

Module 5 : Design of Deep Foundations

Lecture 20 : Introduction [Section 20.1 : Introduction]

Objectives

In this section you will learn the following

- Introduction

Module 5 : Design of Deep Foundations

Lecture 20 : Introduction [Section 20.1 : Introduction]

INTRODUCTION

The shallow foundations are used in case of small buildings or structures, which carry lesser loads, and hence the loads are dissipated into the soil mass at much lower depth. However when we are considering large structures, which carry heavy loads, the loads are dissipated at greater depths where usually the soil bearing capacity is quite high. One guideline of differentiating between the shallow and deep foundations is that in case of the deep foundations the depth of foundations is more than the dimension of the structure (usually the width is considered as the dimension).

DIFFERENT TYPES OF DEEP FOUNDATION

Deep foundations are of the following types:

- Deep footings.
- Piles.
- Piers.
- Caissons /Well foundations.

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Lecture 20 : Introduction [Section 20.1 : Introduction]

Recap

In this section you have learnt the following.

- Introduction

Module 5 : Design of Deep Foundations

Lecture 20 : Introduction [Section 20.2 : Design Methodology For Piles]

Objectives

In this section you will learn the following

- Requirement For Deep Foundations
- Classification Of Piles
- Points To Be Considered For Choosing Piles
- Pile Classification
- Piles In Clay
- Piles In Sand
- Settlement Of Pile Groups
- Codal Provision

Module 5 : Design of Deep Foundations

Lecture 20 : Introduction [Section 20.2 : Design Methodology For Piles]

DESIGN METHODOLOGY FOR PILES

The detailed design methodology of piles is described in the following sections.

REQUIREMENT FOR DEEP FOUNDATIONS

Generally for structures with load $>10 \text{ t/m}^2$, we go for deep foundations. Deep foundations are used in the following cases:

- Huge vertical load with respect to soil capacity.
- Very weak soil or problematic soil.
- Huge lateral loads eg. Tower, chimneys.
- Scour depth criteria.
- For fills having very large depth.
- Uplift situations (expansive zones)
- Urban areas for future large and huge construction near the existing building.

CLASSIFICATION OF PILES

1. Based on material

- Timber piles
- Steel piles
- Concrete piles
- Composite piles (steel + concrete)

2. Based on method of installation

- Driven piles ---- (i) precast (ii) cast-in-situ.
- Bored piles.

3. Based on the degree of disturbance

- Large displacement piles (occurs for driven piles)
- Small displacement piles (occurs for bored piles)

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Lecture 20 : Introduction [Section 20.2 : Design Methodology For Piles]

POINTS TO BE CONSIDERED FOR CHOOSING PILES

- Loose cohesion less soil develops much greater shaft bearing capacities if driven large displacement piles are used.
- Displacement effect enhanced by tapered shafts.
- Potential increased of shaft capacities is undesirable if negative friction is to be feared. (Negative friction is also called drag down force)
- High displacement piles are undesirable in stiff cohesive soils, otherwise excessive heaving takes place.
- Encountered with high artesian pressures on cased piles should be excluded. (Mainly for bridges and underwater construction)
- Driven piles are undesirable due to noise, damage caused by vibration, ground heaving.
- Heavy structures with large reactions require high capacity piles and small diameter cast-in-situ piles are inadequate.

PILE CLASSIFICATION

- Friction piles.
- End bearing piles.
- Compaction piles. (Used for ground movement, not for load bearing)
- Tension piles/Anchored piles.(To resist upliftment)
- Batter piles (Inclined) --- +ve and -ve.

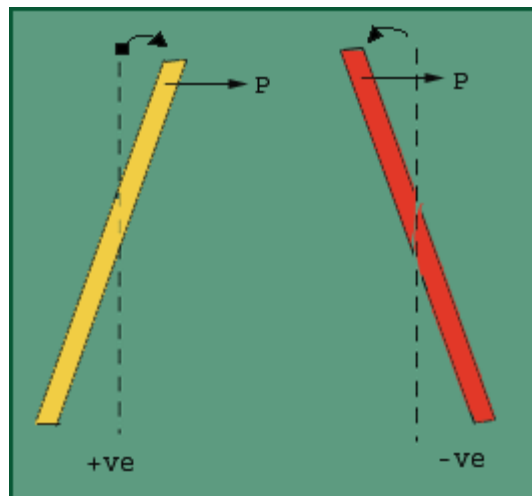


Fig. 5.1 Direction of load is same as the direction of batter. (Rotation of pile)

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Lecture 20 : Introduction [Section 20.2 : Design Methodology For Piles]

- Raymond piles. (Driven cast-in-situ piles, first tapered shell is driven and then cast)
- Franki Piles (Driven cast-in-situ piles, first casing is driven upto 2m depth, then cast a block within that casing and then drive the block. When it reaches the particular depth, take out the casing and cast the piles.)
- Underreamed piles (bored cast-in-situ piles, bulbs used, hence not possible to install in loose sand and very soft clays.)

PILES IN CLAY

Zone of influence

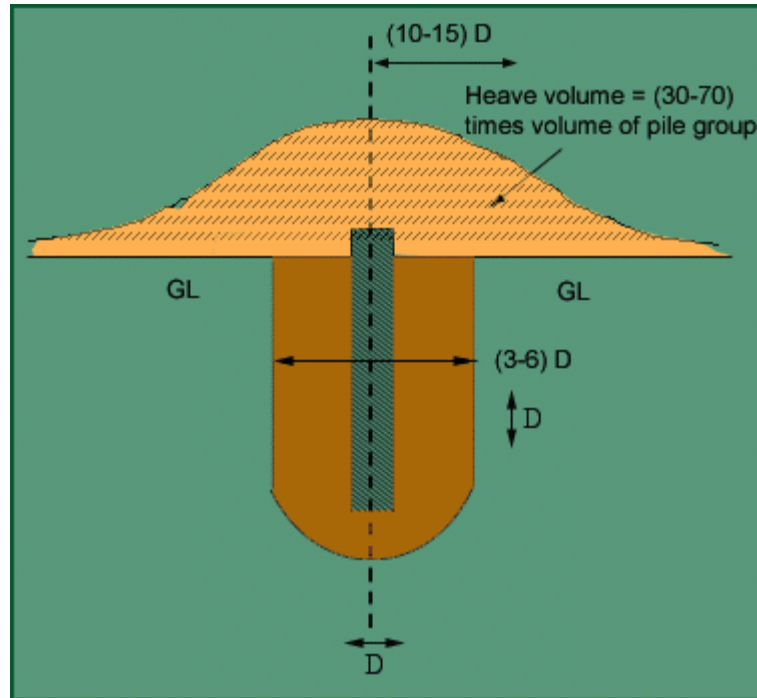


Fig.5.2 Driven piles in clay

The heaving effect can be felt upto $(10 - 15) D$ from the centerline of the pile. Due to driving load, pressure is generated and as a result heaving occurs. Afterwards with time, the heaved part gets consolidated and strength gradually increases as the material regains shear strength within 3 – 6 months time after the installation of the pile. This regain of strength is called thixotrophy.

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Lecture 20 : Introduction [Section 20.2 : Design Methodology For Piles]

On the first day some part of the pile will be driven and on the second day some part of the pile may move up due to the gain of shear strength. This is known as the *wakening of the pile*. By the driving force, the extra pore pressure generated is $(5 - 7)$ times the C_u of the soil. Bearing capacity of the pile is $9 C_u$. Hence due to this property, maximum single length of the pile theoretically can be upto 25m but 10-12m is cast at a time. Then by splicing technique the required hired length of the pile is obtained. Special types of collars are used so that the splices become weak points. Concrete below the grade M20 is never used.

| Pile Diameter | Maximum length (m) |
|---------------|--------------------|
| 250 | 12 |
| 300 | 15 |
| 350 | 18 |
| 400 | 21 |
| 450 | 25 |

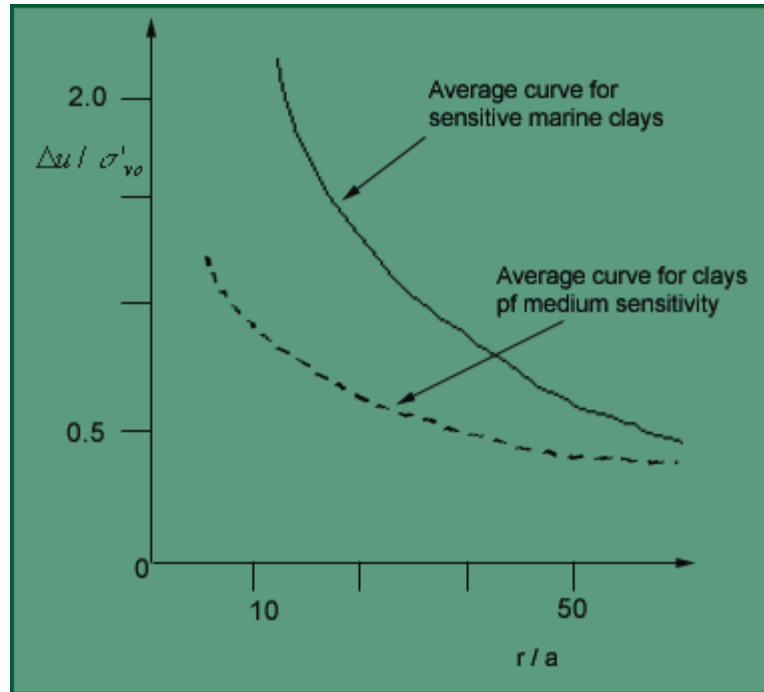
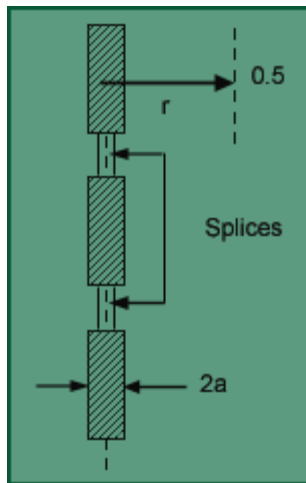


Fig.5.3 Generation of

PILES IN SAND

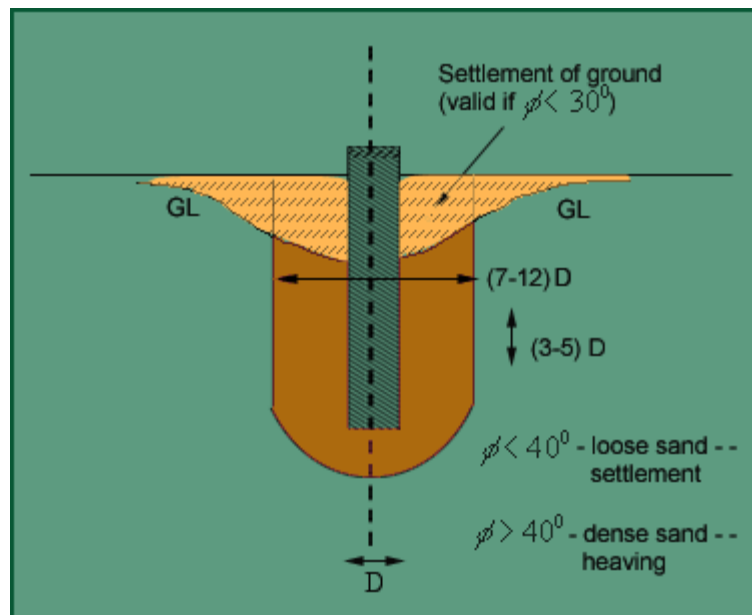


Fig.5.4a Driven piles in loose sand

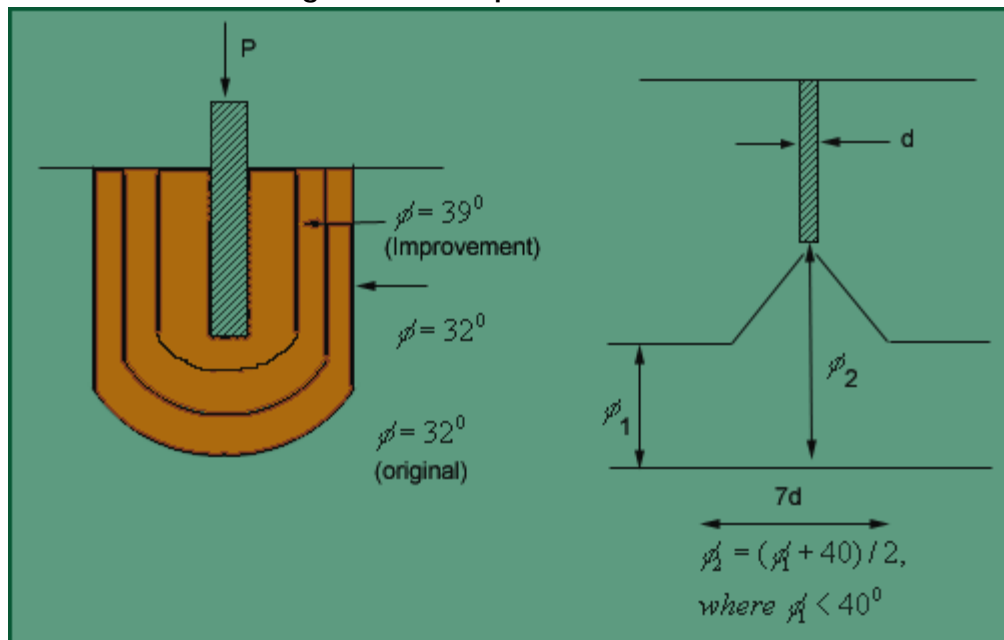


Fig.5.4b Improvement in f due to pile driving

SETTLEMENT OF PILE GROUPS

Assume 2V:1H dispersion for settlement of pile groups.

