

UNIT-VI: DESIGN OF DEEP FOUNDATIONS OR PILE FOUNDATIONS

INTRODUCTION

- Deep or Pile foundation is defined as deep foundation with $D > B$ or $D > 3\text{m}$.
- Deep or Pile foundation always more expensive than shallow foundation but will overcome problems of soft surface soils by transferring load to stronger, deeper stratum, thereby reducing settlements.
- Deep foundations are employed when the soil strata immediately beneath the structure are not capable of supporting the load with tolerable settlement or adequate safety against shear failure.
- Piles are relatively long, slender members that are driven into the ground or cast-in situ. Piers, caissons or wells are larger, constructed by excavation and are sunk to the required depth; these usually permit visual examination of the soil or rock on which they rest.
- They are normally used to carry very heavy loads such as those from bridge piers or multi-storeyed buildings.

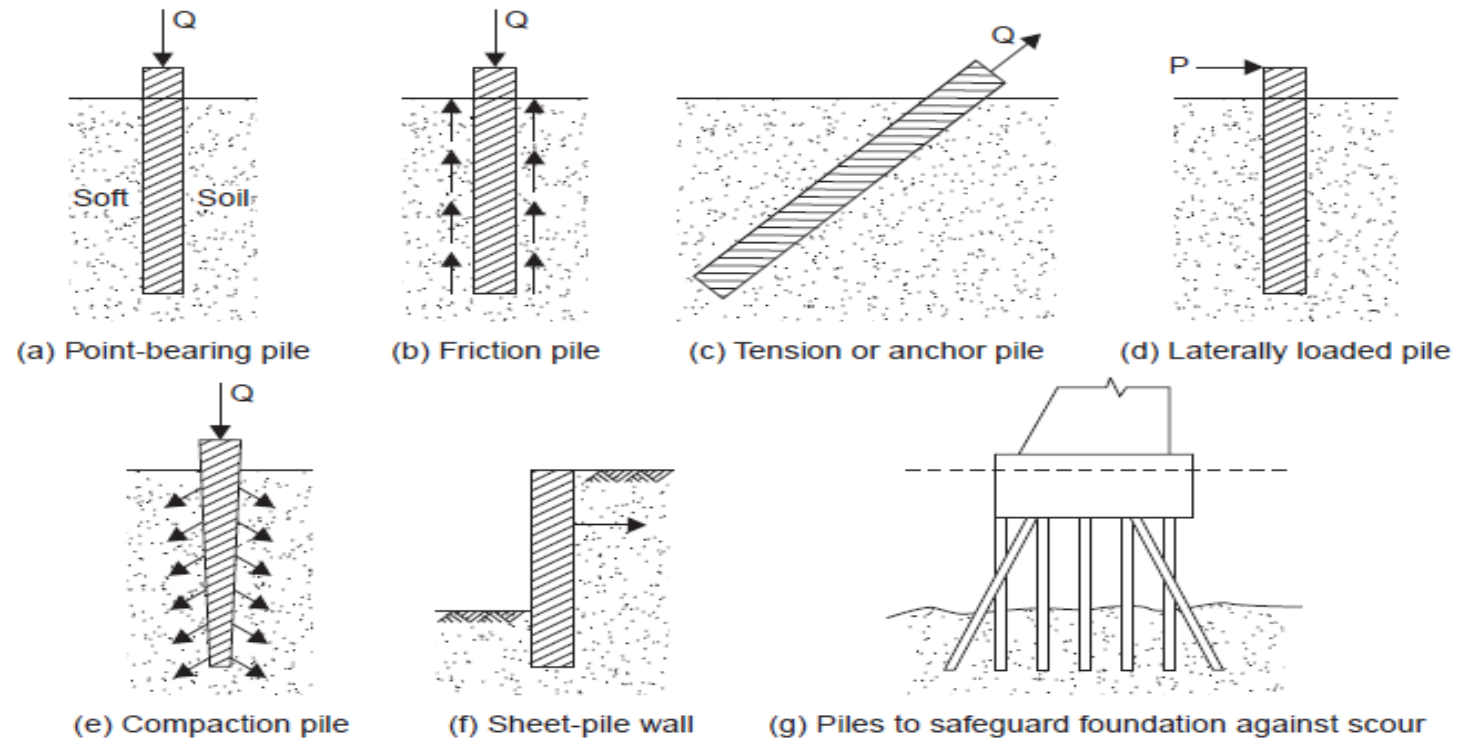
NECESSITIES OF Deep or PILE FOUNDATION

- 1] Top layers of soil are highly compressible for it to support structural loads through shallow foundations.
- 2] Rock level is shallow enough for end bearing pile foundations provide a more economical design.
- 3] Lateral forces are relatively prominent.
- 4] In presence of expansive and collapsible soils at the site.
- 5] Offshore structures
- 6] Strong uplift forces on shallow foundations due to shallow water table can be partly transmitted to Piles.
- 7] For structures near flowing water (Bridge abutments, etc.) to avoid the problems due to erosion.

USES OF PILE FOUNDATIONS

The important ways in which piles are used are as follows:

- (i) To carry vertical compressive loads,
- (ii) To resist uplift or tensile forces, and
- (iii) To resist horizontal or inclined loads.



TYPES OF PILE FOUNDATIONS

- Steel Piles

- 1] Pipe piles

- 2] Rolled steel H-section piles

- Concrete Piles

- 1] Pre-cast Piles

- 2] Cast-in-situ Piles

- 3] Bored-in-situ piles

- Timber Piles

- Composite Piles



STEEL PILES:

- Usual length: 15m – 60m
- Usual Load: 300kN – 1200kN

Advantage:

- Relatively less hassle during installation and easy to achieve cut off level.
- High driving force may be used for fast installation
- Good to penetrate hard strata
- Load carrying capacity is high

Disadvantage:

- Relatively expensive
- Noise pollution during installation
- Corrosion



CONCRETE PILES:

Pre-cast Piles:

- Usual length: 10m – 45m
- Usual Load: 7500kN – 8500kN

Cast-in-situ Piles:

- Usual length: 5m – 15m
- Usual Load: 200kN – 500kN

Advantage:

- Relatively cheap
- It can be easily combined with concrete superstructure
- Corrosion resistant
- It can bear hard driving

Disadvantage:

- 1)Difficult to transport
- 2)Difficult to achieve desired cut off



TIMBER PILES

- Timber is most suitable for long cohesion piling and piling beneath embankments. The timber should be in a good condition and should not have been attacked by insects.
- For timber piles of length less than 14meters, the diameter of the tip should be greater than 150mm.If the length is greater than 18 meters a tip with a diameter of 125mm is acceptable.
- It is essential that the timber is driven in the right direction and should not be driven into firm ground. As this can easily damage the pile. Keeping the timber below the ground water level will protect the timber against decay and putrefaction.
- To protect and strengthen the tip of the pile, timber piles can be provided with toe cover.



COMPOSITE PILES

Combination of different materials in the same of pile. As indicated earlier, part of a timber pile which is installed above ground water could be vulnerable to insect attack and decay. To avoid this, concrete or steel pile is used above the ground water level, whilst wood pile is installed under the ground water level.



DESIGN OF PILE FOUNDATIONS

- The design of a pile foundation consists of assuming a design, then checking the proposed design for safety and revising it until it is satisfactory.
- The final design is selected on the basis of cost and time available for construction.

DESIGN STEPS

- **Selection of Length of Piles**

Selection of the approximate length of the pile is made from a study of the soil profile and the strength and compressibility of the soil strata.

- **Selection of Type of Pile and Material of Pile**

The points to be considered in the selection of type of a pile and material of pile are: (i) the loads, (ii) time available for completion of the job, (iii) the characteristics of the soil strata involved, (iv) the ground water conditions, (v) the availability of equipment, and (vi) the statutory requirements of building codes.

- **Pile Capacity**

The pile capacity both for an individual pile and for groups of piles shall be determined in accordance with the procedures outlined earlier.

- **Pile Spacing**

The piles are placed so that the capacity of the pile group acting as a unit is equal to the sum of the capacities of the individual piles.

- **Inspection and Records**

Competent engineering inspection and keeping complete records of the driving of every pile is an essential part of any important job.

PILE CAPACITY

What is meant by load carrying capacity of the pile foundation?

The amount of load the pile can carry without undergoing continuous displacements for insignificant load increments by virtue of its boundary condition (soil condition) and not by virtue of its structural strength.

The assumption for this definition is –the failure of surrounding soil occurs prior to the failure of the pile material especially in the case of concrete piles.

Typical loading diagram on a single pile embedded in soil

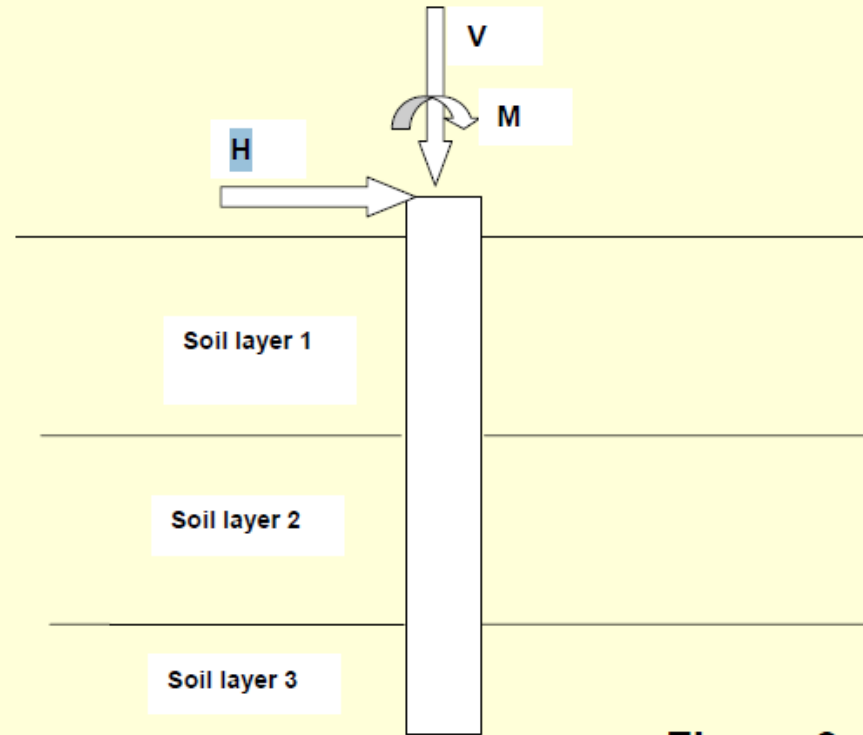


Figure 3

Note: V can be either downward or upward depending on the pile is subjected to compressive or tensile load

METHODS OF DETERMINING ULTIMATE LOAD BEARING CAPACITY OF A SINGLE VERTICAL PILE

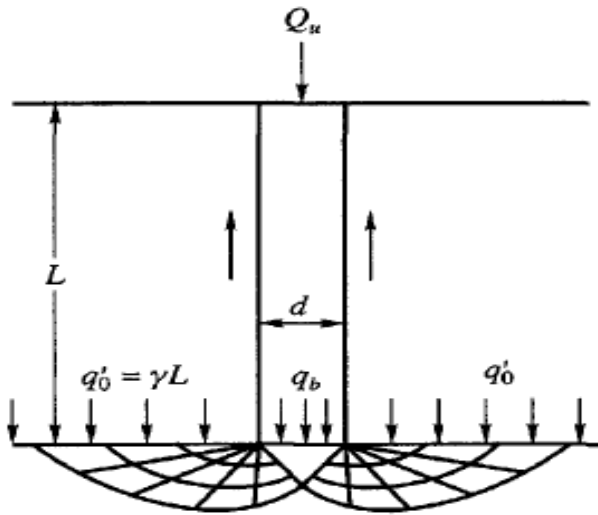
The ultimate bearing capacity, Q_u , of a single vertical pile may be determined by any of the following methods.

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.

