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Advanced Foundation Engineering

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CHAPTER 7: PILE FOUNDATION

Chapter 7

Pile Foundation

7.1 Introduction

Deep foundations are required when the soil at shallow depth is not capable of supporting structural loads. Deep foundation will be adopted if a firm stratum having desired bearing capacity cannot be reached by open excavation.

The purpose of pile foundations is to transmit a super structure load to deeper load bearing strata, to withstand lateral, vertical, uplift load and to minimize the settlement. A structure can be founded on piles if the soil immediately beneath its base does not have adequate bearing capacity to withstand the desired structural loads. If the results of site investigation show that the soil at shallow depth is unstable or if the estimated settlement is beyond acceptable limits, a pile foundation will be adopted.

7.2 Classification of Piles

Classification based on materials or composition:

1. Timber piles: Timber piles are made from tree trunks and are well seasoned, straight and free from all defects. Usually available length will be 4 to 6m. Timber piles are used where good bearing stratum is available at a relatively shallow depth.
2. Concrete piles: Concrete piles are either precast or cast in-situ. Precast piles are cast and cured at the casting yard and then transported to the site for installation. These piles are adequately reinforced to withstand handling stresses along with working stress. Precast piles are generally used for short lengths. Cast-in-situ piles are constructed by drilling hole in the ground and then filling that hole with freshly prepared concrete after placing the reinforcement.
3. Steel Piles: Steel piles are usually of rolled H-sections or thick pipe sections. These piles are used to withstand large impact stresses and where fewer disturbances from driving is desired. These piles are also used to support open excavations and to provide seepage barrier.

4. Composite piles: A pile made up of two different materials like concrete and timber or concrete and steel is called composite pile. Composite piles are mainly used where a part of the pile is permanently under water. The part of the pile which will be under water can be made of untreated timber and the other part can be of concrete.

Classification based on the function:

1. End bearing piles: Piles which transfer structural load to a hard and relatively incompressible stratum such as rock or dense sand are known as end bearing piles. These piles derive the required bearing capacity from end bearing at tip of the pile.

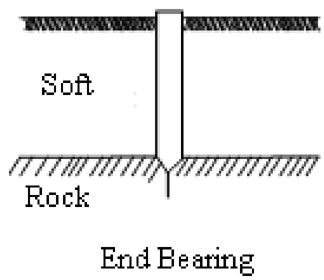

Fig.7.1a End bearing pile

2. Friction piles: These are piles which derive carrying capacity from skin friction or adhesion between the pile surface and surrounding soil.

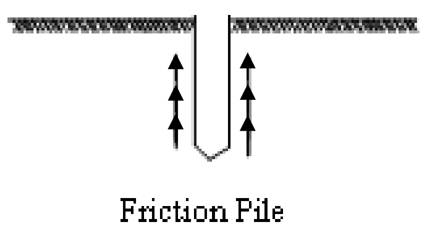

Fig.7.1b End bearing pile

3. Tension pile: These piles are also called as uplift piles. Generally it can be used to anchor down the structures which are subjected to uplift pressure due to hydrostatic force.

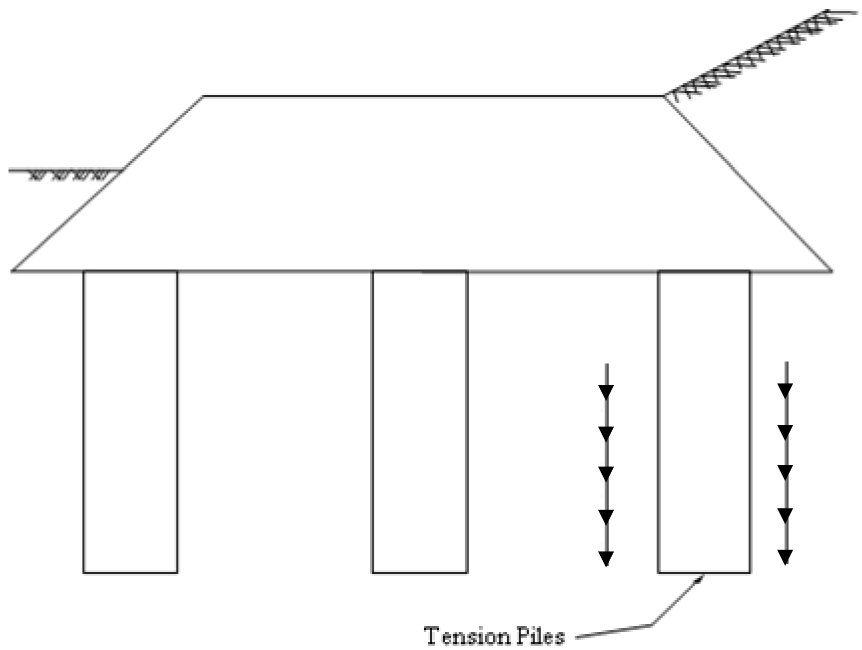

Fig.7.1c Tension pile

4. Compaction piles: These piles are used to compact loose granular soil to increase its bearing capacity. Compaction piles do not carry load and hence they can be of weaker material. Sand piles can be used as compaction piles.