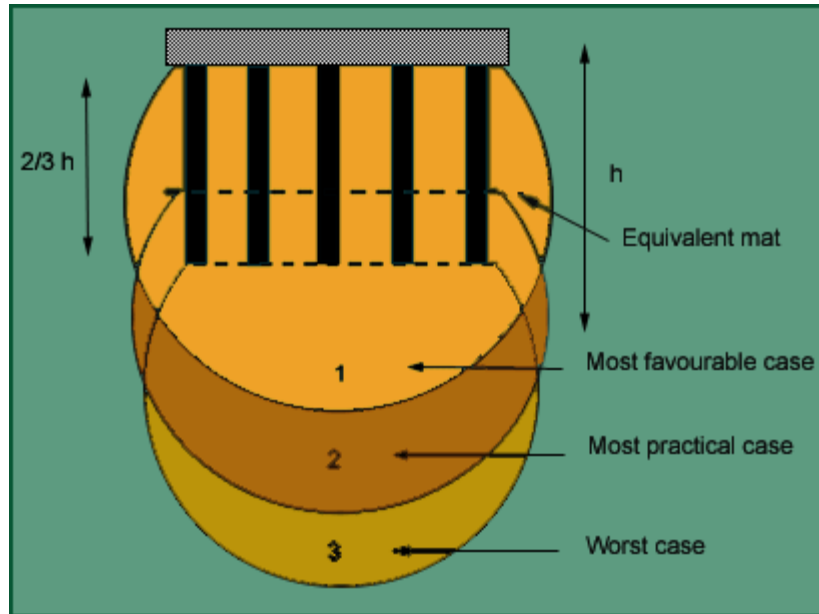


Fig.5.5 Settlement of pile groups

CODAL PROVISION

SAFE LOAD ON PILES/PILE GROUPS (Ref. IS: 2911 Part IV 1979)

Single pile:

1. Safe load = Least of the following loads obtained from routine tests on piles :
 - 2/3 of the final load at which total settlement is 12mm.
 - 50% of the final load at which settlement is 10% of the pile dia. (for uniform dia. piles) and 7.5% of bulb dia. (for Underreamed piles)
 - 2/3 of the final load at which net settlement is 6mm.
 - Consider pile as column and find the total compressive load depending on the grade of concrete and dimensions. Eg. Consider a 300mm dia pile made of M20 concrete. $\sigma_{cc} = 5 \text{ N/mm}^2$.

Therefore, ultimate load = $\frac{\pi}{4} \times 300^2 \times 5 = 353.4 \text{ KN}$.

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Lecture 20 : Introduction [Section 20.2 : Design Methodology For Piles]

Group of piles:

2. Safe load = Least of the following loads obtained from
 - Final load at which total settlement of pile group attains a value of 25mm.
 - 2/3 of the final load at which the total settlement attains a value of 40mm.

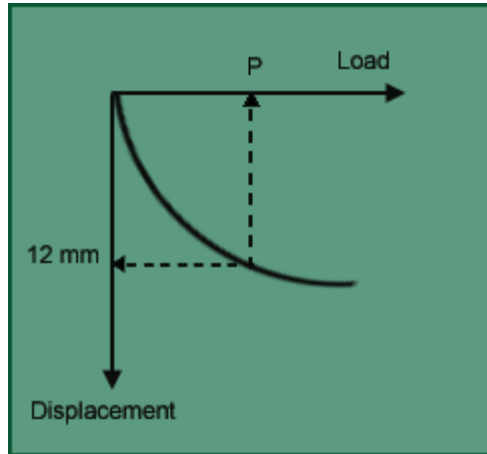


Fig.6 Load vs Displacement Curve for piles

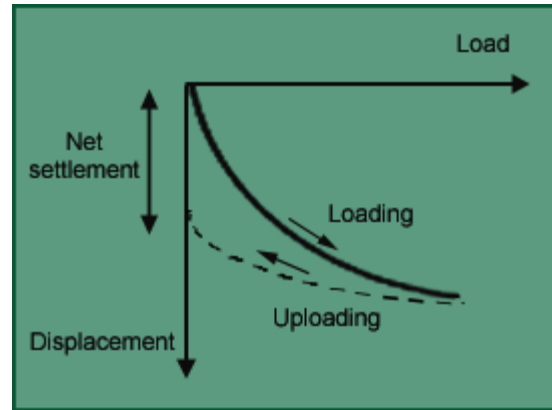


Fig.5.7 Loading and unloading curve for piles

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Lecture 20 : Introduction [Section 20.2 : Design Methodology For Piles]

Recap

In this section you have learnt the following.

- Requirement For Deep Foundations
- Classification Of Piles
- Points To Be Considered For Choosing Piles
- Pile Classification
- Piles In Clay
- Piles In Sand
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- Codal Provision

Congratulations, you have finished Lecture 20. To view the next lecture select it from the left hand side menu of the page

Module 5 : Design of Deep Foundations

Lecture 21 : Types of pile load test [Section 21.1 : Different types of pile load test]

Objectives

In this section you will learn the following

- Terminologies
- Maintained Load Test
- Constant Rate Of Penetration Test

Module 5 : Design of Deep Foundations

Lecture 21 : Types of pile load test [Section 21.1 : Different types of pile load test]

Terminologies

- Test pile -pile chosen for pile load test only. It may or may not be a working pile. It can work as a working pile if routine load test is carried out on it is up to half of the safe load.
- Initial test -it is done on test piles to determine the safe load or allowable load or ultimate load bearing capacity.
- Trial pile -minimum two initial test should be conducted and half percentage of the total number of piles used in a project must be subjected to routine test. For largely varying strata and for important structure, it is up to 2%.

Different types of pile load test

1. Maintained Load Test

Load is maintained in 10 or 8 steps and maintained for sometime to exact settlement.(step load test).To test a single pile at least 3 dial gauges should be used and 4 dial gauges for group piles.loading rate is 0.1mm/hr. For clay it is 0.02mm/hr.loadf is maintained for atleast 2hrs. safe load is maintained for 24 hrs.

2. Constant Rate Of Penetration Test

Generally used for soft clays and penetration is done for constant rate of 0.75mm/min.

Cyclic load test

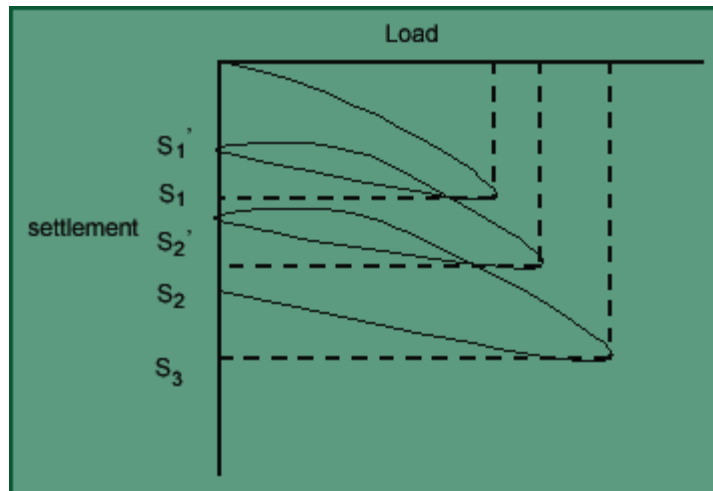


Fig. 5.8 Load vs settlement

Lecture 21 : Types of pile load test [Section 21.1 : Different types of pile load test]

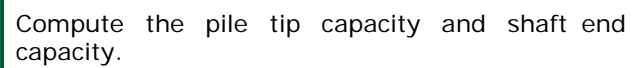


Fig-5.10 Q Calculate the Value of Tip Resistance and Shaft Resistance in Cyclic Pile Load Test.

Module 5 : Design of Deep Foundations

Lecture 21 : Types of pile load test [Section 21.1 : Different types of pile load test]

Recap

In this section you have learnt the following.

- Terminologies
- Maintained Load Test
- Constant Rate Of Penetration Test

Module 5 : Design of Deep Foundations

Lecture 21 : Types of pile load test [Section 21.2 : Problem]

Objectives

In this section you will learn the following

- Problem

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Lecture 21 : Types of pile load test [Section 21.2 : Problem]

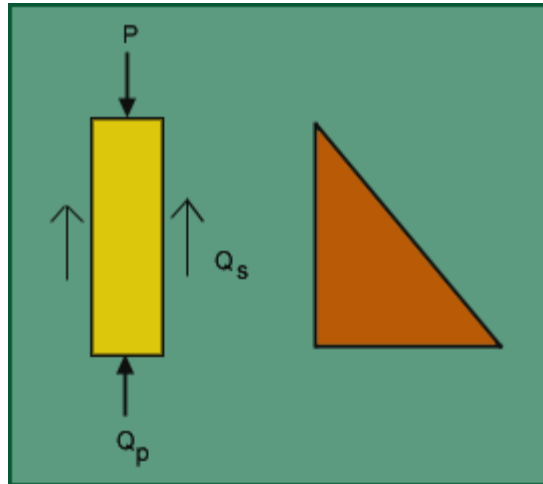


Fig. 5.11 Variation Of Q_s with Depth Of pile

Here Q_s = Shaft Resistance, P = Applied Load, Q_p = Tip Resistance, S_{es} Settlement in soil, S_{ep} = Settlement in Pile.

Given Data

| Loads (Ton) | Total Settlement (mm) | Net Settlement (mm) | Elastic Settlement (mm) |
|-------------|-----------------------|---------------------|-------------------------|
| 0 | 0 | 0 | 0 |
| 5 | 2.5 | 0.5 | 2 |
| 10 | 4 | 1.25 | 2.75 |
| 20 | 9.5 | 3.75 | 5.75 |
| 30 | 16.5 | 8 | 8.5 |
| 40 | 27 | 14 | 13 |
| 50 | 40.5 | 21 | 19.5 |
| 60 | 61 | 31 | 30 |

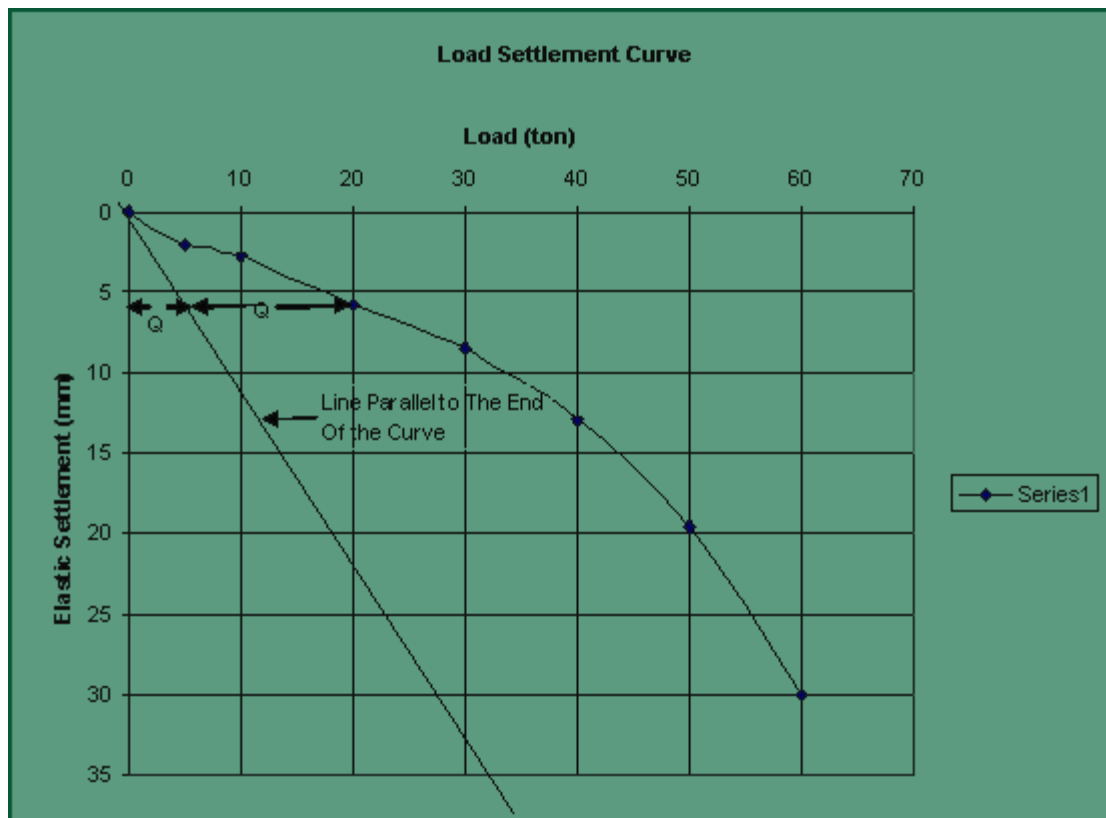
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Lecture 21 : Types of pile load test [Section 21.2 : Problem]

| Initially | $\Delta L = 0$ | $S_{es} = S_{ep}$ | | |
|-----------|----------------|-------------------|---------------------------------|-------------------------|
| Loads | Q_p (ton) | Q_s (ton) | $\Delta L = (P(Q_s / 2)).L/AE$ | Elastic Settlement (mm) |
| 0 | 0 | 0 | 0.00 | 0.00 |
| 5 | 1.75 | 3.25 | 0.35 | 1.65 |
| 10 | 2.5 | 7.5 | 0.66 | 2.09 |
| 20 | 5.25 | 14.75 | 1.33 | 4.42 |
| 30 | 8.25 | 21.75 | 2.01 | 6.49 |
| 40 | 12.5 | 27.5 | 2.76 | 10.24 |
| 50 | 18.75 | 31.25 | 3.61 | 15.89 |
| 60 | 29 | 31 | 4.67 | 25.33 |

Load Vs Elastic Settlement Curve (Total Elastic Recovery of the Pile Top)

