BALASORE COLLEGE OF ENGINEERING AND TECHNOLOGY SERGARH, BALASORE

A
Lecture Notes
On
FOUNDATION ENGINEERING

3rd Year **6th Semester**

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Asst. Professor Civil Engineering Department

CHECKED BY

MODULE WISE DISTRIBUTION OF LOADS

Module	Chapter title	Assigned Hour (as per BPUT)	Actual Session Needed	Range of Marks of Questions to be being asked (BPUT)
I	Lateral Earth Pressure and Retaining Structures	08	11	25-35
II	Bearing Capacity	10	17	30-40
III	Deep Foundations	10	11	25-30
IV	Subsoil Exploration	08	12	20-25
TOTAL		**36**	**51**	**100**

Lecture Note On Foundation Engineering
6^{th} sem civil Eng.

Module – I (8 Hours)

Lateral Earth Pressure and Retaining Structures: Concept of earth pressure, Earth pressure at rest,active and passive earth pressure for both cohesionless and cohesive soils, Earth pressure theories:Rankine's theory, Coulomb's Wedge theory, Graphical methods: Rebhan's and Culmann's graphicalsolutions, Stability conditions for retaining walls.

Module: II (10HOURS)

Bearing Capacity: Definitions, Rankine's analysis, Types of failures: General and local shear failure,Terzaghi's Analysis, Brinch-Hansen analysis, Meyerhof's analysis, Vesics's bearing capacityequation, Effect of water table on bearing capacity, IS code method for computing bearing capacity, Field Methods: Plate load test and its limitations, Standard penetration test. Shallow Foundations: Types of foundations: Spread footing, combined and strap footing, mat or raft footing, Settlement of footings.

Module: III (10HOURS)

Deep Foundations: Difference between shallow and deep foundations, Types of deep foundations. Pile Foundations: Types of piles, pile driving, load carrying capacity of piles-static and dynamic formulae, Pile load test and its limitations, correlation with penetration tests, Group action in piles settlement and efficiency of pile groups in clay, negative skin friction, Under reamed pile foundation. Basics of well foundation - types, component parts and ideas about the forces acting on a well foundation.

Module: IV (8 HOURS)

Subsoil Exploration: Necessity and planning for subsoil exploration, Methods - direct (test pits and trenches), indirect (sounding, penetration tests and geophysical methods). Soil sampling – types of samples, standard penetration test, static and dynamic cone penetration test, in-situ vane shear test, Rock coring, soil exploration report.

Books:

1. Principles of Foundation Engineering by B. M. Das, Cenage Learning
2. Basic and Applied Soil Mechanics by Gopal Ranjan and A. S. R. Rao, New Age International Publishers
3. Geotechnical Engineering by C. Venkatramiah, New Age International Publishers
4. Geotechnical Engineering by S. K. Gulati & Manoj Gupta, Mc Graw Hill
5. Soil Mechanics and Foundations by B. C. Punmia et al., Laxmi Publications
6. Soil Mechanics & Foundation Engineering by B.N.D. Narasinga Rao, Wiley.

Digital Learning Resources:

Course Name : FOUNDATION ENGINEERING
Course Link : https://nptel.ac.in/courses/105/105/105105176/
Course Instructor: PROF. KOUSIK DEB Department of Civil Engineering IIT Kharagpur.

MODULE-I
Lecture-1

RETAINING WALL

Retaining walls are structures used to retain earth or water or other materials such as coal, ore, etc; where conditions do not permit the mass to assume its natural slope. The retaining material is usually termed as backfill. The main function of retaining walls is to stabilize hillsides and control erosion. When roadway construction is necessary over rugged terrain with steep slopes, retaining walls can help to reduce the grades of roads and the land alongside the road. Some road projects lack available land beside the travel way, requiring construction right along the toe of a slope. In these cases extensive grading may not be possible and retaining walls become necessary to allow for safe construction and acceptable slope conditions for adjacent land uses. Where soils are unstable, slopes are quite steep, or heavy runoff is present, retaining walls help to stem erosion. Excessive runoff can undermine roadways and structures, and controlling sediment runoff is a major environmental and water quality consideration in road and bridge projects. In these situations, building retaining walls, rather than grading excessively, reduces vegetation removal and reduces erosion caused by runoff. In turn, the vegetation serves to stabilize the soil and filter out sediments and pollutants before they enter the water source, thus improving water quality.

In this section you will learn the following

- Gravity walls
- Semi Gravity Retaining Wall
- Flexible walls
- Special type of retaining walls

Different Types of Retaining Structures On the basis of attaining stability, the retaining structures are classified into following:

1. Gravity walls :

Gravity walls are stabilized by their mass. They are constructed of dense, heavy materials such as concrete and stone masonry and are usually reinforced. Some gravity walls do use mortar, relying solely on their weight to stay in place, as in the case of dry stone walls. They are economical for only small heights.