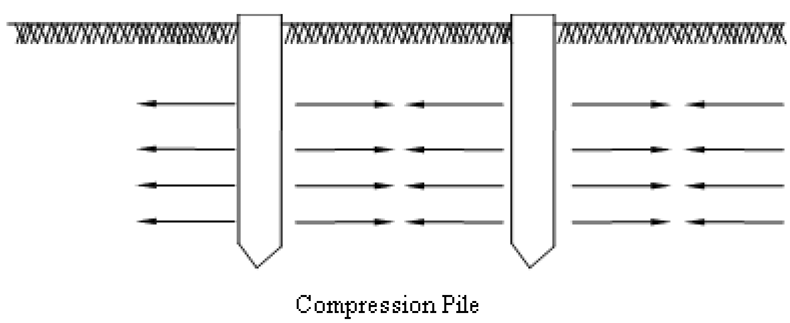

Fig.7.1d Compaction pile

5. Anchor piles: These piles are generally used to provide anchorage against horizontal pull from sheet piling.

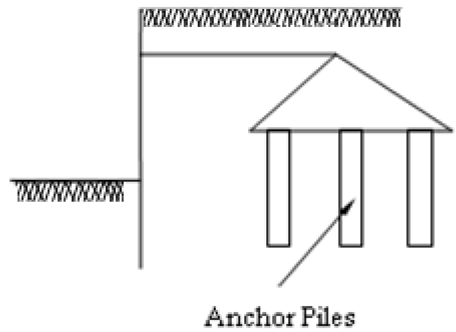

Fig.7.1e Compaction pile

6. Fender piles and dolphins: Fender piles and dolphins are used to protect water front structure from impact of any floating object or ships.

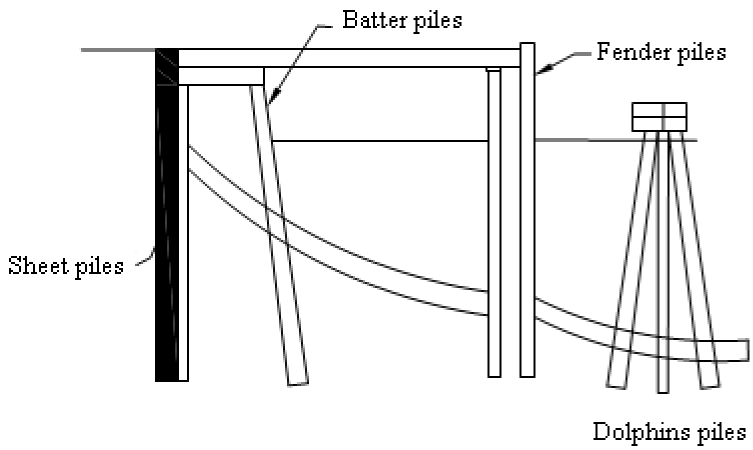

Fig.7.1f Compaction pile

Classification based on method of installation:

1. Bored piles: Bored piles are constructed in pre-bored holes either using a casing or by circulating stabilizing agent like bentonite slurry. The borehole is filled with concrete after placing or lowering reinforcement. The main advantage in bored piles is no damage due to handling and driving which is common in driven piles. The different types of bored piles are: small diameter piles up to 600mm diameter: Large diameter pile greater than 600mm; Under-reamed piles generally 300 to 450mm diameter.
2. Driven piles: Driven piles may be of concrete, steel or timber. These piles are driven into the soil strata by the impact of a hammer. Generally boring is not used in these cases.

When pile is driven into granular soils it densifies the soil and increases stiffness (strength) of soil.

3. Driven and Cast-in-Place Piles: These piles are formed by driving a tube with a closed end into the soil strata, and then filling the tube with freshly prepared concrete. The tube may or may not be withdrawn afterwards.

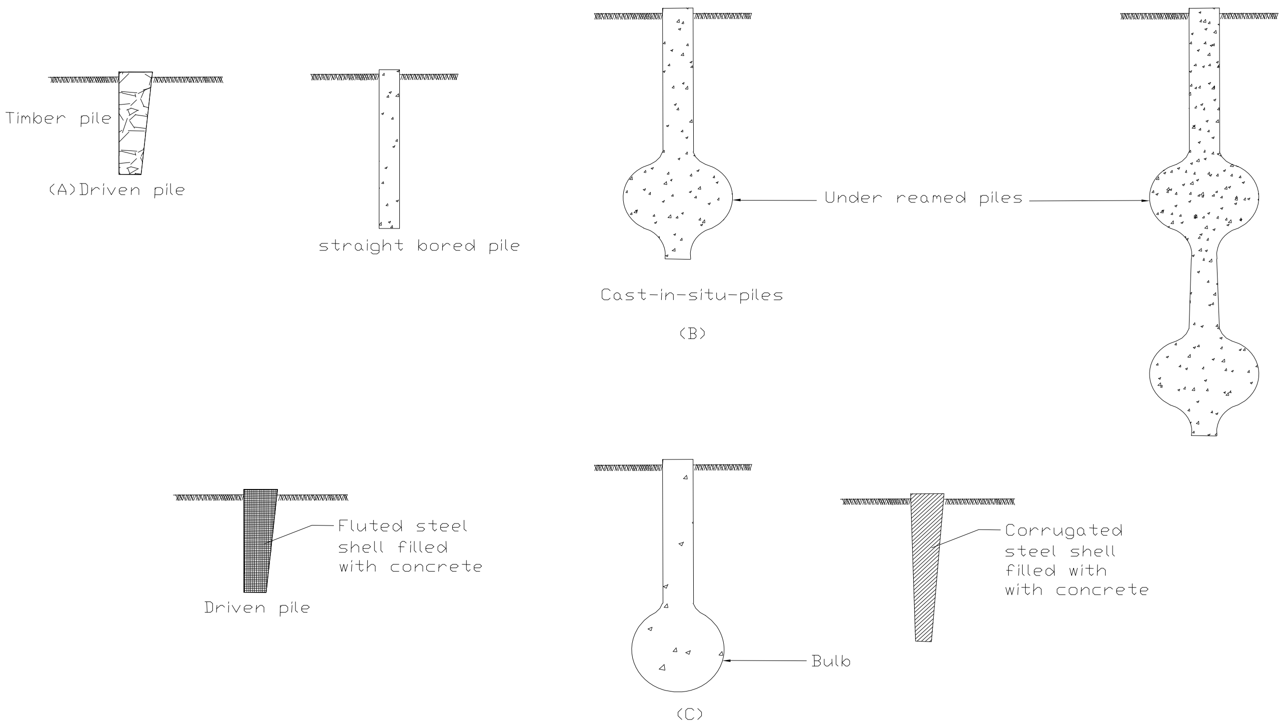

Fig. 7.2: Driven and Cast-in-situ piles

7.3 Load Transfer Mechanism

To understand the mechanism of distribution of applied load into skin resistance along length of pile and base resistance at top of pile, we consider the pile shown in Fig. 7.3 (a). If the pile is instrumented and the load on pile increased gradually, we can determine the load distribution along the pile at different stages of loading and plot the same as shown in Fig 7.3 (b).

