

## 8 PILE FOUNDATION

### 8.1 Introduction

- ❖ It is a slender member structural member having its cross sectional dimensions very smaller than length.
- ❖ Deep foundation which transfers loads to great depths .

#### Necessity of pile foundation

- ❖ When strata below the ground surface is highly compressive or very weak to support the load transmitted by the structure .
- ❖ To reduce the differential settlement.
- ❖ To transfer the load below the active zones i.e shrinks or swells and through deep strata to firm strata .
- ❖ On ill conditions soils such as washout , erosion and scour etc.
- ❖ When the foundation subjected to uplift .

### 8.2 Types and uses of Piles

#### According to material used

- a. Steel piles : Thick pipes or steel sections
- b. Concrete piles : Precast or cast in situ
- c. Timber piles: Made up of trunk of tree after proper trimming .Timber should be straight ,sound and free from defects . .
- d. Composite piles: made up of two materials ( i.e lower portion steel and upper portion concrete).

#### According to mode of transfer of loads

1. End bearing pile
2. Friction pile
3. Combined end bearing and friction

#### According to method of installation

- i. Driven piles
- ii. Driven and cast in situ pile
- iii. Bored and cast in situ pile
- iv. Screwed pile
- v. Jacked pile

#### According to displacement of pile

1. Displacement pile
2. Non – Displacement pile

#### According to based on use

- i. Load bearing pile
- ii. Tension Pile
- iii. Compaction pile
- iv. Sheet pile
- v. Anchor piles

#### Uses of pile

- i. To transfer loads to strong / less compressible strata
- ii. To compact loose granular soil
- iii. To provide foundation below the scour depth
- iv. To carry horizontal and vertical forces from abutments and retaining walls
- v. To carry up lifts forces

### 8.3 Construction of Piles

The construction of a pile foundation involves two steps, namely the installation of piles and the making of pile caps. The second step is relatively simple and is similar to the construction of footings.

Installation of piles would depend upon whether they are driven or cast-in-place. Some details regarding the equipment required to install piles by driving them into soil have already been given. Water jetting is used to assist penetration of the piles.

Cast-in-place piles are mostly concrete piles of standard types such as the Raymond pile and the Franki pile, so called after the piling firms which standardized their construction.

Damage due to improper driving may be avoided if driving is stopped when the penetration reaches the desired resistance.

Some degree of tolerance in alignment has to be permitted since piles can never be driven absolutely vertical and true to position.

A pile may be considered defective if it is damaged by driving or is driven out of position, is bent or bowed along its length. A defective pile must be withdrawn and replaced by another pile. It may be left in place and another pile may be driven adjacent to it.

Pile driving may induce subsidence, heave, compaction, and disturbance of the surrounding soil. These effects are to be carefully studied so as to understand their bearing on the capacity of the pile.

The method of installing a pile at a site depends upon the type of pile. The equipment required for this purpose varies. The following types of piles are normally considered for the purpose of installation

#### 1. Driven piles

The piles that come under this category are,

- a. Timber piles,
- b. Steel piles, H-section and pipe piles,
- c. Precast concrete or prestressed concrete piles, either solid or hollow sections.

#### 2. Driven *cast-in-situ* piles

This involves driving of a steel tube to the required depth with the end closed by a detachable conical tip. The tube is next concreted and the shell is simultaneously withdrawn. In some cases the shell will not be withdrawn.

#### 3. Bored *cast-in-situ* piles

Boring is done either by auguring or by percussion drilling. After boring is completed, the bore is concreted with or without reinforcement.

### 8.4 Selection of Pile

The selection of the type, length and capacity is usually made from estimation based on the soil conditions and the magnitude of the load. In large cities, where the soil conditions are well known and where a large number of pile foundations have been constructed, the experience gained in the past is extremely useful. Generally the foundation design is made on the preliminary estimated values. Before the actual construction begins, pile load tests must be conducted to verify the design values. The foundation design must be revised according to the test results. The factors that govern the selection of piles are:

1. Length of pile in relation to the load and type of soil
2. Character of structure
3. Availability of materials
4. Type of loading
5. Factors causing deterioration

6. Ease of maintenance
7. Estimated costs of types of piles, taking into account the initial cost, life expectancy and cost of maintenance
8. Availability of funds

All the above factors have to be largely analyzed before deciding up on a particular type.

### 8.5 Types of Foundation to suit subsoil conditions (Gopal Ranjan and ASR Rao)

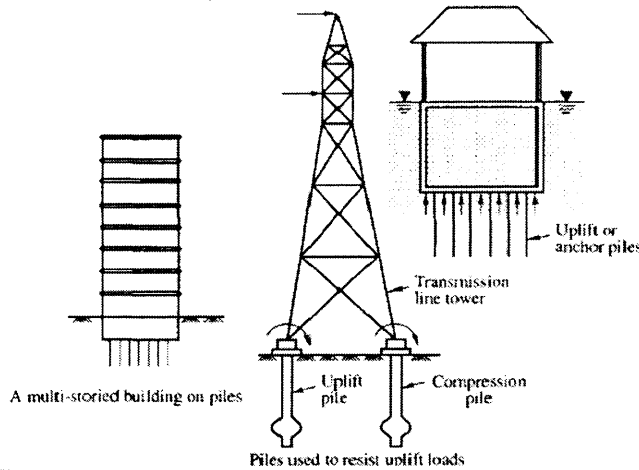


Figure (a) Principles of floating foundation; and a typical rigid raft foundation

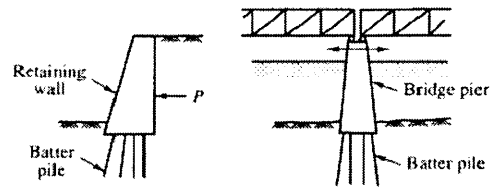


Figure (b) Piles used to resist lateral loads

### 8.6 Pile Driving Formula

### 8.7 Static Pile Load Formula

The ultimate bearing capacity,  $Q_u$ , of a single vertical pile may be determined by any of the following methods.

1. By the use of static bearing capacity equations.
2. By the use of SPT and CPT values.
3. By field load tests.
4. By dynamic method.

