Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.1 : Procedure for ultimate pile capacity : Static analysis]

Objectives

In this section you will learn the following

- Static analysis
    - Piles in granular soils (sands and gravel)
    - Bored cast in situ piles
    - Piles in clays

Procedure for ultimate pile capacity

1. Static analysis
2. Dynamic formulae
3. Pile load test

Static analysis

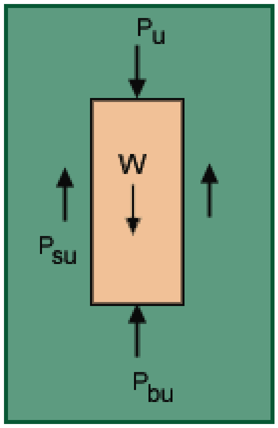


Fig –5.12 Forces on pile

$$P_{all} = \frac{P_u}{F.S} = \frac{P_{su}}{1} + \frac{P_{bu}}{3} \qquad \text{----------(1)}$$

For piles in granular soil, the design is based on an effective stress analysis. In clays, it is common to use a total stress analysis in which the load capacity is related to the undrained shear strength, c_u .

Ultimate load capacity,

$$P_u = P_{bu} + P_{fs} \qquad \text{----------(2)}$$

Where $P_{bu} = q_{bu}\, A_b$ & $P_{fs} = f_s\, A_s$

Where q_{bu} is the point bearing load

A_b is the cross sectional area of pile

f_s is the unit skin friction resistance

A_s is the surface area of the pile in contact with the soil

- **Piles in granular soils (sands and gravel)**

Driven piles

Point bearing in granular soil,

$q_{nu} = \bar{\sigma} N_q$ ---------(3)

Where s is the effective overburden pressure at the tip of the pile, equal to γL

L is the length of the embedment of the pile

For driven piles in sands, a value of $\phi = \dfrac{\phi_1 + 40^o}{2}$ may be taken, where ϕ_1 is the in situ value of the angle of hearing resistance

Unit skin friction,

$f_s = \sigma_h \tan\delta = K\bar{\sigma}\tan\delta$ ----------(4)

Where K is the lateral earth pressure coefficient and d is the angle of internal friction between the pile and the soil.

Ultimate skin friction resistance,

$P_f = f_{sav} A_s$, $P_f = K\bar{\sigma}_{av}\tan\delta$ ----------(5)

σ_{av} = effective overburden pressure over the embedded length of the pile

Table-5.1 Values of K and δ

Pile material	δ	Values of K	
		Loose sand	**Dense sand**
Steel	20	0.5	1.0
Concrete	0.75 Φ	1.0	2.0
Timber	0.67 Φ	1.5	4.0

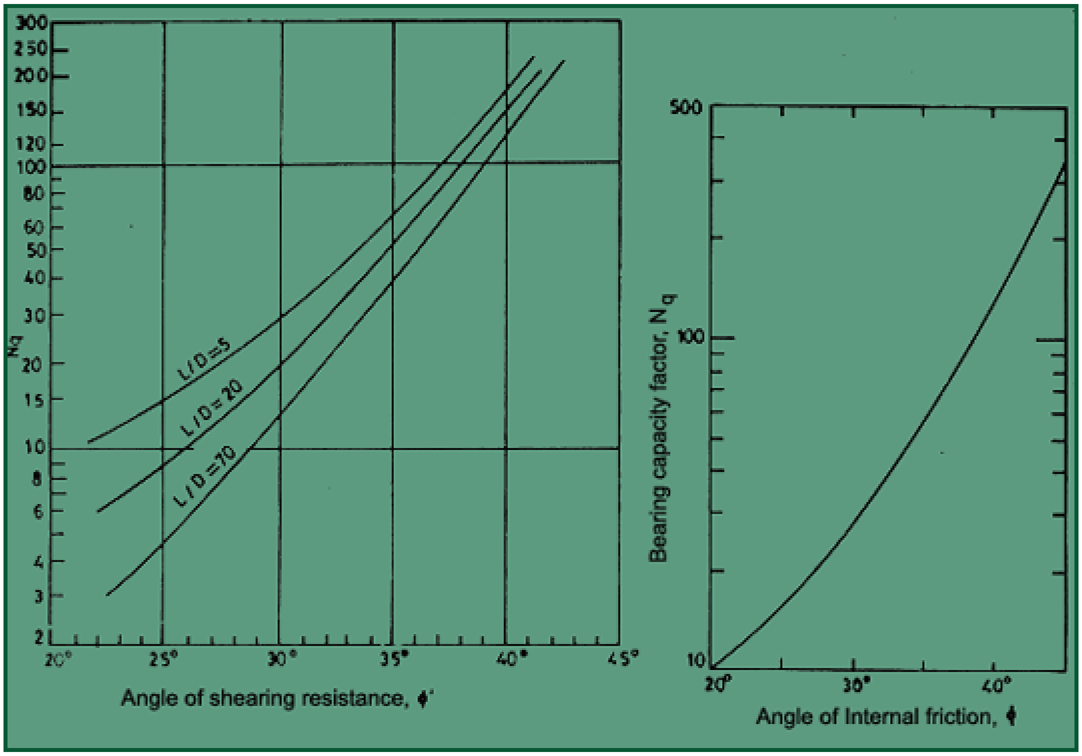


Fig-5.13 Values of N_q for pile formula (after Berezantzev et al, 1961) and N_q for driven piles (IS: 2911 Part I-1979)

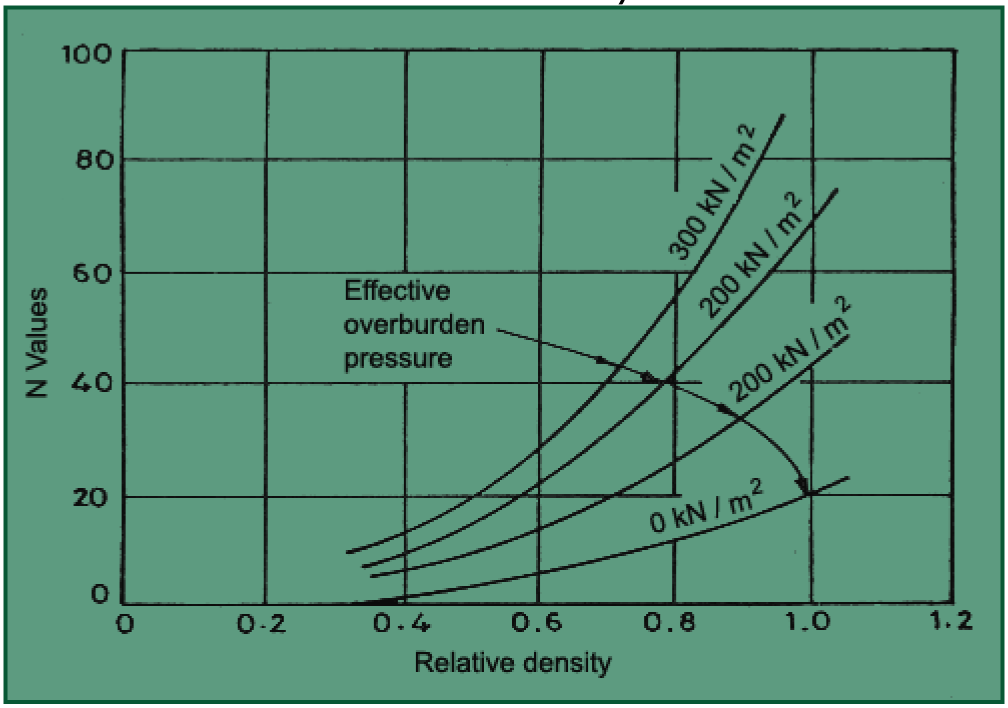


Fig-5.14 Relative density obtained from N values (After Gibbs and Holtz, 1966)

- **Bored cast in situ piles**

  The load carrying capacity of a bored cast in situ pile will be much smaller than that of a driven pile in sand. The angle of shearing resistance of the soil is reduced by 30, to account for the loosening of the sand due to the drilling of the hole.

  The value of, $K \approx 1 - \sin\phi$. K is generally varying from 0.3 to 0.75, with a medium value of 0.5. d can be taken equal to ϕ for bored piles excavated in dry soil and reduced value of d if slurry has been used during excavation.