When a load Q_1 is applied on the pile head, the axial load at pile top level is also Q_1 but at some level A_1 at distance L_1 below pile top the axial load is zero. The entire load Q_1 is thus distributed as skin resistance along pile length L_1. The lower section A_1B of pile is not affected by the load Q_1. We increase the load at pile top to Q_2, such that the axial load at the top of pile is just zero. The total load applied Q_2 is distributed as skin resistance along whole length L of pile. Any applied load greater than Q_2 will be distributed as skin resistance and point resistance. Both the components increase as the applied load is increased. But at some load level Q_m, the skin resistance reaches an ultimate value Q_f. further increase in applied load above Q_m will only result in increase in point load Q_p until the soil at base of pile fails by punching shear.

For any applied load the relative proportions of skin resistance and base resistance, mobilized depends on the shear strength and elasticity of soil. In general it is found that the vertical movement of pile which is required to mobilize full base resistance is much greater than that required to mobilize full skin resistance. For instance, in the case of bored cast insitu piles full skin resistance is mobilized at settlement of 0.5 to 1 percent of pile diameter, full base resistance is mobilized at settlement of 10 to 20 percent of pile diameter.

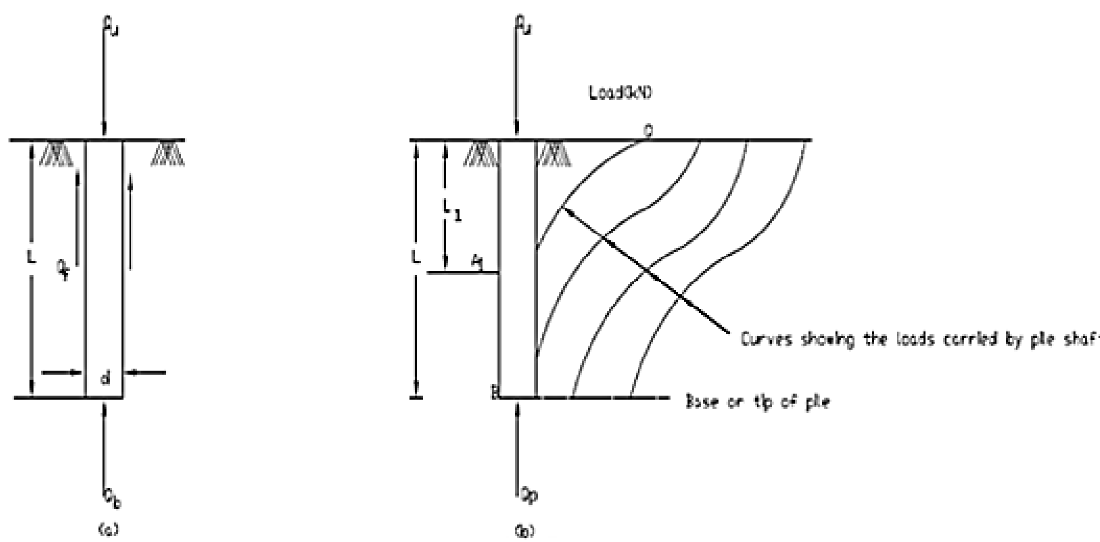

Fig.7.3: Load transfer mechanism

7.4 Load Carrying Capacity of Piles

The load carrying capacity of a single pile can be estimated using

1. Static formulae
2. Dynamic formulae
3. Correlations with penetration test data
4. Load tests

7.4.1 Static Formulae

The static formulae for ultimate load carrying capacity of pile based on soil propertiesand pile geometry are as given in Eq 7.1 and Eq 7.2 for piles in granular soils and cohesive soils respectively.

Piles in granular soils: The ultimate load Q_u is given by

Q_u = End bearing resistance Q_p+ Skin resistance Q_s

$$Q_u = A_p\left(0.5\gamma D N_\gamma\right) + A_p\left(\sigma' N_q\right) + \sum_{i=1}^{n} kA_{si}(\sigma_i' tan\delta) \qquad \text{(Eq.7.1)}$$

where

A_p = Cross section area of pile.

D = Stem diameter of pile.

N_γ = Bearing capacity factor taken for general shear.

N_q = Bearing capacity factor.

σ'= Effective overburden pressure (Critical depth taken as 15D for ø ≤30° and 20D for Ø ≥40°)

k= Co-efficient of earth pressure.

σ_i'= Effective over burden pressure at middle of corresponding layer.

δ = Angle of wall friction usually taken as ¾ ø of soil.

A_{si} = Surface area of pile.

Piles in cohesive soils

The ultimate bearing Q_u of piles in cohesive soils is given by the following formula

Q_u = End bearing resistance Q_p + Skin resistance Q_S

$$Q_u = A_p\, N_c C_p + \sum_{i=1}^{n} \alpha\, A_{si}\, C_{si}$$

(Eq.7.2)

where

N_C = Bearing capacity factor in clays which is taken as 9 (See Skempton's curve)

c_P = Average cohesion at pile toe.

$\acute{\alpha}_i$ = Adhesion factor.

c_i = Average cohesion of the i^{th} layer on the side of the pile.