1. By the use of static bearing capacity equations.

Total ultimate load capacity of a pile foundation is it's sum of base or resistance and shaft resistance.

$$Q_u = Q_b + Q_s + W$$

$$= q_b A_b + q_s A_s$$

Where;  $Q_u$  = load at failure applied to the pile,

$Q_b$  = base resistance,

$Q_s$  = shaft (skin friction) resistance,

$W$  = wieight of pile

$A_b$  = Area of Base

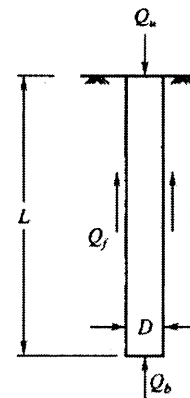
$A_s$  = Area of Shaft

#### For single Pile

In sandy soil shaft resistance is given by friction component only. Hence the ultimate load carrying capacity of pile in sand is given by

$$Q_b = A_b(cN_c + \sigma_{vb}N_q + 0.5\gamma bN_\gamma)$$

$$Q_s = q_s A_s = f_s A_s = \int_0^L P \tau_a dz$$



where

$$\tau_a = c_a + \sigma_n \tan \phi_a$$

$\tau_a$  = pile-soil shear strength

$c_a$  = adhesion

$\sigma_n$  = normal stress between pile and soil

$\phi_a$  = angle of friction between pile and soil

Normal (vertical load) can be given as

$$\sigma_n = K_s \sigma_v$$

Where,

$K_s$  = coefficient of lateral pressure

$\sigma_{vb}$  = Vertical stress in the soil at pile base

Thus,

$$\tau_a = c_a + \sigma_v K_s \tan \phi_a$$

$$\begin{aligned} A_s f_s &= \int_0^L P \tau_a dz \\ &= \int_0^L P (c_a + \sigma_v K_s \tan \phi_a) dz \end{aligned}$$

Where,  $P$ =Pile peremeter ,  $L$ = Length of pile

Thus the final general equation will be

$$Q_u = \int_0^L P (c_a + \sigma_v K_s \tan \phi_a) dz + A_b (c_u N_c + \sigma_{vb} N_q + 0.5 \gamma b N_\gamma) - W$$

Weight of pile is normally neglected in the case of end bearing pile and considered in the case of friction piles in practical use.

### Piles in Clay

If the clay is saturated, the undrained angle of friction  $\phi_u=0$ , and  $f_a$  may also be taken as zero, In addition  $N_q=1$  and  $N_\gamma=0$  for  $\phi=0$  so the general equation

reduces to

$$Q_u = \int_0^L P c_a dz + A_b (c_u N_c + \sigma_{vb}) - W$$

Where,

$c_u$  = undrained cohesion of soil at level of pile base

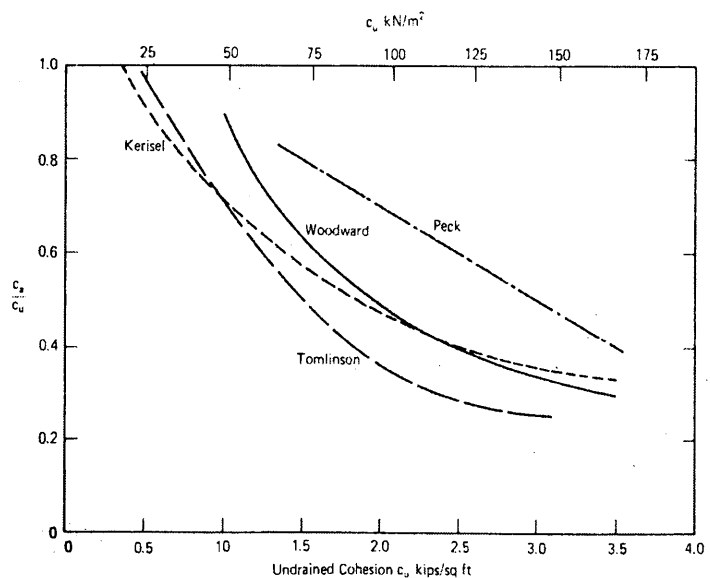
$c_a$  = undrained pile-soil adhesion

Further simplification is possible in many cases, since for piles without an enlarged base,  $A_b \sigma_{vb} \approx W$ , in which case,

$$Q_u = \int_0^L P c_a dz + A_b c_u N_c$$

For driven piles, typical relationships between  $c_a/c_u$  and  $c_u$ , based on the summary provided

by McClelland (1974), are shown in the figure below. It is generally agreed that for soft clays ( $c_u \leq 24$  kPa),  $c_a/c_u$  is 1 or more for different cases (driven piles in stiff clays).



### Piles in Sand

For the case of piles in sand, if the pile-soil adhesion  $c_a$  and term  $cN_c$  are taken as zero, and the term  $0.5\gamma bN\gamma$  is neglected as being small in relation to the term involving  $Nq$ , the ultimate load capacity of a single pile in sand may be expressed as follows:

$$Q_u = \int_0^L P\sigma'_v K_s \tan \phi'_a dz + A_b \sigma'_{vb} N_q - W$$

Where,

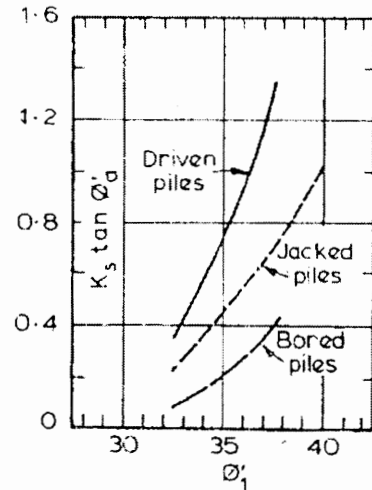
$\sigma'_v$  = effective vertical stress along shaft

$\sigma'_{vb}$  = effective vertical stress at level of pile base

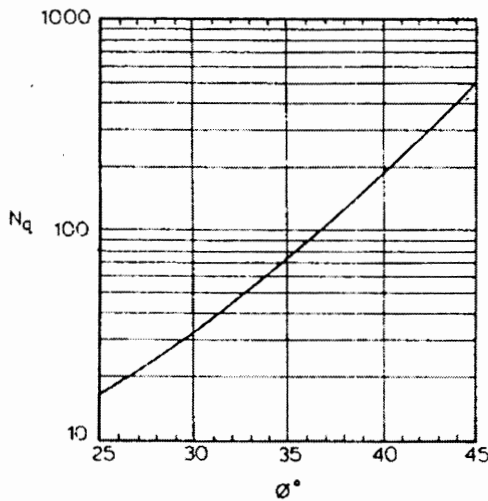
The values of  $\phi'_a$  is taken as  $0.75\phi'$

The bearing capacity factor  $Nq$  and  $f$  is based on those derived by Berzantzev et al. (1961) and the values appear to fit the available test data best. The solutions given by Berzantzev et al. indicate only a small effect of relative embedment depth  $L/d$ , and represents an average of this small range. The curves given by meryerhof (1967) show a large effect of  $L/d$ , however the curve given by Berzantzev also lies near the middle of Meyerhof's range.