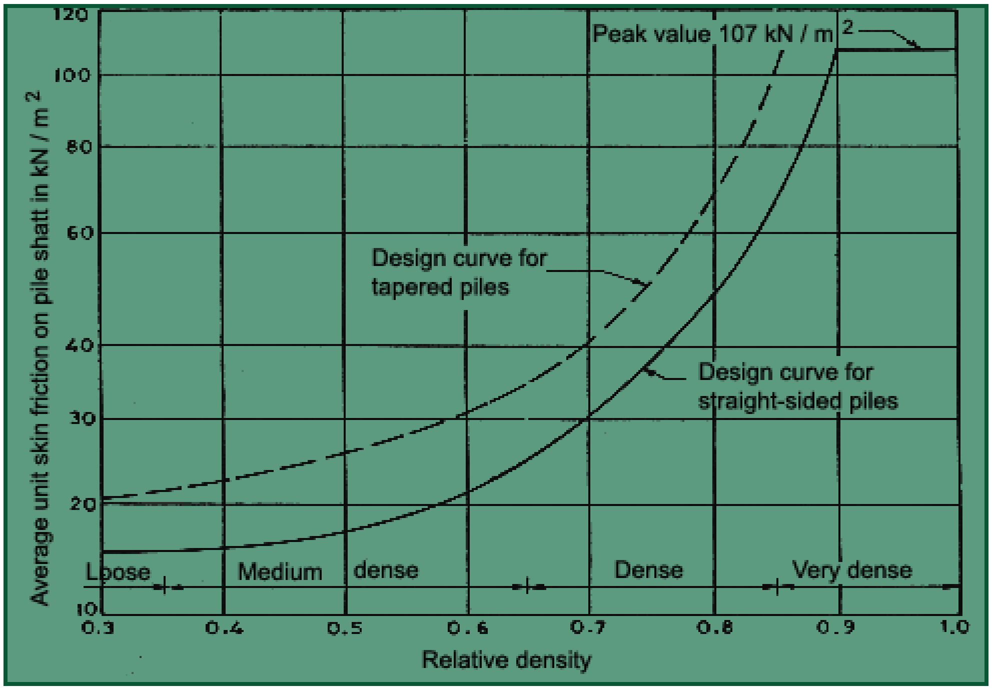


Fig-5.15 Average unit skin friction on driven piles in cohesion less soils

- **Piles in clays**

The ultimate load capacity of the pile is estimated by, $P_u = q_{bu}\ A_b + f_s\ A_s$

In clays, $q_{bu} = c_{ub} N_c$ and $c_a = \alpha c_u$; thus,

$$P_u = c_{ub}\ N_c\ A_b + \alpha c_u\ A_s \qquad \text{----------(6)}$$

c_{ub} is the undrained cohesion at the base of the pile

N_c is the bearing capacity factor for deep foundation, generally taken as 9

α is the adhesion factor

c_u undrained cohesion in the embedded length of the pile

Table : 5.2 Values of Reduction Factor, α

Consistency	N value	Bored piles	Driven cast in situ piles
Soft to very soft	<4	0.7	1.0
Medium	4-8	0.5	0.7
Stiff	8-15	0.4	0.4
Stiff to hard	>15	0.3	0.3

Recap

In this section you have learnt the following.

- Static analysis
    - Piles in granular soils (sands and gravel)
    - Bored cast in situ piles
    - Piles in clays

Objectives

In this section you will learn the following

- Pile load test
- Determination of Ultimate Load of pile Pile Load Test
    - Single Tangent method
    - Double Tangent Method
    - Log-Log method
    - Rectangular Hyperbola method
    - Vander Veen's method (1953)
    - Maazurkiewicz parabola method (1972)

Pile load test

1. Pile load test is the most reliable of all the approaches to determine the allowable load on the pile.
2. Pile load test are very useful for cohesion less soil. However, incase of cohesive soils, the data from the pile load test should be used with caution on account of disturbance due to pile driving, development of pore pressure and the in adequate time allowed of consolidation settlement.
3. Three types of pile tests are generally carried out.
   - Vertical load test
   - Lateral load test
   - Pull out test

IS: 2911 Part IV (1979) details the procedure for carrying out the load tests and assessing the allowable load. According to the code, the test shall be carried out by applying a series of vertical downward loads on a RCC cap over the pile. The load shall preferably be applied by means of a remote controlled hydraulic jack taking reaction against a loaded plot form. The test shall be applied in increments of about 20% of the assumed safe load. Settlement shall be recorded with at least three dial gauges of sensitivity 0.02 mm. each stage of loading shall be maintained till the rate of movement of pile top is not more than 0.1 mm per hours which ever is later.

The loading shall be continued up to twice the safe load or the load at which the total settlement of the pile top/ cap equals the appropriate value as indicated in the criterion stated below:

1. 2/3 the final load at which the total settlement attains a value of 12mm.
2. Fifty percent of the final load at which the total settlement equals 10% of piles diameter in case of uniform diameter piles and 7.5% of bulb diameter in case of under reamed piles.

The allowable load on a group of piles shall be the lesser of the following:

1. Final load at which the total settlement attains a value of 25mm, unless a total settlement different from 25mm is specified in a given case on the basis of the nature and type of structure.
2. Two-thirds the final load at which the total settlement attains a value of 40 mm.

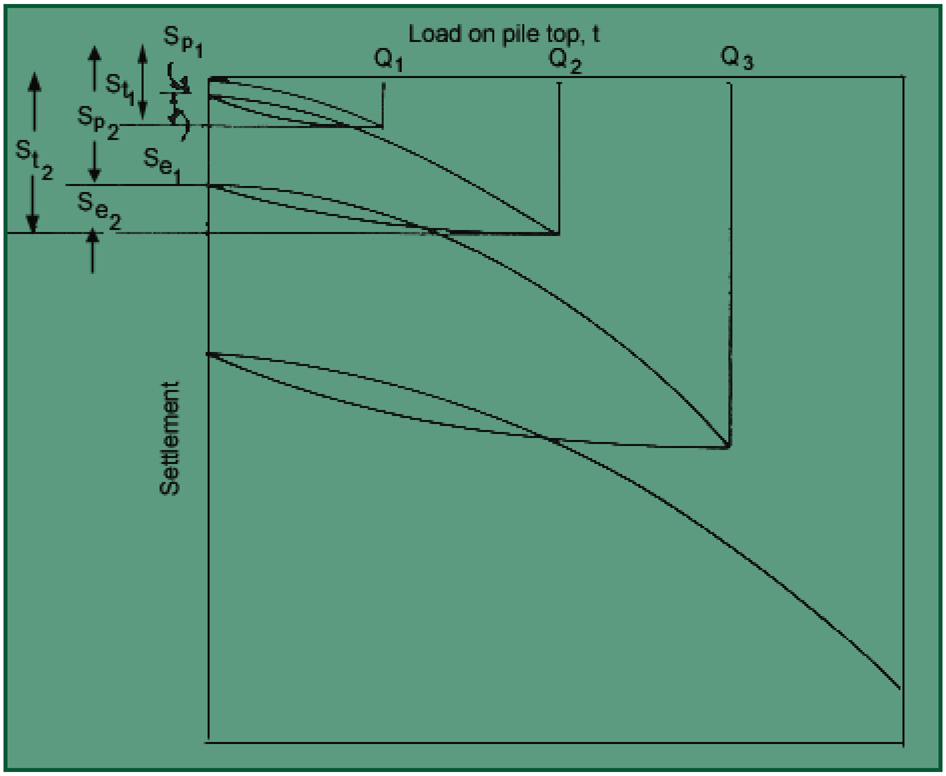


Fig –16 Typical load settlement plot from pile load test

The total settlement S of a pile obtained from a pile load test comprises of two components, namely, elastic settlement, S_e and plastic settlement, S_p .

$$S = S_e + S_p$$

The elastic settlement, S_e is due to the elastic recovery of the pile material and the elastic recovery of the soil at the base of the pile, S_e .

The total settlement of the pile, S at any load level can be written as S= S_b + Δ_1

Where S_b is the compression of the soil at the base and Δ_1 is the compression of the pile.