A_{si} = Surface area of pile stem in the i^{th} layer.

$\acute{\alpha}_i\, c_i$ = Adhesion between shaft of pile and clay.

Piles in C-Ø soils

Where the soil has large values of both c and ø (as for a true c- ø), we should use the conservative Terzaghi's bearing capacity factors to determine the load carrying capacity

$$Q_u = A_p\, CN_c + \sigma_{vb} N_q + 0.5\gamma D N_\Upsilon + \sum_{i=1}^{n} A_s\, \alpha_c + k(\sigma_v tan\varphi)$$

(Eq.7.3)

where

N_c, Nq, $N\gamma$ = Terzaghi's bearing capacity factors

σ_{vb}, σ_v = Effective overburden pressure at base and pile shaft, irrespective of the critical depth.

7.4.2 Dynamic Formulae

For Piles driven in soils there are a set of formulae based on the so-called Engineering News (1888) formula.

$$Q_u = \frac{WHn}{S+C} \qquad \text{(Eq.7.4)}$$

where, Q_u= Ultimate load capacity of the driven pile.

W= Hammer weight (tons)

H= Height of fall of hammer (cm)

S= Final set (cm/blow)

C= a constant depending on type of hammer (2.54 for drop hammer, 0.254 for steam hammer)

η_h = efficiency of hammer(0.65 for steam hammer, 1.0 for drop hammer)

For double-acting steam hammer

The hammer weight W is replaced by W+ap,

where 'a' is the area of the piston (cm^2) and p is the steam pressure (kg/cm^2).

$$Q_a = \frac{Q_u}{F}$$

where, Q_a= allowable load

F is taken as 6.

7.4.2.2 Hiley's Modification of Wellington's formula

$Q_u = \frac{WHn_h n_b}{S+\left(\frac{C}{2}\right)}$ (Eq.7.5)

where, the terms W, H, η_h andS are the same as before.

η_b is the efficiency of the hammer blow.

$\eta_h = \frac{W + Pe^2}{W + Pe}$ if W > ep

$\eta_b = \frac{W + Pe^2}{W + Pe}$ if W < ep

Here P is the pile weight and e is the co-efficient of restitution, whose value is 0.4 for concrete and 0.5 for steel.

The quantity C in Hiley's formula is total elastic compression given by

$C = C_1 + C_2 + C_3$

where, C_1, C_2, C_3 are the compression of pile cap, pile shaft and soil respectively.

Dynamic formulae are generally found to be less reliable than static formulae.

7.4.3 Load Carrying Capacity from Penetration Test Data

Static cone penetration test data and standard penetration test data are often used to determine the pile load capacity.

The point resistance of driven piles in sand including H piles, can also be determined using N values as per the below equation.

$$q_{pu} = 40N\left(\frac{L}{D}\right) \text{ kN/m}^2$$

where *N* is thestandard penetration resistanceas observed in the field for bearing stratum without the overburden corrections

Data from a static cone penetration test can be used to estimate the unit skin friction.

$$f = aq_c$$

where, q_c= static cone resistance in kg/cm^{2}and a is coefficient whose value depends on the soil type(0.04 to 0.08 for clays, 0.01 to 0.04 for silty sands, 0.01 to 0.02 for sands).

The maximum unit skin friction for steel H-piles is taken as 0.5kg/cm^2 and for driven concrete piles it is 1.0kg/cm^2

For driven steel or concrete piles the point resistance may be obtained from the SPT N value

$$q = 4N\left(\frac{l}{d}\right)$$

For bored concrete piles

$$q = 1.4N\left(\frac{l}{d}\right)$$

7.4.4 Load Tests on Piles

Before finalizing the design, load tests are carried out on piles installed for the purpose on the site. These are called initial load tests. They are useful in determining the general suitability of the proposed pile foundation, comparing the load capacity obtained from formulae, and for a general check on the piling equipment to be used as well as on soil properties.

Procedure for pile load test

The pile head is chipped off to natural horizontal plane till sound concrete is met. The projecting reinforcement is cut off suitably and the top is finished smooth & level with plaster of Paris.Loading platform of 6.2m x 6.2m is constructed by using 2nos. of ISMB 500 as main girders and 21nos of ISMB 300 as secondary girders.

Fig. 7.4 (a): Pile load Test Setup

The CG of platform is made to coincide with centre of pile. Platform thus constructed is loaded with sand bags for required weight. A 20mm thick mild steel plate is placed on the top of pile head, Hydraulic jack of 250T Capacity is placed centrally on top of the plate. The gap between the top of jack and bottom of main girders is filled with steel packing materials. The Hydraulic pump is connected to jack by flexible pressure hose. Calibrated pressure gauge is connected to hydraulic pump. Datum bars of heavy sections were placed very near to pile head and are supported on ends at a distance of 2m on either side from face of the pile. Two numbers of settlement gauges are placed on pile head at diametrical opposite locations with the help of magnetic bases fixed on datum bars.

Fig. 7.4(b): Pile load Test Setup

The pump is operated till the ram of jack touches the bottom of main girders. At this stage the pressure gauge reading is zero and dial gauge reading are adjusted for zero loading. The loads are then applied in increments of 20% of safe load. For each increment of load the dial gauge reading are taken at intervals of 15 minutes, till the rate of settlement is less than 0.1 mm in the first half hour or 0.2 mm in one hour of for a maximum period of 2hrs. Then the next increment of load is applied and the procedure repeated till the test load is reached. This load is maintained for 24 hours and hourly settlement readings are noted. At the end of 24 hours, unloading is done gradually till the entire load is released.

Allowable load from single pile load test data

There are different methods for determining the allowable loads on a single pile which can be determined by making use of load test data. If the ultimate load can be determined from load-settlement curves, allowable loads are found by dividing the ultimate load carried by a pile by suitable factor of safety which varies from 2 to 3. Normally a factor safety is 2.5 is recommended.