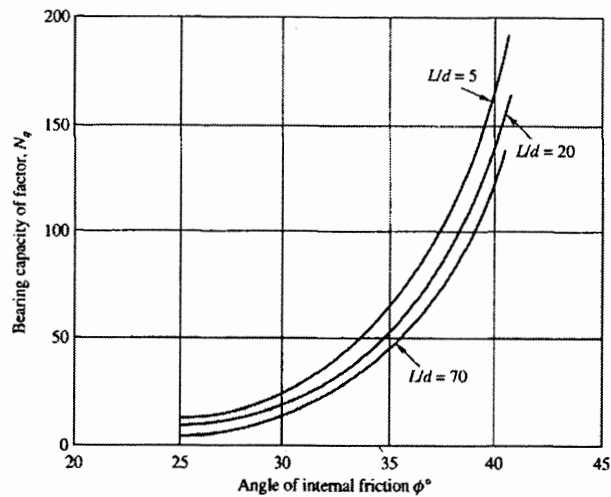
Values of  $K_s \tan \phi'_a$  Base on Meyerhof (1976)



where  $\phi'_1$  = angle of internal friction prior to installation of pile



Relationship between  $N_q$  and  $\phi$  (after Berzantzev et al., 1961)



Berezantsev's bearing capacity factor,  $N_q$  (after Tomlinson, 1986)

S. No.	Material of pile	Consistency of clay	Cohesion ( $kN/m^2$ )	Adhesion factor
1.	Wood and concrete	Soft	0-35	0.90 to 1.00
		Medium	35-70	0.60 to 0.90
		Stiff	70-140	0.45 to 0.60
2.	steel	Soft	0-35	0.45 to 1.00
		Medium	35-70	0.10 to 0.50
		Stiff	70-140	0.50

Adhesion factors for piles in clay (Tomlinson, 1969)

The adhesion  $c_a$  may be expressed as

$$c_a = \alpha \cdot c$$

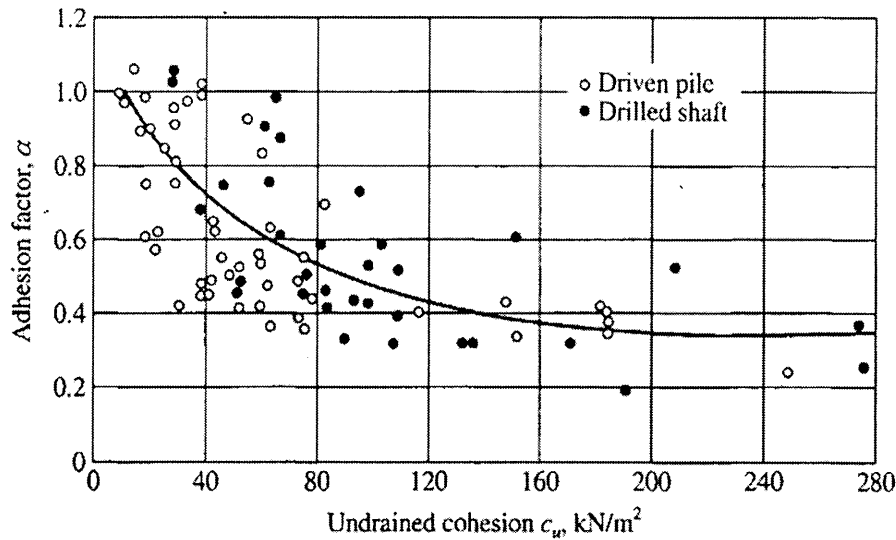


Figure- Adhesion factor for piles with penetration lengths less than 50 m  
(Data from Dennis and Olson 1983 Stas and Kulhawy, 1984)

**For a layered system in sand:**

$$Q_u = \sigma_v (N_q - 1) A_b + P \sum \sigma_v' K \tan \delta \Delta t$$

Where,

- $\sigma_v$  = effective overburden pressure at the base level of pile.
- $\sigma_v'$  = effective overburden pressure (taken average) over the embedded depth in a particular layer of soil.
- $K_s$  = Lateral earth pressure coefficient.
- $A_b$  = Base area of the pile.
- $A_s$  = Shaft area of the pile. =  $P \cdot \Delta t$
- $P$  = Perimeter of the pile.
- $\Delta t$  = Thickness of the layer
- $\delta$  = Angle of internal friction of pile.
- $N_c, N_q$  = Bearing capacity factors for cohesion and surcharge
- $N_c = 9$ , for deep foundation and
- $N_q$  can be found from the charts of bearing capacity factors

**Single Pile in Clay**

In clayey soil shaft resistance is due to adhesion of surrounding soil with the pile shaft. So the ultimate load carrying capacity of pile in clay is given by,

$$Q_u = Q_b + Q_f = c_b N_c A_b + \alpha c_u A_s$$

**For a layered soil:**

$$Q_u = c_b N_c A_b + p \sum \alpha c_u P \Delta t$$

Where,

- $c_b$  = undrained shear strength of clay at base of pile
- $c_u$  = Average undrained shear strength of clay at side of pile..
- $N_c$  = bearing capacity factor. = 9 for pile foundations.
- $A_b$  = Base of the pile.
- $\alpha$  = Adhesion factor which can be found from following figure
- $A_s$  = Shaft area of the pile.
- $P$  = perimeter of the pile.
- $\Delta t$  = thickness of the layer clay.