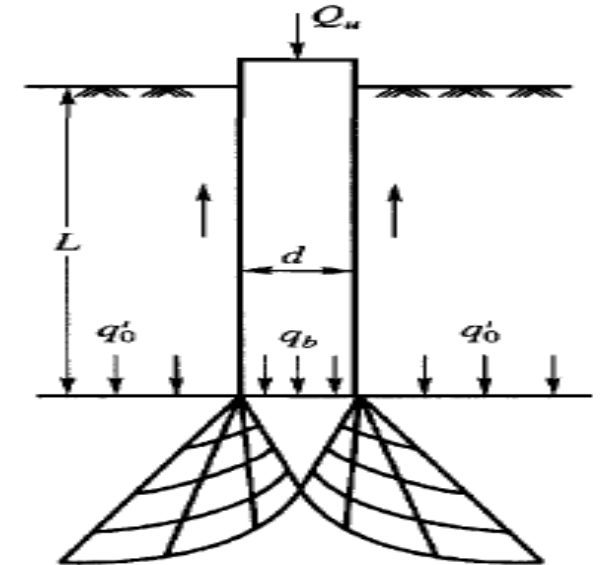
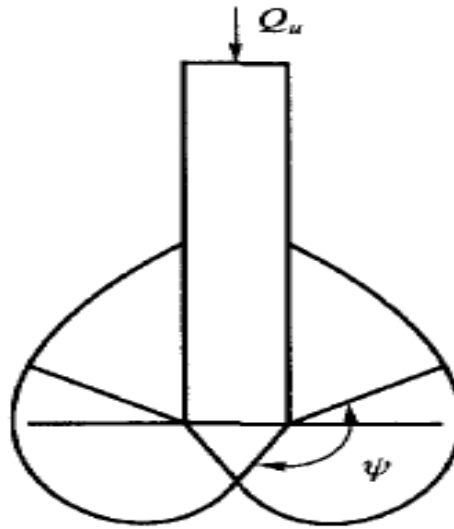
The determination of the ultimate point bearing capacity, q_b , of a deep foundation on the basis of theory is a very complex one since there are many factors which cannot be accounted for in the theory. The theory assumes that the soil is homogeneous and isotropic which is normally not the case. All the theoretical equations are obtained based on plane strain conditions. Only shape factors are applied to take care of the three-dimensional nature of the problem.

GENERAL THEORY FOR ULTIMATE BEARING CAPACITY

According to Vesic (1967), only punching shear failure occurs in deep foundations irrespective of the density of the soil so long as the depth-width ratio L/d is greater than 4 where L = length of pile and d = diameter (or width of pile). The types of failure surfaces assumed by different investigators are shown in Fig. for the general shear failure condition. The detailed experimental study of Vesic indicates that the failure surfaces do not revert back to the shaft as shown in Fig. The shapes of failure surfaces at the tips of piles as assumed by (a) Terzaghi, (b) Meyerhof, and (c) Vesic.



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The total failure load $\bar{Q}_u$ may be written as follows

$$\bar{Q}_u = Q_u + W_p = Q_b + Q_f + W_p$$

where Q_u = load at failure applied to the pile

Q_b = base resistance

Q_f = shaft resistance

W_p = weight of the pile.

The general equation for the base resistance may be written as

$$Q_b = cN_c + q'_o N_q + \frac{1}{2} \gamma d N_\gamma A_b$$

where d = width or diameter of the shaft at base level

q'_o = effective overburden pressure at the base level of the pile

A_b = base area of pile

c = cohesion of soil

γ = effective unit weight of soil

N_c, N_q, N_γ = bearing capacity factors which take into account the shape factor.

Cohesionless Soils

For cohesionless soils, $c = 0$ and the term $1/2\gamma dN_\gamma$ becomes insignificant in comparison with the term $q_o N_q$ for deep foundations. Therefore Eq. (15.5) reduces to

$$Q_b = q'_o N_q A_b = q_b A_b \quad (15.6)$$

Eq. (15.4) may now be written as

$$\bar{Q}_b = Q_u + W_p = q'_o N_q A_b + W_p + Q_f \quad (15.7)$$

The net ultimate load in excess of the overburden pressure load $q_o A_b$ is

$$Q_u + W_p - q_o A_b = q'_o N_q A_b + W_p - q'_o A_b + Q_f \quad (15.8)$$

If we assume, for all practical purposes, W_p and $q'_o A_b$ are roughly equal for straight side or moderately tapered piles, Eq. (15.8) reduces to

$$Q_u = q'_o N_q A_b + Q_f$$

or
$$Q_u = q'_o N_q A_b + A_s \bar{q}'_o \bar{K}_s \tan \delta$$

where A_s = surface area of the embedded length of the pile
 $\bar{q}'_o$ = average effective overburden pressure over the embedded depth of the pile
 $\bar{K}_s$ = average lateral earth pressure coefficient
 δ = angle of wall friction.

Cohesive Soils

For cohesive soils such as saturated clays (normally consolidated), we have for $\phi = 0$, $N_q = 1$ and $N_\gamma = 0$.

$$\bar{Q}_b = (c_b N_c + q'_o) A_b$$

The net ultimate base load is

$$(\bar{Q}_b - q'_o A_b) = Q_b = c_b N_c A_b$$

Therefore, the net ultimate load capacity of the pile, Q_u , is

$$Q_u = c_b N_c A_b + Q_f$$

or
$$Q_u = c_b N_c A_b + A_s \alpha \bar{c}_u$$

where α = adhesion factor

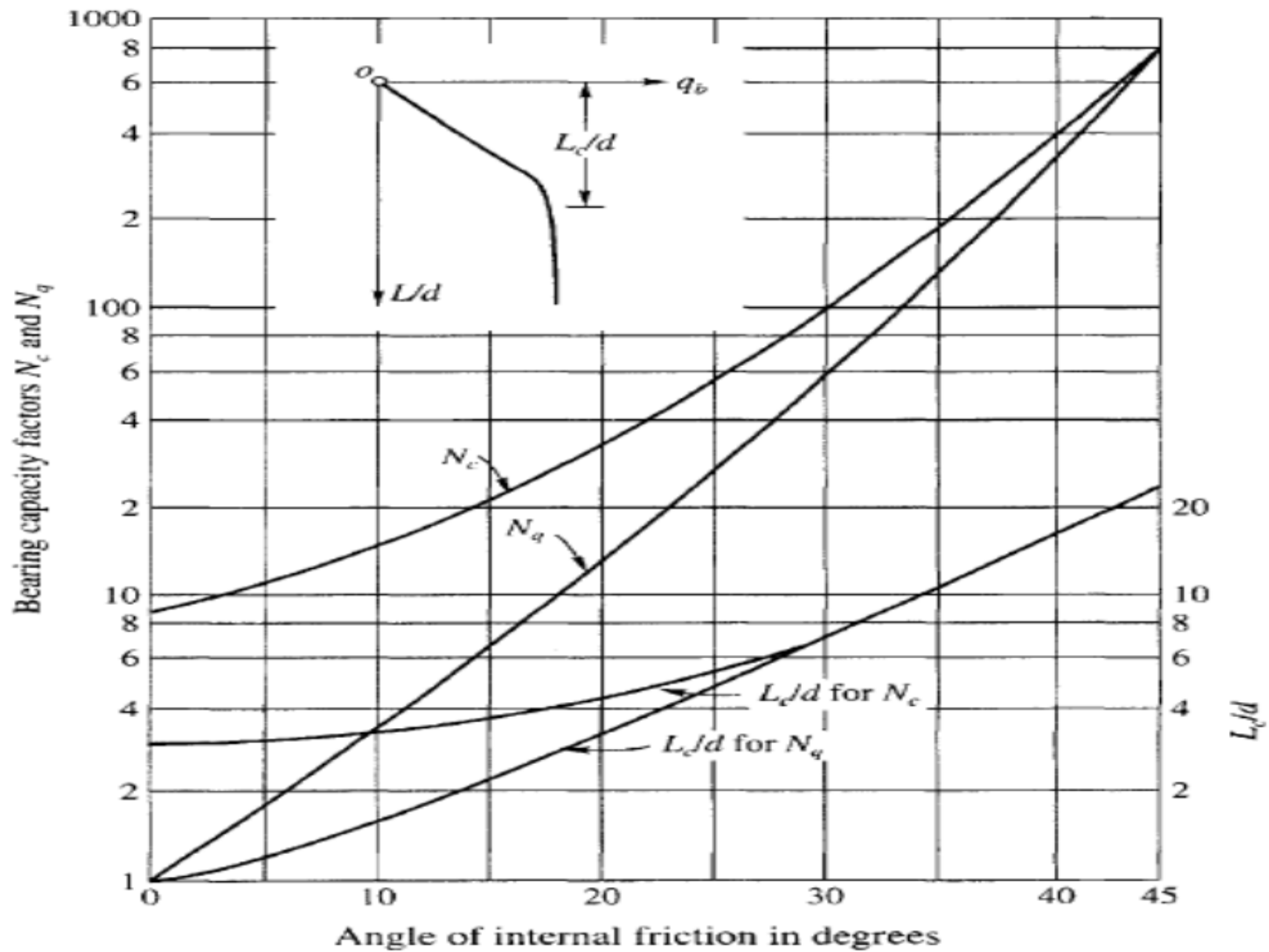
$\bar{c}_u$ = average undrained shear strength of clay along the shaft

c_b = undrained shear strength of clay at the base level

N_c = bearing capacity factor

MEYERHOF'S METHOD OF DETERMINING Q_b FOR PILES IN SAND

Meyerhof (1976) takes into account the critical depth ratio (L_c / d) for estimating the value of Q_b , Fig. shows the variation of L_c / d for both the bearing capacity factors N_c and N_q as a function of Φ . According to Meyerhof, the bearing capacity factors increase with L_b / d and reach a maximum value at L_b / d equal to about 0.5 (L_c / d) where L_b is the actual thickness of the bearing stratum.



Bearing capacity factors and critical depth ratios L_c/d for driven piles

As per Fig the value of L_c/d is about 25 for ϕ equal to 45° and it decreases with a decrease in the angle of friction ϕ . Normally, the magnitude of L_b/d for piles is greater than 0.5 (L_c/d) so that maximum values of N_c and N_q may apply for the calculation of q_b , the unit bearing pressure of the pile. Meyerhof prescribes a limiting value for q_b , based on his findings on static cone penetration resistance. The expression for the limiting value, q_{bl} , is

$$\text{for dense sand: } q_{bl} = 50 N_q \tan \phi \text{ kN/m}^2$$

$$\text{for loose sand: } q_{bl} = 25 N_q \tan \phi \text{ kN/m}^2$$

where ϕ is the angle of shearing resistance of the bearing stratum.

The equation for base resistance in sand may now be expressed as

$$Q_b = q'_o N_q A_b \leq q_{bl} A_b$$

where q'_o = effective overburden pressure at the tip of the pile L_c/d and N_q = bearing capacity factor

Clay Soil ($\phi = 0$)

The base resistance Q_b for piles in saturated clay soil may be expressed as

$$Q_b = N_c c_u A_b = 9 c_u A_b$$

where $N_c = 9$, and c_u = undrained shear strength of the soil at the base level of the pile.

VESIC'S METHOD OF DETERMINING Q_b

The unit base resistance of a pile in a $(c - \phi)$ soil may be expressed as (Vesic, 1977)

$$q_b = cN_c^* + q'_o N_q^*$$

where

c = unit cohesion

q'_o = effective vertical pressure at the base level of the pile

N_c^* and N_q^* = bearing capacity factors related to each other by the equation

$$N_c^* = (N_q^* - 1) \cot \phi$$

As per Vesic, the base resistance is not governed by the vertical ground pressure q'_o but by the mean effective normal ground stress σ_m expressed as

$$\sigma_m = \frac{1 + 2K_o}{3} q'_o$$

in which K_o = coefficient of earth pressure for the at rest condition = $1 - \sin \phi$.

$$q_b = cN_c^* + \sigma_m N_\sigma^*$$

$$N_\sigma^* = \frac{3N_q^*}{1 + 2K_o}$$

Vesic has developed an expression for N_{σ}^* based on the ultimate pressure needed to expand a spherical cavity in an infinite soil mass as

$$N_{\sigma}^* = \alpha_1 e^{\alpha_2} N_{\phi} (I_{rr})^{\alpha_3}$$

where $\alpha_1 = \frac{3}{3 - \sin \phi}$, $\alpha_2 = \left(\frac{\pi}{2} - \phi \right) \tan \phi$, $\alpha_3 = \frac{1.33 \sin \phi}{(1 + \sin \phi)}$, and $N_{\phi} = \tan^2(45^\circ + \phi / 2)$

According to Vesic

$$I_{rr} = \frac{I_r}{1 + I_r \Delta}$$

where $I_r = \text{rigidity index} = \frac{E_s}{2(1 + \mu)(c + q'_o \tan \phi)} = \frac{G}{(c + q'_o \tan \phi)}$

where

- I_{rr} = reduced rigidity index for the soil
- Δ = average volumetric strain in the plastic zone below the pile point
- E_s = modulus of elasticity of soil
- G = shear modulus of soil
- μ = Poisson's ratio of soil

The values of rigidity index can be computed knowing the values of shear modulus G_s and the shear strength $s (= c + q'_o \tan \phi)$.

When an undrained condition exists in the saturated clay soil or the soil is cohesionless and is in a dense state we have $\Delta = 0$ and in such a case $I_r = I_{rr}$.