LECTURE-2

Semi Gravity Retaining Wall

These walls generally are trapezoidal in section. This type of wall is constructed in concrete and derivesits stability from its weight. A small amount of reinforcement is provided for reducing the mass of the concrete. This can be classified into two:

- Cantilever retaining wall
- Counter fort retaining wall
- Cantilever retaining wall

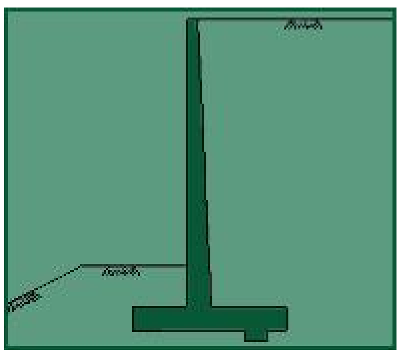


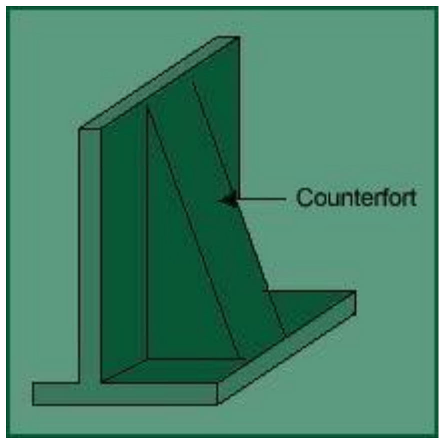
Fig 6.3. Semi Gravity Retaining Wall

This is a reinforced concrete wall which utilizes cantilever action to retain the backfill. This type is suitable for retaining backfill to moderate heights (4m-7m). In cross section most cantilevered walls look like or inverted. To ensure stability, they are built on solid foundations with the base tied to the vertical portion of the wall with reinforcement rods. The base is then backfilled to counteract forward pressure on the vertical portion of the wall. The cantilevered base is reinforced and is designed to prevent uplifting at the heel of the base, making the wall strong and stable. Local building codes, frost penetration levels and soil qualities determine the foundation and structural requirements of taller cantilevered walls. Reinforced concrete cantilevered walls sometimes have a batter. They can be faced with stone, brick, or simulated veneers. Their front faces can also be surfaced with a variety of textures. Reinforced Concrete CantileveredWalls are built using forms. When the use of forms is not desired, Reinforced Concrete Block CantileveredWalls are another option. Where foundation soils are poor, Earth Tieback Retaining Walls are another choice. These walls are counterbalanced not only by a large base but also by a series of horizontal bars or strips extending out perpendicularly from the vertical surface into the slope. The bars or strips, are made of wood, metal, or synthetic materials such as geotextiles. Once an earth tieback retaining wall is backfilled, the weight and friction of the fill against the horizontal members anchors the structure.

LECTURE-3

Counterfort retaining wall

When the height of the cantilever retaining wall is more than about 7m, it is economical to providevertical bracing system known as counter forts. In this case, both base slab and face of wall span horizontally between the counter forts.



Counter fort retaining wall

3. **Flexible walls:** there are two classes of flexible walls.

A. Sheet pile walls
and
B. Diaphragm wall

A. **Sheet Pile Walls**

Sheet piles are generally made of steel or timber. The use of timber piles is generally limited to temporary structures in which the depth of driving does not exceed 3m. for permanent structures and for depth of driving greater than 3m, steel piles are most suitable. Moreover, steel lies are relatively water tight and can be extracted if required and reused. However, the cost of sheet steel piles is generally more than that of timber piles. Reinforced cement concrete piles are generally used when these are to be jetted into fine sand or driven in very soft soils, such as peat. For toughersoils, the concrete piles generally break off.

Based on its structural form and loading system, sheetpile walls can be classified into 2 types:

(i)Cantilever Sheet Piles and

(ii)Anchored Sheet Piles

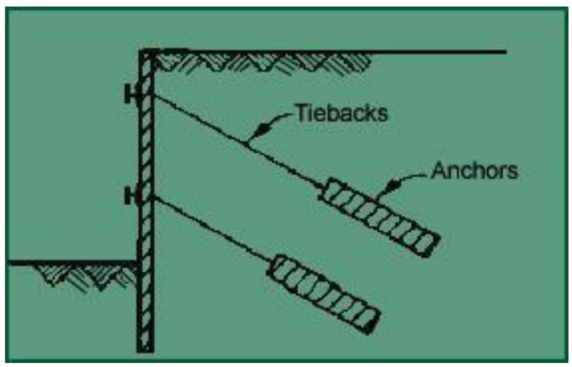
Free cantilever sheet piles

It is a sheet pile subjected to a concentrated horizontal load at its top. There is no back fill above the dredgelevel. The free cantilever sheet pile derives its stability entirely from the lateral passive resistance of the soil below the dredge level into which it is driven.

1. Cantilever Sheet Pile Wall with Backfill

A cantilever sheet pile retains backfill at a higher level on one side. The stability is entirely from the lateral passive resistance of the soil into which the sheet pile is driven, like that of a free cantilever sheet pile.