## 8.8 Load Test on Piles

Load test on a pile is one of the best methods of determining the load-carrying capacity of a pile. It may be conducted on a driven pile or cast-in-situ pile, on a working pile or a test pile, and on a single pile or a group of piles. A working pile is one which forms part of the foundation, while a test pile is one which is used primarily to check estimated capacities (as predetermined by other methods).

The test should be conducted only after a lapse of a few weeks in clays and at least a few days in sands, in order that the results obtained be more meaningful for design.

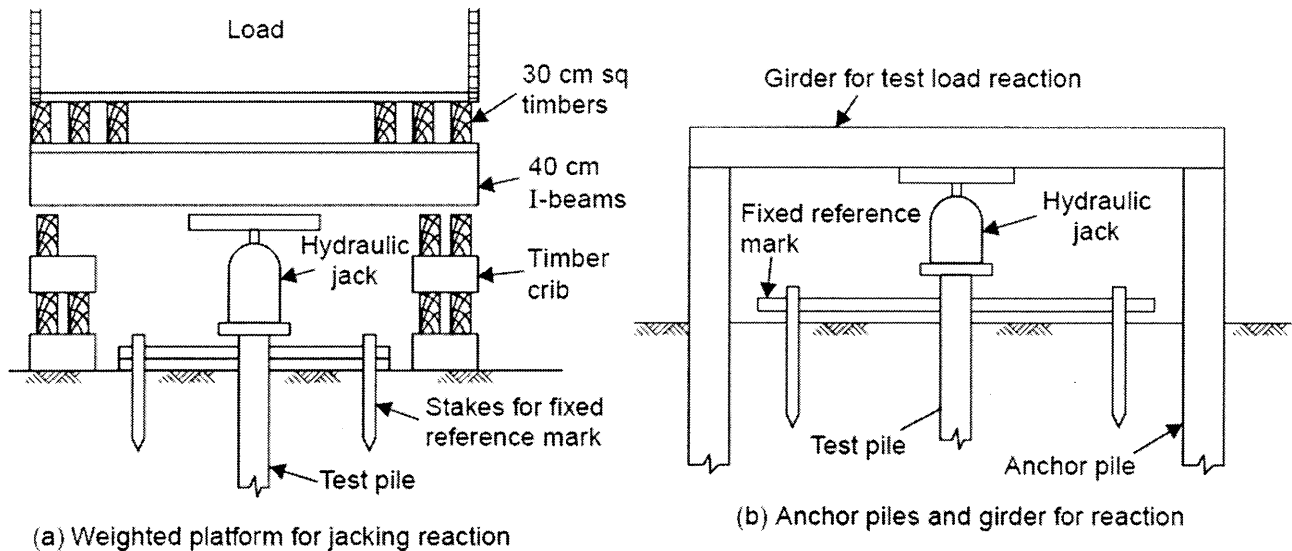


Fig. Typical pile load test arrangements

Load may be applied by using a hydraulic jack against a supported platform (above Fig. *a*), or against a reaction girder secured to anchor piles (above Fig. *b*). Sometimes a proving ring is preferred for better accuracy in obtaining the load. Instead of reaction loading, gravity loading may also be used; but the former is given better uniformity in loading. Measurement for pile settlement is related to a fixed reference mark. The support for the reference mark has to be located outside the zone that could be affected by pile movements.

The most common procedure is the test in which the load is maintained slowly.

- About five to eight equal increments are used until the load reaches about double the design value.
- Time-settlement data are recorded for each load increment.
- Each increment is maintained until the rate of settlement becomes a value less than 0.25 mm per hour.
- The final load is maintained for 24 hours.

Another procedure is the constant-strain rate method. In this method,

- The load is increased such that the settlement occurs at a predetermined rate such as 0.5 mm per minute.
- This test is considerably faster than the other approach.

The load-settlement curve is obtained from the data. Often the definition of 'failure load' is arbitrary. It may be taken when a predetermined amount of settlement has occurred or where the load-settlement plot is no longer a straight line. If the ultimate load could be found, a suitable factor of safety—2 to 3—may be used to determine the allowable load.

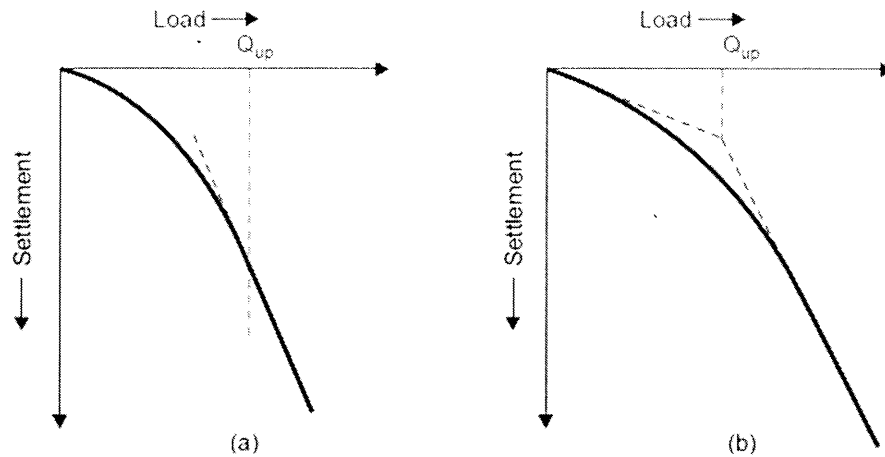


Fig. Determination of ultimate load from load-settlement curve for a pile

The ultimate load may be determined as the abscissa of the point where the curved part of the load-settlement curve changes to a steep straight line (Fig. a). Alternatively, the ultimate load is the abscissa of the point of intersection of initial and final tangents of the load settlement curve (Fig. b).

Another method in use for the slow test is to plot both load and settlement values on logarithmic scale. The results typically plot as two straight lines (Fig. c). The intersection of the straight lines is taken as failure load for design purposes although this may not be the actual load at which failure occurs.

The allowable load on a single pile may be obtained as one of the following [I.S: 2911 (Part I)-1974]:

1. 50% of the ultimate load at which the total settlement is equal to one-tenth the diameter of the pile.
2. Two-thirds of the load which causes a total settlement of 12 mm.
4. Two-thirds of the load which causes a net (plastic) settlement of 6 mm (total settlement minus elastic settlement).