Fig. 7.5: Determination of Ultimate load from load-settlement curve

1. The ultimate load, Q_u can be determined as the abscissa of the point where the curved part of the load-settlement curve changes to falling straight line, Fig. 7.5(a)
2. Q_u is the abscissa of the point of intersection of the initial and final tangents of the load-settlement curve, Fig.7.5(b)
3. The allowable load Q_a is 50 percent of the ultimate load at which the total settlement amounts to one-tenth of the diameter of the pile.
4. The allowable load Q_a is sometimes taken as equal to two-thirds of the load which causes a total settlement of 12mm.
5. The allowable load Q_a is sometimes taken as equal to two-thirds of the load which causes a net settlement of 6mm.

7.5 Pile settlements

Pile settlement can be estimated as follows.

1. Compute the average pile axial force in each segment of length ΔL, average cross-section & A_{av} and shaft modulus of elasticity E_p from the pile butt to point. That is.

$$\Delta Hs, s = \frac{P_{av} \times \Delta L}{A_{av} \times E_p}$$

and sum the several values to obtain the axial total compression

$$\Delta H_a = \sum \Delta Hs, s$$

2. Compute the point settlement using the equation below.

$$\Delta H_{pt} = \Delta qD\left(\frac{1-\mu^2}{Es}\right)mI_S I_F F_1$$

Where,

mI_s= 1

I_f= Fox embedment factor, with values as follows:

If=0.55 if L/D≤5

=0.5 L/D>5

D = diameter of the pile

μ =Poisson's ratio

Δq= bearing pressure at point = input load/ A_p

E_s = Young's modulus

SPT: Es= 500 (N+15)

CPT: 3-6 q_c

F1 is the reduction factor as follows

0.25 if the axial skin resistance reduces the point load Pp ≤0

0.5 if the point load Pp >0

0.75 if the point bearing

7.6Negative Skin Friction

When a weak, compressible soil layer is sandwiched between hard layers, a pile passing through such a stratum may be subjected to an additional load due to compression of the weak layer. This compression may be caused by consolidation, fill placing, remolding during driving, or lowering of the water table. The portion of the pile within this layer is subjected to draw down force in addition to the structural loads. This force should be taken into account when designing the pile foundation.

An approximate estimate of the force can be made by empirical formulae such as following

F_d= force due to negative skin friction

F_d= (perimeter*soil depth)*C_u [for clays]

F_d= 0.5(perimeter*(soil depth)^2*$\gamma K\tan\delta$) [for sands]

C_u= undrained shear strength

γ= unit weight of soil

K= coefficient of earth pressure, δ = angle of internal friction.

7.7Under-Reamed Piles

These are bored, cast in-situ, concrete piles with one or more bulbs formed by enlarging the pile stem. They are suitable for loose and filled up sites, or where soils are weak or expansive like black cotton soil.

The bulbs are located at depths where good bearing strata are available but they should not be placed too near the ground level. Bulb size is usually 2 to 3 times the pile stem diameter. The bulb provides a large bearing area, increasing the pile load capacity. They are also effective in resisting the downward drag due to the negative skin friction that arises in loose or expansive soils. Bulb spacing should not exceed 1.5 times the bulb diameter.

7.7.1 Procedure for Construction of Under-Reamed Piles

The hole is drilled to the full required depth using augers. The under reaming tool consists of a link mechanism attached to a vertical rod with a handle at the top and connected to a bucket at the bottom. The link mechanism incorporates cutting blades. The under reaming tool is inserted into the hole. When the central rod is pressed by the handle the mechanism actuates the cutting blades to open out. The mechanism is now made to rotate keeping the handle under pressure. The blades now scrap the soil from the sides of the hole which falls into the bucket below. The rotation under pressure is continued until the full amount of soil forming the bulb is removed which is identified by the free rotation of the mechanism. The volume of the bucket is such that it gets filled when the bulb is fully formed. The handle is now tightened which makes the link mechanism to collapse back into the position. The under reamed tool is now withdrawn, the reinforcement cage inserted and the hole concreted.

7.8Group Action

Piles are generally used in groups with a common pile cap. A group may consist of two or three, or as many as ten to twelve piles depending on the design requirement. The load carrying capacity of a group of piles is given by

$$(Q_u)_g = Nq_u n \qquad \text{(Eq.7.6)}$$

where,

$(Q_u)_g$= Load carrying capacity of pile group

N = number of piles

q_u= allowable load per pile

n = group efficiency

Its value for bearing or friction piles at sites where the soil strength increases with depth is found to be 1.

For friction piles in soft clays the value on n is less than 1. The actual value of n depends on soil type, method of pile installation, and pile spacing.

When piles are driven in loose, sandy soils, the soil is densified during driving, and n >1 in such cases.

It has been observed that if the spacing between piles is more than 2.5 times the pile diameter, the group efficiency is not reduced.

The large pile to pile spacing will increase the overall cost of construction. The reduction in load capacity due to the group effect can be estimated empirically.

The use of Feld's rule is probably the simplest. It states that the load capacity of each pile in a group is reduced by 1/16 on account of the nearest pile in each diagonal or straight row.