First Trial



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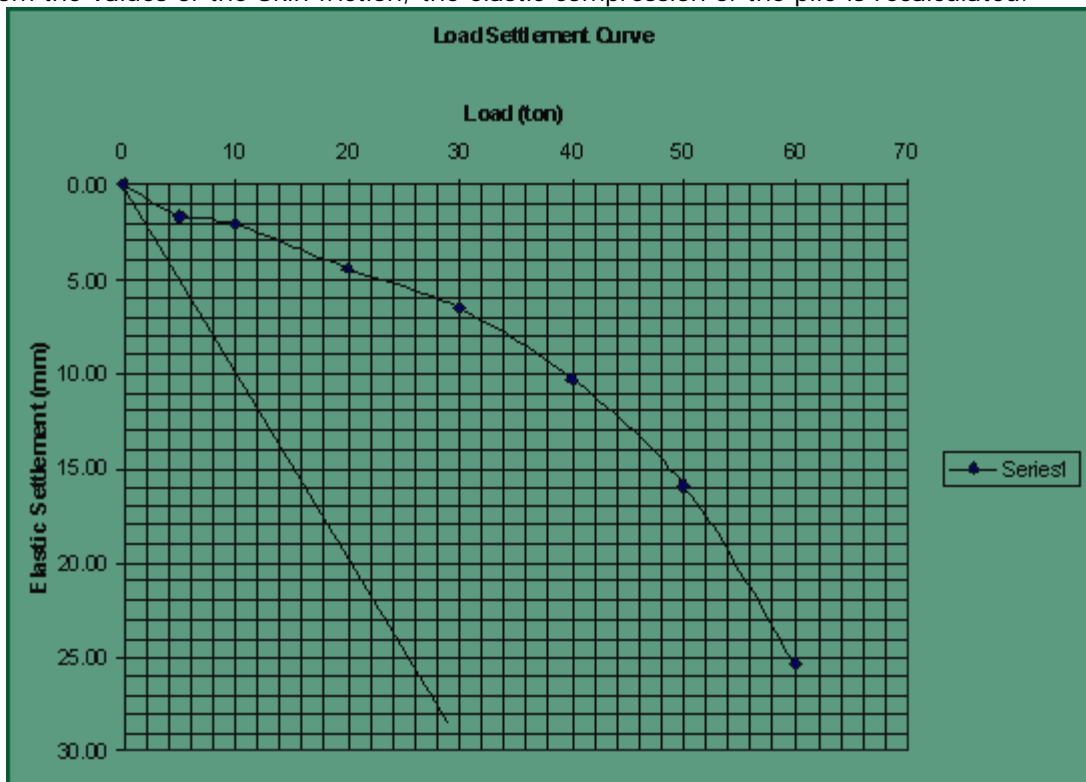
Lecture 21 : Types of pile load test [Section 21.2 : Problem]

Second Trial

| Loads (ton) | Q_p (ton) | Q_s (ton) | $\Delta L = (P - (Q_s / 2)) \cdot L / AE$ | Elastic Settlement (mm) |
|-------------|-------------|-------------|-------------------------------------------|-------------------------|
| 0 | 0 | 0 | 0.00 | 0.00 |
| 5 | 1.5 | 3.5 | 0.34 | 1.66 |
| 10 | 2 | 8 | 0.63 | 2.12 |
| 20 | 4.5 | 15.5 | 1.29 | 4.46 |
| 30 | 6.5 | 23.5 | 1.92 | 6.58 |
| 40 | 10.25 | 29.75 | 2.64 | 10.36 |
| 50 | 16 | 34 | 3.47 | 16.04 |
| 60 | 25.5 | 34.5 | 4.49 | 25.51 |

Load Vs Elastic Settlement of soil Curve For Second Trial

Second Curves gives new values of skin friction and point resistance which are more accurate than earlier values. From the values of the Skin friction, the elastic compression of the pile is recalculated.



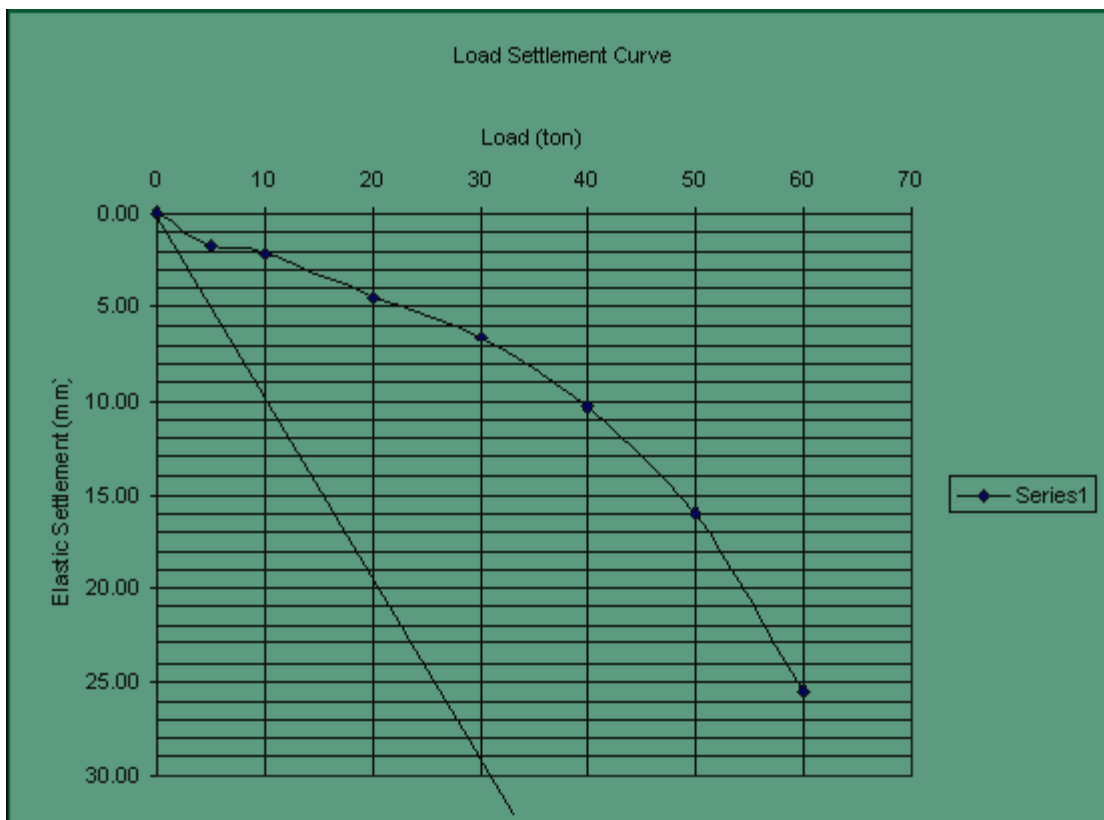
Module 5 : Design of Deep Foundations

Lecture 21 : Types of pile load test [Section 21.2 : Problem]

Third Trial

| Load (ton) | Q_p (ton) | Q_s (ton) | $\Delta L = (P - (Q_s / 2)) \cdot L / AE$ | Elastic Settlement (mm) |
|------------|-------------|-------------|-------------------------------------------|-------------------------|
| 0 | 0 | 0 | 0.00 | 0.00 |
| 5 | 1.75 | 3.25 | 0.35 | 1.65 |
| 10 | 2.25 | 7.75 | 0.64 | 2.11 |
| 20 | 4.75 | 15.25 | 1.30 | 4.45 |
| 30 | 6.75 | 23.25 | 1.93 | 6.57 |
| 40 | 10.75 | 29.25 | 2.66 | 10.34 |
| 50 | 16.75 | 33.25 | 3.50 | 16.00 |
| 60 | 25.5 | 34.5 | 4.49 | 25.51 |

Load Vs Elastic Settlement Curve For Third Trial



Final Value Of Tip Resistance & Skin Resistance

Ans

| Q_p (ton) | Q_s (ton) |
|-------------|-------------|
| 0 | 0 |
| 1.75 | 3.25 |
| 2.25 | 7.75 |
| 4.75 | 15.25 |
| 6.75 | 23.25 |
| 10.75 | 29.25 |
| 16.75 | 33.25 |
| 25.5 | 34.5 |

Module 5 : Design of Deep Foundations

Lecture 21 : Types of pile load test [Section 21.2 : Problem]

Recap

In this section you have learnt the following.

- Problem

Congratulations, you have finished Lecture 21. To view the next lecture select it from the left hand side menu of the page

Module 5 : Design of Deep Foundations

Lecture 25 : Pile Groups [Section 25.1 : Different types of pile groups]

Objectives

In this section you will learn the following

- Different types of pile groups

Module 5 : Design of Deep Foundations

Lecture 25 : Pile Groups [Section 25.1 : Different types of pile groups]

Different types of pile groups

The different types of pile groups are shown below:

Fig.5.41 Different Pile groups

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Lecture 25 : Pile Groups [Section 25.1 : Different types of pile groups]

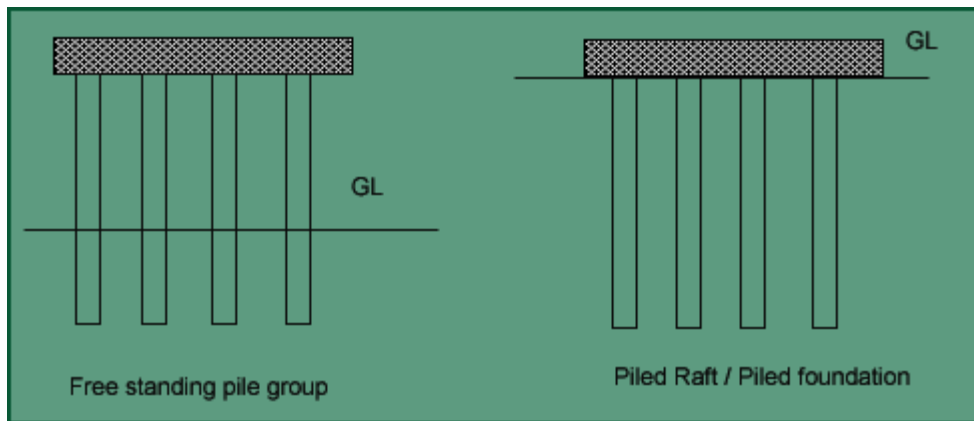


Fig. 5.42 Pile Groups

Usually the GL and the base of the piled foundation are made flush with one another. As a result the Pile foundation has a higher load bearing capacity and lesser settlement. This is because the pile in piled raft foundation are in contact with more soil than the free standing pile group.

Module 5 : Design of Deep Foundations

Lecture 25 : Pile Groups [Section 25.1 : Different types of pile groups]

Recap

In this section you have learnt the following.

- Different types of pile groups

Module 5 : Design of Deep Foundations

Lecture 25 : Pile Groups [Section 25.2 : Group Deficiency Factor (η) ; Block Failure Criteria]

Objectives

In this section you will learn the following

- Group Deficiency Factor
- Block Failure Criteria

Module 5 : Design of Deep Foundations

Lecture 25 : Pile Groups [Section 25.2 : Group Deficiency Factor (η) ; Block Failure Criteria]

GROUP DEFICIENCY FACTOR (η)

For bearing capacity,

η = (Ultimate load capacity of group) / Sum of ultimate load capacity of individual piles)

$$\eta = P_{ug} / nP_u$$

The aim of the designer is to make η = 100% or 1.0

However for sandy soils, $\eta > 1.0$

This is because, the soils gets densified due to driving of piles in sandy soil and hence soil strength properties increase. Therefore while starting a design we start with

$P_{ug} = nP_u$. Since P_u remains same, therefore calculate nP_u based on the no. of piles to be used. Compare with P_{ug} . In design take whichever value is minimum.

Alternatively, P_{ug} is expressed as function of spacing of piles (s) and equated with nP_u .

Calculate 's' for η = 1.0. Usually, 's' is taken as 3d to 8d depending on the soil and pile parameters. Lower the value of s, higher is the lower is the efficiency.

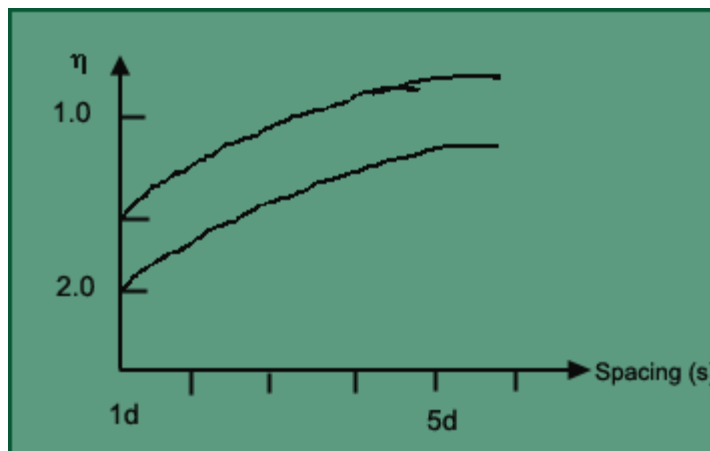


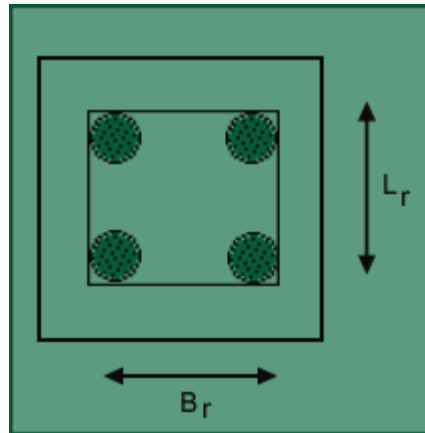
Fig. 5.43 Variation of efficiency of pile group with spacing of piles

Module 5 : Design of Deep Foundations

Lecture 25 : Pile Groups [Section 25.2 : Group Deficiency Factor (η) ; Block Failure Criteria]

Block Failure Criteria

In this case it is assumed that the pile group fails as a block.



$$P_{ug} = B_r L_r c N_c + (B_r + L_r) L_r c' \quad \text{----- (45)}$$

Where, c is the average cohesion at base.

c' is the average cohesion along pile length.

The first term denotes the P_{ubg} (base capacity)

and the 2 nd term denotes P_{usg} (shaft resistance) .

$$\xi = 0.5$$

$$N_c = 9, \text{ depends on } L / B_r .$$

Module 5 : Design of Deep Foundations

Lecture 25 : Pile Groups [Section 25.2 : Group Deficiency Factor (77) ; Block Failure Criteria]

Recap

In this section you have learnt the following.