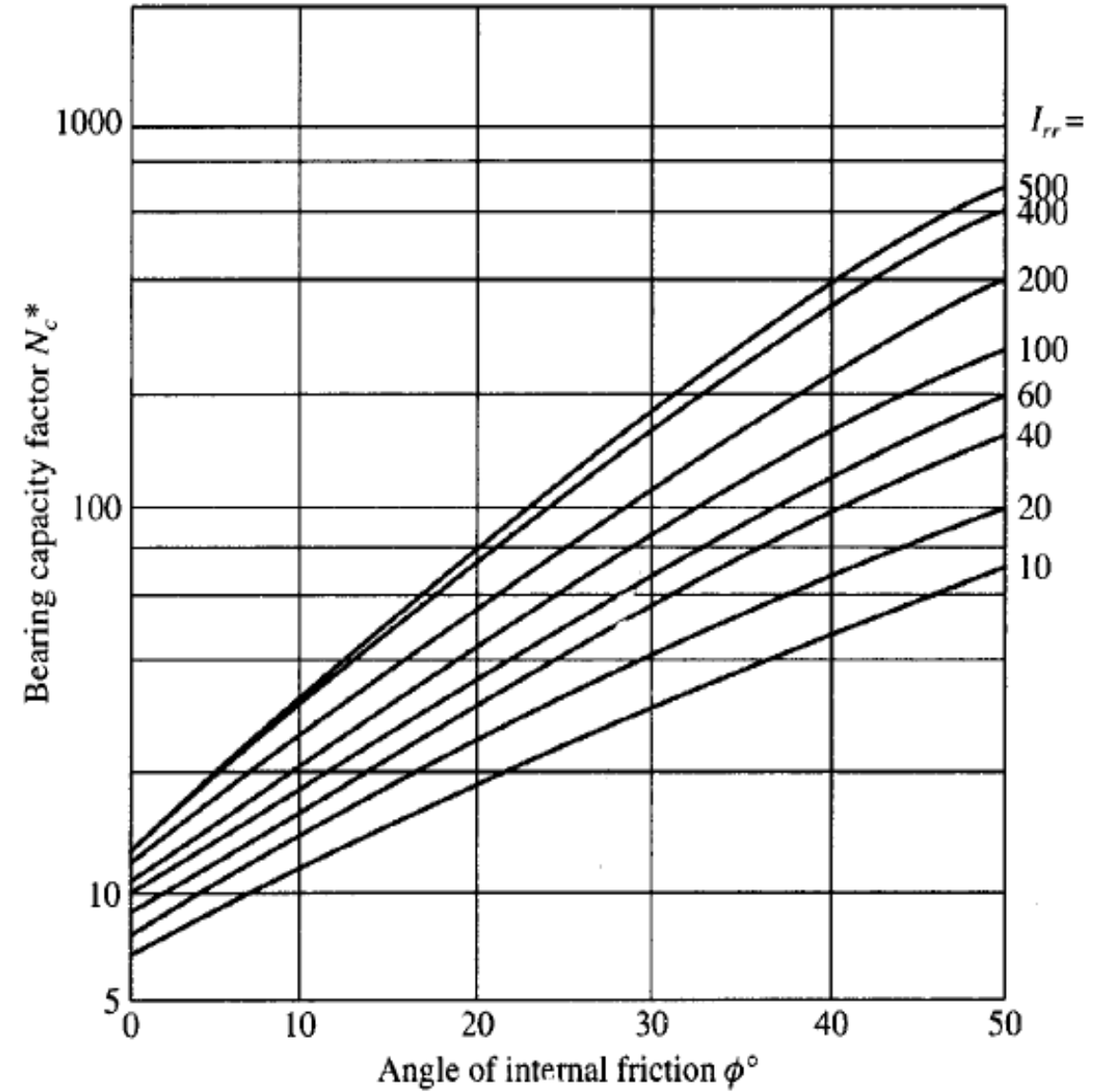
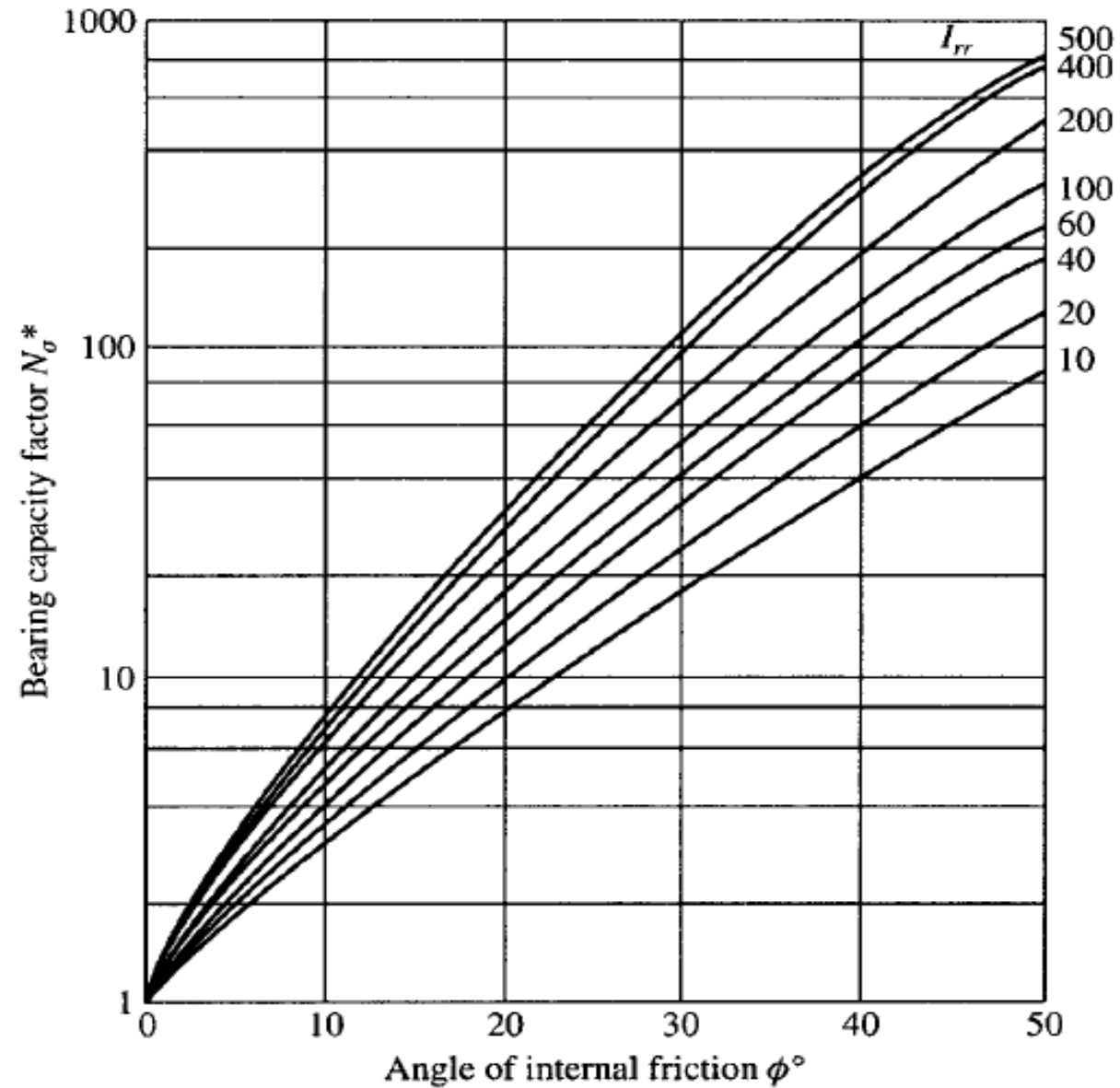
For $\phi = 0$ (undrained condition), we have

$$N_c^* = 1.33(\ln I_{rr} + 1) + \frac{\pi}{2} + 1$$

The value of I_r depends upon the soil state, (a) for sand, loose or dense and (b) for clay low, medium or high plasticity. For preliminary estimates the following values of I_r may be used.

Soil type	I_r
Sand ($D_r = 0.5-0.8$)	75-150
Silt	50-75
Clay	150-250

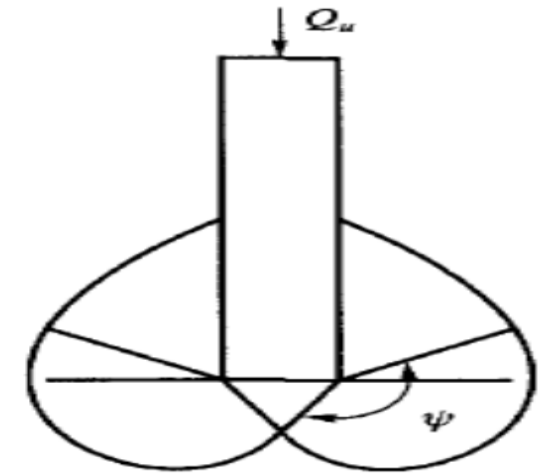
Bearing capacity factors



JANBU'S METHOD OF DETERMINING Q_b

The bearing capacity equation of Janbu (1976) is

$$Q_b = (cN_c^* + q'_o N_q^*) A_b$$



$$N_q^* = \left[\tan \phi + \sqrt{1 + \tan^2 \phi} \right]^2 e^{2\psi \tan \phi}$$

where ψ = angle as shown in Fig. This angle varies from 60° in soft compressible soil to 105° in dense sand. The values for N_c^* used by Janbu are the same as those given by Vesic. Table gives the bearing capacity factors of Janbu.

Bearing capacity factors N_q^* and N_c^* by Janbu

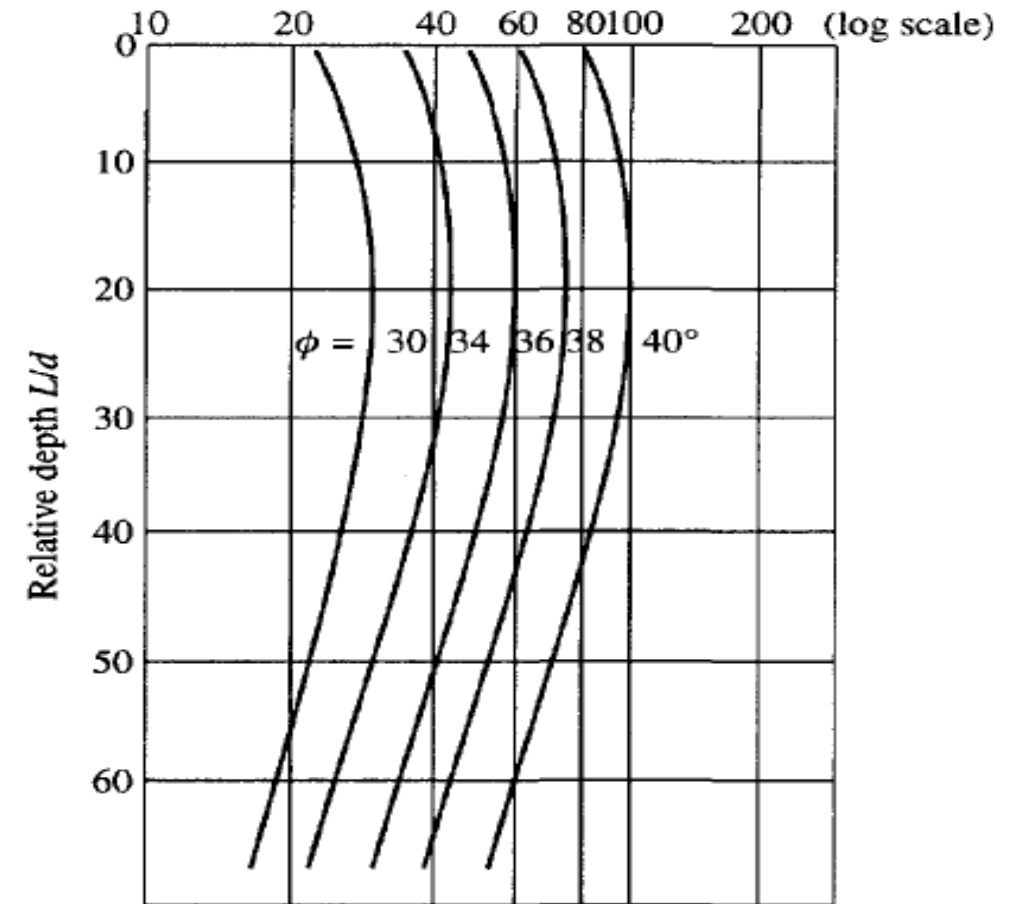
$\psi =$	75°		90°		105°	
ϕ°	N_q^*	N_c^*	N_q^*	N_c^*	N_q^*	N_c^*
0	1.00	5.74	1.00	5.74	1.00	5.74
5	1.50	6.25	1.57	6.49	1.64	7.33
10	2.25	7.11	2.47	8.34	2.71	9.70
20	5.29	11.78	6.40	14.83	7.74	18.53
30	13.60	21.82	18.40	30.14	24.90	41.39
35	23.08	31.53	33.30	46.12	48.04	67.18
40	41.37	48.11	64.20	75.31	99.61	117.52
45	79.90	78.90	134.87	133.87	227.68	226.68