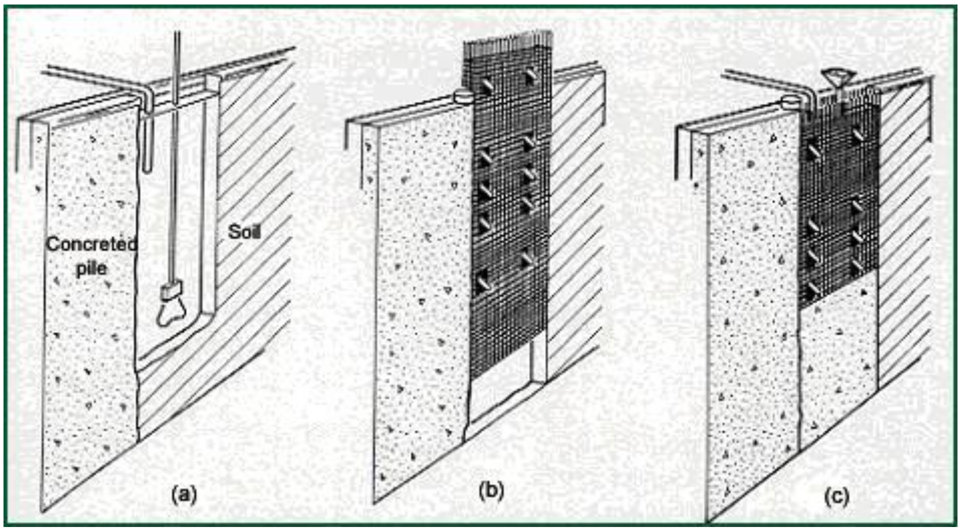
2. Anchored sheet pile walls Anchored sheet pile walls are held above the driven depth by anchors provided suitable level. The anchors provided for the stability of the sheet pile, in addition to the lateral passive resistance of the soil into which the sheet piles are driven. The anchored sheet piles are also of two types.



Anchored sheet pile wall

- **Free earth support piles.** An anchored pile is said to have free earth support when the depth of embedment is small and the pile rotates at its bottom tip. Thus there is a point of contra flexure inthe pile.
- **Fixed earth support piles.** An anchored sheet pile has fixed earth support when the depth of embedment is large. The bottom tip of the pile is fixed against rotations. There is a change in the curvature of the pile, and hence, an inflection point occurs.
- **Diaphragm Walls** Diaphragm walls are commonly used in congested areas for retention systems and permanent foundation walls. They can be installed in close proximity to existing structures, with minimal loss of support to existing foundations. In addition, construction dewatering is not required, so there is no associated subsidence. Diaphragm walls have also been used as deep groundwater barriers through and under dams.

Diaphragm walls are constructed by the slurry trench technique which was developed in Europe, and has been used in the United States since the l940's. The technique involves excavating a narrow trench that is kept full of an engineered fluid or slurry. The slurry exerts hydraulic pressure against the trench walls and acts as shoring to prevent collapse. Slurry trench excavations can be performed in all types of soil, even below the ground water table. Cast in place; diaphragm walls are usually excavated under bentonite slurry. The construction sequence usually begins with the excavation of discontinuous primary panels. Stop-end pipes are placed vertically in each end of the primary panels, to form joints for adjacent secondary panels. Panels are usually 8 to 20 feet long, with widths varying from 2 to 5 feet. Once the excavation of a panel iscomplete, a steel reinforcement cage is placed in the center of the panel. Concrete is then poured in one continuous operation, through one or several tremie pipes that extend to the bottom of the trench. The tremie pipes are extracted as the concrete raises in the trench, however the discharge of the tremie pipe always remains embedded in the fresh concrete. The slurry, which is displaced by the concrete, is saved and reused for subsequent panel excavations. When the concrete sets, the end pipes are withdrawn. Similarly, secondary panels are constructed between the primary panels, and the process continues to create a continuous wall. The finished walls may cantilever or require anchors or props for lateral support.



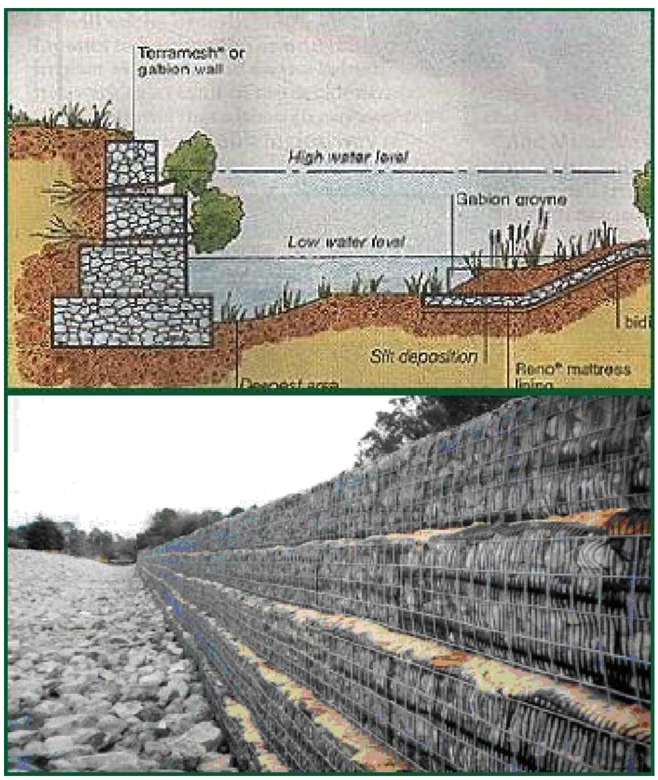
Construction Stages of a Diaphragm Wall using Slurry Trench Technique.

LECTURE-4

4. Special type of retaining walls
Gabion walls

Gabion walls are constructed by stacking and tying wire cages filled with trap rock or native stone on top of one another. They can have a continuous batter (gently sloping) or be stepped back (terraced) with each successively higher course.

This is a good application where the retaining wall needs to allow high amounts of water to pass through it,as in the case of riverbank stabilization. It is important to use a filter fabric with the gabion to keep adjacent soil from flowing into or through the cages along with the water. As relatively flexible structures, they are useful in situations where movement might be anticipated. Vegetation can be re-established around the gabions and can soften the visible edges allowing them to blend into the surrounding landscape.For local roads, they are a preferred low-cost retaining structure.