- Group Deficiency Factor
- Block Failure Criteria

Module 5 : Design of Deep Foundations

Lecture 25 : Pile Groups [Section 25.3 : Design value of ultimate load ; Settlement of a pile]

Objectives

In this section you will learn the following

- Design value of ultimate load
- Settlement of a pile

Module 5 : Design of Deep Foundations

Lecture 25 : Pile Groups [Section 25.3 : Design value of ultimate load ; Settlement of a pile]

Design value of ultimate load

For piled raft foundation : The minimum of block failure value and the sum of individual pile capacities.

For free standing pile : The minimum of block failure value and 2/3 rd the sum of individual pile capacities.

Minimum of block failure value & 2/3 of sum of individual pile capacity

Bearing capacity of block plus bearing capacity of

1. Extra width of pile group and times bearing capacity of each pile + bearing capacity of (area of cap – area of pile, n)

$$2. P_{ug} = n(c_a A_s + A_b c_b N_c) + n c_c (B_c L_c - \frac{n \pi d^2}{4}) \quad \text{-----(46)}$$

where N_c^c is the N_c value at cap base, c_c is the cohesion, and B_c , L_c are width and length of cap

Settlement of a pile

$$\text{At centre of pile, } w_i = \frac{(3 - 4\nu)(1 - \nu)}{4E_s(1 - \nu)} p a \quad \text{-----(47)}$$

where p is the perimeter of the pile and a is the radius from centre of the pile

In reality, settlement of a single pile is, $w = w_s + w_{ps} + w_{pp}$

where w is the total settlement, w_s is the settlement due to axial deformation of pile shaft because of pile material, w_{ps} is the settlement of a pile tip or point due to load transmitted along pile shaft, and w_{pp} is the settlement pile tip due to load transmitted at pile tip.

Module 5 : Design of Deep Foundations

Lecture 25 : Pile Groups [Section 25.3 : Design value of ultimate load ; Settlement of a pile]

$$w_s = \frac{(P_{ub} + \xi P_{us})L}{A_p E_p} \quad w_{pp} = \frac{q_{ub} d}{E_{soil}} (1 - \xi^2) I_{ub} \quad \text{----- (48)}$$

$$w_{ps} = \frac{P_{us}}{P L} \frac{d}{E_s} (1 - \xi^2) I_{us} \quad \text{----- (49)}$$

$$q_{ub} = \frac{P_{ub}}{\text{area}} = \text{ultimate at base} \quad \text{----- (50)}$$

d is the diameter of the pile and I_{ub} is the influence factor for tip of the pile, p is the perimeter

Generally $\xi = 0.5$ and it is a function of skin friction (f_s)

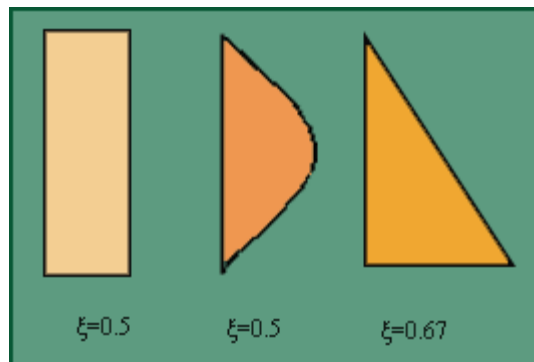


Fig 5.44 Variation of ξ

Module 5 : Design of Deep Foundations

Lecture 25 : Pile Groups [Section 25.3 : Design value of ultimate load ; Settlement of a pile]

Compression pile

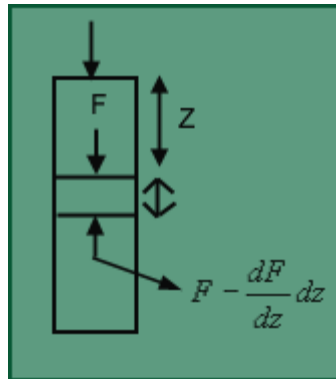


Fig 5.45 Free body diagram of a compression pile

Z is at any depth, w is the settlement

$$\text{Strain} = \frac{dw}{dz}$$

$$\text{Stress} = E \frac{dw}{dz}$$

$$\text{Force} = EA \frac{dw}{dz}$$

$$F_s = Ps = \epsilon dz = sk_s w dz \text{ Where } s \text{ is the perimeter}$$

Module 5 : Design of Deep Foundations

Lecture 25 : Pile Groups [Section 25.3 : Design value of ultimate load ; Settlement of a pile]

$$\text{Elemental force} = \frac{dF}{dz} dz$$

$$\frac{dF}{dz} dz = sk_s w dz \quad \text{or} \quad \frac{dF}{dz} = sk_s w$$

$$\text{Now, } \frac{dF}{dz} = \frac{d}{dz} \left(EA \frac{dw}{dz} \right) = EA \frac{d^2 w}{dz^2}$$

$$EA \frac{d^2 w}{dz^2} = sk_s w$$

$$EA \frac{d^2 w}{dz^2} - sk_s w = 0 \quad \frac{d^2 w}{dz^2} - \frac{sk_s w}{EA} = 0$$

$$\frac{d^2 w}{dz^2} - \lambda^2 w = 0$$

where $\lambda = \sqrt{\frac{sk_s}{EA}}$ where k_s is the sub grade modulus, E is elasticity modulus of pile material and A is the cross sectional area.

General solution is $w = c_1 e^{-\lambda z} + c_2 e^{-\lambda z}$ the values of c_1 & c_2 are obtained by applying boundary conditions.

Module 5 : Design of Deep Foundations

Lecture 25 : Pile Groups [Section 25.2 : Group Deficiency Factor (η) ; Block Failure Criteria]

Recap

In this section you have learnt the following.

- Design value of ultimate load
- Settlement of a pile

Module 5 : Design of Deep Foundations

Lecture 25 : Pile Groups [Section 25.4 : Problem]

Objectives

In this section you will learn the following

- Problem

Module 5 : Design of Deep Foundations

Lecture 25 : Pile Groups [Section 25.4 : Problem]

Problem

Determine the total settlement for the pile group shown in the fig. 5.46

Ans :

The load is assumed to be distributed in a ratio of 1H:4V on the equivalent mat at a distance of (2/3)rd of the pile length ie (2/3*9 =) 6m. Ref. fig.5.46.

$$\text{Width of equivalent map} = 3 + 2 * (6/4) = 6\text{m}$$

$$\text{Length of equivalent map} = 2.1 + 2 * (6/4) = 5.1\text{m}$$

$$\text{Udl acting on the equivalent map} = 150/(6 * 5.1) = 4.902 \text{ t/m}^2$$

Below the equivalent map it is analyzed as shallow foundation, by dividing it into no. of layers as shown in fig. 5.47.

The Effective overburden pressure at center of each layer:

$$\sigma_1' = (16.4 \times 2) + (2 \times 19.1) + (5.5 \times (19.1 - 9.81)) = 122.095 \text{ KN / m}^2$$

$$\sigma_2' = 122.095 + (3 \times (19.1 - 9.81)) = 149.965 \text{ KN / m}^2$$

$$\sigma_3' = 149.965 + (1.5 \times (19.1 - 9.81)) + (1.25 \times (20 - 9.81)) = 176.64 \text{ KN / m}^2$$

$$\sigma_4' = 176.64 + (1.25 \times (20 - 9.81)) = 189.375 \text{ KN / m}^2$$

Module 5 : Design of Deep Foundations

Lecture 25 : Pile Groups [Section 25.4 : Problem]

The settlement at center of each layer (s) =
$$\frac{C_c}{1+e_0} \Delta H \frac{\sigma' + \Delta \sigma'}{\sigma'}$$

Where,

C_c = compression index of that layer,

e_0 = initial void ratio,

ΔH = height of the layer,

σ' = effective overburden pressure at center of the layer,

$\Delta \sigma'$ = increase in the effective pressure due to load.

Total settlement will be the summation of the settlement at center of each layer. The calculations are given in tabular form in table 7.

Table: 5.7 Final settlements

| Layer no. | Depth of the center of the layer (Z) (m) | Effective overburden pressure (s') (KN/ m^2) | Increase in effective pressure $\Delta \sigma' = \frac{1500}{(6 + Z)(5.1 + Z)}$ (KN / m^2) | ΔH (m) | Settlement at center of each layer (mm) |
|-----------|----------------------------------------------|----------------------------------------------------|--------------------------------------------------------------------------------------------------|-------------------|--------------------------------------------|
| 1 | 1.5 | 122.095 | 30.3 | 3 | 38.94 |
| 2 | 4.5 | 149.965 | 14.88 | 3 | 16.62 |
| 3 | 7.25 | 176.64 | 9.17 | 2.5 | 6.58 |
| 4 | 9.75 | 189.375 | 6.14 | 2.5 | 4.1 |

Total settlement = 66.24 mm

Module 5 : Design of Deep Foundations

Lecture 25 : Pile Groups [Section 25.4 : Problem]

Ans Total settlement is 66.24 mm.

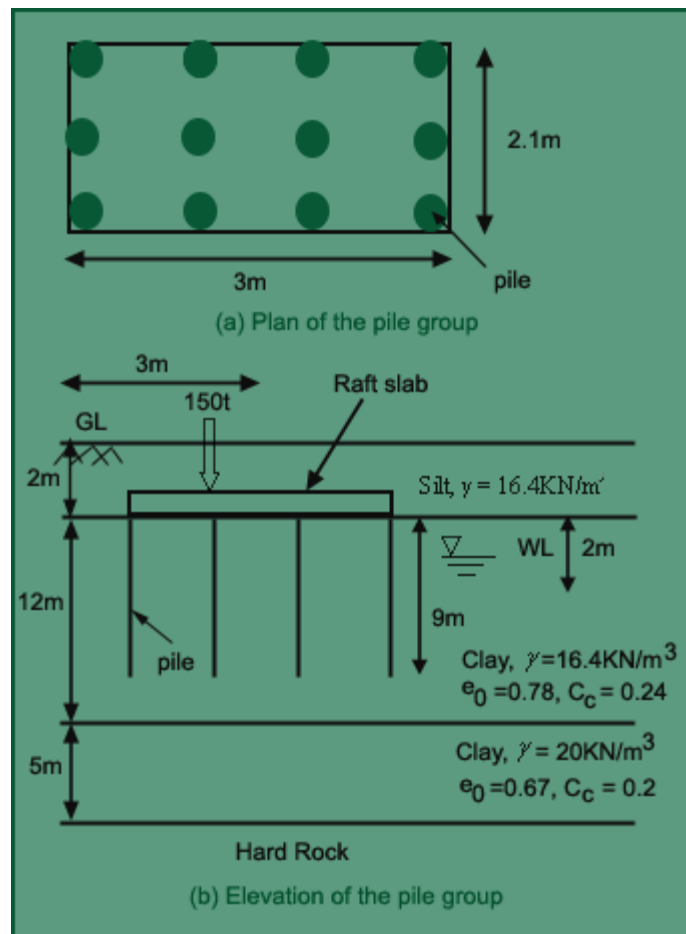


Fig. 5.46 Pile group

Module 5 : Design of Deep Foundations

Lecture 25 : Pile Groups [Section 25.4 : Problem]

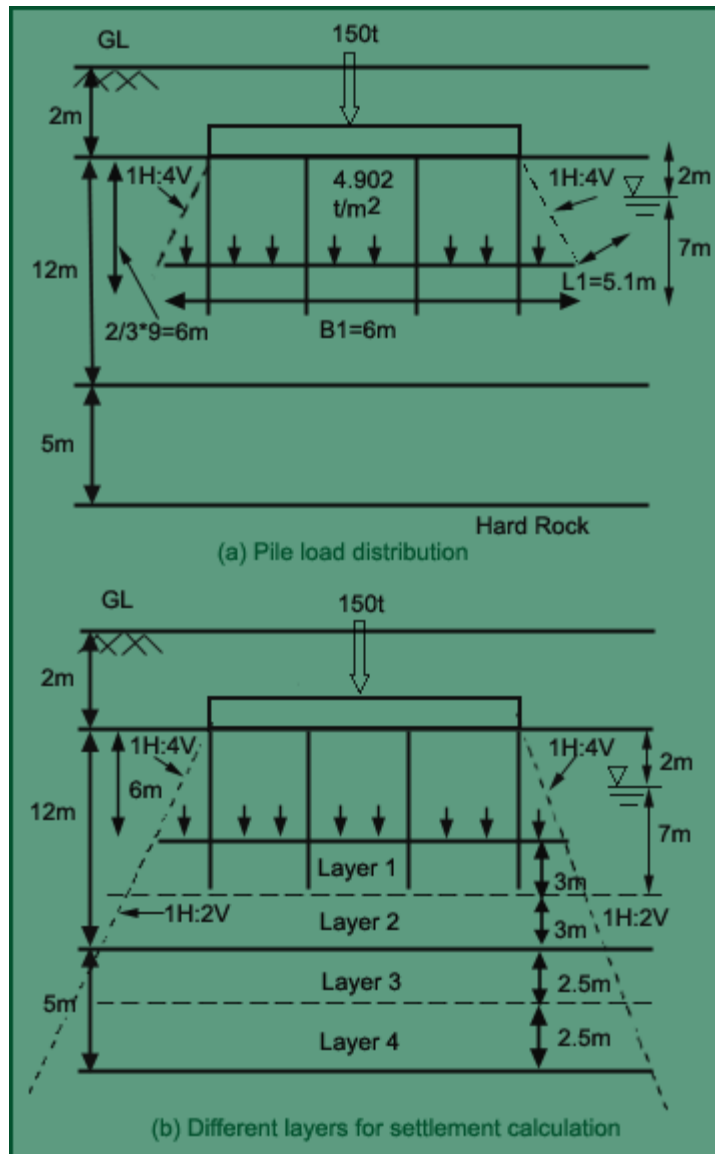


Fig. 547 Pile group settlement analysis

Module 5 : Design of Deep Foundations

Lecture 25 : Pile Groups [Section 25.2 : Group Deficiency Factor (η) ; Block Failure Criteria]

Recap

In this section you have learnt the following.