COYLE AND CASTELLO'S METHOD OF ESTIMATING Q_b IN SAND

Coyle and Castello (1981) made use of the results of 24 full scale pile load tests driven in sand for evaluating the bearing capacity factors which may be expressed as

$$Q_b = q'_o N_q^* A_b$$

where q'_o = effective overburden pressure at the base level of the pile
 N_q^* = bearing capacity factor



N_q^* versus L/d (after Coyle and Castello, 1981)

THE ULTIMATE SKIN RESISTANCE OF A SINGLE PILE IN COHESIONLESS SOIL

Skin Resistance (Straight Shaft)

The ultimate skin resistance in a homogeneous soil .

$$Q_f = A_s \bar{q}'_o \bar{K}_s \tan \delta$$

In a layered system of soil $\bar{q}'_o$, $\bar{K}_s$ and δ vary with respect to depth. Equation then be expressed as

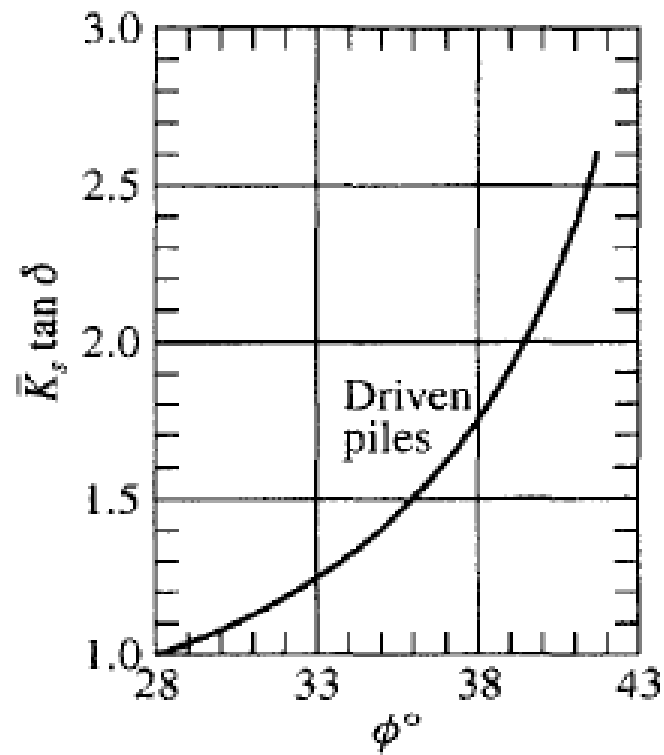
$$Q_f = \int_0^L P \bar{q}'_o \bar{K}_s \tan \delta dz$$

where $\bar{q}'_o$, $\bar{K}_s$ and δ refer to thickness dz of each layer and P is the perimeter of the pile.

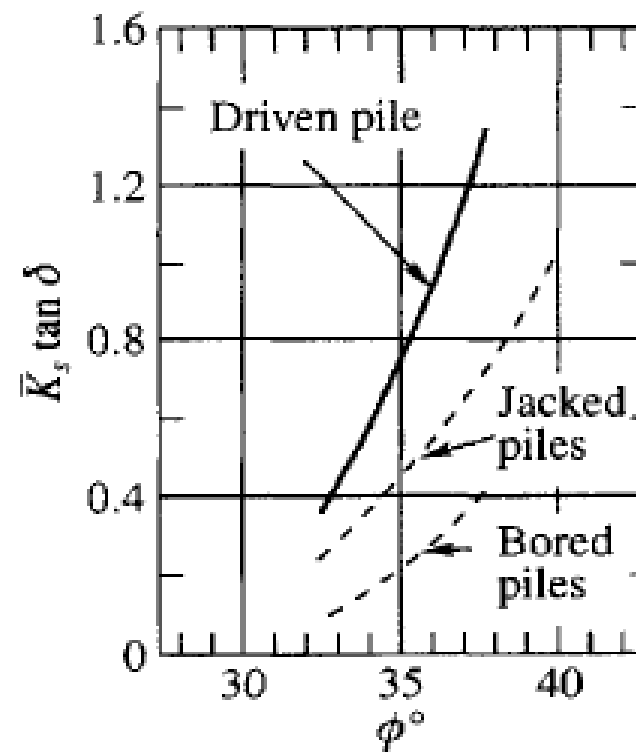
the effective overburden pressure does not increase linearly with depth and reaches a constant value beyond a particular depth L_c , called the critical depth which is a function of ϕ . It is therefore natural to expect the skin resistance f_s also to remain constant beyond depth L_c . The magnitude of L_c may be taken as equal to $20d$.

Values of $\bar{K}_s$ and δ (Broms, 1966)

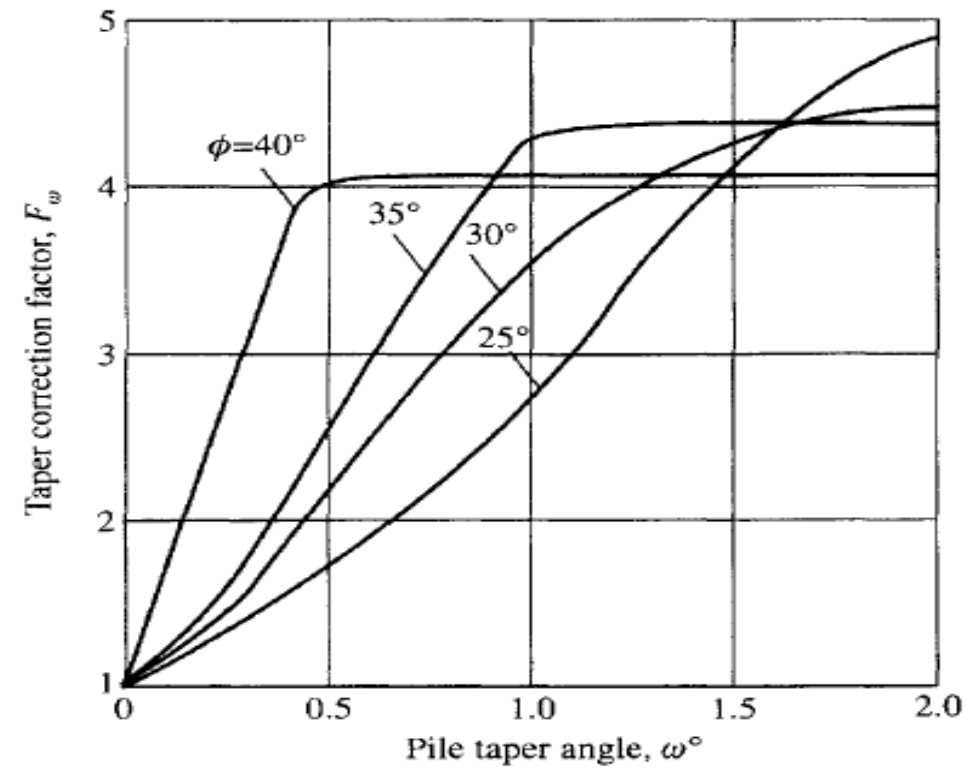
Pile material	δ	Values of $\bar{K}_s$	
		Low D_r	High D_r
Steel	20°	0.5	1.0
Concrete	$3/4\phi$	1.0	2.0
Wood	$2/3\phi$	1.5	4.0



(a)



(b)



(c)

Skin Resistance on Tapered Piles

Nordlund (1963) has shown that even a small taper of 1° on the shaft gives a four fold increase in unit friction in medium dense sand under compression loading. Based on Nordlund's analysis, curves have been developed (Poulos and Davis, 1980) giving a relationship between taper angle ω° and a taper correction factor F_ω , which can be used in Eq.

$$Q_f = \int_0^L F_\omega P \bar{q}'_o \beta dz$$

SKIN RESISTANCE Q_f BY COYLE AND CASTELLO METHOD (1981)

For evaluating frictional resistance, Q_f for piles in sand, Coyle and Castello (1981) made use of the results obtained from 24 field tests on piles.

They developed a chart giving relationships between $\bar{K}_s$ and ϕ for various L/d ratios. The angle of wall friction δ is assumed equal to 0.8ϕ . The expression for Q_f is

$$Q_f = A_s \bar{q}'_o \bar{K}_s \tan \delta$$

where $\bar{q}'_o$ = average effective overburden pressure and δ = angle of wall friction = 0.8ϕ .

STATIC BEARING CAPACITY OF PILES IN CLAY SOIL

Equation for Ultimate Bearing Capacity

The static ultimate bearing capacity of piles in clay

$$Q_u = Q_b + Q_f = c_b N_c A_b + \alpha \bar{c}_u A_s$$

For layered clay soils where the cohesive strength varies along the shaft, written as

$$Q_u = c_b N_c A_b + \int_0^L \alpha \bar{c}_u A_s$$

Bearing Capacity Factor N_c

The value of the bearing capacity factor N_c that is generally accepted is 9 which is the value proposed by Skempton (1951) for circular foundations for a L/B ratio greater than 4. The base capacity of a pile in clay soil may now be expressed as

$$Q_b = 9c_b A_b$$

Skin Resistance by α -Method

Tomlinson (1986) has given some empirical correlations for evaluating α for different types of soil conditions and L/d ratios.

$$Q_u = c_b N_c A_b + \int_0^L \alpha \bar{c}_u A_s$$

Skin Resistance by λ -Method

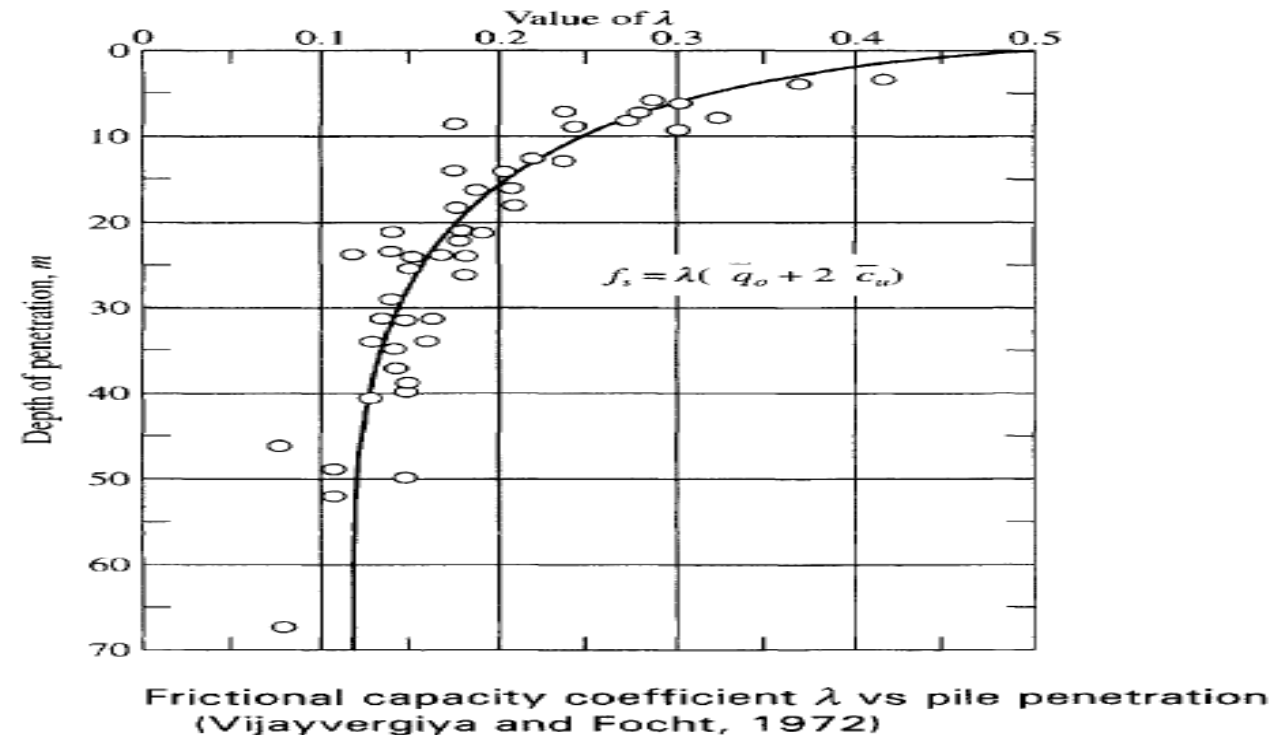
Vijayvergiya and Focht (1972) have suggested a different approach for computing skin load Q_f for steel-pipe piles on the basis of examination of load test results on such piles. The equation is of the form

$$Q_f = \lambda(\bar{q}'_o + 2\bar{c}_u)A_s$$

where λ = frictional capacity coefficient,

$\bar{q}'_o$ = mean effective vertical stress between the ground surface and pile tip.