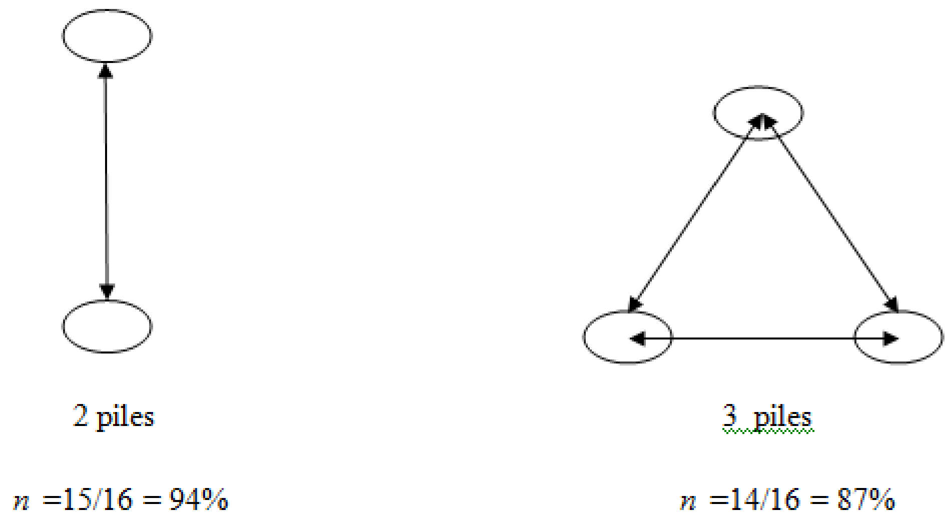

Fig. 7.6: Group action of piles- Feld's rule

A group of piles may fail as a block, i.e., by sinking into the soil and rupturing it at the periphery of the group Fig. 7.7.

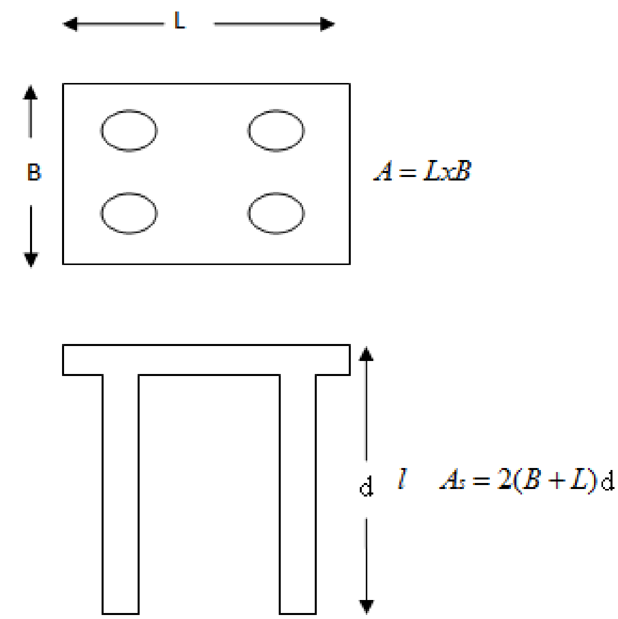

Fig.7.7: Failure of a pile group as a block

7.8.1 Ultimate Load Carrying Capacity for the Pile Group

The ultimate load carrying capacity for the pile group taken as a block is given by

$(Q_u)_g = C_u N_c A_b + C_u A_p$(Eq7.7) where A_p and A_b are the area of the base and the surface area of block. i.e. A_b = LB

where, L and B are the dimensions of the pile cap.

A_p is the perimeter of the block times the embedded length of the pile.

The Ultimate load capacity for the group is also evaluated as

(Eq.7.8)

Here each pile is assumed to individually carry the same load, whether in group or as a single pile. The load carrying capacity of a pile group is taken to be the smaller of the two values obtained from Eqs.7.7 and 7.8.

7.8.2 Efficiency of a Pile Group

The efficiency of a pile group is defined as

$$\eta_g = \frac{Ultimate\ bearing\ capacity\ of\ the\ group}{n \times ultimate\ bearing\ capacity\ of\ \sin gle\ pile\ in\ the\ group}$$

where n = number of piles in the group

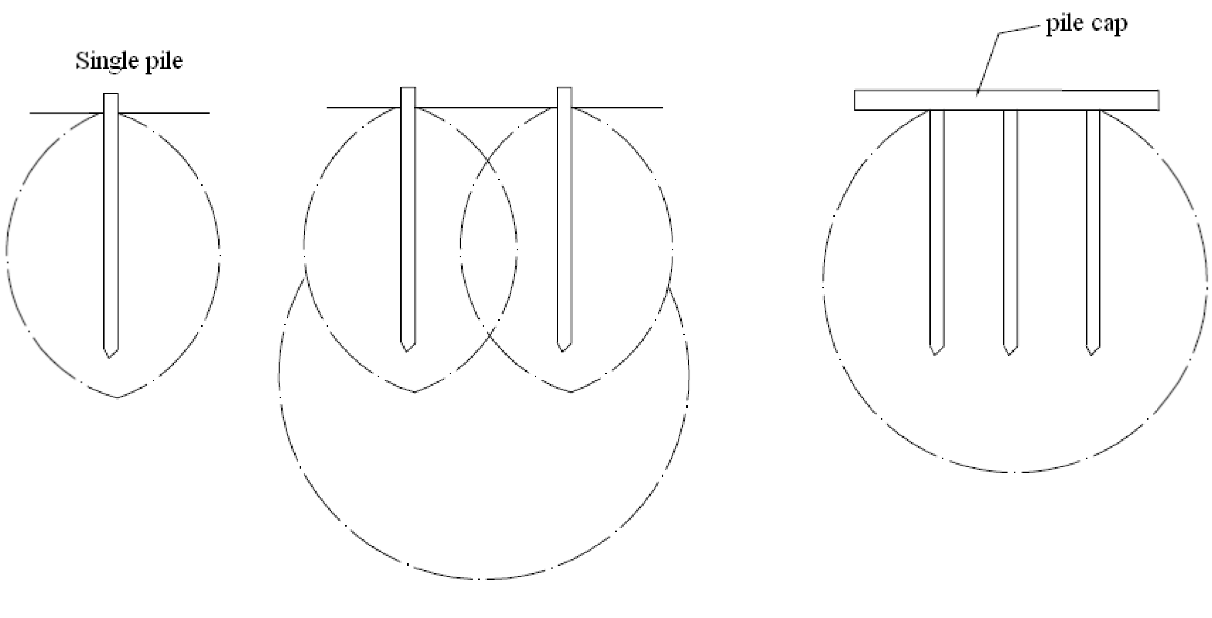

Fig. 7.8: Group action of Piles

7.8.3 Settlement of Pile Groups

Due to group action, both immediate and consolidation settlement values of a pile group are greater than those for a single pile.