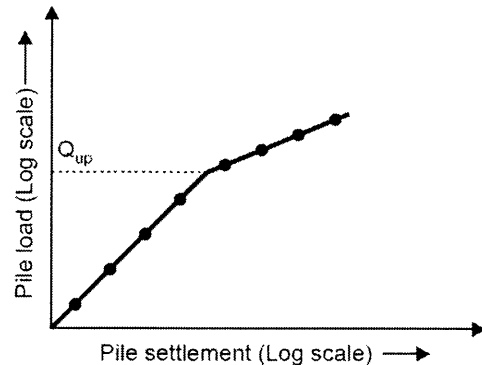


Fig. C Log-Log of pile load-settlement curve and determination of failure

## 8.9 Dynamic Pile Formulae

### Pile capacity from pile driving Formulae

The relationship between dynamic resistance and pile during driving and static load carrying capacity of piles are called pile driving formula.

1. Engineering News Record (ENR) formula
2. Janbu's formula
3. Hiley formula



### Engineering News Record (ENR) formula:

The 'Engineering News' formula (Wellington, 1886) was derived from observations of driving of timber piles in sand with a drop hammer. The general form of this equation is as follows:

$$\text{Where } Q_u = \frac{W_h H}{(s + C)}$$

- $s$  = final penetration (set) per blow. It is taken as average penetration per blow for the last 5 blows or 20 blows depending on whether the hammer is a drop hammer or steam hammer,
- $C$  = empirical constant (representing the temporary elastic compression of the helmet, pile and soil)

A factor of safety of 6 was introduced to make up for any inaccuracies arising from the use of arbitrary values for the constant, while arriving at the allowable load on the pile.

$$Q_a = \frac{W_h \cdot H}{6(s + C)}$$

The value of  $C$  (in cm) is taken as 2.5 for drop hammer, and 0.25 for steam hammer.

$$Q_a = \frac{500 W_h \cdot H}{3(s + 25)} \text{ for drop hammer}$$

$$Q_a = \frac{500 W_h \cdot H}{3(s + 2.5)} \text{ for steam hammer}$$

$$Q_a = \frac{(W_h + ap) \cdot H}{6(s + 2.5)} \text{ For double acting steam hammers}$$

where

- $W_h$  = weight of hammer (newtons),
- $a$  = effective area of piston ( $\text{mm}^2$ ),
- $p$  = mean effective steam pressure ( $\text{N/mm}^2$ ),
- $H$  = height of fall of hammer (metres)
- $s$  = final penetration of pile per blow (mm), and
- $Q_a$  = allowable load on the pile (kN).

(Note: This equation has mixed units).

### ENR Formula

The earliest pile driving formula assumes that for a given hammer blow, the resistance increases in an elastic manner as the pile is displaced, remains constant for further displacement, and finally falls to zero as the pile rebounds. Equating the energy supplied to the work done, the following formula was obtained.

$$\underline{WH = Q_u(S + C)}$$

- where  $W$  = weight of hammer (ton),
- $H$  = fall of hammer (ft),
- $S$  = penetration per blow (in),
- $R$  = pile resistance (ton), and
- $C$  = constant which accounts for elastic settlement of pile-soil system (1.0 in for drop hammer and 0.1 in for single acting steam hammer)

Equation has subsequently been revised to a more generalized form,

$$Q_u = \frac{EWH}{S + C} \frac{W + n^2 W_p}{W + W_p}$$

- where  $E$  = hammer efficiency (0.7–0.9)
- $C$  = 0.1 in (if  $S$  and  $H$  are in inches)
- $W$  = weight of hammer (ton)
- $W_p$  = weight of pile (ton)
- $n$  = coefficient of restitution (0.4–0.5)

**Janbu's Formula (Janbu, 1953)**

$$Q_u = \frac{EH}{K_u S}$$

where

$$K_u = C_d \left( 1 + \sqrt{1 + \frac{\lambda}{C_d}} \right);$$

$$C_d = 0.75 + 0.14 \left( \frac{W_p}{W} \right), \text{ and}$$

$$\lambda = \frac{EHL}{A_p E_p S^2}$$

Where,

$E = \eta W$  = Hammer efficiency

$\eta$  = efficiency factor

= 0.7, for good driving condition

= 0.5, for average driving condition

= 0.4, for difficult or bad driving condition

$H$  = Height of fall

$W$  = Weight of Hammer

$W_p$  = weight of pile

$L$  = Length of pile

$A_p$  = Cross-sectional of Pile

$E_p$  = Modulus of elasticity of pile

$S$  = final set

**Hiley Formula**

$$Q_u = \frac{\eta WH}{S + C/2} \cdot \frac{W + n^2 W_p}{W + W_p}$$

Here,  $Q_u$ ,  $W$ ,  $H$ ,  $n$ ,  $S$ , and  $W_p$  have the same meaning as in Eq. generalized form.

$\eta$  = efficiency of hammer blow (0.75–1.0)

$C$  = a factor which accounts for energy losses due to elastic compression of pile,  $C_1$ , elastic compression of the head assembly,  $C_2$ , and elastic compression of the soil,  $C_3$ , that is,

$$C = C_1 + C_2 + C_3$$

The approximate values of  $C_1$ ,  $C_2$ , and  $C_3$  to be used in Hiley formula for concrete piles are given in Table 9.2.