The other terms are already defined. λ is plotted against pile penetration as shown in Fig.



β -Method or the Effective Stress Method of Computing Skin Resistance

In this method, the unit skin friction f_s is defined as

$$f_s = \bar{K}_s \tan \delta \bar{q}'_o = \beta \bar{q}'_o$$

where $\beta = \text{the skin factor} = \bar{K}_s \tan \delta$,
 $\bar{K}_s = \text{lateral earth pressure coefficient,}$

$\delta = \text{angle of wall friction,}$

$\bar{q}'_o = \text{average effective overburden pressure.}$

Burland (1973) discusses the values to be used for β and demonstrates that a lower limit for this factor for normally consolidated clay can be written as

$$\beta_o = \bar{K}_o \tan \phi'$$

As per Jaky (1944)

$$K_o = (1 - \sin \phi')$$

therefore $\beta = (1 - \sin \phi') \tan \phi'$

where $\phi' = \text{effective angle of internal friction.}$

Since the concept of this method is based on effective stresses, the cohesion intercept on a Mohr circle is equal to zero. For driven piles in stiff overconsolidated clay, $\bar{K}_s$ is roughly 1.5 times greater than $\bar{K}_o$. For overconsolidated clays $\bar{K}_o$ may be found from the expression

$$\bar{K}_o = (1 - \sin \phi') \sqrt{R_{oc}}$$

where $R_{oc} = \text{overconsolidation ratio of clay.}$

For clays, ϕ' may be taken in the range of 20 to 30 degrees. In such a case the value of β varies between 0.24 and 0.29.

Meyerhof's Method (1976)

Meyerhof has suggested a semi-empirical relationship for estimating skin friction in clays.

For driven piles:

$$f_s = 1.5c_u \tan \phi'$$

For bored piles:

$$f_s = c_u \tan \phi'$$

By utilizing a value of 20° for ϕ' for the stiff to very stiff clays, the expressions reduce to

For driven piles:

$$f_s = 0.55c_u$$

For bored piles:

$$f_s = 0.36c_u$$

In practice the maximum value of unit friction for bored piles is restricted to 100 kPa.

BEARING CAPACITY OF PILES IN GRANULAR SOILS BASED ON SPT VALUE

Meyerhof (1956) suggests the following equations for single piles in granular soils based on SPT values.

For displacement piles:

$$Q_u = Q_b + Q_f = 40 N_{cor} (L/d) A_b + 2 \bar{N}_{cor} A_s$$

for *H*-piles:

$$Q_u = 40 N_{cor} (L/d) A_b + \bar{N}_{cor} A_s$$

where $q_b = 40 N_{cor} (L/d) \leq 400 N_{cor}$

For bored piles:

$$Q_u = 133 N_{cor} A_b + 0.67 \bar{N}_{cor} A_s$$

where Q_u = ultimate total load in kN

N_{cor} = average corrected SPT value below pile tip

$\bar{N}_{cor}$ = corrected average SPT value along the pile shaft

A_b = base area of pile in m² (for *H*-piles including the soil between the flanges)

A_s = shaft surface area in m²

In English units Q_u for a displacement pile is

$$Q_u \text{ (kip)} = Q_b + Q_f = 0.80 N_{cor} (L/d) A_b + 0.4 \bar{N}_{cor} A_s \text{ and } 0.80 N_{cor} \left(\frac{L}{d}\right) A_b \leq 8 N_{cor} A_b \text{ (kip)}$$

where A_b = base area in ft² and A_s = surface area in ft²

A minimum factor of safety of 4 is recommended. The allowable load Q_a is

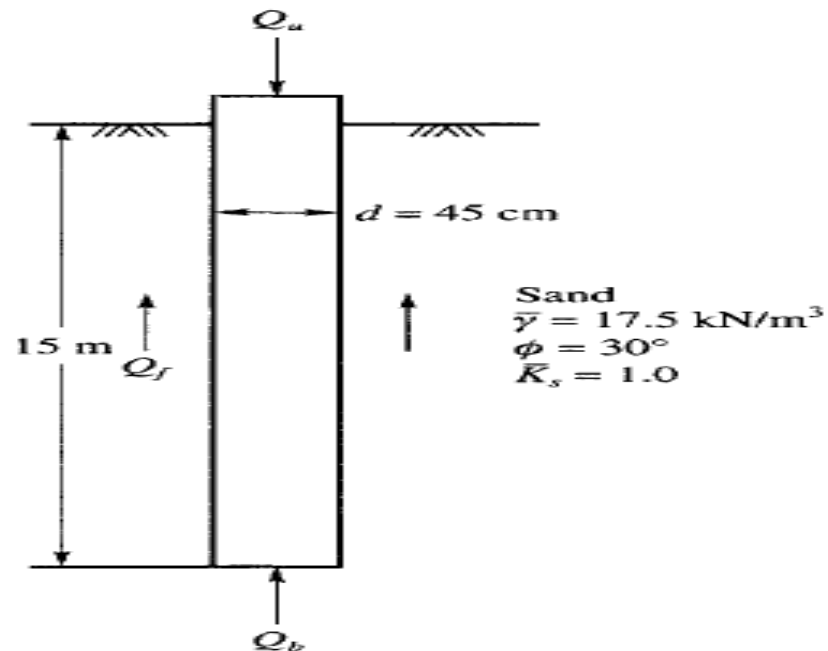
$$Q_a = \frac{Q_u}{4}$$

NUMERICAL PROBLEMS AND DESIGN PROBLEMS

A concrete pile of 45 cm diameter was driven into sand of loose to medium density to a depth of 15 m. The following properties are known:

- (a) Average unit weight of soil along the length of the pile, $\bar{\gamma} = 17.5 \text{ kN/m}^3$, average $\phi = 30^\circ$,
- (b) average $\bar{K}_s = 1.0$ and $\delta = 0.75\phi$.

Calculate (a) the ultimate bearing capacity of the pile, and (b) the allowable load with $F_s = 2.5$. Assume the water table is at great depth. Solve the problem by Meyerhof's method.



Solution

Point bearing capacity

$$q_b = q'_o N_q \leq q_{bl}$$

$$q_{bl} = 25 N_q \tan \phi \text{ kN/m}^2$$

Now From Fig. $N_q = 60$ for $\phi = 30^\circ$

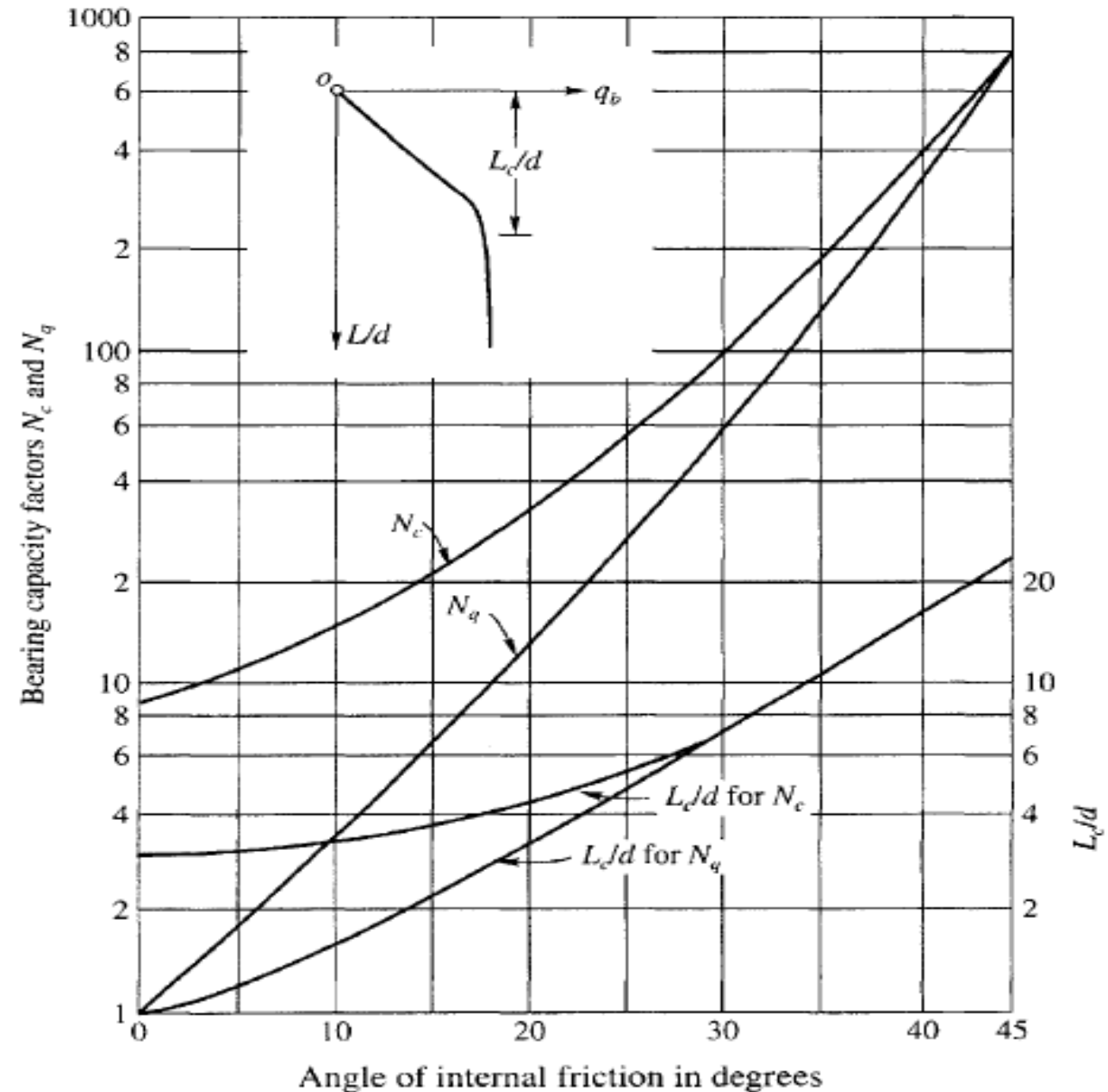
$$q'_o = \gamma L = 17.5 \times 15 = 262.5 \text{ kN/m}^2$$

$$q_b = 262.5 \times 60 = 15,750 \text{ kN/m}^2$$

$$q_{bl} = 25 \times 60 \times \tan 30^\circ = 866 \text{ kN/m}^2$$

Hence the limiting value for $q_b = 866 \text{ kN/m}^2$

$$\text{Now, } Q_b = A_b q_b = A_b q_b = \frac{3.14}{4} \times (0.45)^2 \times 866 = 138 \text{ kN}$$



Example

A concrete pile of 45 cm diameter was driven into sand of loose to medium density to a depth of 15 m. The following properties are known:

- (a) Average unit weight of soil along the length of the pile, $\bar{\gamma} = 17.5 \text{ kN/m}^3$, average $\phi = 30^\circ$
(b) average $\bar{K}_s = 1.0$ and $\delta = 0.75\phi$.