- Problem

Module 5 : Design of Deep Foundations

Lecture 25 : Pile Groups [Section 25.5 : Negative Skin Friction]

Objectives

In this section you will learn the following

- Negative Skin Friction

Module 5 : Design of Deep Foundations

Lecture 25 : Pile Groups [Section 25.5 : Negative Skin Friction]

Negative Skin Friction

In the weak soil zone pile moves up with respect to surrounding soil. It is same as tension uplift, i.e. surrounding soil moves down with respect to pile. This is known as negative skin friction as shown in figure. It is also known as downdrag phenomenon and occurs when the soil layer surrounding the pile settles more than the pile. Negative skin friction develops when the soft or loose soil strata are located anywhere in between and the pile shaft is subjected to compressive load. Also negative skin friction develops due to increase in effective stresses.

F.S. = (Ultimate Pile capacity – Negative Skin Friction force) / Working Load.

The net effect is to reduce the F.S.

$$F.S. = (Q_u - Q_{nf}) / Q_w \quad \text{-----(45)}$$

$$Q_{nf} = P \cdot D_n \cdot C, \text{ for cohesive soils.}$$

where P is the perimeter, D_n is the depth of compressible layer, C is cohesion.

$$Q_{nf} = 0.5 P D_n^2 \gamma \tan \delta, \text{ for cohesionless soil.}$$

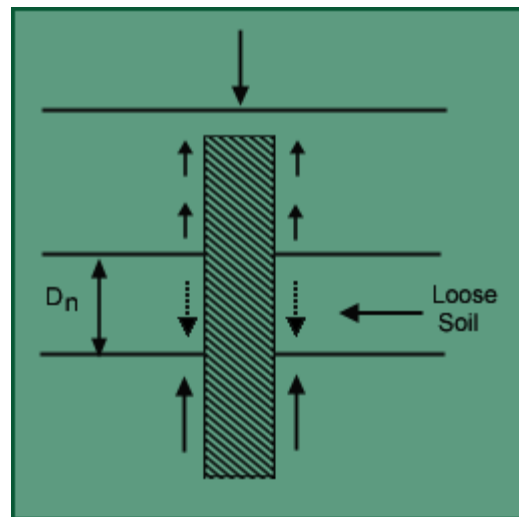


Fig. 5.48 Development of negative skin friction on piles

Module 5 : Design of Deep Foundations

Lecture 24 : Under Reamed Pile [Section 24.1 : Introduction]

Objectives

In this section you will learn the following

- Introduction

Lecture 24 : Under Reamed Pile [Section 24.1 : Introduction]

The diagram illustrates the cross-section of a bulb pile. It features a central vertical stem of diameter D and a wider bulb section of diameter D_u . The stem is labeled 'Stem' and the bulb is labeled 'Bul'. A vertical force P is applied at the top of the stem. The minimum length of the pile is denoted as $l_{min} = l_1$. The length of the bulb section is l_u . The bucket length is l_2 . The diagram also shows the centerlines C_1 and C_2 for the stem and bulb sections respectively.

l_u = Length of Bulb

D_u = Diameter of Bulb

D = Diameter of the Pile

According to IS code

D_u = 2 to 2.5 times of D

The diagram illustrates a pile foundation with the following dimensions and features:

- Ground Level (G.L.):** Indicated by a horizontal line at the top.
- Pile Diameter:** The main shaft of the pile has a diameter D .
- Section 1:** A section of length 2 m (or $2 \times D_u$, whichever is minimum) is shown above the first soil layer.
- Layer 1:** The first soil layer has a thickness of $1.5 D_u$. The pile diameter in this section is D_u . The coefficient of soil resistance is C_1 .
- Section 2:** A section of length $(1.25 \text{ to } 1.5) \times D_u$ is shown within Layer 1.
- Layer 2:** The second soil layer has a thickness of $1.5 D$. The pile diameter in this section is $1.5 D$. The coefficient of soil resistance is C_2 .
- Bearing:** Indicated by upward arrows at the bottom of the pile and at the interface between Layer 1 and Layer 2.

Fig 5.40 Multiple Under Reamed Pile

Module 5 : Design of Deep Foundations

Lecture 24 : Under Reamed Pile [Section 24.1 : Introduction]

Under reamed piles are bored cast-in-situ concrete piles having **one or more** number of **bulbs formed** by enlarging the pile stem. These piles are best suited in soils where considerable **ground** movements occur due to seasonal variations, filled up grounds or in soft soil strata. Provision of under reamed bulbs has the advantage of increasing the bearing and uplift capacities. It also provides better anchorage at greater depths. These piles are efficiently used in machine foundations, over bridges, electrical transmission tower foundation sand water tanks. Indian Standard IS 2911 (Part III) - 1980 covers the design and construction of under reamed piles having one or more bulbs. According to the code the diameter of under reamed bulbs may vary from 2 to 3 times the stem diameter depending upon the feasibility of construction and design requirements. The code suggests a spacing of 1.25 to 1.5 times the bulb diameter for the bulbs. An angle of 45° with horizontal is recommended for all under reamed bulbs. This code also gives Mathematical expressions for calculating the bearing and uplift capacities.

From the review of the studies pertaining to under reamed piles, it can be seen that ultimate bearing capacity of piles increases considerably on provision of under- reamed bulbs (Neumann and P&g, 1955, Subash Chandra and Kheppar, 1964, Patnakar, 1970 etc.). Pile load capacity was found to vary with the number of bulbs and with the spacing ratio S / D_u or S/d adopted (where S = distance between the piles, D_u = diameter of under reamed bulbs and d = diameter of piles). Table summarizes the various recommendations made for the selection of S / D_u and S/d for the optimum pile load capacity. It can be seen that some of these recommendations differ from those given in IS 2911 (Part III), 1980.

Table: 5.6 of recommendations for S / D_u and S/d for the optimum pile load capacity

| Recommendations of S / D_u & S/d values for under reamed piles | | | |
|--------------------------------------------------------------------|----------------------------|-------------------------------------------------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------------------------------|
| s.no. | Reference | No. of Bulbs | Spacing |
| 1. | Patnakar (1970) | Pile capacity for one bulb increases 25 percent, for two bulbs 600 percent, and for three bulbs 700 percent over simple pile. | For optimum capacity two bulbs $S / D_u = 6$ or $S/d = 15$, for three bulbs, $S / D_u = 5$ or $S/d = 12$. |
| 2 | Agarwal and Jain (1971) | - | For optimum capacity $S / D_u = 1.25$ to 1.5 |
| 3 | Sonapal and Thakkar (1977) | - | For optimum capacity $S / D_u = 2.5$ |
| 4 | IS 2911(Part III 1980) | More than two bulbs are not advisable | $S / D_u = 1.25$ to 1.5 |
| 5 | Ray and Raymond (1983) | - | Maximum value of $S / D_u = 1.24$ to 1.5 |

Module 5 : Design of Deep Foundations

Lecture 24 : Under Reamed Pile [Section 24.1 : Introduction]

The choice of an under-reamed pile in unstable or water-bearing ground is generally to be avoided. There is a danger of collapse of the under-ream, either when personnel are down the hole, or during concreting.

Important Notes: On the basis of limited experimental studies conducted on model under reamed piles in cohesion less soil the following conclusions are drawn.

1. By providing under reamed bulbs the ultimate load capacities of piles increases significantly.
2. The ultimate load bearing capacities of the under reamed piles with angle of under reamed bulbs of 45° and zero are almost same.
3. Three or more under reamed bulbs are advantageous only when the spacing ratio (S / D_u) is two or less, and when (S / D_u) is greater than two, multi-under reamed piles do not have specific advantages.
4. The ultimate load bearing capacities of piles are maximum when the spacing between two under reamed bulb is 2.5 times the diameter of the under reamed bulb. It appears that the spacing between two under reamed bulbs suggested in (1.25 to 1.5 times) IS 2911(1980) is not the optimum,
5. The expression suggested in IS 2911(1980) can be used for predicting the ultimate load carrying capacity of under reamed piles with spacing ratio (S / D_u) less than 2.5.

Module 5 : Design of Deep Foundations

Lecture 23 : Piles In Sand [Section 23.2 : Vesic's method]

Recap

In this section you have learnt the following.

- Introduction

Module 5 : Design of Deep Foundations

Lecture 24 : Under Reamed Pile [Section 24.2 : Bearing Capacity of Under Reamed Pile in Clay]

Objectives

In this section you will learn the following

- Introduction

Module 5 : Design of Deep Foundations

Lecture 24 : Under Reamed Pile [Section 24.2 : Bearing Capacity of Under Reamed Pile in Clay]

Bearing Capacity of Under Reamed Pile in Clay

$$P_U = P_{bu} + P_{su}$$

Tip Resistance

- 1 For Single Under Reamed Pile for Clay

$$P_{bu} = n \left[\frac{\pi}{4} \times (D_u^2 - D^2) \times C_1 \times N_c \right] + \left[\frac{\pi}{4} \times D^2 \times C_c \times N_c \right] \quad \text{-----(42)}$$

Here n = No. Of Bulbs

- 2 For Single Under Reamed Pile for $C - \phi$ Soil

$$P_{bu} = n \left[\frac{\pi}{4} \times (D_u^2 - D^2) \times (C_1 \cdot N_c + \sigma'_1 \cdot N_q) \right] + \left[\frac{\pi}{4} D^2 \times [C_2 \cdot N_c + \sigma'_2 \cdot N_q] \right] \quad \text{-----(43)}$$

Here σ'_1 is effective vertical stress at bulb base.

σ'_2 is effective vertical stress at stem base.

Skin Resistance

$$P_{su} = (\pi \cdot \Delta l_1 \cdot C_1 \cdot \alpha_1) + (\pi \times (D_u - D) \times l_2 \times C_2 \times \alpha_2) + \pi \times \Delta l_2 \times C_2 \times \alpha_3 \quad \text{-----(44)}$$

$(\pi \cdot \Delta l_1 \cdot C_1 \cdot \alpha_1)$ This term will absent in compressive Under Reamed pile.

$(\pi \times \Delta l_2 \times C_2 \times \alpha_3)$ This Term will absent in Tension under Reamed pile.

Module 5 : Design of Deep Foundations

Lecture 24 : Under Reamed Pile [Section 24.2 : Bearing Capacity of Under Reamed Pile in Clay]

Recap

In this section you have learnt the following.

- Introduction

Congratulations, you have finished Lecture 24. To view the next lecture select it from the left hand side menu of the page

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

Module 5 : Design of Deep Foundations

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

Procedure for ultimate pile capacity

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

Static analysis

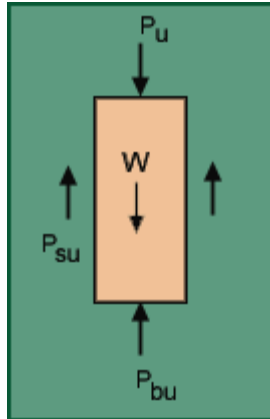


Fig –5.12 Forces on pile

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

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,

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

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

Module 5 : Design of Deep Foundations

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

■ Piles in granular soils (sands and gravel)

Driven piles

Point bearing in granular soil,

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

Where $\bar{\sigma}$ 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 $\phi = \frac{\phi_1 + 40^\circ}{2}$ may be taken, where ϕ_1 is the in situ value of the angle of bearing resistance

Unit skin friction,

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

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

Ultimate skin friction resistance,

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

$\bar{\sigma}_{av}$ = 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 | 0.75Φ | 1.0 | 2.0 |
| Timber | 0.67Φ | 1.5 | 4.0 |

Module 5 : Design of Deep Foundations

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

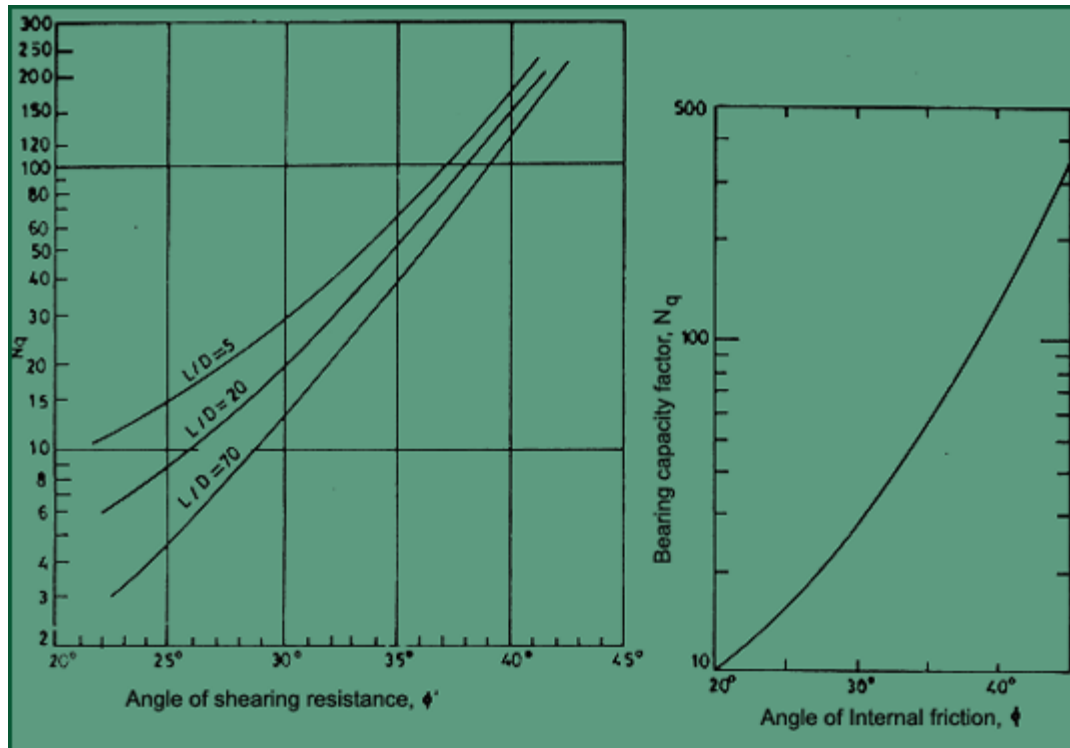


Fig-5.13 Values of N_q for pile formula (after Berezantzev et al, 1961) and N_q for driven piles (IS: 2911 Part I-1979)