S_b can be written as, $S_b = S_c + S_{b1}$

Where S_{b1} is the plastic compression of the soil at the base

Total settlement is S= Δ_1 + S_e + S_{b1}

But, S= S_{e1} + S_p

$S_{e1} + S_p = \Delta_1 + S_e + S_{b1}$

$S_e = (S_p - S_{b1}) + S_{e1} - \Delta_1$

$S_e = S_{e1} - \Delta_1$

Since S_{e1} is known, Se can be determined if Δ_1 is given by equation

$$\Delta l = \frac{(Q - Q_f / 2)L}{AE}$$

where Q is the load on the pile, Q_f is the frictional load, L is the length of the pile, A is the average cross sectional area of the pile and E is the modulus of elasticity of the pile material.

Determination of Ultimate Load of pile Pile Load Test

1. **Single Tangent method**

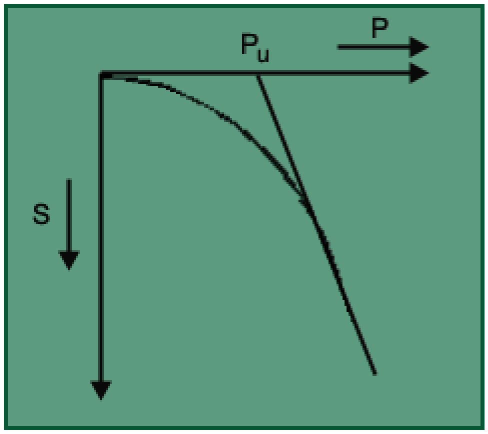


Fig-5.17 Single Tangent method

2. **Double Tangent Method**

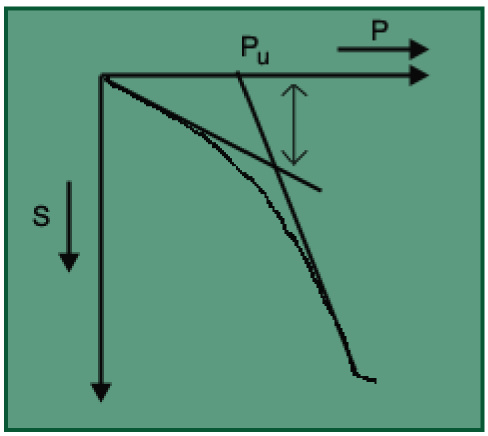


Fig-5.18 Double Tangent Method

3. Log-Log method

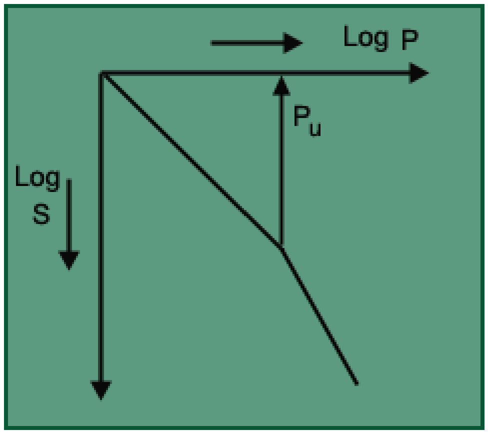


Fig-5.19 Log-Log method

4. Rectangular Hyperbola method

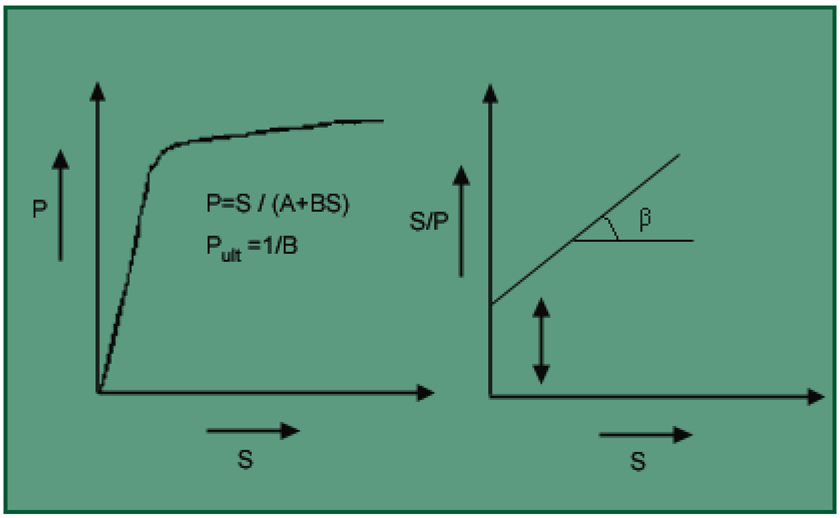


Fig-5.20 Rectangular Hyperbola method

$$P = \frac{S}{A + BS}, \quad P_{ult} = 1/B \qquad \text{----------(7)}$$

5. Vander Veen's method (1953)

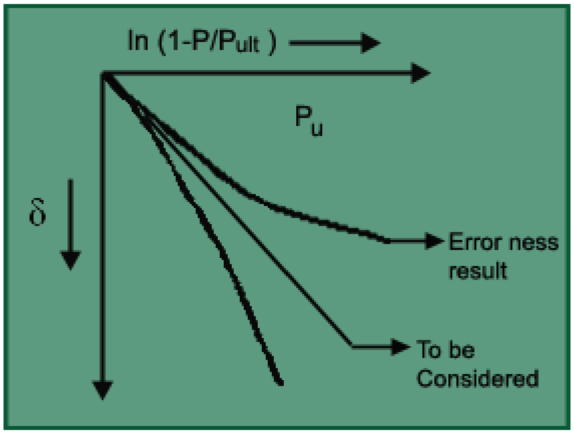


Fig-5.21 Vander Veen's method

$$P_{\text{at any load level}} = P_{ult}(1 - e^{-\alpha\delta_{pile}}) \qquad \text{---------(8)}$$

δ pile=settlement corr. to load P, and a is the factor relates load and deformation

$$P = P_{ult} - P_{ult}\, e^{-\alpha\delta_{pile}}, \qquad P_{ult}\, e^{-\alpha\delta} = P_{ult} - P = P_{ult}(1 - \frac{P}{P_{ult}}) \qquad \text{----------(9)}$$

$$e^{-\alpha\delta} = 1 - \frac{P}{P_{ult}}$$

$$-\alpha\delta = \ln(1 - \frac{P}{P_{ult}})$$

6. Maazurkiewicz parabola method (1972)

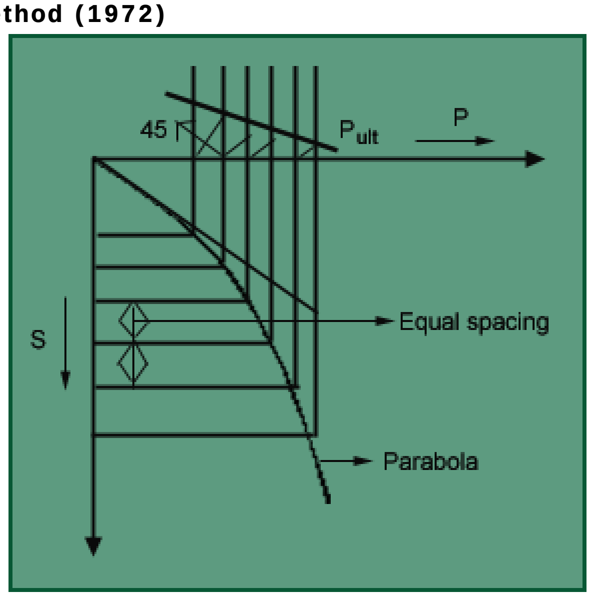


Fig-5.22 Maazurkiewicz parabola method

Recap

In this section you have learnt the following.

- Pile load test
- Determination of Ultimate Load of pile Pile Load Test
  - Single Tangent method
  - Double Tangent Method
  - Log-Log method
  - Rectangular Hyperbola method
  - Vander Veen's method (1953)
  - Maazurkiewicz parabola method (1972)

Objectives

In this section you will learn the following

- Introduction
- Engineering news formula (A.M.Wellington)
- Modified Hilley Formula
- Usefulness of dynamic formulae for pile capacity

Dynamic formulae

These are based on the laws governing the impact of elastic bodies. The input energy of the hammer blow is equated to the work done in overcoming the resistance of the ground to the penetration of the pile. Allowance is made for the losses of energy due to elastic contractions of the pile, pile cap, and subsoil and also the losses due to the inertia of the pile.