Gabion Wall

Design Requirement for Gravity walls

Gravity Retaining walls are designed to resist earth pressure by their weight. They are constructed of the mass, concrete, brick or stone masonry. Since these materials can not resist appreciable tension, the design aims at preventing tension in the wall. The wall must be safe against sliding and overturning. Also the maximum pressure exerted on the foundation soil should exceed the safe bearing capacity of the soil.

So before the actual design, the soil parameters that influence the earth pressure and the bearing capacity ofthe soil must be evaluated. These include the unit weight of the soil, the angle of the shearing resistance, the cohesion intercept and the angle of wall friction. Knowing these parameters, the lateral earth pressure and bearing capacity of the soil determined.

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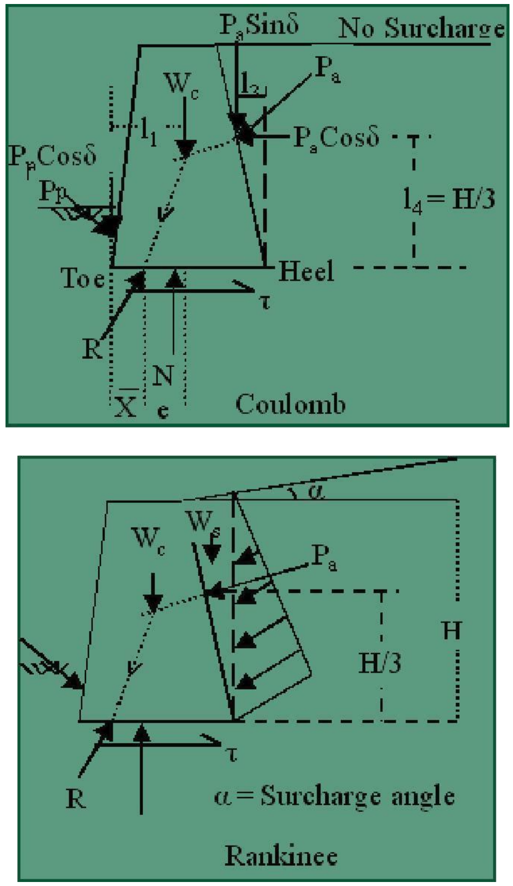


Fig. 6.12a shows a typical trapezoidal section of a gravity retaining wall.

The forces acting on the wall per unit length are:

- Active Earth pressure P_a.
- The weight of the wall (W_c)
- The Resultant soil reaction R on the base. (or Resultant of weight W_c & P_a).Strike the base at point D. There is equal and opposite reaction R' at the base between the wall and the foundation.
- Passive earth pressure P_p acting on the lower portion of the face of the wall, which usually small and usually neglected for design purposes. The full mobilization of passive earth pressure not occurs at the time of failure so we not consider it. If we consider it then it shows resistance against instability. So if we ignore it then we will be in safer side.

First decide which theory we want to apply for calculating the active earth pressure. Normally wecalculate earth pressure using Rankine’s theory or Coulomb’s Earth pressure theory.

For using Rankine’s theory, a vertical line AB is drawn through the heel point (Fig 6.12-b). It is assumedthat the Rankine active condition exist along the vertical line AB. While checking the stability, the weight of the soil (W_s) above the heel in the zone ABC should also be taken in to consideration, in addition to the Earth pressure (P_a) and weight of the wall (W_c). But Coulomb’s theory gives directly the lateral pressure (P_a) on the back face of the wall, the forces to be considered only P_a (Coulomb) and the Weight of the wall (W_c). In this case, the weight of soil (W_s) is need not be considered.

$$F.S._{Overturning} = \frac{\sum M_R}{\sum M_o} \geq 1.5$$

$$F.S._{Overturning} = \frac{W_c.l_1 + P_a.Sin(\delta).l_2 + P_p.Cos(\delta).l_3}{P_a.Cos(\delta).l_4}$$

Once the forces acting on the wall have been determined, the Stability is checked using the procedure discussed in the proceeding section. For convenience, the section of the retaining wall is divided in to rectangles & triangles for the computation of the Weight and the determination of the line of action of the Weight.

$$\bar{X} = \frac{\left[\sum M_R\right] - \left[\sum M_o\right] (\text{net moment})}{\sum V (\text{vertical force})}$$

LECTURE-5

$$\text{Therefore} \quad \left[e = \frac{B}{2} - \bar{X}\right] \leq \frac{B}{6}$$

For a safe design, the following requirement must be satisfied.

No Sliding

Horizontal forces tend to slide the wall away from the fill. This tendency is resisted by friction at the base.

$$p_{(max)} = \frac{\sum V}{B}\left[1 + \frac{6e}{B}\right] \qquad p_{min} = \frac{\sum V}{B}\left[1 - \frac{6e}{B}\right]$$

$$F.S_{sliding} = \frac{\sum \text{Resisting Force} \quad (\sum V)}{\sum \text{Sliding Force} \quad (\sum H)} > 1.5 (\text{for Stability}) \quad (\text{General})$$

$$F.S._{sliding} = \frac{[\mu.\sum W] + P_P.COS(\delta)}{P_a.COS(\delta)}$$

$$F.S._{sliding} = \frac{\mu.W_C + \mu.[P_a.Sin(\delta)] + P_P.COS(\delta)}{P_a.COS(\delta)} \quad \therefore [\mu = \tan(\delta)] \quad (\text{Coulomb})$$

μ = Coefficient of friction between the base of the wall and soil (= tan δ).