**Table 9.2** Values of  $C_1$ ,  $C_2$ ,  $C_3$  in Hiley formula

$C_1 = 0.075-0.10$ in, for hard driving	
$C_2 = R_u L / A E_p$	where $R_u$ = ultimate test load, $L$ = Length of pile, $A$ = Cross sectional area of pile, and $E_p$ = Elastic modulus of pile material.
$C_3 = 0.1$ in (0 for hard soil and 0.2 for resilient soil)	

### 8.10 Pile Capacity from In-situ Tests

Meyerhoff suggested following relation for ultimate bearing capacity of pile:

- ❖ Displacement pile

$$Q_u = 400NA_b + 2N'A_s$$

- ❖ H- pile

$$Q_u = 400NA_b + N'A_s$$

- ❖ Bored pile:

$$Q_u = 133NA_b + 0.67 N'A_s$$

$N$  = SPT value below pile tip,  $N'$  = Avg SPT along the pile shaft,  $A_b$  = base area,  $A_s$  = Shaft area and  $Q_u$  = Ultimate load.

$$\text{Allowable Load } Q_a = Q_u / \text{Fos}, \text{ Fos} = 4$$

### BC based on CPT result

#### Vandeer- Veen 's method for pile in cohesion less soil

$$Q_u = Q_b + Q_s$$

Base resistance

$$Q_b = A_b \cdot q_p'$$

$A_b$  = Base area of pile

$q_p'$  = avg cone resistance over depth  $4D$  (look curve from RKP book)

#### Shaft Resistance :

Meryerhoff suggested following values

Large displacement pile,  $Q_s = A_s q'_c / 2$

Small displacement pile,  $Q_s = A_s q'_c / 4$

$$Q_a = (Q_b + Q_s) / 2.5$$

### Pile Group subjected to eccentric vertical loading

For symmetrical pile system and pile cap being rigid and very thick, when the total vertical load centrally placed be  $Q_g$  and the number of piles be  $n$ , then load to be transmitted to each pile,

$$Q_i = Q_g / n$$

But for a centrally loaded pile cap.

$$\text{❖ } Q_i = \left\{ (Q_g/n) \pm (M_y \cdot x / \sum x^2) \pm (M_x \cdot y / \sum y^2) \right\}$$

Where,

$M_x = Q_g \cdot e_y$  = Moment about X- axis

$M_y = Q_g \cdot e_x$  = Moment about Y- axis

$Q_i$  = Load transmitted to particular pile

$Q_g$  = Total Vertical load acting on the pile cap centrally

$n$  = No. of pile in group

$e_x, e_y$  = Eccentricity along X and Y direction

$\sum x^2$  = Summation of squares of distance of all piles from Y-axis

$\sum y^2$  = Summation of squares of distance of all piles from X-axis

### 8.11 Group Action of Piles

#### Group action of piles:

A structure is never founded on a single pile. Piles are ordinarily closely spaced beneath structures; consequently, the action of the entire pile group must be considered. This is particularly important when purely friction piles are used.

The bearing capacity of a pile group is not necessarily the capacity of the individual pile multiplied by the number of piles in the group; the phenomenon by virtue of which this discrepancy occurs is known as 'Group action of piles'.

#### Number of Piles and Spacing

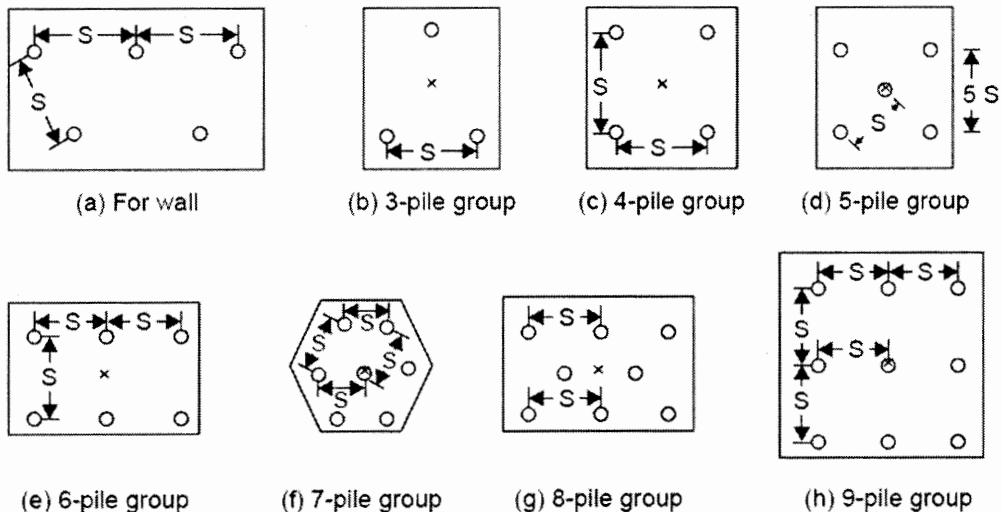
Point-bearing piles may be more closely spaced than friction piles. The minimum spacing of piles is usually specified in building codes.

The spacing may vary from  $2d$  to  $6d$  for straight uniform **cylindrical piles**,  $d$  being the diameter of the pile.

For **friction piles**, the recommended minimum spacing is  $3d$ .

For **point-bearing piles** passing through relatively compressible strata, the minimum spacing is  $2.5d$  when the piles rest in compact sand or gravel; this should be  $3.5d$  when the piles rest in stiff clay.

The minimum spacing may be  $2d$  for compaction piles.



When piles are used in groups then the capacity of pile group does not necessarily become equal to single pile capacity multiplied by number of piles.

When piles are applied in loose sand then sand gets compacted and the bearing capacity of the group becomes greater than summation of individual pile capacity on the other hand when piles are inserted into loose clay it can lead to group failure or individual pile failure depending upon the spacing of the piles.

Hence summation of individual pile capacity becomes greater than group capacity in cohesive soil. Spacing should be determined based on economical and practical considerations.

**Group Efficiency of Pile Groups**

Group efficiency of a pile group is the ratio of total load carried by a pile group to the summation of individual pile load capacities.

Mathematically, it can be written as,

$$E_r = \{Q_u(g) / n * Q_u(s)\} * 100\%$$

Where;  $E_r$  = Group efficiency ratio.