- **Engineering news formula (A.M.Wellington)**

The dynamic resistance of soil or ultimate pile load capacity, $Q_u = \dfrac{WH}{F(S+C)}$

Where W is the weight of the hammer falling through a height H

S is the real set per blow

C is the empirical factor

F is the factor of safety say 6.

In metric units

Drop hammer, $Q_u = \dfrac{WH}{6(S+2.5)}$ ----------(10)

Single acting steaming hammer, $Q_a = \dfrac{WH}{6(S+0.25)}$ ---------(11)

Where Q_a & H are expressed in kg. H is in cm, S is the final set in cm/blow, usually taken as average penetration for the last 5 blows of a drop hammer, or 20 blows of a steam hammer.

- **Modified Hilley Formula**

It taken in to account more energy losses during driving in a more realistic manner.

$$R = \frac{Wh\eta}{S + C/2} = \frac{Wh\eta}{S + \frac{1}{2}(C_1 + C_2 + C_3)} \quad \text{----------(12)}$$

where R is the ultimate driving resistance in tons

W is the weight of hammer in tons.

H is the effective fall of hammer.

η is the efficiency of the blow that represents the ratio of energy after impact to the striking energy of the ram

S is the final set or penetration per blow in cm

C is the total elastic compression= $C_1 + C_2 + C_3$

C_1 is the temporary elastic compression of the dolly and packing

C_2 is the temporary elastic compression of the pile

C_3 is the temporary elastic compression of the soil

$$C_1 = 1.77\frac{R}{A} = 9.05\frac{R}{A} \qquad \text{---------(13)}$$

1.77 When the driving is without dolly or helmet and cushion about 2.5cm thick.

9.05 When the driving is with short dolly up to 60 cm long, helmet and cushion up to 7.5cm thick.

$$C_2 = 0.657\frac{RL}{A} \qquad \text{----------(14)}$$

$$C_3 = 3.55\frac{R}{A} \qquad \text{----------(15)}$$

where L is the length of the pile in m and A is the cross sectional area of pile.

$$\eta = \frac{W + Pe^2}{W + P} \qquad \text{---------(16)}$$

$$\eta = \frac{W + Pe^2}{W + P} - \left(\frac{W - Pe}{W + P}\right)^2 \qquad \text{---------(17)}$$

Where P is the weight of pile, anvil, helmet and follower in tons and

e is the coefficient of restitution of the materials under impact. Values are:

For steel ram of double-acting hammer striking on steel anvil and driving reinforced concrete pile, e=0.5

For cast-iron ram of single acting or drop hammer striking on head of reinforced concrete pile, e=0

for single acting or drop hammer striking a well-conditioned driving cap and helmet with hard wood dolly while driving reinforced concrete piles or directly on head of timber pile, e=0.25

For a deteriorated condition of the head of pile or of dolly, e=0

Table: 5.3 values of η in relation to *e* and P / W

Ratio of P / W	e = 0.5	e = 0.4	e = 0.32	e = 0.25	e = 0
0.5	0.75	0.72	0.70	0.69	0.67
1	0.63	0.58	0.55	0.53	0.50
1.5	0.55	0.50	0.47	0.44	0.40
2.0	0.5	0.44	0.40	0.37	0.33
2.5	0.45	0.40	0.36	0.33	0.28
3.0	0.42	0.36	0.33	0.30	0.25
3.5	0.39	0.33	0.30	0.27	0.22
4	0.36	0.31	0.28	0.25	0.20
5	0.31	0.27	0.24	0.21	0.16
6	0.27	0.24	0.21	0.19	0.14
7	0.24	0.21	0.19	0.17	0.12
8	0.22	0.20	0.17	0.15	0.11

Usefulness of dynamic formulae for pile capacity

These formulae are based on the assumption of the impact of two free elastic bodies. Pile is not a free body. Dynamic formula may be used with confidence in free-draining materials such as coarse sand, but are not likely to yield useful results in the case of cohesive soil deposits. Further, in saturated sand deposits, vibrations during driving are likely to cause liquefaction.

Recap

In this section you have learnt the following.

- Introduction
- Engineering news formula (A.M.Wellington)
- Modified Hilley Formula
- Usefulness of dynamic formulae for pile capacity

Objectives

In this section you will learn the following

- Introduction

Pile capacity

For circular shallow footing,

$$q_{ult} = 1.3CN_c + qN_q + 0.3\gamma BN_\gamma \qquad \text{----------(18)}$$

$$q_{ult} = CN_c F_{cs} F_{cd} + qN_q\ F_{qs} F_{qd} + 0.5\gamma BN_\gamma F_{\gamma s} F_{\gamma d} \qquad \text{----------(19)}$$

for deep footings, $P + P_{bu} + P_{su}$

$$= q_{ub}.A_b + f_s.A_s \qquad \text{---------(20)}$$

where q_{ub} is the ultimate bearing capacity, A_b is the area of pile base, f_s is the unit skin friction and A_s is the shaft area (perimeter*length)

for piles,

$$q_{ult} = CN_c{*} + qN_q{*} + 0.5\gamma BN\gamma{*} \qquad \text{---------(21)}$$

for clays, $\Phi = 0$,

therefore,

$$P_{bu} = A_b(CN_c{*} + qN_q{*}) \qquad \text{---------(22)}$$

here the unit weight term is neglected because $P_u = P_{bu} + P_{su} - W$

$$= (P_{bu} - W) + P_{su} \qquad \text{--------(23)}$$

Determination of P_{bu} :

- Meyerhof's method
- Vesic method
- Janbu Method

Determination of P_{su}

- α-Method
- λ-Method
- β-Method

Recap

In this section you have learnt the following.

- Introduction

Objectives

In this section you will learn the following

- Meyerhof's Method
- Vesic method to compute P_{bu} :
- Janbu's method to compute P_{bu} **:**

The Frictional Resistance Q_s is obtained from above eq after estimating the unit skin friction f_s. The unit friction for a straight side pile depends up on the soil pressure acting normal to the pile surface & the coefficient of the friction between the soil and the pile material in fig.

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

$$\sigma_h = K.\sigma_v$$

Where K = Earth pressure coefficient, σ_V' = Effective vertical pressure at that depth.

The Unit Skin Friction f_s acting at any depth can be written as

$$f_s = \sigma_h.\tan\delta$$

$$f_s = K.\sigma_V'.\tan\delta \qquad \text{----------(33)}$$

Selection of value of **K** & require good engineering judgment depend up on the loose sand & medium sand.

In General Dense & Loose sand depend on the initial relative density and the method of installations. The larger the volume of the soil displacement, the higher the value of the resulting friction. For high displacement driven piles, the soil is considered dense. For driven in cast in place piles, the soil is considered medium dense if the casing is left in place or if the concrete is compacted as the casing is withdrawn. The sand is considered to be loose, if the concrete is not compacted. Tapered soil develops greater unit friction than the straight piles. Further the value of K is greater if the pile is driven in to undisturbed soil than the one for installed in a pre drilled holes.

The effective vertical Pressure increases with depth only up to the critical depth. Below the critical depth the value of σ_V Constant.

The ultimate frictional resistance can be expressed as,

$$P_{SU} = \sum P\times(\Delta L)\times f_s \qquad \text{----------(34)}$$

Where P = Perimeter, ΔL = Segmented Length f_s = Unit skin friction, σ_V' = Vertical stress at the centre of the segmented length.

Recap

In this section you have learnt the following.