$\sum W$ = Sum of the all vertical forces i.e. vertical component of inclined active force.A minimum factor of safety of 1.5 against sliding is recommended.

No Overturning

The wall must be safe against overturning about toe.

Lecture-6

Lateral Earth Pressure

At – Rest, Active and Passive Pressures :-

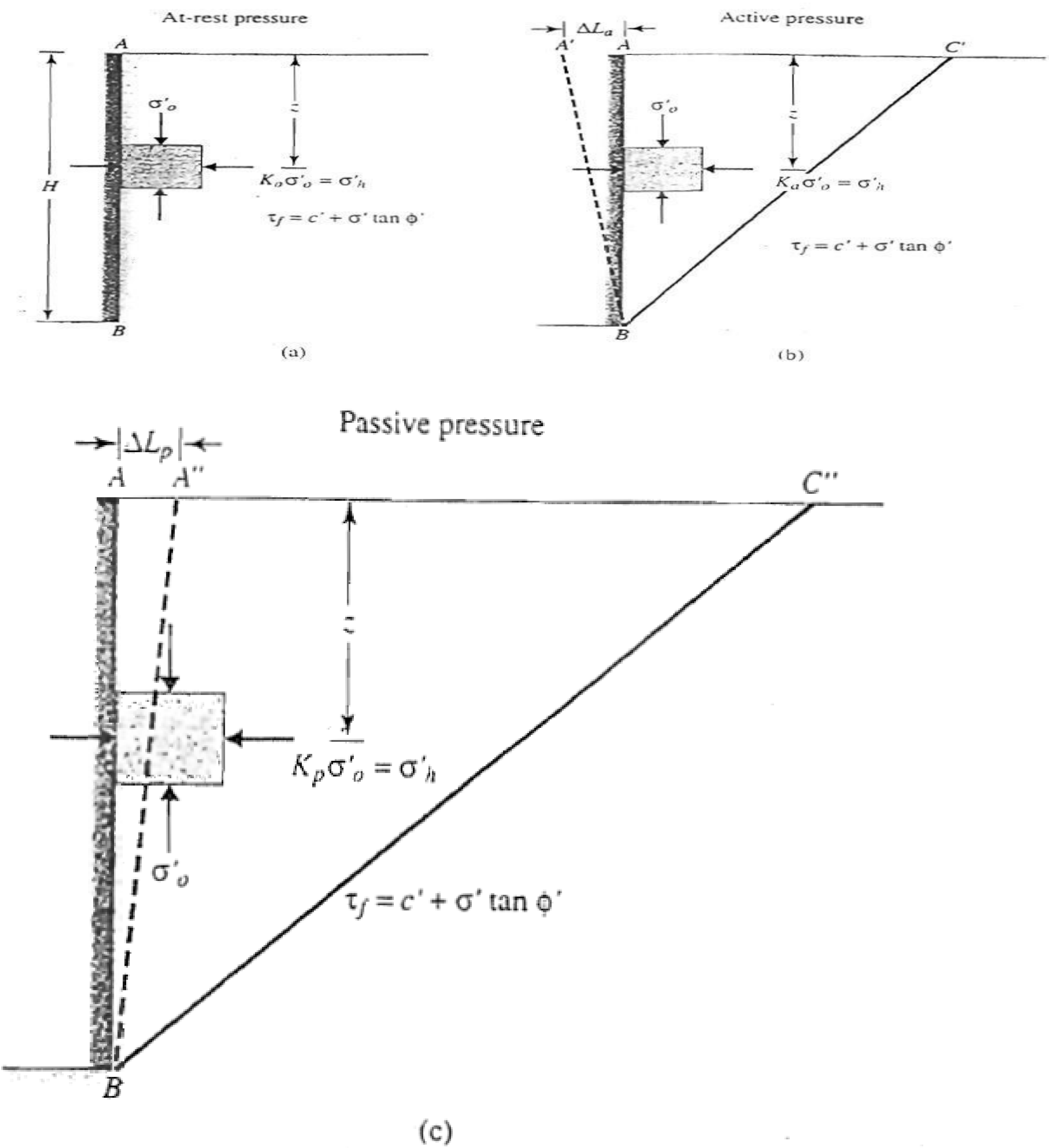


Fig. Definition of at – rest, and passive pressures (Note : Wall AB is frictionless)

Consider a mass of soil shown in Fig. 7.3(a). The mass, is bounded by a frictionless wall of height AB. A soil element located at a depth „z" is subjected to a vertical effective pressure σ'_0 and a horizontal effective pressure σ'_n. There are no shear stresses on the vertical and horizontal planes of the soil element. Let us define the ratio of σ'_n to σ'_0 as a non-dimensional quantity k, or

$$K = \frac{\sigma'_n}{\sigma'_0}.$$

Now, three possible cases may arise concerning the retaining wall and they are described.

Case 1 :-

If the wall AB is static – that is, if it does not move either to the right or to the left of its initial position- the soil mass will be in a state of static equilibrium. In that case, σ'_n is referred to as the at rest earth pressure or $K = K_0 = \dfrac{\sigma'_n}{\sigma'_0}$.

where K_0 = at rest earth pressure coefficient.