For bearing piles the total foundation load is assumed to act at the base of the piles on an imaginary foundation of the same size as the plan of the pile group as show in Fig 7.9 (b)

For friction piles it is virtually impossible to determine the level at which the structural load is effectively transferred to the soil. The level used in design is at a depth of two-thirds the penetration depth.

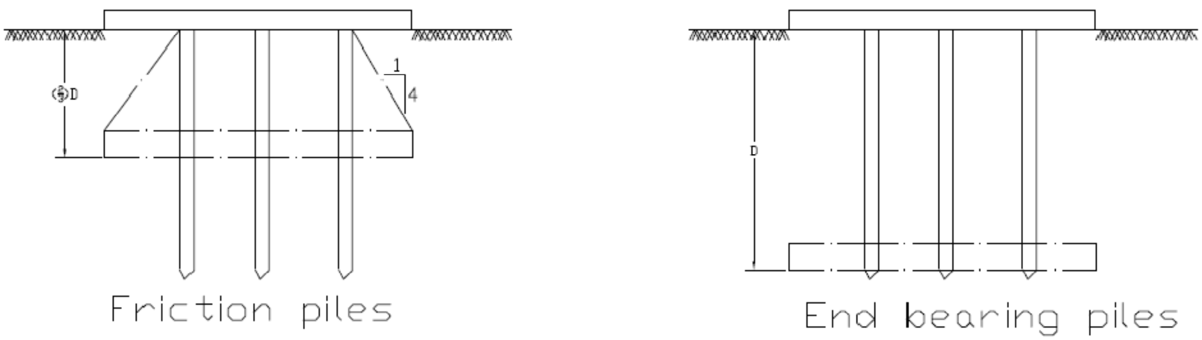

Fig.7.9: Equivalent foundations for pile

7.8.4 Multi-Layered Deposits

Driven piles through a multi-layer deposit can be calculated by their load capacities from both skin and point resistance and these capacities may need to be confirmed by load tests. Whenever possible, piles should be driven to a layer of sufficient strength and thickness that they derive their load capacity entirely from the layer.

$$q' = \frac{Q}{(B+1.15H')(L+1.15H')} \qquad \text{(Eq.7.9)}$$

for a side slope of 30 degrees,

Or $$q' = \frac{Q}{(B+H')(L+H')} \qquad \text{(Eq.7.10)}$$

for a side slope of 2:1.

If the strength of the underlying clay layer is c, the margin of safety against a punching failure will be sufficient if:

$$q' \leq 3c$$

7.9Eccentric and Inclined Loads on Pile Groups

When horizontal force acting on a pile group is accompanied by a vertical load, due to the weight of the pile cap or some supported structure. In a pile cap acted upon by an eccentric inclined load which will distribute itself into the piles.

If there is no horizontal load and if the vertical load is concentric with the centroid of the pile group, the load in each pile is simply taken to be equal to the total load divided by the total number of piles.

$$Q_p = \frac{Q_v}{n}$$

Q_p= total vertical load

Q_v= vertical load per pile

n= number of piles

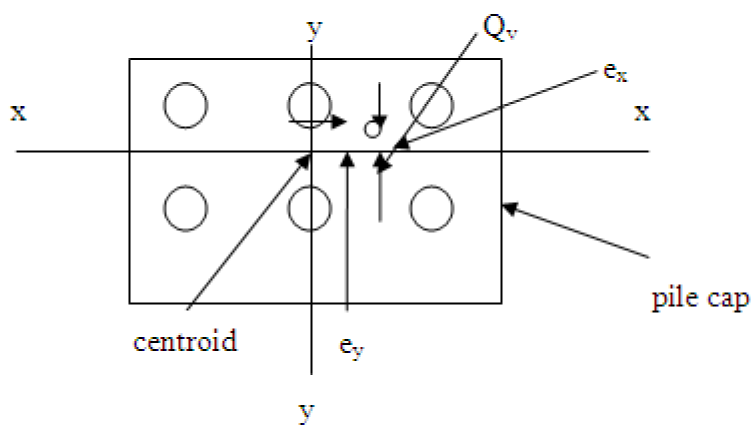

Fig. 7.10: Pile group with eccentric vertical load

Consider a pile in the group at distance X and Y from the centroid of the group.

From the theory of bending of beams, tensile and compressive stresses at a distance Y, from the neutral axis given by

$$f_b = \pm \frac{M_y}{I}$$

where,

M= applied moment

I= moment of inertia of beam section

Total vertical load induced in the pile can be expressed by

$$Q_p = \frac{Q_v}{n} \pm \frac{A_b Y M_x}{I_x} \pm \frac{A_b X M_y}{I_y}$$

$$I_x = I_o + A_b \sum Y^2 = A_b \sum Y^2$$

Since, I_o= moment of inertia of the pile section in negligible.

$$I_y = A_D \sum X^2$$

$M_x = Q_v e_y$ And $M_y = Q_v e_x$

Hence

$$Q_p = \frac{Q_v}{n} \pm \frac{A_b Y M_x}{I_x} \pm \frac{A_b X M_y}{I_y}$$

$Q_p = Q_v \left(\frac{1}{n} + \frac{X e_x}{\sum X^2} + \frac{Y e_y}{\sum Y^2}\right)$(Eq.7.11)

7.10 Laterally Loaded piles

Structures supported on pile foundation are often subjected to lateral loads and moments in addition to vertical loads. The sources of lateral loads are traffic, seismic events, wind, waves, and earth pressure. Moments may arise due to the eccentricity of the vertical force, fixity of the superstructure to the foundation and the location of the resultant lateral force on the pile with reference to the ground surface.