- Meyerhof's Method
- Vesic method to compute P_{bu}
- Janbu's method to compute P_{bu}

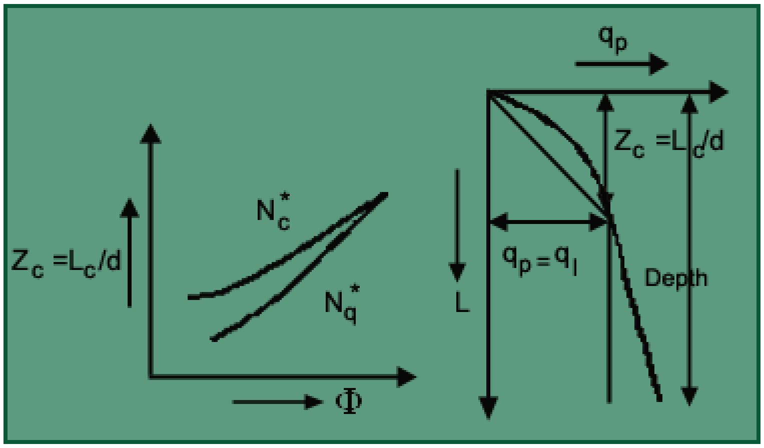
Determination of P_{bu} :

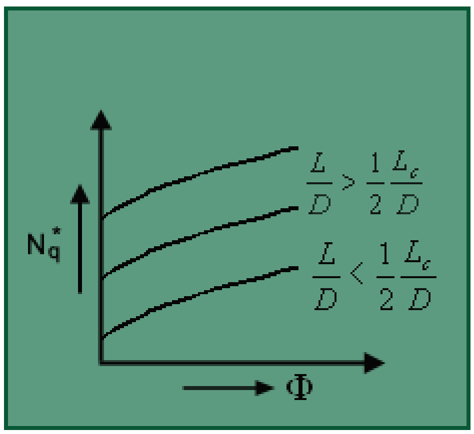
1. **Meyerhof's Method :** Good for sands

$P_{bu} = A_b(CN_c{}^* + qN_q{}^*)$

For sands C=0,

therefore $P_{bu} = A_b q N_q{}^* = A_b\, q_p$





After certain depth q_p becomes constant and that particular value should not exceed limiting value(q_l)

q_p =50*N*q tan ϕ

q_l =11 M_x / M^2

q_l =4N for driven piles & 1.2 N for bored piles.

Where N is the SPT value

q_p is the minimum of q_p or q_l

Meyerhoff's method of finding pile tip resistance in layered soil

- **For two layers**

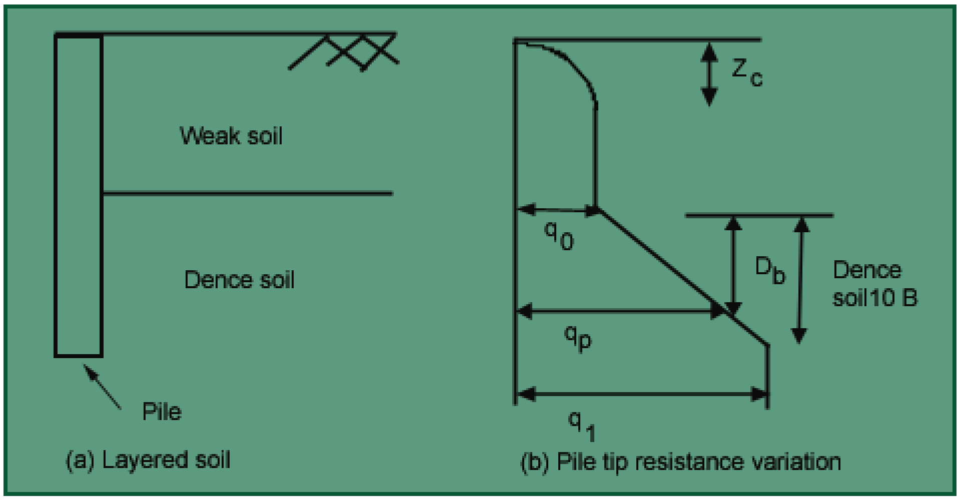


Fig.5.23 Pile tip resistance for layered soil

Where,

q_o is the point resistance per unit area at the base of first layer,

q_p is the point resistance per unit area at the pile tip,

q_l is the limiting point resistance per unit area,

z_c is the depth upto portion of nonlinearity, B is the width of the pile or width of the pile, as shown in the fig.

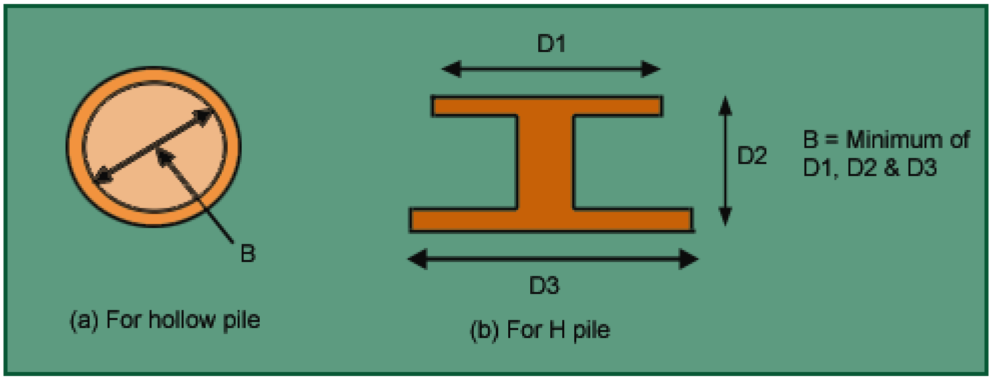


Fig. 5.24 Different B values

$$q_p = q_0 + \frac{(q_l - q_0)}{10B} D_b \le q_l \qquad \text{----------(24)}$$

where,

q_l values are given by Meyerehoff as given in earlier section.

- **For three layers**

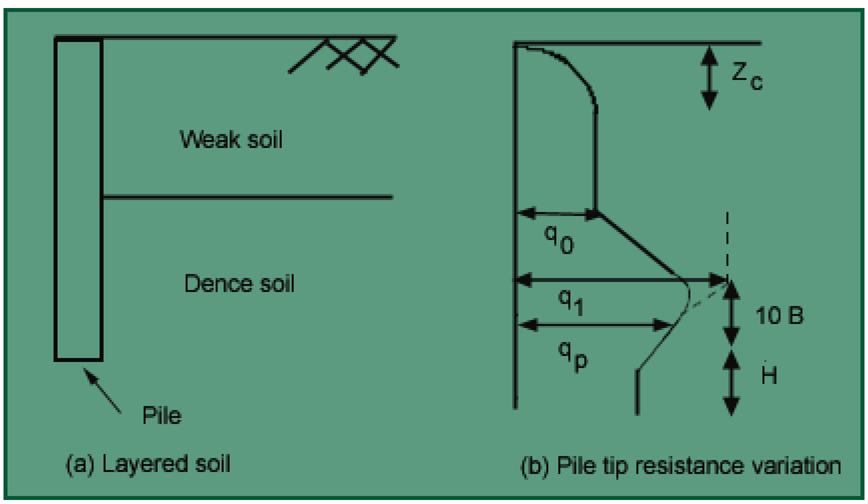


Fig 5.25 Pile tip resistance for layered soil

H<10B,

$$q_p = q_0 + \frac{(q_l - q_0)}{10B} \le q_l \qquad \text{----------(25)}$$

Example: For an end bearing pile of cross sectional area 1.17 m^2 and D = 1.22m.

γ= 7.85 KN/ m^3, $\phi_{avg} = 29^0$,

According to Meyerhoff's chart, for $\phi = 29^0$, (L_c /D) = 7. Compute the pile capacity for (L/D) =10.

Ans:

$\because$ Pile length L = 1.22 x 10 =12.2 m.

From fig. 26, for $\phi = 29^0$, $N_q{*} = 55$,

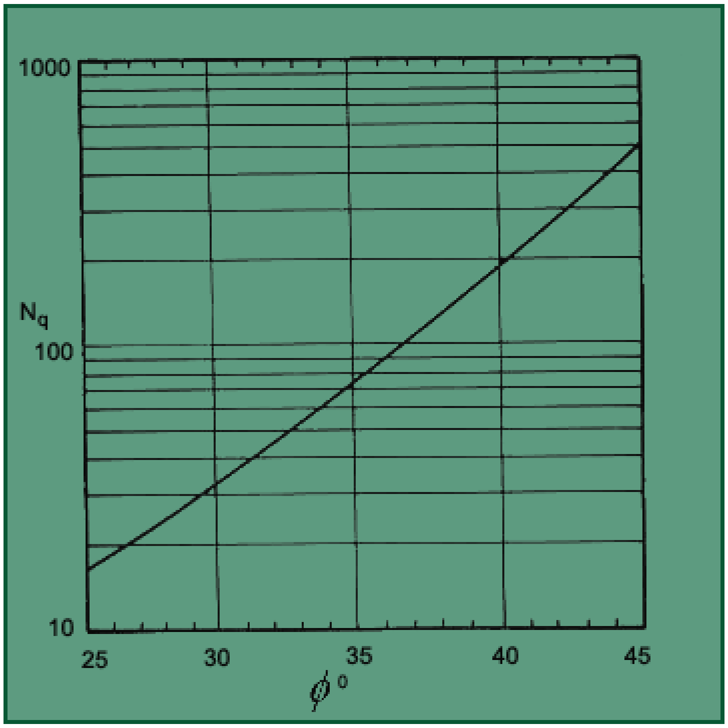


Fig 5.26 Meyerhoof's chart of N_q *

$q_p = \gamma$ (7 x D) x N_q
= 7.85 x 7 x 1.22 x 55
= 3687.145 KN/
q_l = 50 N_q * tan ϕ
= 50 x 55 x tan 29 0
= 1524.35 KN/ m^2 < q_p

Design value of q_p is 1524.35 KN/ m^2 .