Case 2 :-

If the frictionless wall rotates sufficiently about its bottom to a position of A`B (Fig. 7.3 b), then a triangular soil mass ABC` adjacent to the wall will reach a state of plastic equilibrium and will fail sliding down the plane BC`. At this time, the horizontal effective stress, $\sigma'_n = \sigma'_a$, will be referred to as active pressure. Now,

$$K = \frac{\sigma'_n}{\sigma'_0} = \frac{\sigma'_a}{\sigma'_0} = K_a$$

where K_a = active earth pressure coefficient.

LECTURE 7

Case 3 :-

If the frictionless wall rotates sufficiently about its bottom to a position $A'B$ (Fig.7.3(c)), then a triangular soil mass ABC' will reach a state of plastic equilibrium and will fail sliding upward along the plane BC'. The horizontal effective stress at this time will be $\sigma_n' = \sigma_p'$, the so called passive pressure. In this case.

$$K = \frac{\sigma_n'}{\sigma_0'} = \frac{\sigma_p'}{\sigma_0'} = K_p$$

Where K_p = passive earth pressure coefficient.
Rankine"s Theory of Active Pressure

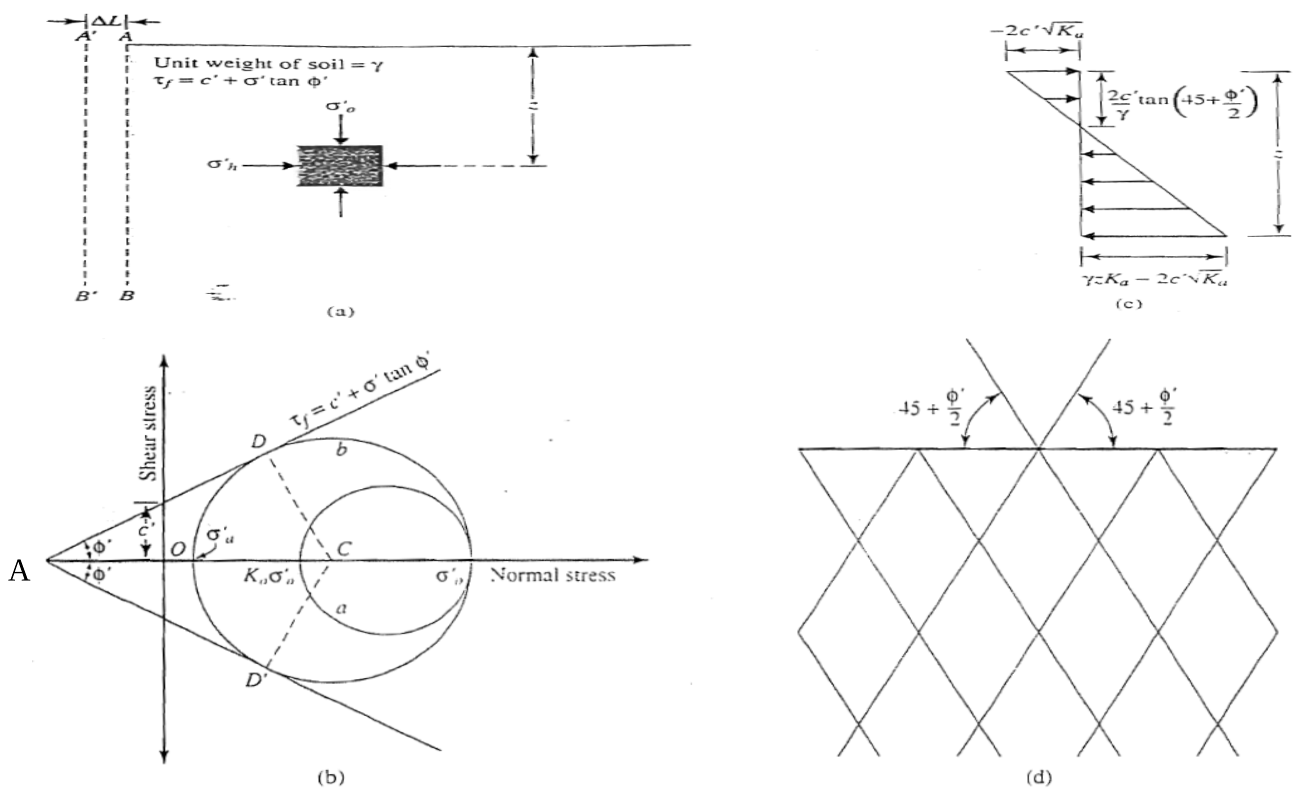


Fig. Rankine’s active earth pressure

$$\sin\phi' = \frac{CD}{AC} = \frac{CD}{AO + OC}$$

But CD = radius of the failure circle.

$$= \frac{\sigma_0' - \sigma_a'}{2}$$

AO = $c'\cot\phi'$ and OC = $\frac{\sigma_0' + \sigma_a'}{2}$

$$\text{So } \sin\phi' = \frac{\dfrac{\sigma_0' - \sigma_a'}{2}}{c'\cot\phi' + \dfrac{\sigma_0' + \sigma_a'}{2}}$$

$$\text{Or } c'\cos\phi' + \frac{\sigma_0' + \sigma_a'}{2}\sin\phi' = \frac{\sigma_0' - \sigma_a'}{2}$$

$$\sigma_a' = \sigma_0' \frac{1-\sin\phi'}{1+\sin\phi'} - 2c'\frac{\cos\phi'}{1+\sin\phi'} \quad \text{------------------------------} \quad 2.1$$