$Q_u(g)$  = Ultimate bearing capacity of pile group

$Q_u(s)$  = Ultimate bearing capacity of single pile

**Converse-Labarre formula**

$$\eta_g = 1 - \frac{\phi}{90} \left[ \frac{m(n - 1) + n(m - 1)}{mn} \right]$$

where  $\eta_g$  = efficiency of pile group,

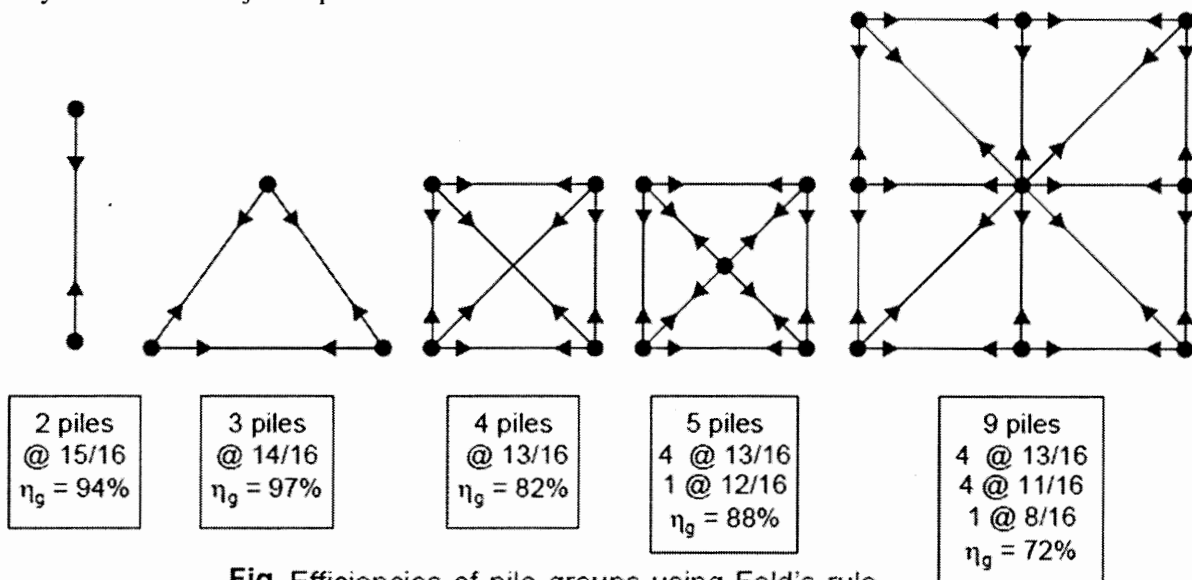
$\phi = \tan^{-1} \frac{d}{s}$  in degrees,  $d$  and  $s$  being the diameter and spacing of piles,

$m$  = number of rows of piles, and

$n$  = number of piles in a row

(interchangeable)

- Other formulae **Fled's rule** is sometimes applied for determining pile efficiency in which efficiency decreases by 1/16 for each adjacent pile.



**Fig.** Efficiencies of pile groups using Feld's rule

**Pile Group in Cohesionless soil.**

- For piles driven in loose sand and gravel, the soil around 3 times diameter gets compacted and hence pile group acts as a block together as pier foundation having larger base area contained by the piles. So in pile group efficiency increases and may reach greater than unity but for design purpose efficiency is generally taken as unity.

So, we can write,  $Q_{gn} = n * Q_u$

Where,  $n$  = the number of piles in the group.

- When piles rest on compressible soils such as silts or clay then above statement does not hold well because stress transferred to compressible soil from pile group can result in over stressing or excessive consolidation. The carrying capacity of pile in such cases is governed by the shear failure and compressibility of soil criteria rather than group efficiency.

**Pile Groups in Cohesive Soils**

- Due to the possibility of remolding and lifting up of pile due to upheaval of soil in driven piles, so usually bored piles are preferred in cohesive soils. The piles should be driven from centre and then towards edges and should be kept at greater distances apart.
- From various experiments performed on piles in cohesive soils when loaded, it shows that it may fail individually (individual failure) or as a block (block failure). At closer spacing around 2-3 times diameter of piles, usually acts as a group and fails in group known as group failure. For piles kept at distance apart i.e. larger spacing (greater than 8 times diameter of piles), piles in pile group fails individually known as individual failure.

$$Q_{gu} = c_b * N_c * A_g + P_g * L * c_u * \alpha$$

Where,

- $c_b$  = cohesive strength of clay base of the pile group,
- $c_u$  = average cohesive strength of clay around the group,
- $L$  = Length of pile,
- $P_g$  = Perimeter of pile group,
- $A_g$  = Base area of group,
- $N_c$  = Bearing capacity factor, which may be assumed as 9 for deep foundation

**Bearing capacity for individual failure:**

$$Q_{gu} = n * Q_u$$

Where;  $n$  = Number of piles in the group,

$Q_u$  = Bearing capacity of an individual pile.

Terzaghi and Peck recommends the value of load capacity of group is taken as smaller value given by above equations.

**8.12 Negative skin Friction**

When filled up soil starts consolidating under its own overburden pressure it develops a drag on the surface of a pile is called negative skin friction .It may also occurs when the fill is placed over the peat or soft clay strata .

Here, net ultimate load carrying capacity of pile is decreased

$$Q_u' = Q_u - Q_{nsf}$$

$Q_{nsf}$  = negative skin friction

**For cohesive soil:**

$$Q_{nsf} = C_u' * A_s'$$

$A_s'$  = shaft area of pile subjected to a negative skin friction

**For Cohesionless Soil:**

$$Q_{nsf} = k \sigma_v' K \tan \delta * A_s'$$

$A_s'$  = shaft area of pile subjected to negative skin friction

$$\sigma_v' = (0 + \gamma * L_c) / 2$$

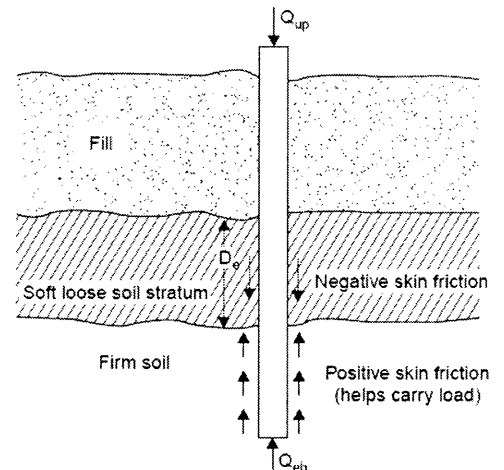


Fig. Negative skin friction on a pile

### Negative skin friction for pile group

$$Q_{ns}(g) = n * Q_{ns}(s)$$

$$Q_{ns}(g) = C_u * A_s'(g) + \gamma * L_c * A_b(g)$$

The greater value from above equation is taken as negative skin friction for pile group.

$A_g(s)$  = perimeter area of pile group subjected to negative skin friction .