The variation of q_p value is given in fig.

P_{bu} = (1524.34) x 1.17 = 1783 KN

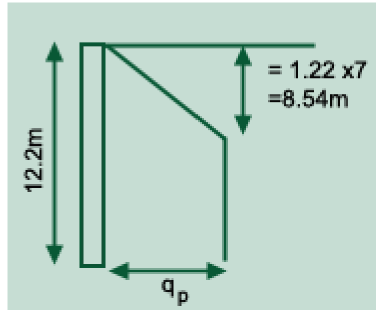


Fig. 5.27 Variation of pile tip resistance

Base resistance in pure clay (ϕ = 0) :

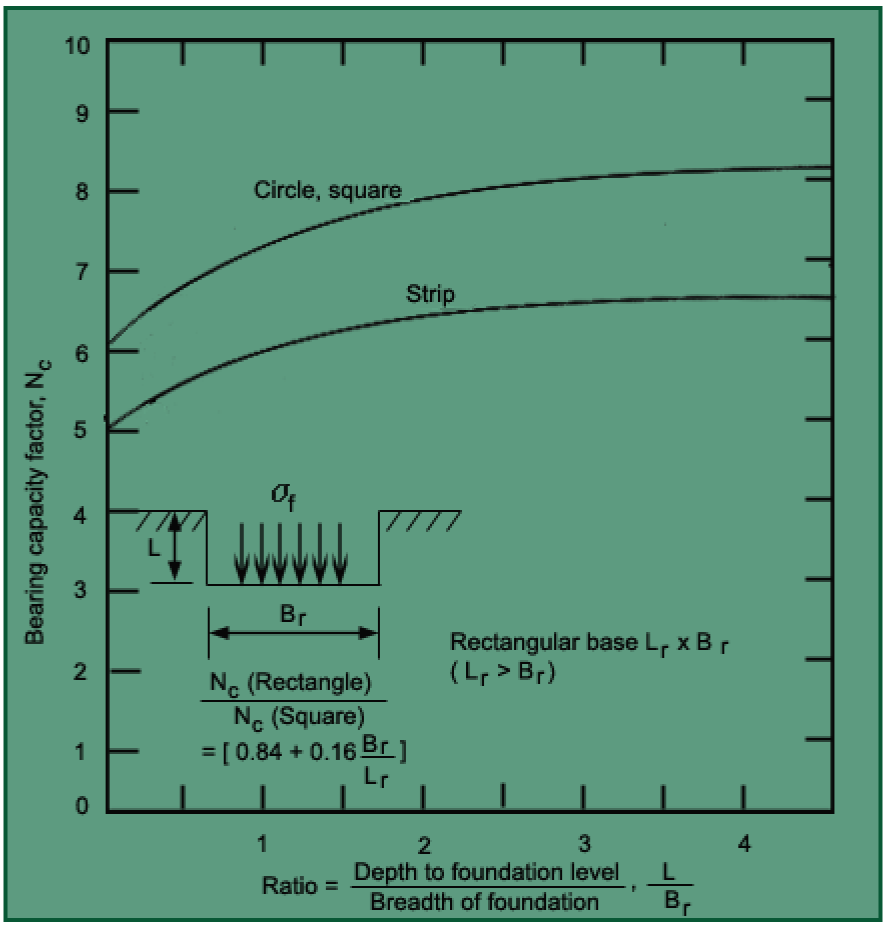


Fig. 5.28 Value of N_c given by Skempton

Pile is considered as circular foundation, referring fig.

$\because$ For (L/B r) > 4, N_c = 9 for clays.

N_c values depend on ,

i) Method of installation,

ii) Stress strain relationship of soil etc.

Typical values of N_c are,

N_c = 5.7 to 8.2 for expansive clays,

N_c = 7.4 to 9.3 for insensitive clays,

N_c = as low as 5.5 for very large value of N_c . •

Unless otherwise stated we should consider N_c = 9 in our design.

Bishop's equation of N_c :

$$N_c = 1 + \frac{4}{3}\left[1 + \ln\left(\frac{E_u}{3\,C_u}\right)\right] \qquad \text{----------(26)}$$

where,

E_u is the undrained modulus of soil from stress-strain curve,

C_u is the undrained cohesion.

Base resistance in $c - \phi$ soil (Meyerhoff's analysis):

$$P_{bu} = A_b\, q_p = A_b\,(cN_c^{*} + q'\,N_q^{*}) \qquad \text{---------(27)}$$

$$N_c^{*} = (N_q^{*} - 1)\cot\phi \qquad \text{---------(28)}$$

where,

q' is the effective overburden pressure,

N_q *can be found from Meyerhoff's chart corresponding to ϕ value.

2. Vesic method to compute P_{bu} :

It based on cavity expansion theory of cylinder.

$$P_{bu} = A_b \,(cN_c + \sigma_0{}' \, N_\sigma{}^*) \qquad \text{----------(29)}$$

where,

$$N_\sigma{}^* = \frac{B\, N_q{}^*}{(1+2K_0)} \qquad \text{---------(30)}$$

where, K_0 is the earth pressure coefficient at rest,

$\sigma_0{}'$ in mean normal stress,

$N_\sigma{}^* = f(I_{\gamma\gamma})$

Where,

$I_{\gamma\gamma}$ is the rigidity coefficient for reduced rigidity for the soil which depends on the elastic modulus of soil.

$$I_{rr} = \frac{I_r}{1 + I_r\,\Delta} \qquad \text{----------(31)}$$

where,

Δ is average volumetric strain,

$$I_r = \frac{E_s}{2(1+\mu_s)(c+q'\tan\phi)} \qquad \text{----------(32)}$$

here,

μ_s is the poission's ratio.

3. Janbu's method to compute P_{bu} :

In this failure plane assumed is as shown in fig. 5.29

$N_c^*, N_q^* = f(\eta', \phi)$

η' = 70 ^{0} for soft clays,

= 105 ^{0} for sand.

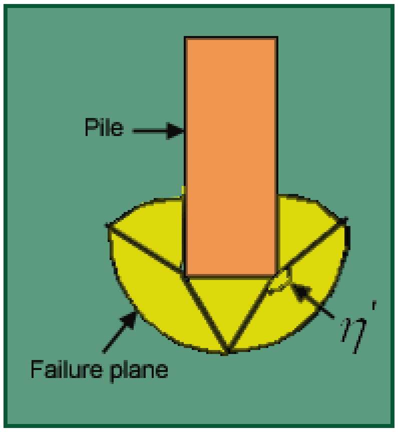


Fig. 5.29 Failure plane assumed by Janbu

Skin Resistance :

The Method of estimating the Ultimate Load carrying capacity of a pile foundation, depending up on the characteristics of the soil, can be found out by Static method from the following eq.

$Q_U = Q_P + Q_S$

Where Q_u = Ultimate Load

Q_p = Point or Base Resistance of the pile

Q_s = Shaft Resistance Developed by the friction (or adhesion) between the soil and the pile shaft.