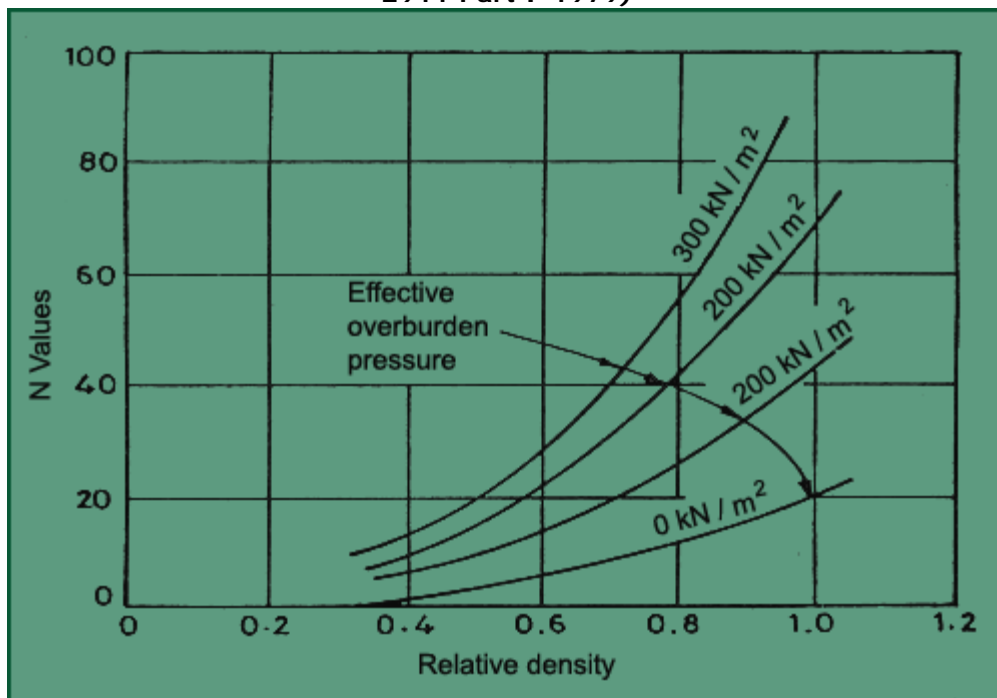


Fig-5.14 Relative density obtained from N values (After Gibbs and Holtz, 1966)

Module 5 : Design of Deep Foundations

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

■ 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.

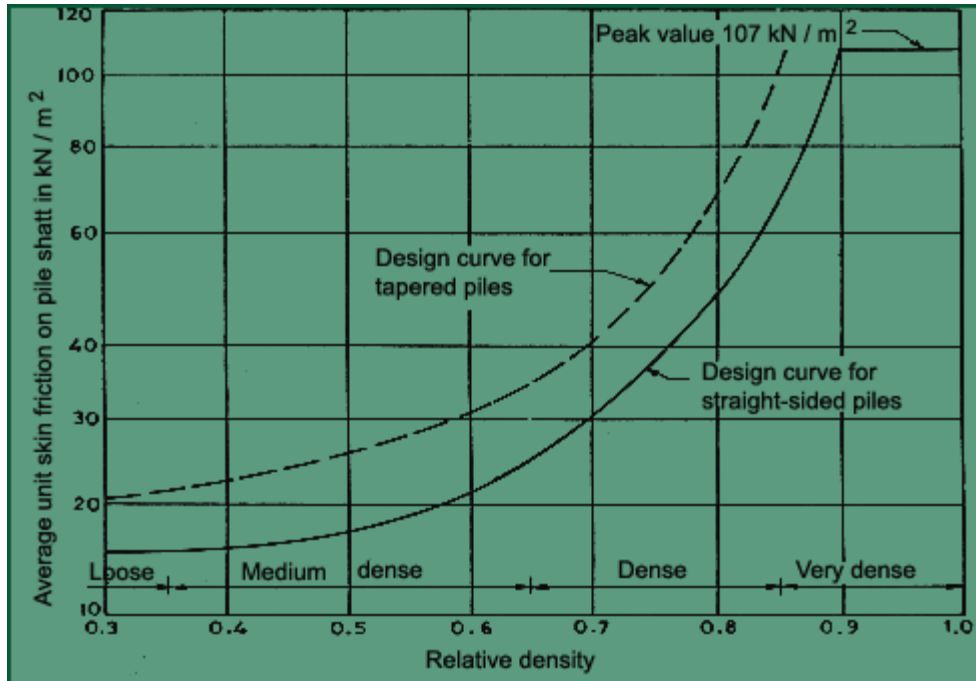


Fig-5.15 Average unit skin friction on driven piles in cohesion less soils

Module 5 : Design of Deep Foundations

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

■ 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 $c_u = \alpha c_u$; thus,

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

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 |

Module 5 : Design of Deep Foundations

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

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

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.2 : Pile load test]

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)

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.2 : Pile load test]

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 inadequate 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.

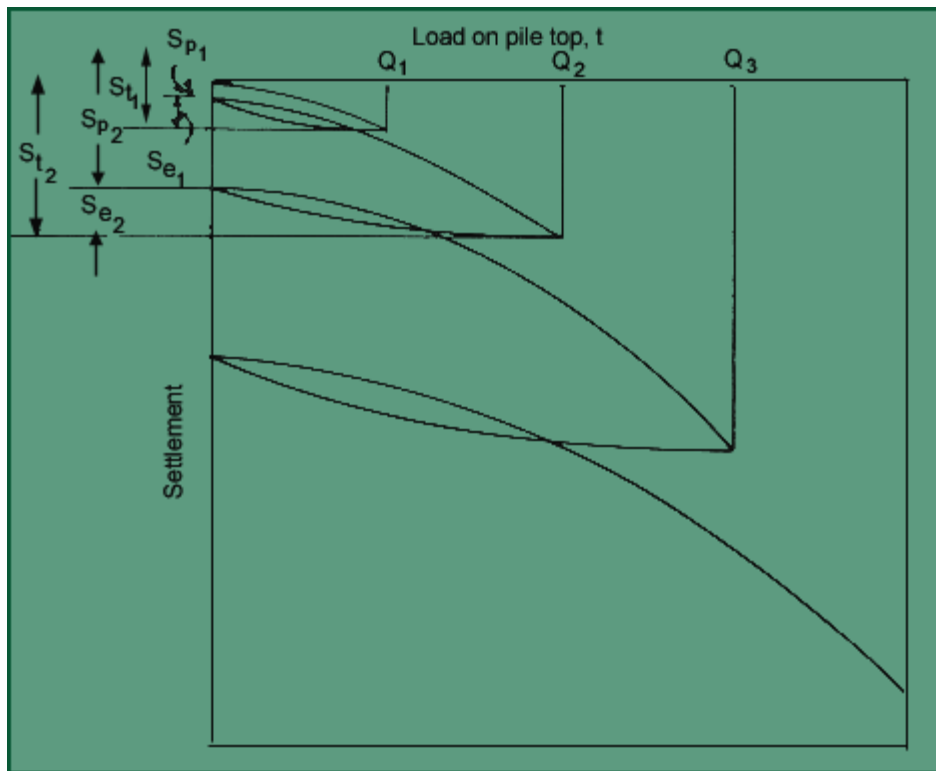


Fig -16 Typical load settlement plot from pile load test

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, $S_{e'}$.

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.2 : Pile load test]

The total settlement of the pile, S at any load level can be written as $S = S_{\delta} + \Delta_1$

Where S_{δ} is the compression of the soil at the base and Δ_1 is the compression of the pile.

S_{δ} can be written as, $S_{\delta} = S_e + S_{\delta 1}$

Where $S_{\delta 1}$ is the plastic compression of the soil at the base

Total settlement is $S = \Delta_1 + S_e + S_{\delta 1}$

But, $S = S_{e1} + S_p$

$$S_{e1} + S_p = \Delta_1 + S_e + S_{\delta 1}$$

$$S_e = (S_p - S_{\delta 1}) + S_{e1} - \Delta_1$$

$$S_e = S_{e1} - \Delta_1$$

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

$$\Delta_1 = \frac{(Q - Q_f / 2)L}{AE}$$

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.

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.2 : Pile load test]

Determination of Ultimate Load of pile Pile Load Test

1. Single Tangent method

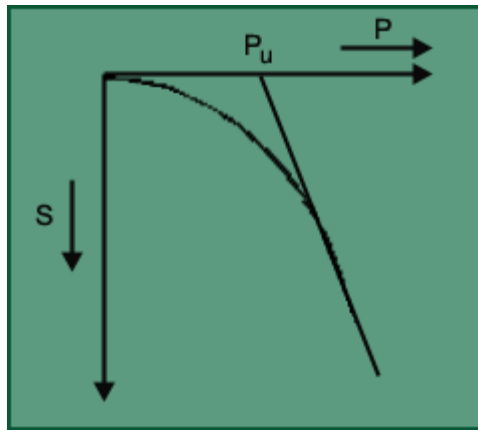


Fig-5.17 Single Tangent method

2. Double Tangent Method

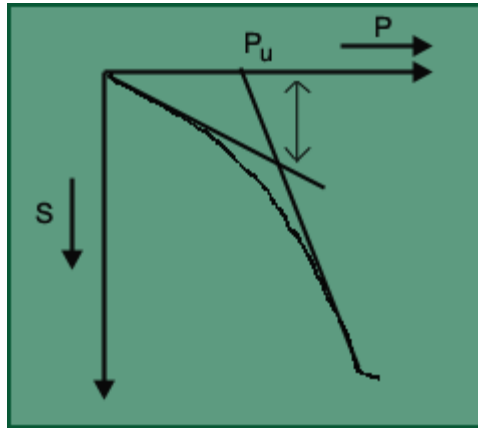


Fig-5.18 Double Tangent Method

3. Log-Log method

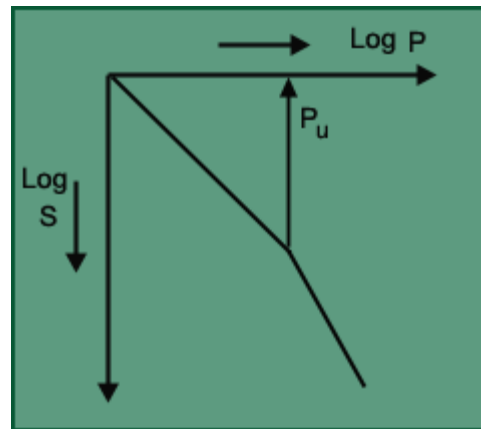


Fig-5.19 Log-Log method

4. Rectangular Hyperbola method

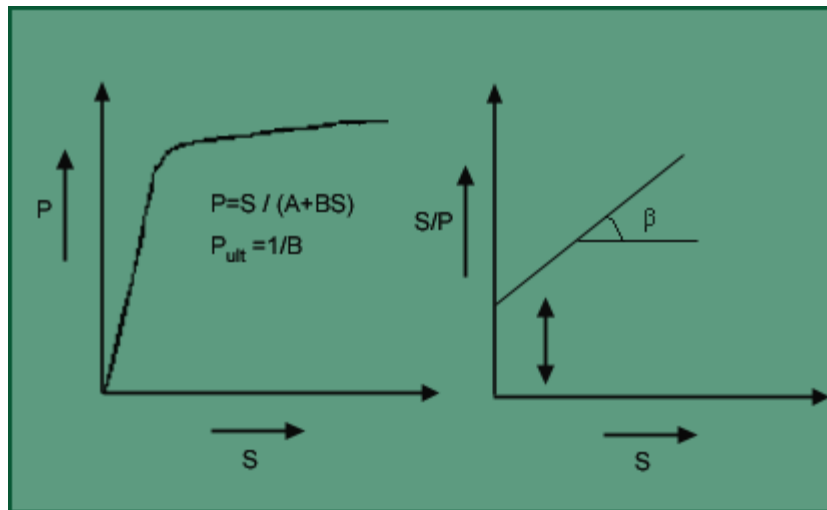


Fig-5.20 Rectangular Hyperbola method

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.2 : Pile load test]

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

5. Vander Veen's method (1953)

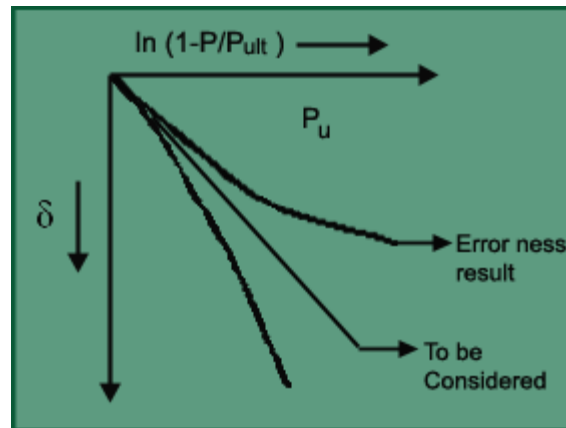


Fig-5.21 Vander Veen's method

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

δ_{pile} = settlement corr. to load P, and α is the factor relates load and deformation

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

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

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

6. Maazurkiewicz parabola method (1972)

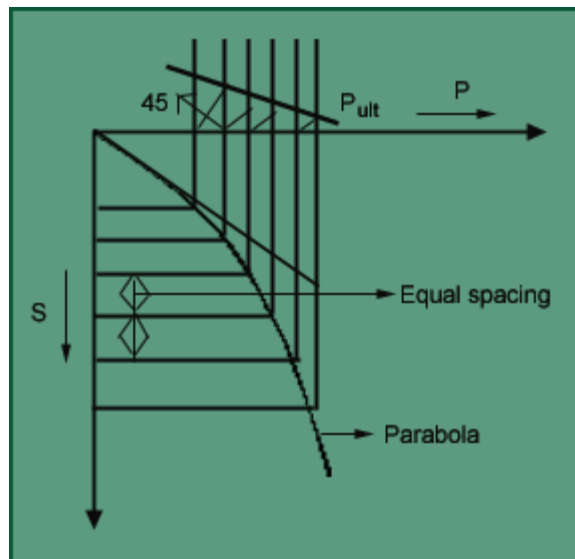


Fig-5.22 Maazurkiewicz parabola method

Module 5 : Design of Deep Foundations

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

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)

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.3 : Dynamic formulae]

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

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.3 : Dynamic formulae]

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, $Q_u = \frac{WH}{F(S + C)}$

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

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

Where Q_u & 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.