$A_b(g)$  = area of group at base

$L_c$  = length of soil under the negative friction

### Remedial measures for negative skin friction

1. By providing small area of cross section of pile shaft
2. By driving pile inside casing and space between pile and casing be filled with a viscous material
3. By coating a pile with bitumen

### 8.13 Laterally Loaded Piles

Piles and pile groups may be subjected to vertical loads, lateral loads or a combination of both. If the lateral loads act at an elevation considerably higher than the base of the foundation, there will be significant moments acting on it.

Vertical piles can resist lateral forces to a certain extent depending on the strength and stiffness of the pile and the soil . According to **IS 2911-1985**, permissible lateral load of a vertical pile is 2 – 5% of the permissible vertical load. For greater horizontal load, additional reinforcement is to be provided in the pile or raker piles may be used.

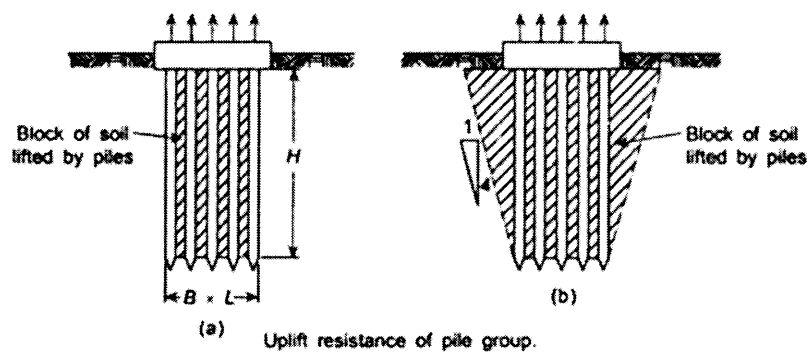
Extensive theoretical and experimental studies have been made on laterally loaded piles by Reese and Matlock (1960), Palmer and Brown (1954), and Murthy (1964). Most of these are based on the concept of coefficient of sub grade reaction, which is the pressure required to cause unit deflection.

### 8.14 Piles Subjected to Uplift Loads

A straight shaft pile, when subjected to uplift forces, derives its ultimate capacity from frictional resistance of the pile which can be determined in the same way as indicated for piles under compression. However for cyclic loading, skin friction may be reduced by the degradation of soil strength at the pile – soil interface under repetitive load. In particular, for sandy soils, reduction in uplift capacity to 50% of the ultimate skin friction has been reported. For cohesive soils, 30 – 50% reduction of uplift capacity in short augured piles has been observed.

As a general rule, a factor of safety of 3 – 4 on the frictional resistance calculated for compression may be applied to determine the uplift capacity of piles. However it should be noted that an upward movement of only 0.5 – 1% of the pile diameter is required to mobilize the peak frictional resistance.

For cohesionless soil, an assumed spread of 1:4 from the pile tip to the ground and the weight of soil block enclosed within the group give the frictional resistance. The submerged weight of the soil below the ground water table should be taken.



For cohesive soils, the uplift resistance of the block may be obtained by summing up the undrained shearing resistance around the periphery of the block and the weight of the soil enclosed by the group as

$$Q_u = 2(L + B)D_f c_u + W$$

- where
- $L$  = length of the pile group,
  - $B$  = width of the pile group,
  - $D_f$  = depth of the pile group,
  - $c_u$  = average undrained shear strength of the clay, and
  - $W$  = weight of the soil enclosed within the block.

A safety factor of 3 should be used to determine the safe uplift capacity of the group.



- Q.1 A square group of 9 piles was driven into soft clay extending to a large depth. The diameter and length of the piles were 30 cm and 9 m respectively. If the unconfined compression strength of the clay is 90 kN/m<sup>2</sup>, and the pile spacing is 90 cm centre to centre, what is the capacity of the group? Assume a factor of safety of 2.5 and adhesion factor of 0.75.

Block failure:

Since, it is a square group, 3 rows of 3 piles each will be used.

$$Q_g = c' \cdot N_c \cdot A_g + c \cdot P_g \cdot L$$

Here, cohesion  $c = c' = 45 \text{ kN/m}^2$

$N_c$  is taken as 9.

$$L = 9 \text{ m} \quad B = 2s + d = 2 \times 0.9 + 0.3 = 2.1 \text{ m}$$

$$P_g = 4B = 8.4 \text{ m}$$

$$A_g = B^2 = 2.1^2 \text{ m}^2 = 4.41 \text{ m}^2$$

Substituting,

$$\begin{aligned} Q_g &= 45 \times 9 \times 4.41 + 45 \times 8.4 \times 9 \\ &= 5,186 \text{ kN.} \end{aligned}$$

Individual pile failure:

$$\begin{aligned} Q_g &= n[q_{cb} \cdot A_b + f_s \cdot A_s] \\ &= n[cN_c A_b + \alpha \cdot c \cdot A_s] \\ &= 9 \left[ 45 \times 9 \times \frac{\pi}{4} \times 0.3^2 + 0.75 \times 45 \times \pi \times 0.3 \times 9 \right] \\ &= 2,835 \text{ kN} \end{aligned}$$

In this case, individual pile failure governs the design. Allowable load on the pile group

$$= \frac{2,835}{2.5} = 1,130 \text{ kN.}$$

- Q2. A 16-pile group has to be arranged in the form of a square in soft clay with uniform spacing. Neglecting end-bearing, determine the optimum value of the spacing of the piles in terms of the pile diameter, assuming a shear mobilization factor of 0.6.

At the optimum spacing, efficiency of the pile group is unity.

Let  $d$  and  $s$  be the diameter and spacing of the piles. Let  $L$  be their length.

Width of the block for a 16-pile square group,

$$B = 3s + d$$

Group capacity for block failure

$$= 4L(3s + d) \times c$$

where  $c$  is the unit cohesion of the soil.

Group capacity based on individual pile failure

$$\begin{aligned} &= n[(0.6c)(\pi dL)] \\ &= 16 \times 0.6 \pi d L c \end{aligned}$$

Equating these two,

$$4Lc(3s + d) = 16 \times 0.6 \pi d L c$$

$$12s + 4d = 9.6 \pi d$$

$$s = \frac{(9.6\pi - 4)}{12} d = 2.18d$$

$\therefore$  The optimum spacing is about 2.2 d.