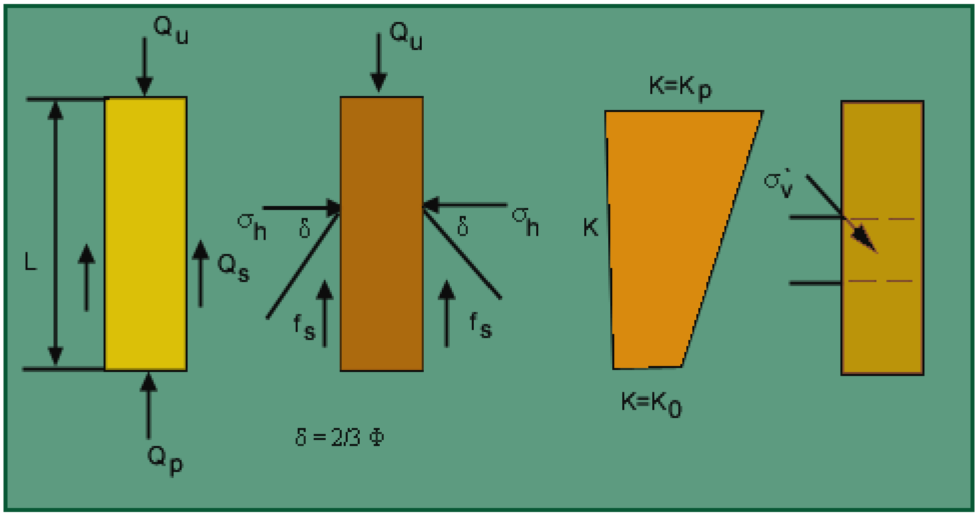


Fig 5.30 Variation of K in Sands.

Objectives

In this section you will learn the following

- α**-**Method for cohesive soil:
- λ **-** Method
- β **-** Method

Determination of P_{su}:

1. **α-method for cohesive soil:**

The ultimate bearing capacity of a pile in cohesive soil may develop up to 80 – 90% of its

capacity through shaft resistance. The α-Method is a total stress analysis where the ultimate capacity of the pile is determined from the undrained shear strength of the cohesive soil. This method assumes that the shaft resistance is independent of the effective overburden pressure. The unit shaft resistance is expressed in terms of an empirical adhesion factor times the undrained shear strength. The unit shaft resistance is equal to the adhesion (c_a) which is the shear stress between the pile and the soil.

αMethod is an empirical adhesion factor to reduce the average undrained shear strength (c_u) of the undisturbed clay along the embedded length of the pile. The coefficient αdepends on the nature and strength of the clay, pile dimensions, method of installation, and time effects.

Step By Step Procedure for α Method in Cohesive Soil

Step 1 Delineate the soil profile into layers and determine the adhesion, *ca.*

Step 2 For each soil layer, compute the unit shaft resistance

Step 3 Compute the shaft resistance in each soil layer and the ultimate shaft resistance,

$$P_{su} = \int_0^L P.c_a dz \qquad \text{----------(36)}$$

Step 4 Compute the ultimate toe resistance, *Rt* .

$qt = 9cu$

Rt = *qt .At*

Step 5 Compute the ultimate pile capacity (kips).

$Q_u = P_{su} + Rt$

Step 6 Compute the allowable design load Q_a (kips).

$Q_a = Q_a$ / *Factor of Safety*

(A) α-method for cohesive soil (Homogenous Layer)

$c_a = \alpha c_u$

$c_a = \alpha c_u$

$\alpha = \dfrac{c_a}{c_u}$ where c_u is the undrained shear strength for a homogenous layer.

For very soft clay, $\alpha = 1$ or slightly more than 1.Kerisel (1966) had shown the variation of α values with undrained shear strength of the soil.

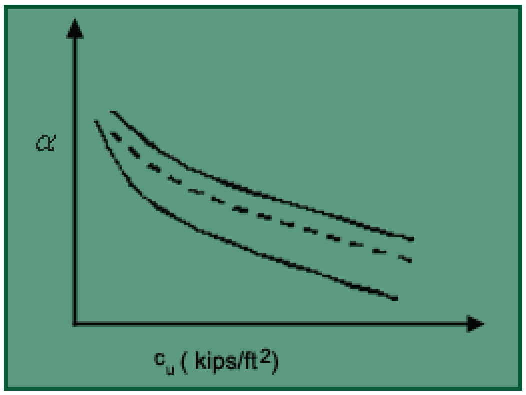


Fig. 5.31 Variation of α with undrained shear strength

Heterogeneous Soil:

Case1 : Sands over lying stiff cohesive clays.

Case2: soft clays/sits overlying stiff clays.

Case3: stiff cohesive soils without any overlying strata.

Table : 5.4 α Values For Different Penetration Ratios

Cases	Penetration ratio	$\alpha = \frac{c_a}{c_u}$
Case1	<20 >20	1.25 fig ( )
Case2	<20(78) >20	0.4 0.7
Case3	<20(78) >20	0.4 fig( )

Where, Penetration ratio= Depth of penetration in stiff clay
Pile diameter

Driven piles:

1. The clay around the pile is displaced both vertically and horizontally. Upward displacement results in heaving of the ground and can cause reduction in the bearing capacity of adjacent piles.
2. The clay in the disturbed zone around the pile is completely remoulded during driving.
3. The excess pore water pressures set up by the driving stresses dissipates within a few months as the disturbed zone is relatively narrow. Thus the skin friction at the end of the dissipation is normally appropriate in design. The adhesion factor a for driven piles is generally correlated to $\frac{c_u}{\sigma_v}$, i.e. the ratio of the undrained shear strength to the existing vertical effective overburden pressure.

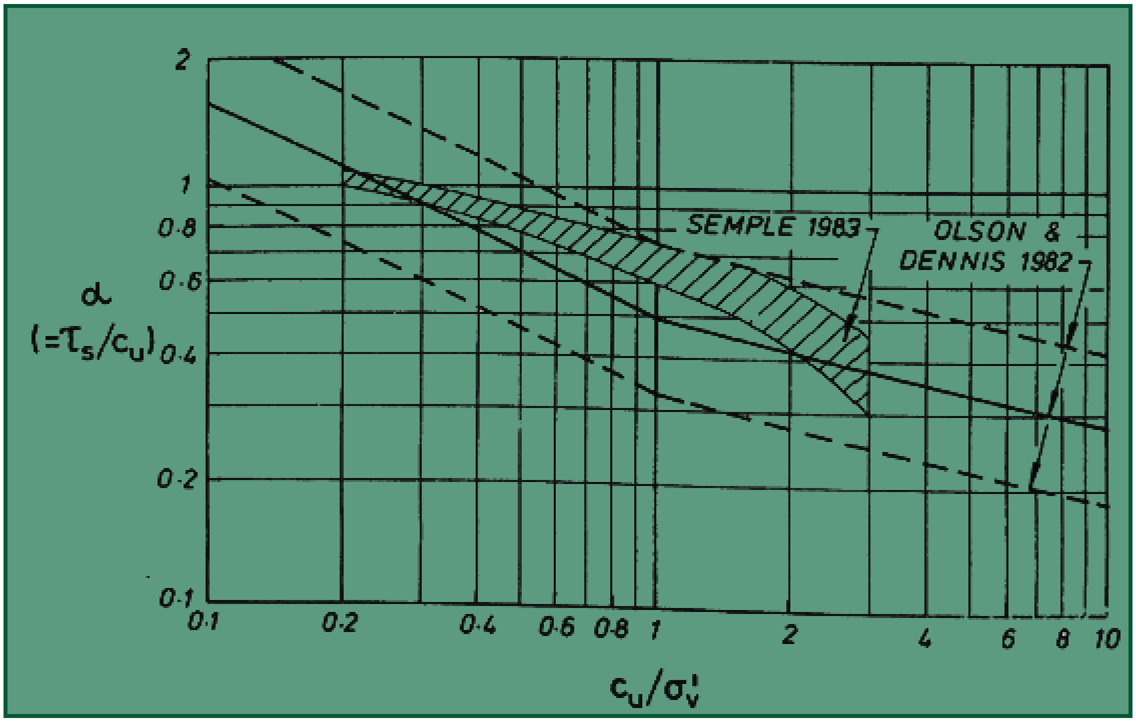


Fig. 5.32 Design Curve for Driven Piles

Bored piles:

1. A thin layer of clay (usually 25mm) immediately adjoining the shaft will be remoulded during boring.
2. Gradual softening of the clay adjacent to the pile will take place due to stress release, pore water seeping from surrounding clay towards the shaft. Water can also be absorbed from wet concrete. This softening is accompanied with reduction in shear strength and a reduction in skin friction. Construction should therefore be completed as soon as possible.