There are two types of piles encountered in practice.

1. Long pile (L/d>30)
2. Short pile (L/d<20)

In the case of long pile: When a pile is greater than a particular length, the length loses its significance. The behavior of the pile will not be affected if the length is greater than this particular length.

Three types of boundary conditions are normally applicable.

1. Fixed-headed pile.
2. Free-head pile.
3. Partially-restrained-head pile.

In the case of free-head piles, the lateral load may act at or above ground level and the pile head is free to rotate without any restraint.

In the case of fixed head piles, the pile is free to move only laterally but rotation is prevented completely, where as a pile with a partially restrained head moves and rotates under restraint.

The partially restrained head is normally encountered in offshore drilling platforms and other similar structures.

The analysis of laterally loaded single piles is based on the following assumptions.

1. The laterally loaded pile behaves as an elastic member and the supporting soil behaves as an ideal elastic material.
2. The theory of sub-grade reaction
3. There is no axial load.

The different methods for solving the problem of laterally loaded piles are;

1. Closed-Form solution
2. Difference equation method
3. Non-dimensional method
4. A direct method
5. Pressure-meter method
6. Broms method
7. Polulos Method

When a horizontal load is applied to the head of a vertical pile which is free to move in any lateral direction, the load is initially carried by the soil close to the ground surface. However the soil compress elastically and there is some transfer of load to the soil at a greater depth. When the horizontal load is increased the soil yields plastically and the load transfer extends to greater depths.

In the case of short pile: the flexural stiffness EI of the material of the pile loses its significance. The pile behaves as a rigid member and rotates as a unit.

Failure occurs by rotation when the passive resistance at the head and toe are exceeded.

Non-dimensional solutions for laterally loaded piles in a soil deposit in which the subgrade modulus increases linearly with depth, have been developed by Reese and Matlock (1956). The solutions have been developed for long piles when L/T > 5, where L is the length of the pile and T is the relative stiffness factor given by below equation.

$T = (EI/n_h)^{1/5}$

Where E and I are the modulus of elasticity and the moment of inertia of the soil respectively; n_h is the unit modulus of the subgrade reaction.

For a vertical pile length L, subjected to horizontal load $Q_g$at the ground level and a moment Mg at the ground level, the solution for deflection y may be expressed as a function of various quantities.

$y = f(x,T,L,K,EI,Q_g,M_g)$

Using the principal of superposition,

$y = y_A + y_B$

y= total deflection of the pile at any depth

$y_{A=}$ deflection due to the horizontal load Q_R

y_B= deflection due to moment M_g at ground level

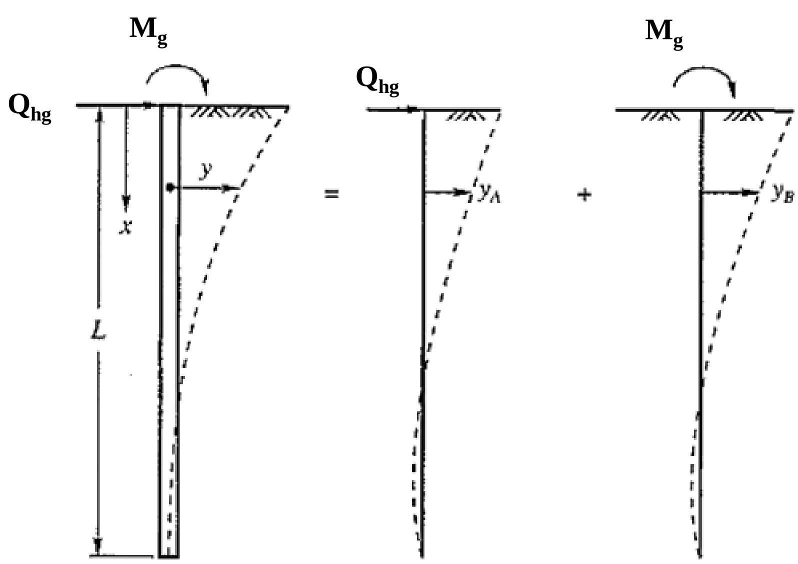

Fig. 7.11. Principle of superposition in laterally loaded piles

$$y = \left(\frac{Q_g T^3}{EI}\right) A_y + \left(\frac{M_g T^2}{EI}\right) B_y$$

Where

$$A_y = \left(\frac{y_A EI}{Q_g T^3}\right) \text{and } B_y = \left(\frac{y_B EI}{M_g T^2}\right)$$

7.11Piles on a Rocky Bed

Piles are required to be driven through weak layers of soil until the tips meet a hard stratum for bearing. If the bearing strata found to be rock, the piles are to be driven to refusal to obtain the maximum load carrying capacity from the piles. If the rock is found to be strong at its surface, the pile will refuse further driving at a negligible penetration. In these cases the load carrying capacity of the piles is governed by the strength of the pile shaft. If the soil mass through which the piles are driven happens to be stiff clays or sands, the piles can be regarded as being supported on all sides from buckling as a strut. In these cases, the capacity of a pile is calculated from the safe load on the material of the pile at the point of minimum cross-section.

If piles are driven to weak rocks, working loads as calculated by the available stress on the material of the pile shaft may not be possible. In these cases frictional resistance developed over the penetration into the rock and the end bearing resistance are required to be calculated.

Tomlinson (1986) suggested an equation for calculating the end bearing of piles resting on rock strata.

$$q_u = 2N_\varphi q_{ur}$$

where $N_\varphi = \tan^2(45 + \varphi / 2)$, q_u= Unconfined compressive strength of the rock.