But σ_0' = Vertical effective overburden pressure = rz

$$\frac{1-\sin\phi'}{1+\sin\phi'} = \tan^2\left(45 - \frac{\phi'}{2}\right) \text{ and } \frac{\cos\phi'}{1+\sin\phi'} = \tan\left(45 - \frac{\phi'}{2}\right)$$

Substituting the preceding values into equation 2.1 we get

$$\sigma_a' = rz\tan^2\left(45 - \frac{\phi'}{2}\right) - 2c'\tan\left(45 - \frac{\phi'}{2}\right)$$

$$K_a = \frac{\sigma_a'}{\sigma_0'} = \tan^2\left(45 - \frac{\phi'}{2}\right)$$

Theory of Rankine's passive pressure :-

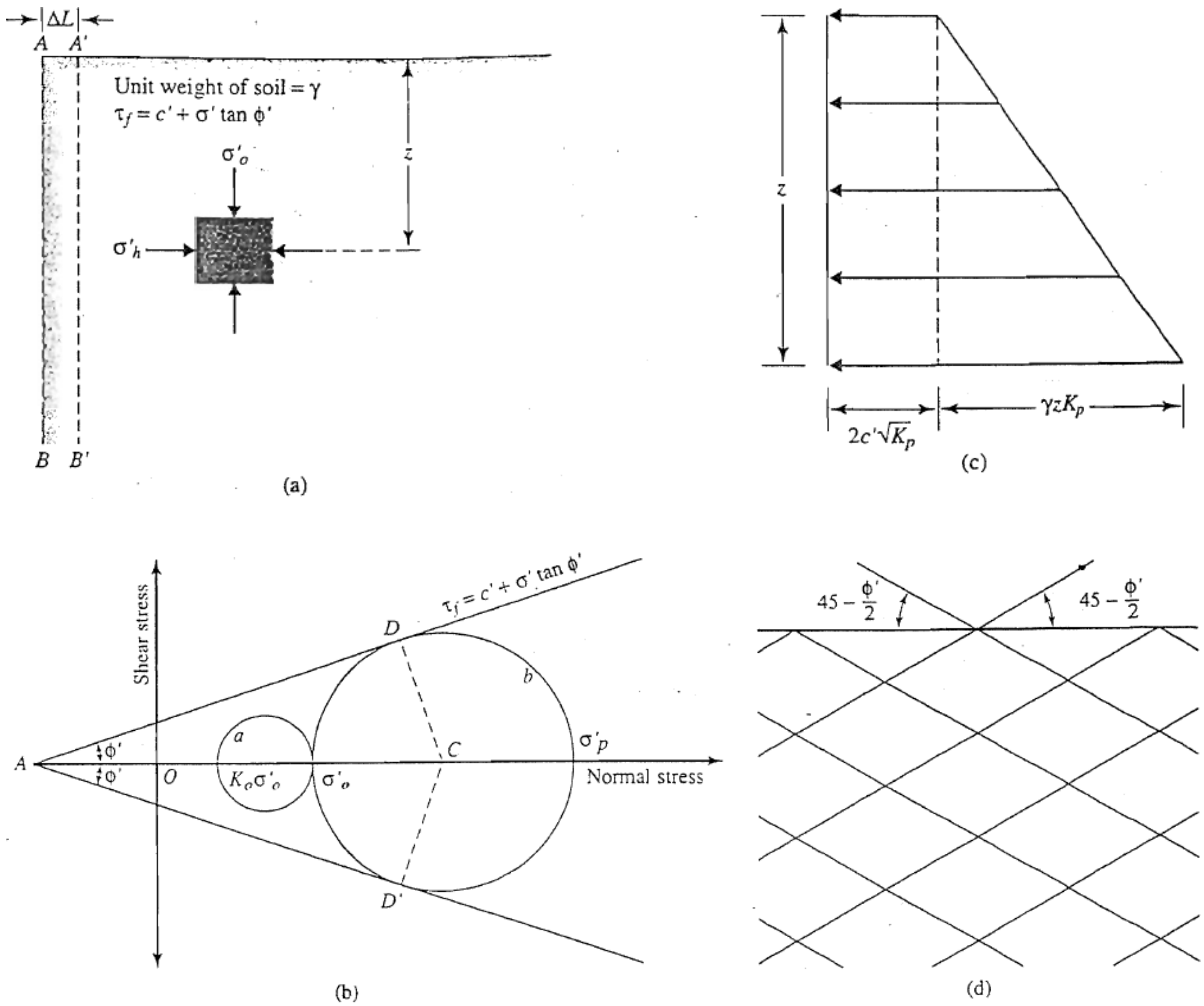


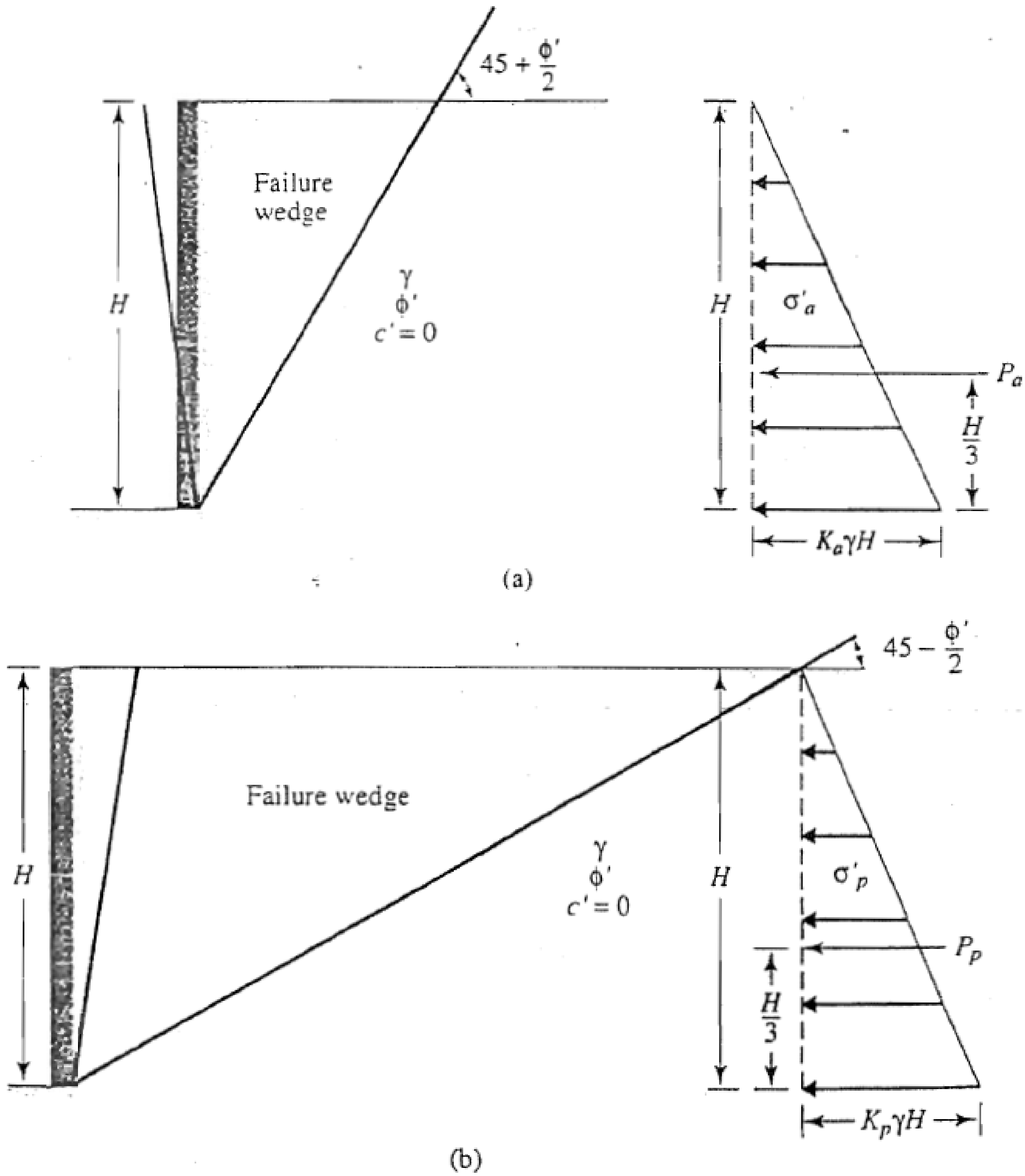
Fig. Rankine's passive earth pressure