The value of a for bored piles in clay is usually lower than those for driven piles. Most of the recommendations of the values of α come from experience. For example, London clay has been extensively studied and the recommended value of a is 0.45. For short piles in weathered London clay the value drops to 0.3. For Indian clays it is 0.5. For other clays, Weltman and Healy (1978) produced a variation of a with c_u reproduced in Figure

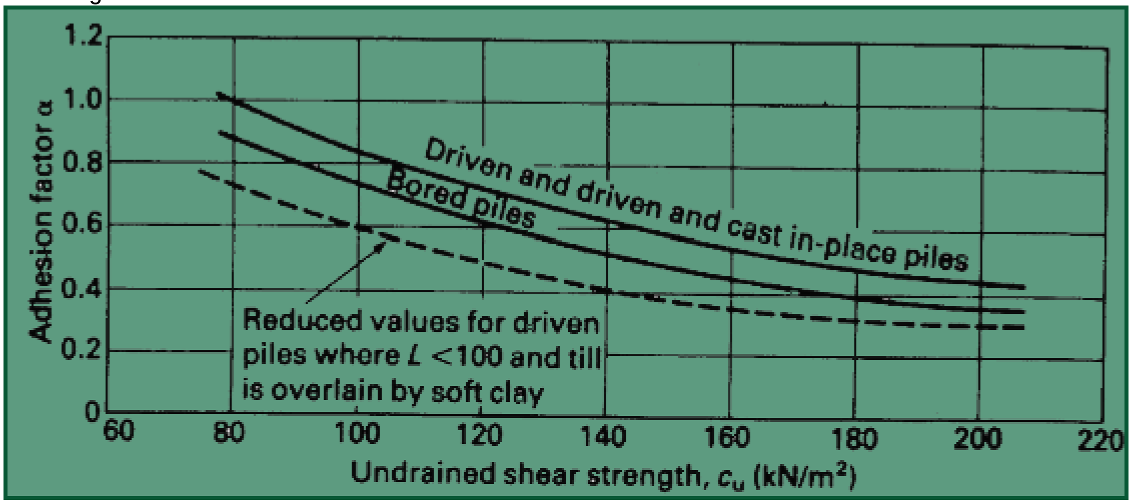


Fig. 5.33 α values for bored and driven piles (based on c_u value)

Table: 5.5 α values for various types of piles (based on c_u value)

Pile type	$C_u < 1000 psf = 0.5 kg/cm^2$	$C_u < 1000 psf = 0.5 kg/cm^2$
Steel	0.5 c_u	2000 p_{sf}
Concrete	0.8 c_u	600 p_{sf}
Wood	1.0 c_u	1000 p_{sf}

2. $\lambda - method$ **:**

An alternative and entirely empirical method has been proposed by Vijayvergiya and Focht (1972) for the estimation of the side resistance of long steel pipe piles founded in clay. This method is used fairly frequently in the design of heavily loaded offshore foundations. Because these piles are long and slender, the great majority of capacity is derived from the shaft and, therefore, the end bearing component can be insignificant. This method is not commonly used for land-based piles, and should only be applied where an assumption of normal consolidation is appropriate . The authors simply established a correlation between ultimate shaft resistance, P_{su} , determined from a large number of load tests on steep pipe piles, the mean effective vertical stress between ground and pile toe, $\sigma_m^{'}$, and the mean undrained cohesion along the pile shaft, c_m as follows:

$$P_{su} = \lambda(\sigma_m^{'} + 2c_m)A_s \qquad \text{----------(37)}$$

λ = Dimensionless coefficient

$\sigma_m^{'}$ = mean effective vertical stress between ground surface and pile tube.

c_m = average undrained cohesion along the pile.

A_s = pile surface area.

It follows then that λ is a function of pile penetration and decreases to a reasonably constant value for very large penetrations. It is possible to compare the conventional adhesion factor, α, with λ from a comparison of the relevant equations.

$$\alpha = \frac{c_a}{c_m} = \lambda\left[\frac{\sigma_m^{'}}{cm} + 2\right]$$

5.5.7.3 $\beta - method$:

$$P_{su} = \int_0^L P\,\sigma_v^{'}(k_s \tan\phi_a^{'})dz$$

$$\beta = k_s \tan\phi_a^{'} \qquad \text{---------(38)}$$

k_s = Earth pressure coefficient

$\phi_a^{'}$ = pile soil interfacial friction angle.

$\sigma_v^{'}$ = mean vertical effective stress

3 β method

- **Burland method**

The β method developed by Burland (1973) shows comparable values to the actually measured skin resistances. This method intensely counts on the soil-pile interaction parameters such as the angle of soil-pile friction angle (δ) and the coefficient of earth pressure (k_s). Burland method for predicting the pile skin resistance tends to over predict the capacity of the piles.

β is (1 - sin ϕ) tan ϕ

ϕ ranges from 20 0 -30 0

β ranges from 0.24 to 0.29

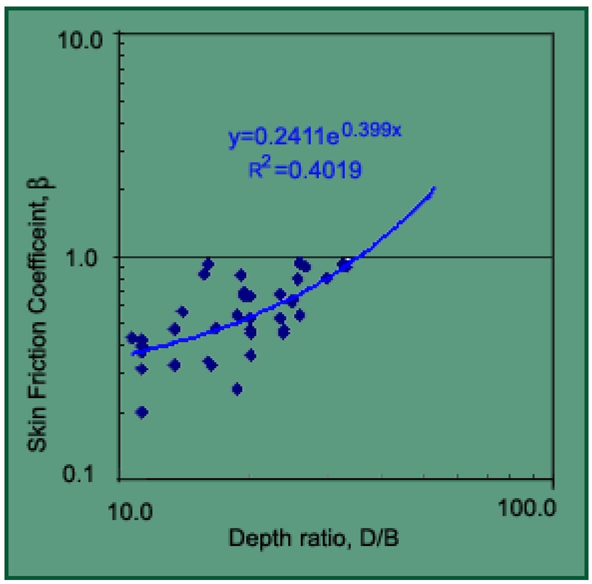


Fig. 5.34 Relation between Depth Ratio D/B and Skin Friction Coefficients as predicted by Burland.

- **Meyerhoff method**

  δ depends on ϕ as well as depth of penetration

  β approximately 0.15 for depth >60m

Stiff Clay

K_s = k_0 as per Burland

ϕ_R' =Remolded angle of friction of soil

Meyerhof (1976) has proposed values of K $\tan\delta$ for driven, jacked and bored piles. The shaft resistance values reflect the likely changes of stress state in the soil due to the method of installation. The values for bored piles are based on an assumed reduced friction angle ϕ of 75% of its undisturbed value. In using this chart, the undisturbed value is used in all cases. These values are combined in the Meyerhof method with the full calculated effective overburden pressure . Meyerhof demonstrated that for driven piles in stiff clay, K_s =1.5 K_0, while for bored piles, K_s = 0.75 K_0. Meyerhof proposed the following expression for K_0 .

$f_s = \frac{\overline{N}}{50}$ for driven pile

$f_s = \frac{\overline{N}}{100}$ for bored piles

$\overline{N}$ =average N value over pile length.

$f_l = 1 kg/cm^2$

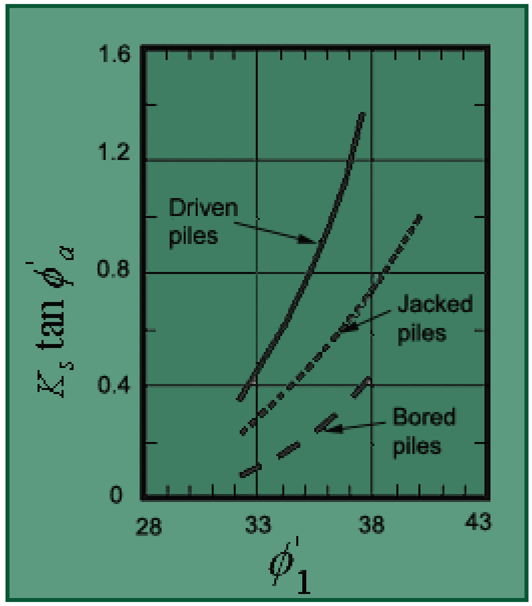


Fig. 5.35 Values of $K_s \tan\phi$

Recap

In this section you have learnt the following.

- α-Method for cohesive soil:
- λ - Method
- β - Method

Congratulations, you have finished Lecture 22. To view the next lecture select it from the left hand side menu of the page