \pi/4 \times (2.0)^2 \times 11 \times 19 + \pi/4 \times (0.4)^2 \times 11 \times (23 - 19)$$

or $$W = 662 \text{ kN}$$

From Table 25.3, $$s_f = 1.3.$$

$$K_u = \frac{1+\sin\phi}{1-\sin\phi}\left(\tan\frac{2}{3}\times\phi\right) = \frac{1+\sin 25°}{1-\sin 25°}\left(\tan\frac{2}{3}\times 25°\right) = 0.737$$

Therefore, $P_u = \pi \times 2.0 \times 20 \times 6 + 1.3 \times 19\,(\pi/2 \times 2)\,(2 \times 11 - 6) \times 6 \times 0.737 \tan 25° + 662$

or $P_u = 3976$ kN

Allowable pull, $P_a = \frac{3976}{3} = $ **1325 kN**

Illustrative Example 25.13. *A group of friction piles of 30 cm diameter is subjected to a net load of 2000 kN, as shown in Fig. E-25.13. Estimate the consolidation settlement.*

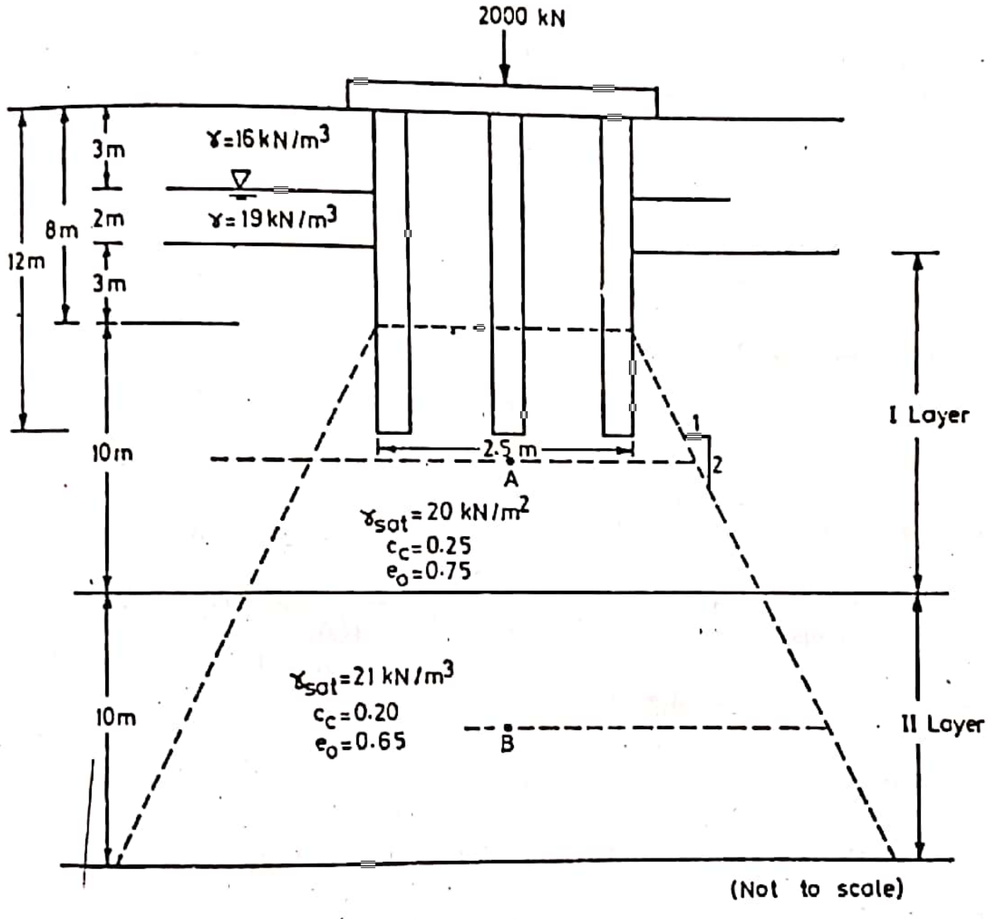


Fig. E-25.12.

Solution. σ_0 at point A, middle of I layer

$$= 3 \times 16 + 2 \times (19 - 10) + 8 \times 10.0 = 146 \text{ kN/m}^2$$

σ_0 at point B, middle of II layer $= 3 \times 16 + 2 \times 9.0 + 13 \times 10.0 + 5 \times 11 = 251$ kN/m²

$$\text{Cross-sectional area at } A = \left(2.5 + 2 \times 5 \times \frac{1}{2}\right) = 7.5 \text{ m}^2$$

$$\Delta\sigma = \frac{2000}{7.5 \times 7.5} = 35.56 \text{ kN/m}^2$$

$$\text{Cross-sectional area at } B = \left(2.5 + 15 \times 2 \times \frac{1}{2}\right) = 17.5 \text{ m}^2$$

$$\Delta\sigma = \frac{2000}{17.5 \times 17.5} = 6.53 \text{ kN/m}^2$$

$$\text{Settlement of I layer} = C_c\left(\frac{H}{1+e_0}\right)\log\frac{\sigma_0+\Delta\sigma}{\sigma_0}$$

$$= 0.25 \times \frac{10}{1+0.75}\log\frac{146+35.56}{146} = 0.135 \text{ m}$$

$$\text{Settlement of II layer} = 0.20 \times \frac{10}{1+0.65}\log\frac{251+6.53}{251} = 0.014$$

$$\text{Total settlement} = 0.135 + 0.014 = \mathbf{0.149\ m}$$

PROBLEMS