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.3 : Dynamic formulae]

■ 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

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.3 : Dynamic formulae]

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

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.

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

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

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

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

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

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

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.3 : Dynamic formulae]

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.

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.3 : Dynamic formulae]

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

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.4 : Pile capacity]

Objectives

In this section you will learn the following

- Introduction

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.4 : Pile capacity]

Pile capacity

For circular shallow footing,

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

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

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

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

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,

$$q_{ult} = CN_c^* + qN_q^* + 0.5\gamma BN_\gamma^* \quad \text{-----(21)}$$

for clays, $\Phi = 0$,

therefore,

$$P_{du} = A_b (CN_c^* + qN_q^*) \quad \text{-----(22)}$$

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

$$= (P_{du} - W) + P_{su} \quad \text{-----(23)}$$

Determination of P_{du} :

- Meyerhof's method
- Vesic method
- Janbu Method

Determination of P_{su}

- α -Method
- λ -Method
- β -Method

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.4 : Pile capacity]

Recap

In this section you have learnt the following.

- Introduction

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.5 : Determination of P_{bu}]

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} :

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.5 : Determination of 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

$$\sigma_h = K \cdot \sigma_v$$

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

$$\begin{aligned} f_s &= \sigma_h \cdot \tan \phi \\ f_s &= K \cdot \sigma_v' \cdot \tan \phi \end{aligned} \quad \text{-----(33)}$$

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,

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

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

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.5 : Determination of P_{bu}]

Recap

In this section you have learnt the following.

- Meyerhof's Method
- Vesic method to compute $P_{\phi u}$
- Janbu's method to compute $P_{\phi u}$

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.5 : Determination of P_{bu}]

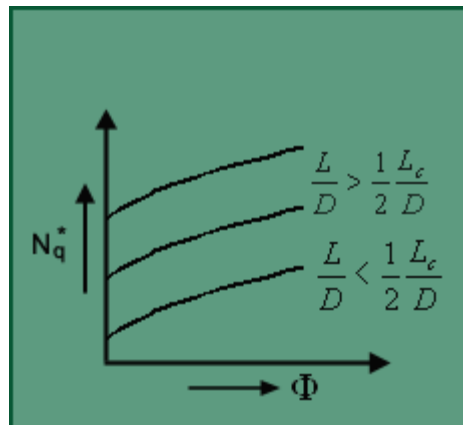
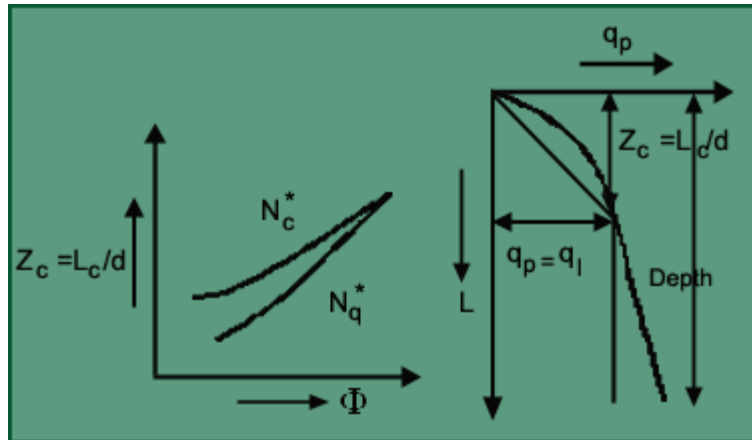
Determination of P_{bu} :

1. Meyerhof's Method : Good for sands

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

For sands $C=0$,

$$\text{therefore } P_{bu} = A_p q N_q^* = A_p q_p$$



Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.5 : Determination of P_{bu}]

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

$$q_p = 50 \cdot N \cdot q \tan \phi$$

$$q_i = 11 \frac{M_n}{M^2}$$

$q_i = 4N$ 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_i

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

- For two layers

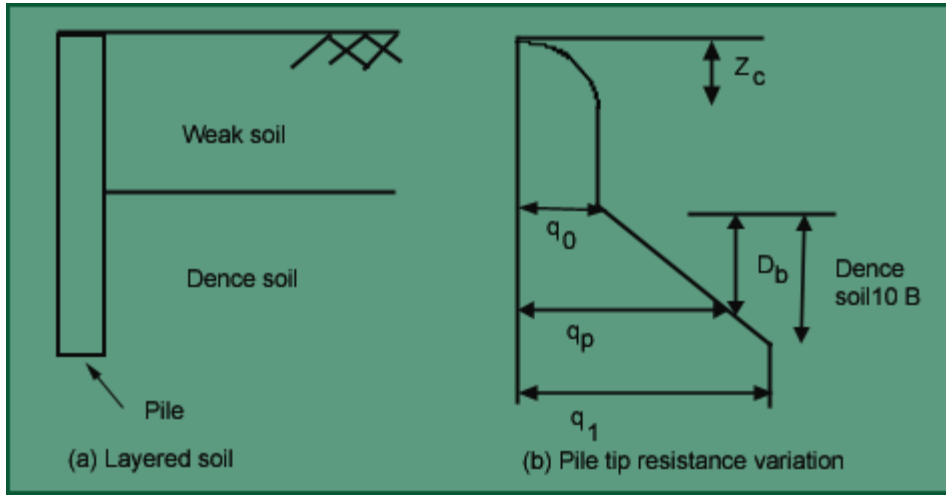


Fig.5.23 Pile tip resistance for layered soil

Where,

q_0 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_i is the limiting point resistance per unit area,

z_c is the depth upto portion of nonlinearity,

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.5 : Determination of P_{bu}]

B is the width of the pile or width of the pile, as shown in the fig.

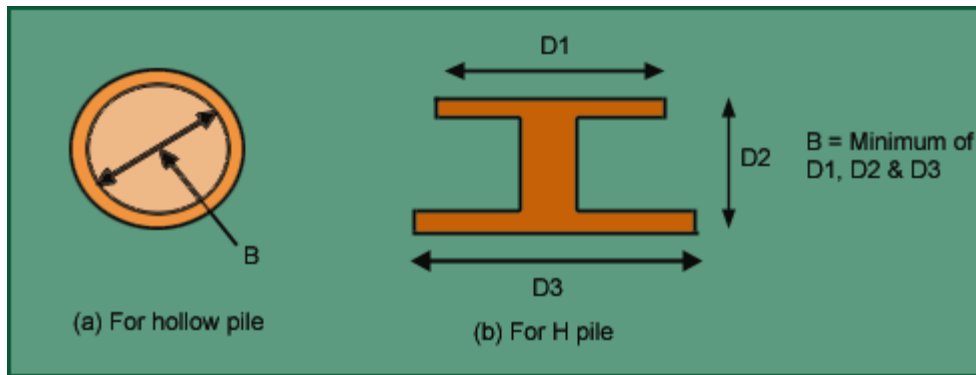


Fig. 5.24 Different B values

$$q_p = q_0 + \frac{(q_i - q_0)}{10B} D_p \leq q_i \quad \text{-----(24)}$$

where,

q_i values are given by Meyerhoff as given in earlier section.

- For three layers

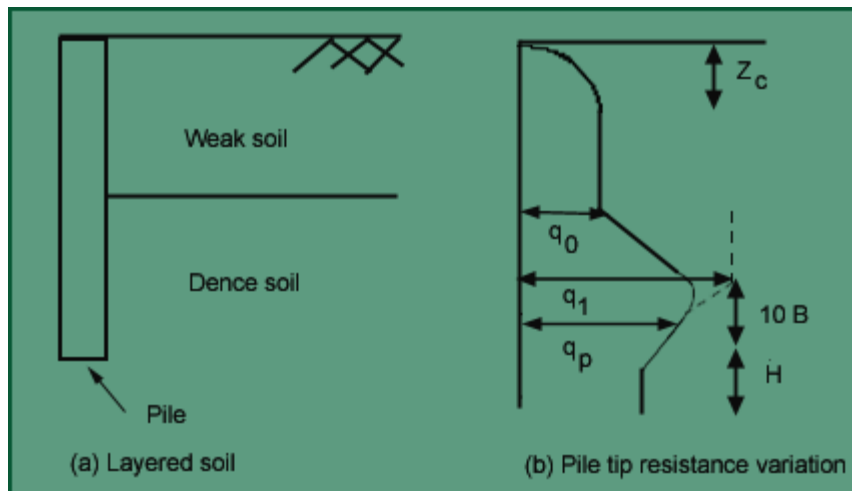


Fig 5.25 Pile tip resistance for layered soil

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.5 : Determination of P_{bu}]

$H < 10B$,

$$q_p = q_0 + \frac{(q_i - q_0)}{10B} \leq q_i \quad \text{-----(25)}$$

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

$$\gamma = 7.85 \text{ KN/m}^3, \phi_{avg} = 29^\circ,$$

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

Ans:

∴ Pile length $L = 1.22 \times 10 = 12.2 \text{ m}$.

From fig. 26, for $\phi = 29^\circ$, $N_q^* = 55$,

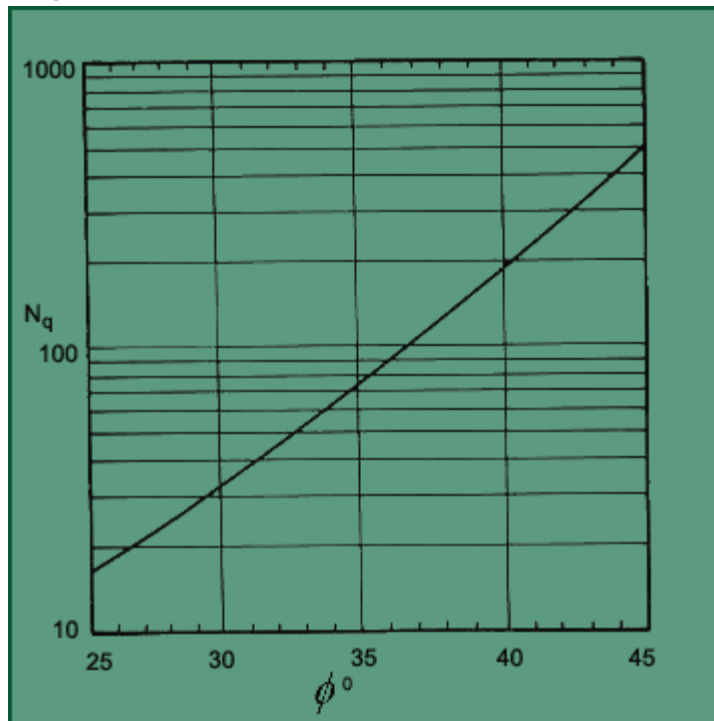


Fig 5.26 Meyerhoff's chart of N_q^*

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.5 : Determination of P_{bu}]

$$q_p = \gamma (7 \times D) \times N_q$$

$$= 7.85 \times 7 \times 1.22 \times 55$$

$$= 3687.145 \text{ KN/}$$

$$q_i = 50 N_q \times \tan \phi$$

$$= 50 \times 55 \times \tan 29^\circ$$

$$= 1524.35 \text{ KN/ m}^2 < q_p$$

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

The variation of q_p value is given in fig.

$$P_{bu} = (1524.34) \times 1.17 = 1783 \text{ KN}$$

Base resistance in pure clay ($\phi = 0$) :

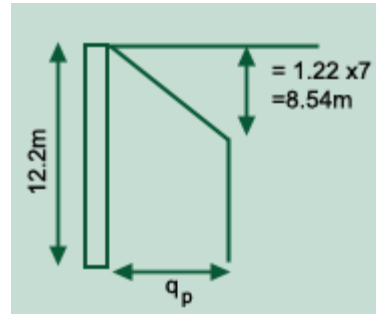


Fig. 5.27 Variation of pile tip resistance

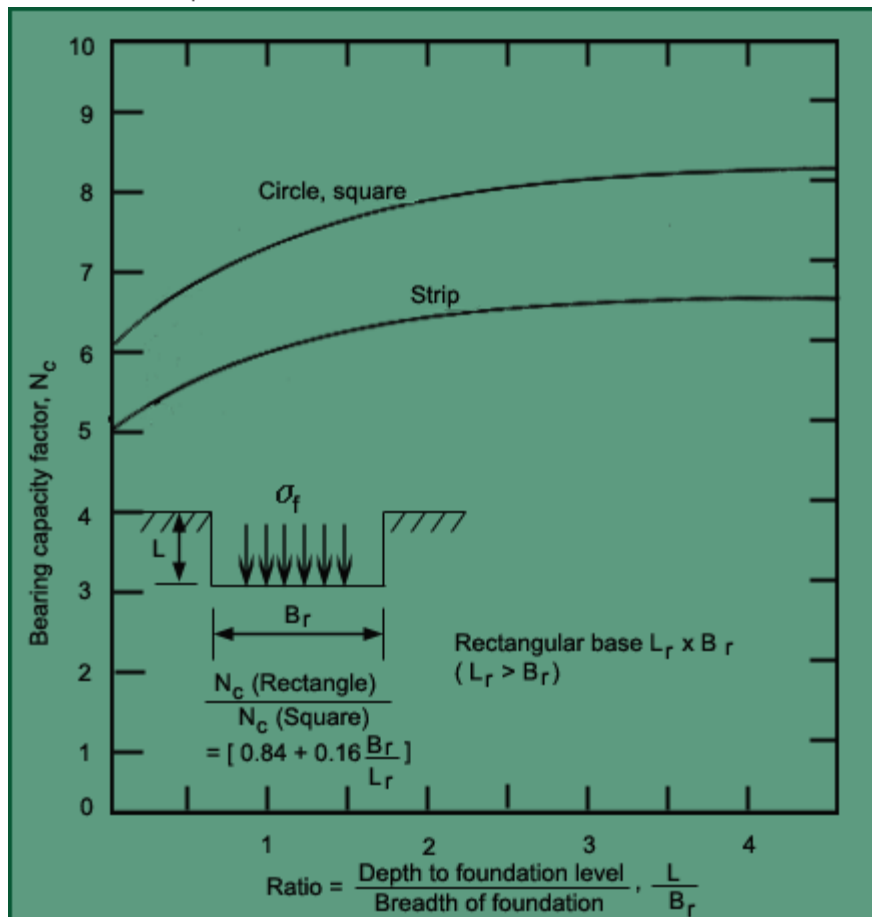


Fig. 5.28 Value of N_c given by Skempton

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.5 : Determination of P_{bu}]

Pile is considered as circular foundation, referring fig.