Estimate Q_b , Q_f , Q_u and Q_a by the Coyle and Castello method

Solution

Base load Q_b

$$q_b = q'_o N_q^*$$

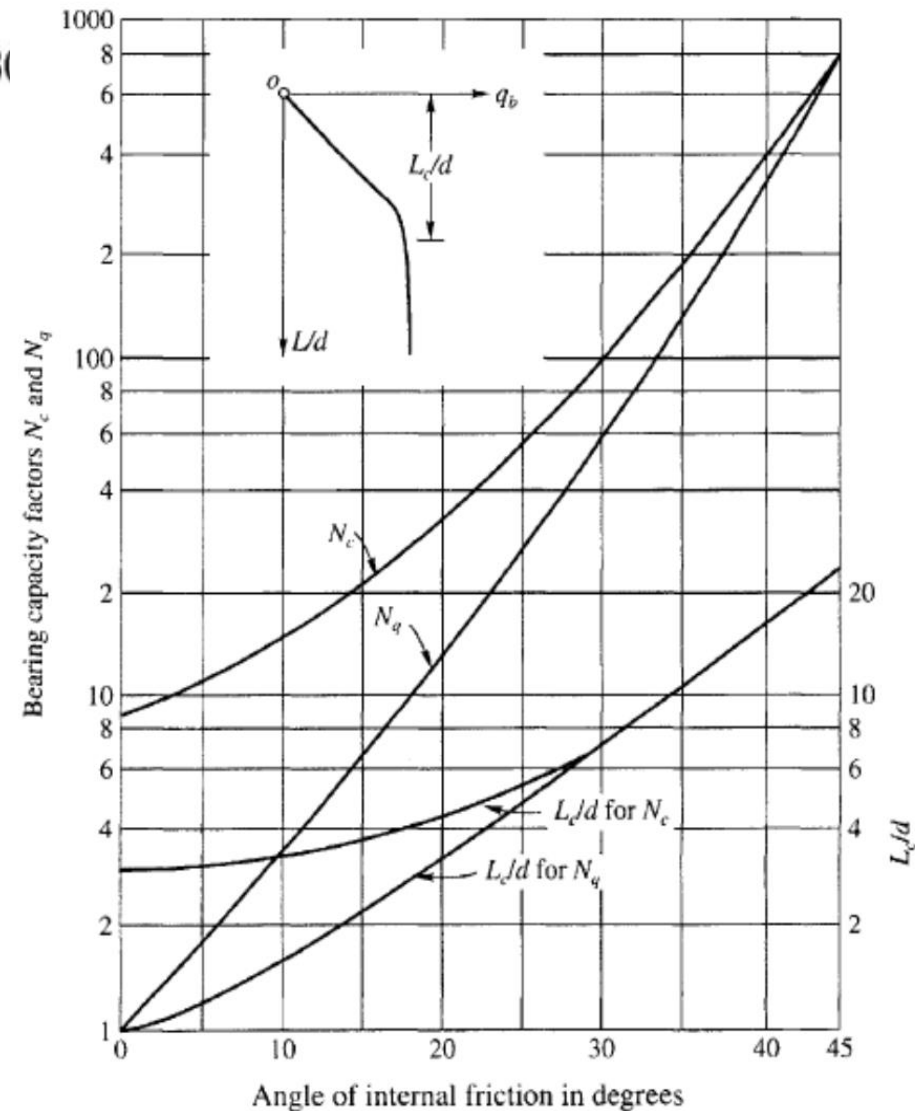
From Fig. $N_q^* = 29$ for $\phi = 30^\circ$ and $L/d = 33.3$

$$q'_o = 262.5 \text{ kN/m}^2$$

Therefore $q_b = 262.5 \times 29 = 7612 \text{ kN/m}^2$

$$Q_b = A_b q_b = 0.159 \times 7612 = 1210 \text{ kN}$$

$$Q_f = A_s \bar{q}'_o \bar{K}_s \tan \delta$$



Frictional resistance

the unit skin resistance f_s is assumed to increase from 0 at ground level to a limiting value of f_{sl} at $L_c = 20 d$ where L_c = critical depth and d = diameter. Therefore $L_c = 20 \times 0.45 = 9 \text{ m}$

$$\text{Now } f_{sl} = q'_o \bar{K}_s \tan \delta = \gamma L_c \bar{K}_s \tan \delta$$

$$\text{Given: } \gamma = 17.5 \text{ kN/m}^3, L_c = 9 \text{ m}, \bar{K}_s = 1.0 \text{ and } \delta = 22.5^\circ \text{ m.}$$

Substituting and simplifying we have

$$f_{sl} = 17.5 \times 9 \times 1.0 \times \tan 22.5 = 65 \text{ kN/m}^2$$

$$\text{The skin load } Q_f = Q_{f1} + Q_{f2} = \frac{1}{2} f_{sl} P L_c + P f_{sl} (L - L_c)$$

$$\begin{aligned} \text{Substituting } Q_f &= \frac{1}{2} \times 65 \times 3.14 \times 0.45 \times 9 + 3.14 \times 0.45 \times 65 (15 - 9) \\ &= 413 + 551 = 964 \text{ kN} \end{aligned}$$

The failure load Q_u is

$$Q_u = Q_b + Q_f = 138 + 964 = 1,102 \text{ kN}$$

with $F_s = 2.5$,

$$Q_a = \frac{1102}{2.5} = 440 \text{ kN}$$

Example

Determine the base load of the problem
Determine Q_a for $F_s = 2.5$ using the Q_f estimated

by Janbu's method. Use $\psi = 90^\circ$.

Solution

$$q'_0 = \gamma L = 17.5 \times 15 = 262.5 \text{ kN/m}^2$$

$$q_b = q'_0 N_q^*$$

For $\phi = 30^\circ$ and $\psi = 90^\circ$, we have $N_q^* = 18.4$

$$q'_0 = 262.5 \text{ kN/m}^2 \text{ as in}$$

Therefore $q_b = 262.5 \times 18.4 = 4830 \text{ kN/m}^2$

$$Q_b = A_b q_b = 0.159 \times 4830 = 768 \text{ kN}$$

$$Q_u = Q_b + Q_f = 768 + 964 = 1732 \text{ kN}$$

$$Q_a = \frac{1732}{2.5} = 693 \text{ kN}$$

Bearing capacity factors N_q^* and N_c^* by Janbu

$\psi =$	75°		90°		105°	
ϕ°	N_q^*	N_c^*	N_q^*	N_c^*	N_q^*	N_c^*
0	1.00	5.74	1.00	5.74	1.00	5.74
5	1.50	6.25	1.57	6.49	1.64	7.33
10	2.25	7.11	2.47	8.34	2.71	9.70
20	5.29	11.78	6.40	14.83	7.74	18.53
30	13.60	21.82	18.40	30.14	24.90	41.39
35	23.08	31.53	33.30	46.12	48.04	67.18
40	41.37	48.11	64.20	75.31	99.61	117.52
45	79.90	78.90	134.87	133.87	227.68	226.68

Example

A precast concrete pile of size 18×18 in is driven into stiff clay. The unconfined compressive strength of the clay is 4.2 kips/ft^2 . Determine the length of pile required to carry a safe working load of 90 kips with $F_s = 2.5$.

Solution

The equation for Q_u is

$$Q_u = N_c c_u A_b + \alpha \bar{c}_u A_s$$

we have

$$Q_u = 2.5 \times 90 = 225 \text{ kips,}$$

$$N_c = 9, c_u = 2.1 \text{ kips/ft}^2$$

$$\alpha = 0.48 \text{ from Fig.} \quad \bar{c}_u = c_u = 2.1 \text{ kips/ft}^2, A_b = 2.25 \text{ ft}^2$$

Assume the length of pile = L ft

$$\text{Now, } A_s = 4 \times 1.5L = 6L$$

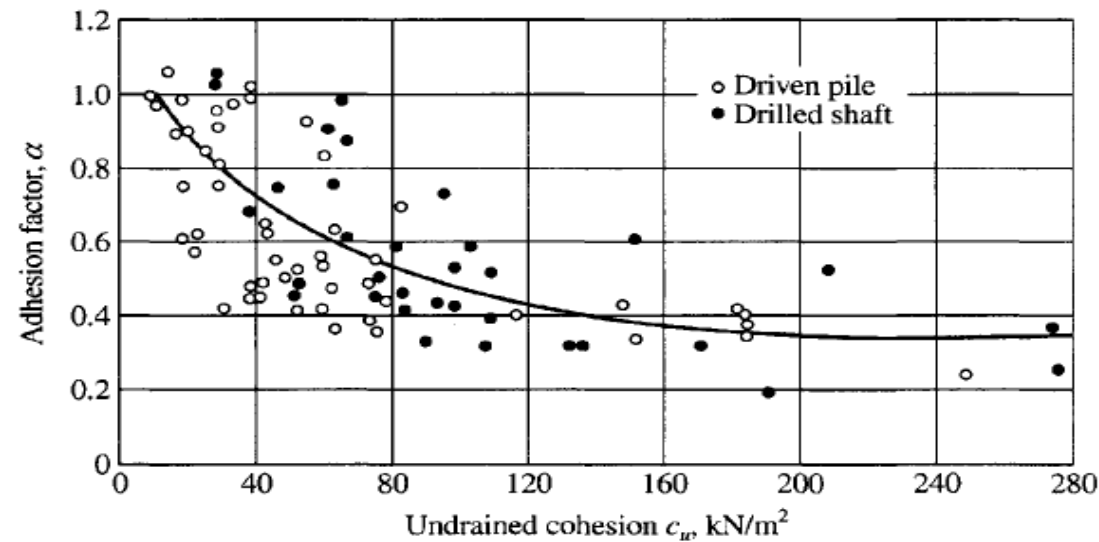
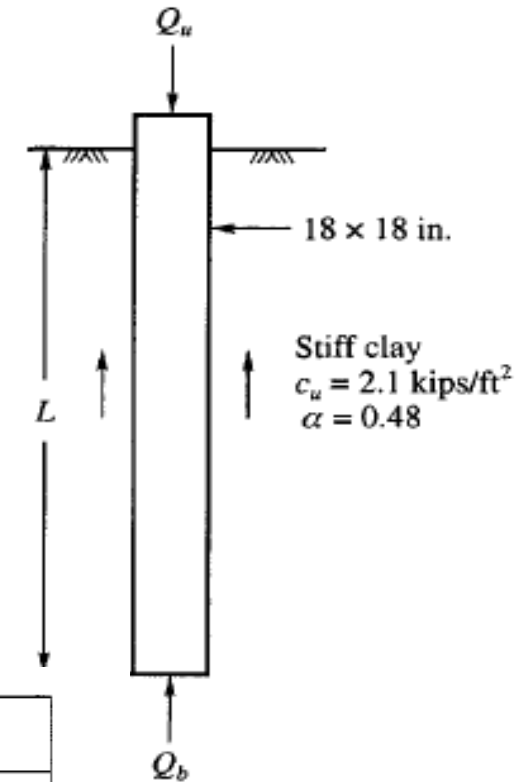
Substituting the known values, we have

$$225 = 9 \times 2.1 \times 2.25 + 0.48 \times 2.1 \times 6L$$

$$\text{or } 225 = 42.525 + 6.05L$$

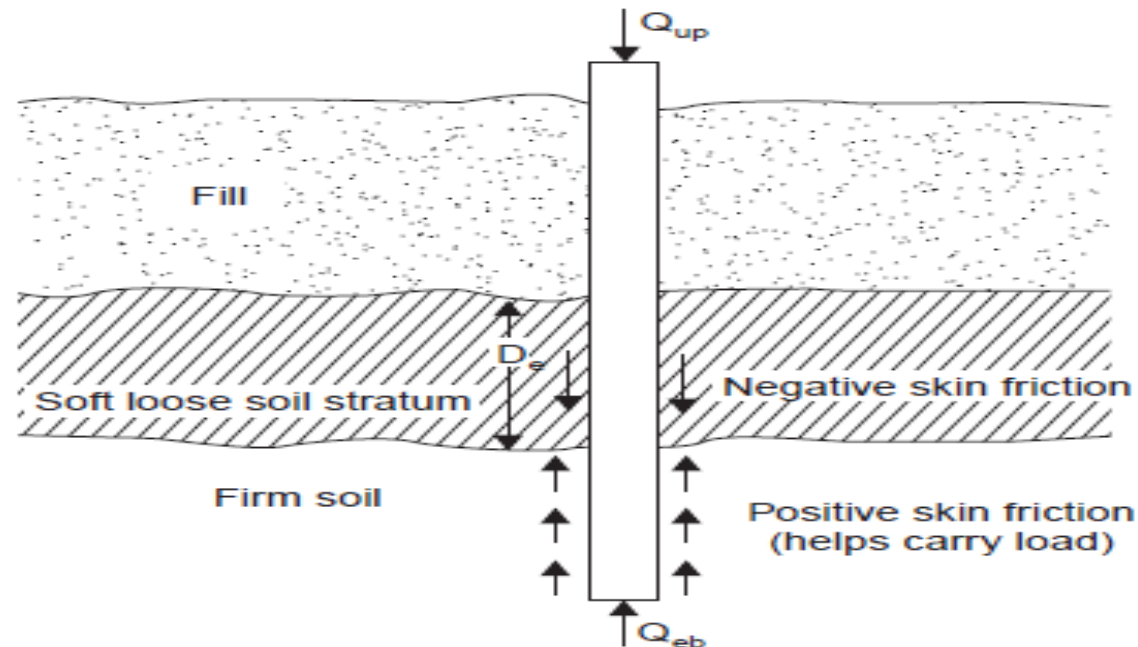
Simplifying, we have

$$L = \frac{225 - 42.525}{6.05} = 30.2 \text{ ft}$$



NEGATIVE SKIN FRICTION

- Negative skin friction' or 'down drag' is a phenomenon which occurs when a soil layer surrounding a portion of the pile shaft settles more than the pile.
- This condition can develop where a soft or loose soil stratum located anywhere above the pile tip is subjected to new compressive loading. If a soft or loose layer settles after the pile has been installed, the skin-friction-adhesion developing in this zone is in the direction of the soil movement, pulling the pile downward.



- Negative skin friction may also occur by the lowering of ground water which increases the effective stress inducing consolidation and consequent settlement of the soil surrounding the pile.
- It is necessary to subtract negative skin friction force from the total load that the pile can support. In such a case the factor of safety will be modified as follows:
- Values of negative skin force are computed in just the same way as positive skin friction.

$$\text{Factor of safety} = \frac{\text{Ultimate pile load capacity}}{\text{Working load} + \text{Negative skin friction force}}$$

Sometimes this may also be written as

$$\text{Factor of safety} = \frac{\text{Ultimate pile load capacity} - \text{Negative skin friction force}}{\text{Working load}}$$

Computation of Negative Friction on a Single Pile

The magnitude of negative friction F_n for a single pile in a fill may be taken as

(a) For cohesive soils

$$F_n = PL_n s.$$

(b) For cohesionless soils

$$F_n = \frac{1}{2} PL_n^2 \gamma K \tan \delta$$

where L_n = length of piles in the compressible material,

s = shear strength of cohesive soils in the fill,

P = perimeter of pile,

K = earth pressure coefficient normally lies between the active and the passive earth pressure coefficients,

δ = angle of wall friction which may vary from $\phi/2$ to ϕ .

Negative Friction on Pile Groups

When a group of piles passes through a compressible fill, the negative friction, F_{ng} , on the group may be found by any of the following methods [Fig. 15.32b].

$$(a) \quad F_{ng} = nF_n$$

$$(b) \quad F_{ng} = sL_nP_g + \gamma L_nA_g$$

where n = number of piles in the group,

γ = unit weight of soil within the pile group to a depth L_n ,

P_g = perimeter of pile group,

A_g = sectional area of pile group within the perimeter P_g ,

s = shear strength of soil along the perimeter of the group.

$$F_{ng} = n(F_{n1} + F_{n2})$$

$$\begin{aligned} F_{ng} &= s_1L_1P_g + s_2L_2P_g + \gamma_1L_1A_g + \gamma_2L_2A_g \\ &= P_g(s_1L_1 + s_2L_2) + A_g(\gamma_1L_1 + \gamma_2L_2) \end{aligned}$$

where L_1 = depth of fill,

L_2 = depth of compressible natural soil,

s_1, s_2 = shear strengths of the fill and compressible soils respectively,

γ_1, γ_2 = unit weights of fill and compressible soils respectively,

F_{n1} = negative friction of a single pile in the fill,

F_{n2} = negative friction of a single pile in the compressible soil.

Example

A square pile group similar to the one shown in Fig. passes through a recently constructed fill. The depth of fill $L_n = 3$ m. The diameter of the pile is 30 cm and the piles are spaced 90 cm center to center. If the soil is cohesive with $q_u = 60$ kN/m², and $\gamma = 15$ kN/m³, compute the negative frictional load on the pile group.

Solution

The negative frictional load on the group is the maximum of

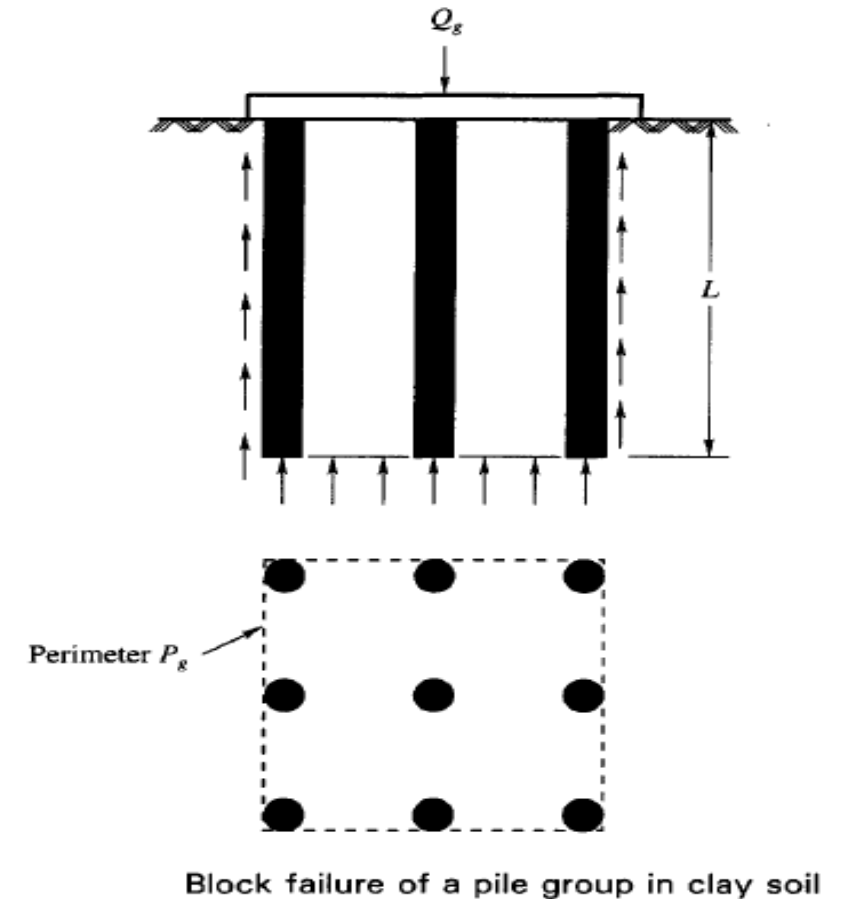
$$(a) \quad F_{ng} = nF_n, \quad \text{and} \quad (b) \quad F_{ng} = sL_n P_g + \gamma L_n A_g,$$

where $P_g = 4 \times 3 = 12$ m, $A_g = 3 \times 3 = 9$ m², $c_u = 60/2 = 30$ kN/m²

$$(a) \quad F_{ng} = 9 \times 3.14 \times 0.3 \times 3 \times 30 = 763 \text{ kN}$$

$$(b) \quad F_{ng} = 30 \times 3 \times 12 + 15 \times 3 \times 9 = 1485 \text{ kN}$$

The negative frictional load on the group = 1485 kN.

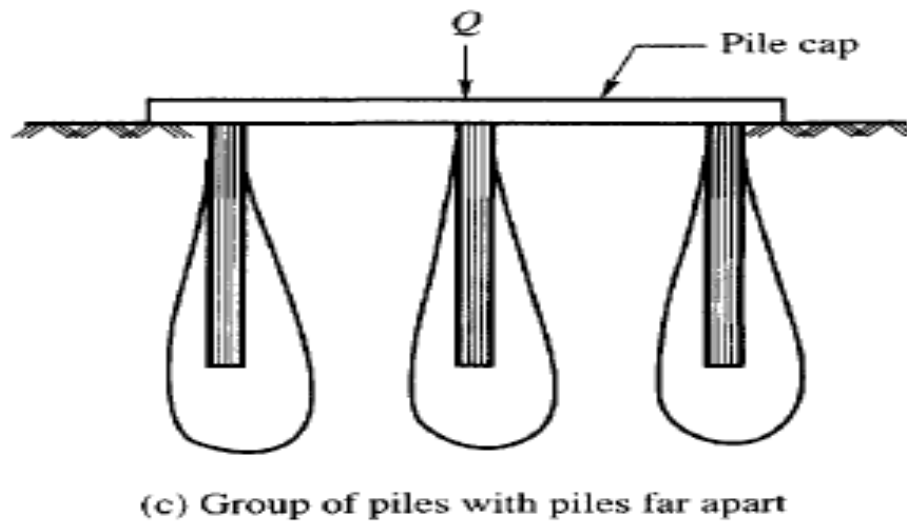
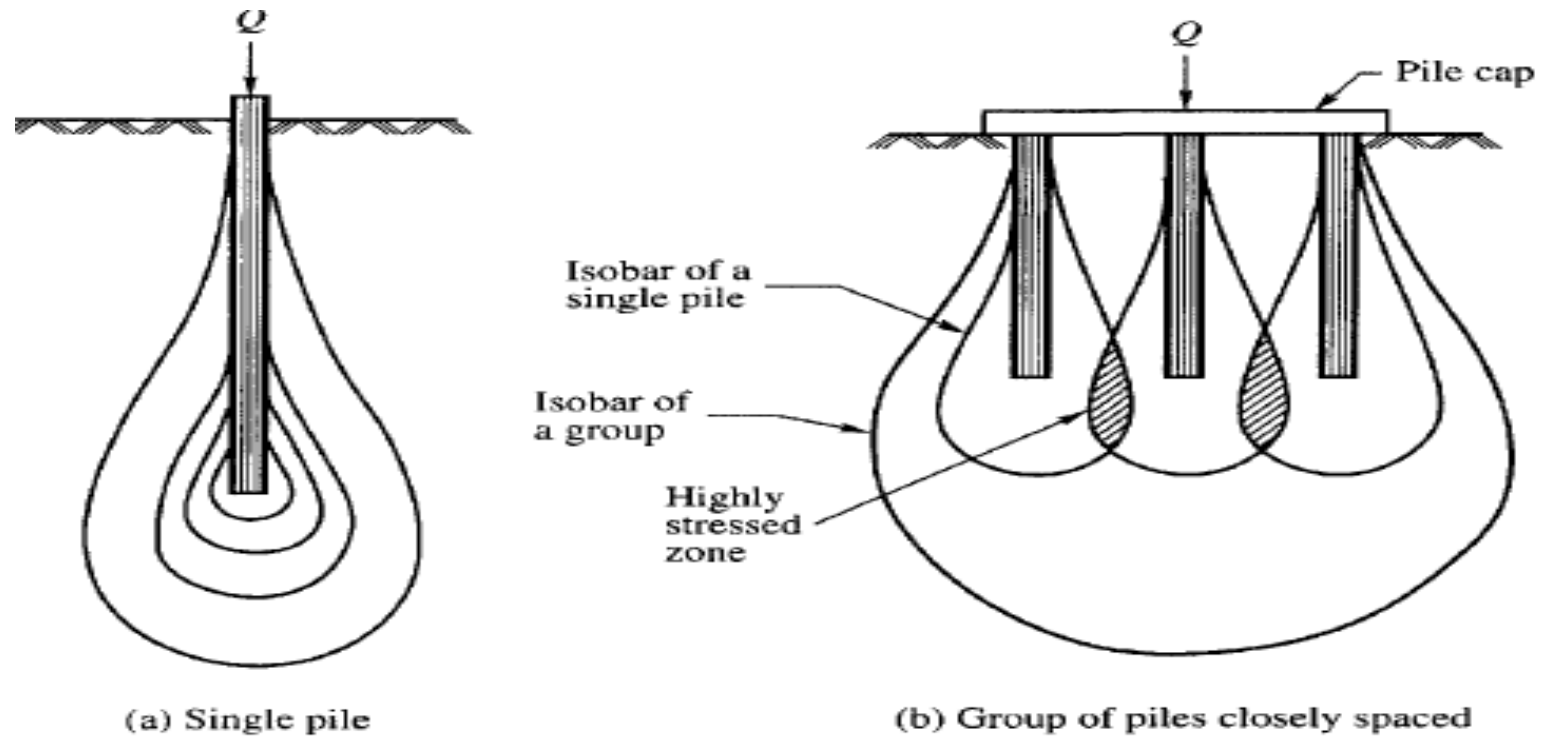


PILE GROUP

NUMBER AND SPACING OF PILES IN A GROUP

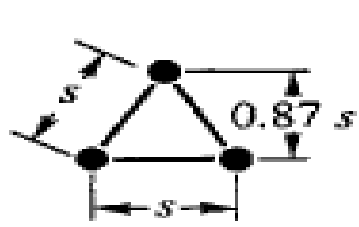
- Very rarely structures are founded on single piles. Normally, there will be a minimum of three piles under a column or a foundation element because of alignment problems.
- The spacing of piles in a group depends upon many factors such as.
 1. Overlapping of stresses of adjacent piles
 2. Cost of foundation
 3. Efficiency of the pile group

PILE GROUP



- The pressure isobars of a single pile with load Q acting on the top are shown in Fig. (a).
- When piles are placed in a group, there is a possibility the pressure isobars of adjacent piles will overlap each other as shown in Fig.(b).
- The soil is highly stressed in the zones of overlapping of pressures.
- With sufficient overlap, either the soil will fail or the pile group will settle excessively since the combined pressure bulb extends to a considerable depth below the base of the piles.
- It is possible to avoid overlap by installing the piles further apart as shown in Fig.(c).
- Large spacing's are not recommended sometimes, since this would result in a larger pile cap which would increase the cost of the foundation.