∴ 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 :

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

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):

$$P_{bu} = A_b q_p = A_b (c N_c^* + q' N_q^*) \quad \text{----- (27)}$$

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

where,

q' is the effective overburden pressure,

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

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.5 : Determination of P_{bu}]

2. Vesic method to compute P_{bu} :

It based on cavity expansion theory of cylinder.

$$P_{bu} = A_b (cN_c + \sigma'_0 N_\sigma) \quad \text{-----(29)}$$

where,

$$N_\sigma = \frac{B N_q}{(1 + 2K_0)} \quad \text{-----(30)}$$

where, K_0 is the earth pressure coefficient at rest,

σ'_0 in mean normal stress,

$$N_\sigma = f(I_{rr})$$

Where,

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

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

where,

Δ is average volumetric strain,

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

here,

μ_s is the poisson's ratio.

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.5 : Determination of P_{bu}]

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

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

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

$$\eta' = 70^\circ \text{ for soft clays,}$$

$$= 105^\circ \text{ for sand.}$$

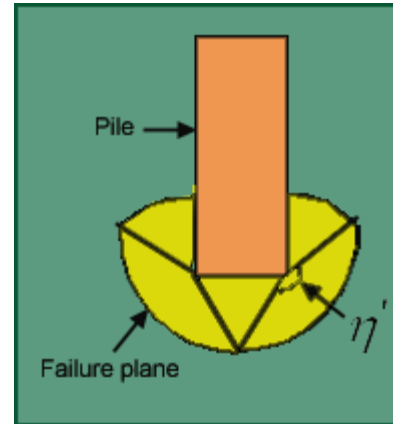


Fig. 5.29 Failure plane assumed by Janbu

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.

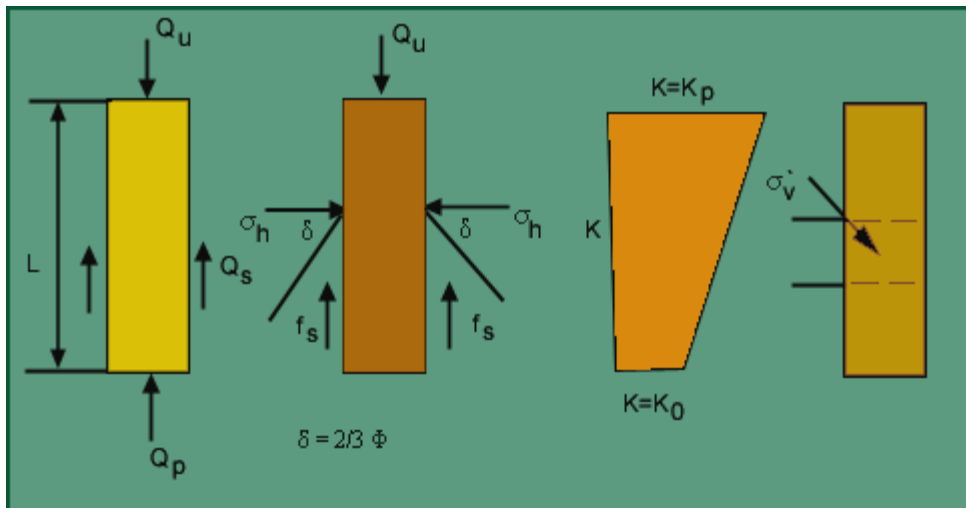


Fig 5.30 Variation of K in Sands.

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.6 : Determination of P_{su}]

Objectives

In this section you will learn the following

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

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.6 : Determination of P_{su}]

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.

α Method 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 α depends 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, c_a .

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,

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

Step 4 Compute the ultimate toe resistance, R_t .

$$q_t = 9c_u$$

$$R_t = q_t . A_t$$

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.6 : Determination of P_{su}]

Step 5 Compute the ultimate pile capacity (kips).

$$Q_u = P_{su} + R_t$$

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

$$Q_a = Q_u / \text{Factor of Safety}$$

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

$$C_a = \alpha c_u$$

$$C_a = \alpha c_u$$

$$\alpha = \frac{C_a}{c_u} \text{ where } c_u \text{ is the undrained shear strength for a homogenous layer.}$$

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.

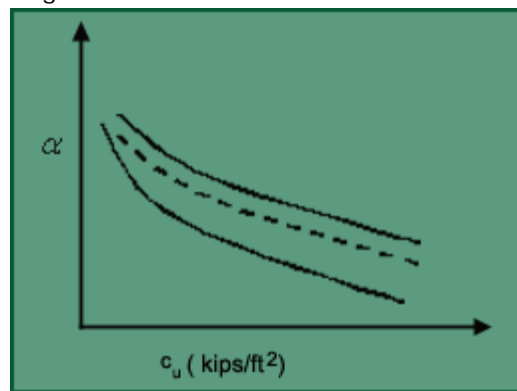


Fig. 5.31 Variation of α with undrained shear strength

Heterogeneous Soil:

Case1 : Sands over lying stiff cohesive clays.

Case2: soft clays/sits overlying stiff clays.

Case3: stiff cohesive soils without any overlying strata.

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.6 : Determination of P_{su}]

Table : 5.4 α Values For Different Penetration Ratios

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

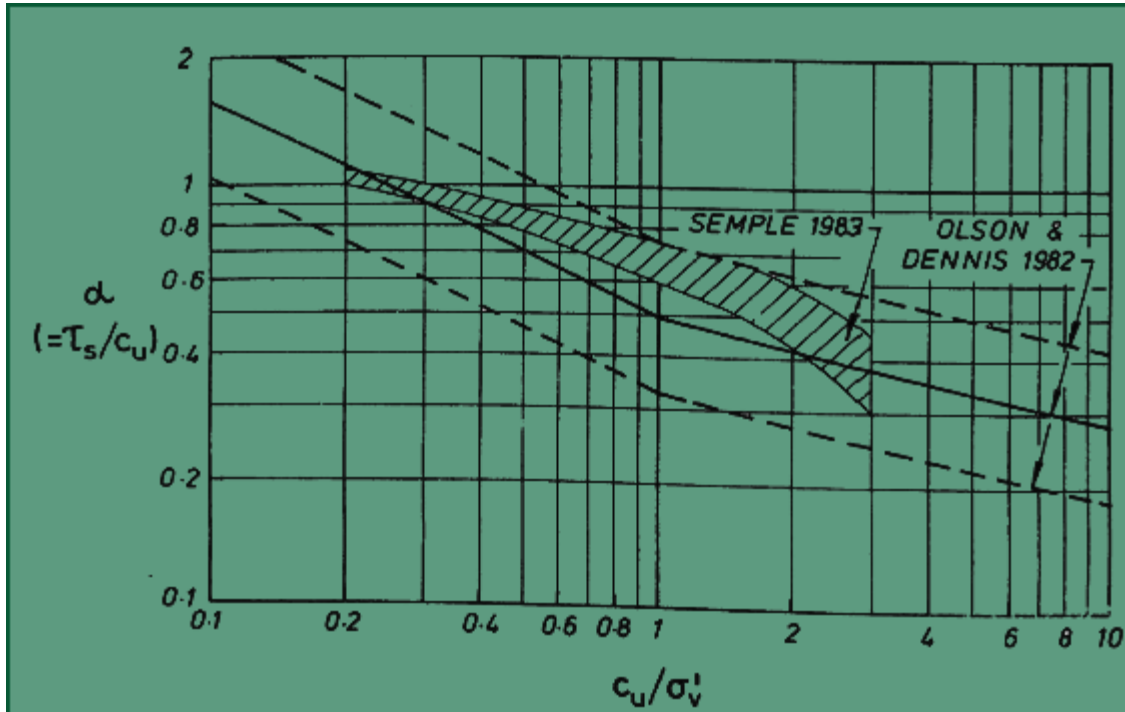


Fig. 5.32 Design Curve for Driven Piles

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.

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.6 : Determination of P_{su}]

The value of α 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 α 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 α with c_u reproduced in Figure

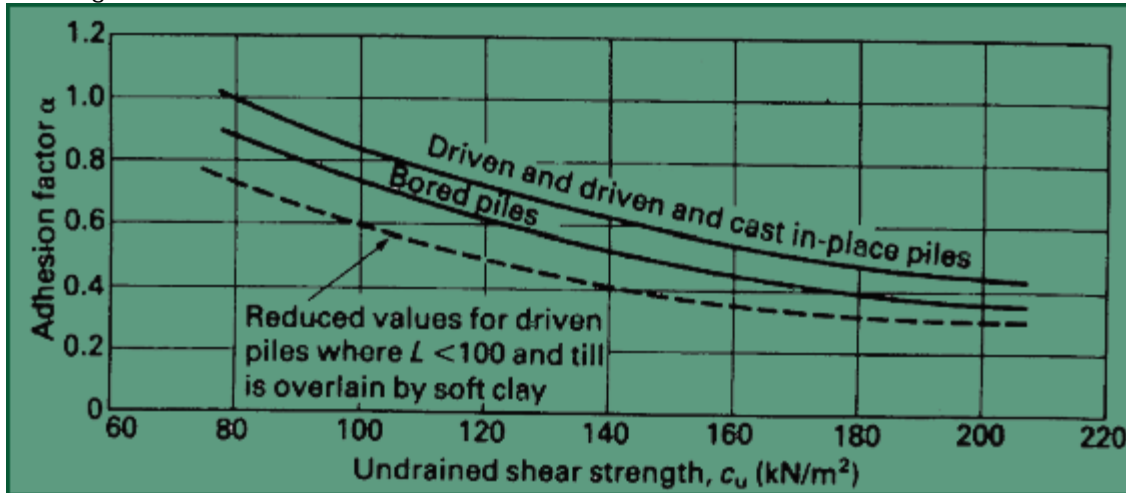


Fig. 5.33 α values for bored and driven piles (based on c_u value)

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

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

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.6 : Determination of P_{su}]

2. λ -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, σ'_m , and the mean undrained cohesion along the pile shaft, c_m as follows:

$$P_{su} = \lambda(\sigma'_m + 2c_m)A_s \quad \text{-----(37)}$$

λ = Dimensionless coefficient

σ'_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.

$$\alpha = \frac{c_a}{c_m} = \lambda \left[\frac{\sigma'_m}{c_m} + 2 \right]$$

5.5.7.3 β -method :

$$P_{su} = \int_0^L P \sigma'_v (k_s \tan \phi'_a) dz$$

$$\beta = k_s \tan \phi'_a \quad \text{-----(38)}$$

k_s = Earth pressure coefficient

ϕ'_a = pile soil interfacial friction angle.

σ'_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.

$$\delta \text{ is } (1 - \sin \phi') \tan \phi'$$

ϕ' ranges from 20° - 30°

k_s ranges from 0.24 to 0.29

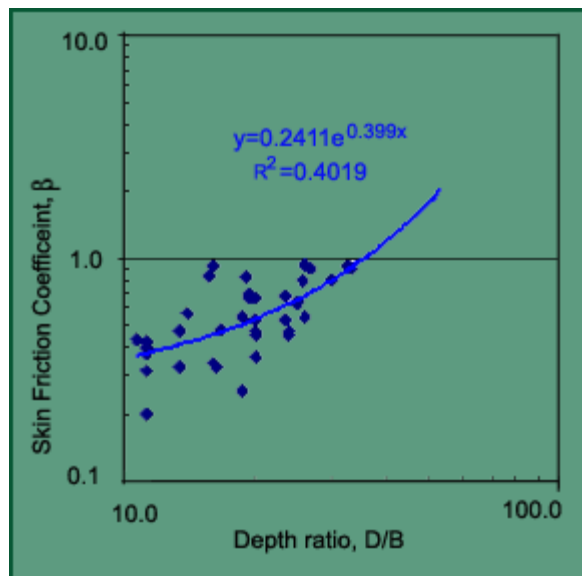


Fig. 5.34 Relation between Depth Ratio D/B and Skin Friction Coefficients as predicted by Burland.

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.6 : Determination of P_{su}]

■ Meyerhoff method

δ depends on ϕ as well as depth of penetration

δ approximately 0.15 for depth >60m

Stiff Clay

$K_s = k_0$ as per Burland

ϕ'_a = 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 .

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

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

\bar{N} = average N value over pile length.

$$f_1 = 1 \text{ kg/cm}^2$$

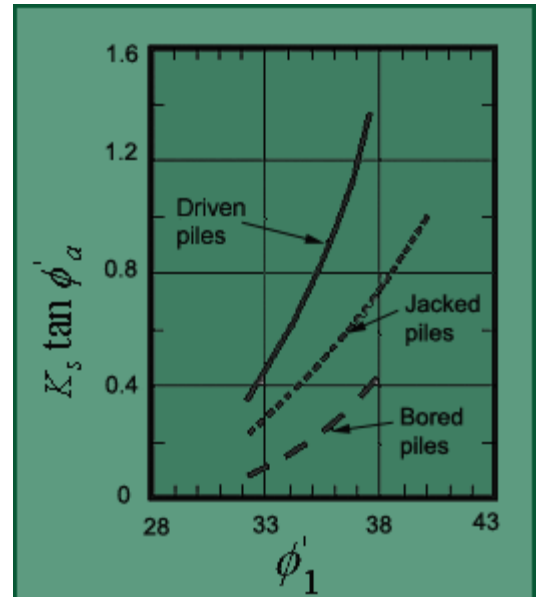


Fig. 5.35 Values of $K_s \tan \phi$

Module 5 : Design of Deep Foundations

Lecture 22 : Ultimate pile capacity [Section 22.6 : Determination of P_{su}]

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