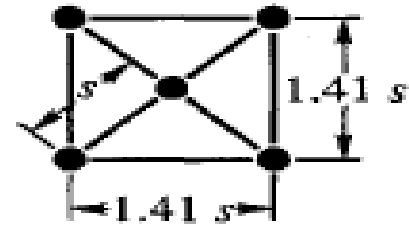
SPACING OF PILES



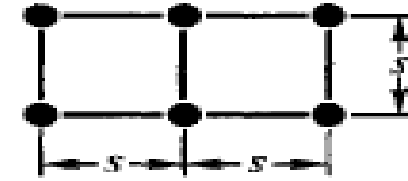
3 pile



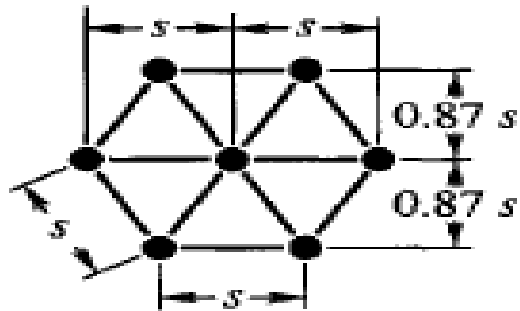
4 pile



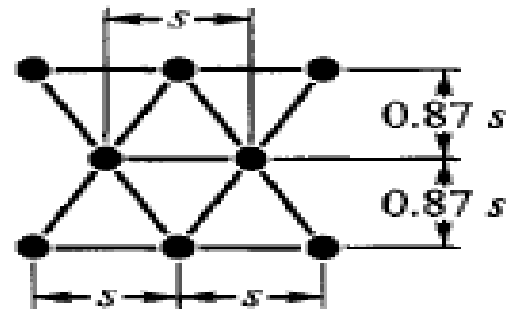
5 pile



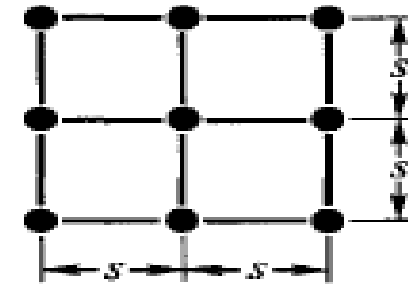
6 pile



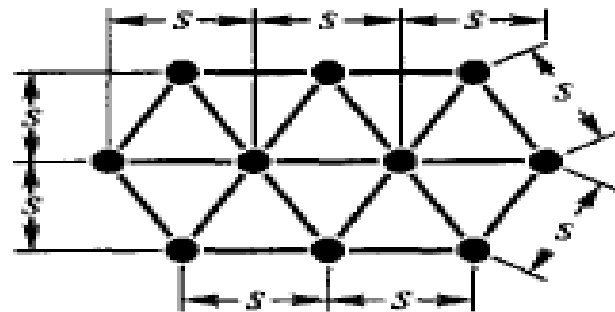
7 pile



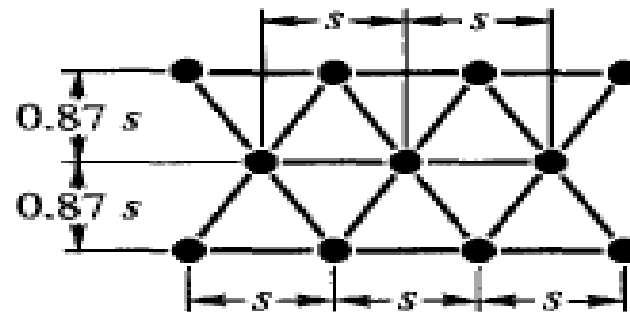
8 pile



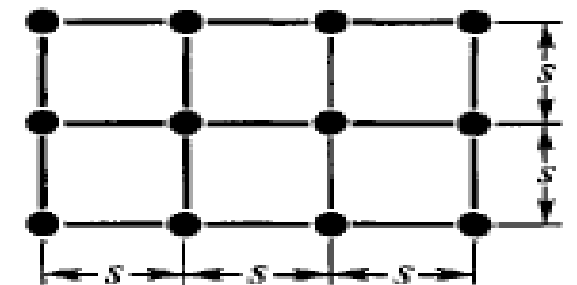
9 pile



10 pile



11 pile



12 pile

SPACING OF PILES

- The spacing of piles depends upon the method of installing the piles and the type of soil.
- The piles can be driven piles or *cast-in-situ* piles. When the piles are driven there will be greater overlapping of stresses due to the displacement of soil.
- If the displacement of soil compacts the soil in between the piles as in the case of loose sandy soils, the piles may be placed at closer intervals. The minimum allowable spacing of piles is usually stipulated in building codes.
- The spacing's for straight uniform diameter piles may vary from 2 to 6 times the diameter of the shaft. For friction piles, the minimum spacing recommended is $3d$ where d is the diameter of the pile. For end bearing piles passing through relatively compressible strata, the spacing of piles shall not be less than $2.5d$. For end bearing piles passing through compressible strata and resting in stiff clay, the spacing may be increased to $3.5d$. For compaction piles, the spacing may be $2d$.

PILE GROUP EFFICIENCY

The spacing of piles is usually predetermined by practical and economical considerations. The design of a pile foundation subjected to vertical loads consists of

1. The determination of the ultimate load bearing capacity of the group Q_{gu} .
2. Determination of the settlement of the group, E_g , under an allowable load Q_{ga} . The ultimate load of the group is generally different from the sum of the ultimate loads of individual piles Q_u .

The pile group factor is given by

$$E_g = \frac{Q_{gu}}{\Sigma Q_u}$$

PILE GROUP EFFICIENCY EQUATION

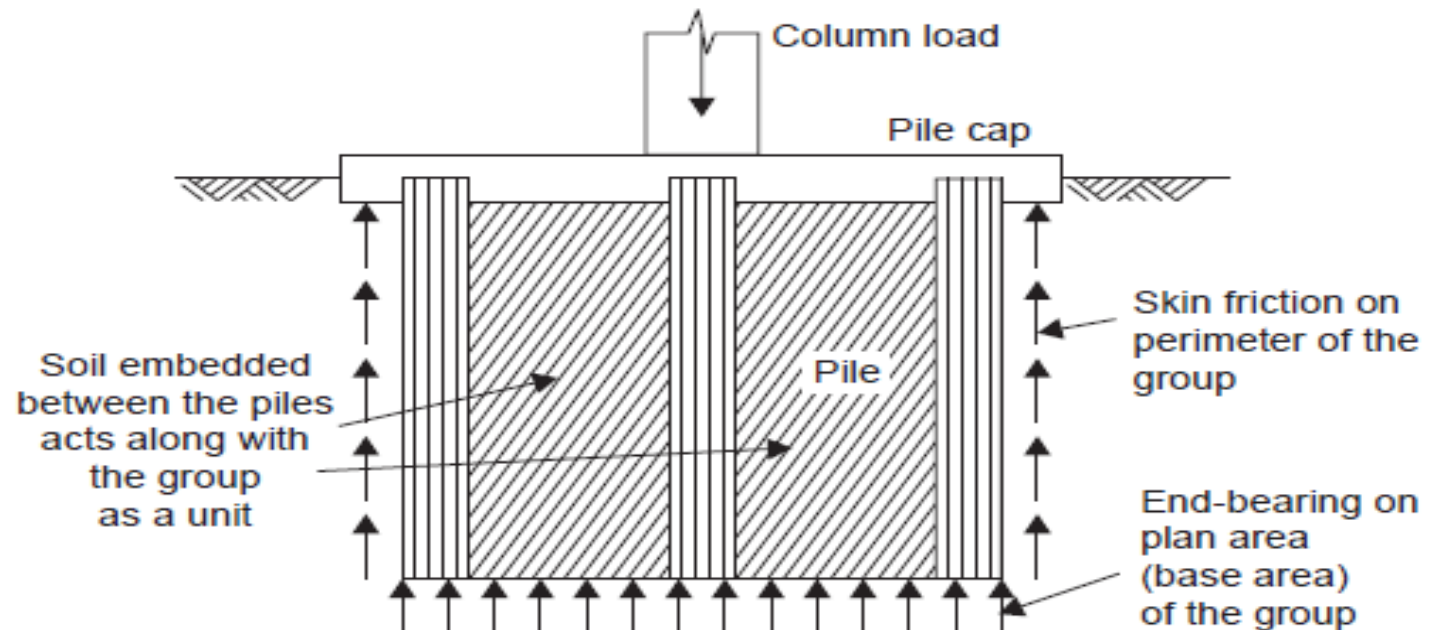
There are many pile group equations. These equations are to be used very cautiously, and may in many cases be no better than a good guess. The Converse Labarre formula is one of the most widely used group-efficiency equations which is expressed as

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

where m = number of columns of piles in a group,
 n = number of rows,
 $\theta = \tan^{-1}(d/s)$ in degrees,
 d = diameter of pile,
 s = spacing of piles center to center.

PILE CAP

- Usually, the load to be supported exceeds the bearing capacity of a single pile, and so a group of similar piles is used.
- The group is capped by a spread footing or a cap to distribute the load to all piles in the group.
- Where there are a large number of closely spaced piles, rather than provide individual caps, it may be more economical to provide just one large cap, thus forming a piled raft.



VERTICAL BEARING CAPACITY OF PILE GROUPS EMBEDDED IN SANDS AND GRAVELS

Driven piles. If piles are driven into loose sands and gravel, the soil around the piles to a radius of at least three times the pile diameter is compacted. When piles are driven in a group at close spacing, the soil around and between them becomes highly compacted. When the group is loaded, the piles and the soil between them move together as a unit. Thus, the pile group acts as a pier foundation having a base area equal to the gross plan area contained by the piles.

Bored Pile Groups In Sand And Gravel

Bored piles are *cast-in-situ* concrete piles. The method of installation involves

- 1 . Boring a hole of the required diameter and depth,
2. Pouring in concrete.

There will always be a general loosening of the soil during boring and then too when the boring has to be done below the water table. Though bentonite slurry (sometimes called as *drilling mud*) is used for stabilizing the sides and bottom of the bores, loosening of the soil cannot be avoided. Cleaning of the bottom of the bore hole prior to concreting is always a problem which will never be achieved quite satisfactorily. Since bored piles do not compact the soil between the piles, the efficiency factor will never be greater than unity.

Pile Groups In Cohesive Soils

The effect of driving piles into cohesive soils (clays and silts) is very different from that of cohesionless soils. It has already been explained that when piles are driven into clay soils, particularly when the soil is soft and sensitive, there will be considerable remoulding of the soil. Besides there will be heaving of the soil between the piles since compaction during driving cannot be achieved in soils of such low permeability. There is every possibility of lifting of the pile during this process of heaving of the soil. Bored piles are, therefore, preferred to driven piles in cohesive soils.

In case driven piles are to be used, the following steps should be favoured:

1. Piles should be spaced at greater distances apart.
 2. Piles should be driven from the centre of the group towards the edges, and
 3. The rate of driving of each pile should be adjusted as to minimize the development of pore water pressure.
- Experimental results have indicated that when a pile group installed in cohesive soils is loaded, it may fail by any one of the following ways:
 1. May fail as a block (called block failure).
 2. Individual piles in the group may fail.

The equation for block failure may be written as

$$Q_{gu} = cN_c A_g + P_g L \bar{c}$$

where c = cohesive strength of clay beneath the pile group,
 $\bar{c}$ = average cohesive strength of clay around the group,
 L = length of pile,
 P_g = perimeter of pile group,
 A_g = sectional area of group,
 N_c = bearing capacity factor which may be assumed as 9 for deep foundations.

The bearing capacity of a pile group on the basis of individual pile failure may be written as

$$Q_{gu} = nQ_u$$

where n = number of piles in the group,
 Q_u = bearing capacity of an individual pile.

Example

A group of 9 piles with 3 piles in a row was driven into a soft clay extending from ground level to a great depth. The diameter and the length of the piles were 30 cm and 10 m respectively. The unconfined compressive strength of the clay is 70 kPa. If the piles were placed 90 cm center to center, compute the allowable load on the pile group on the basis of a shear failure criterion for a factor of safety of 2.5.

Solution

The allowable load on the group is to be calculated for two conditions: (a) block failure and (b) individual pile failure. The least of the two gives the allowable load on the group.

(a) Block failure

$$Q_{gu} = cN_c A_g + P_g L\bar{c} \quad \text{where} \quad N_c = 9, c = \bar{c} = 70/2 = 35 \text{ kN/m}^2$$

$$A_g = 2.1 \times 2.1 = 4.4 \text{ m}^2, \quad P_g = 4 \times 2.1 = 8.4 \text{ m}, \quad L = 10 \text{ m}$$

$$Q_{gu} = 35 \times 9 \times 4.4 + 8.4 \times 10 \times 35 = 4326 \text{ kN}, \quad Q_a = \frac{4326}{2.5} = 1730 \text{ kN}$$

(b) Individual pile failure

$$Q_u = Q_b + Q_f = q_b A_b + \alpha \bar{c} A_s. \text{ Assume } \alpha = 1.$$

$$\text{Now, } q_b = cN_c = 35 \times 9 = 315 \text{ kN/m}^2, \quad A_b = 0.07 \text{ m}^2,$$

$$A_s = 3.14 \times 0.3 \times 10 = 9.42 \text{ m}^2$$

$$\text{Substituting, } Q_u = 315 \times 0.07 + 1 \times 35 \times 9.42 = 352 \text{ kN}$$

$$Q_{gu} = nQ_u = 9 \times 352 = 3168 \text{ kN}, \quad Q_a = \frac{3168}{2.5} = 1267 \text{ kN}$$

The allowable load is 1267 kN.