$$\sigma_p' = \sigma_0' \tan^2\left(45 + \frac{\phi'}{2}\right) + 2C' \tan\left(45 + \frac{\phi'}{2}\right)$$

$$= rz \tan^2\left(45 + \frac{\phi'}{2}\right) + 2C' \tan\left(45 + \frac{\phi'}{2}\right)$$

$$\frac{\sigma_p'}{\sigma_0'} = \tan^2\left(45 + \frac{\phi'}{2}\right) = K_p$$

Backfill – Cohesionless soil with horizontal ground surface.



Pressure distribution against a retaining wall for cohesionless soil backfillwith horizontal ground surface (a) Rankine's active state (b) Rankine's passive state

$$\sigma_a' = K_a \gamma z = K_a \gamma H \quad (Note\ c' = 0)$$

$$P_a = (1/2) K_a \gamma H^2$$

Passive case

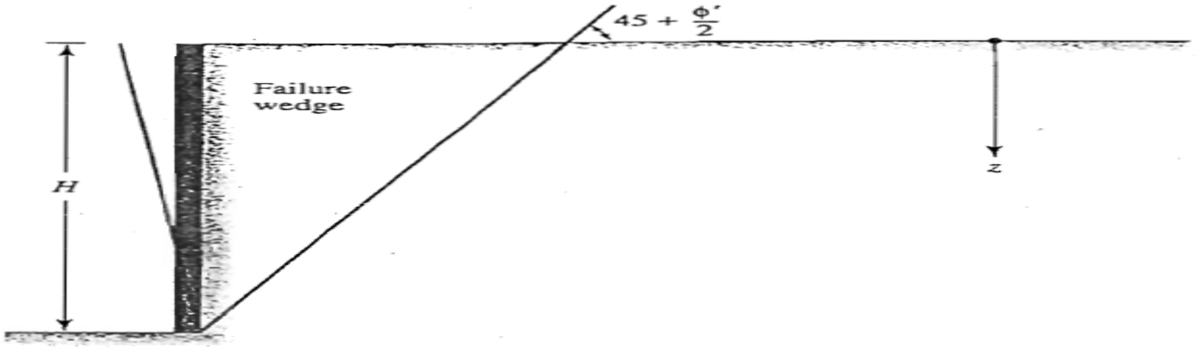
$$\sigma_p' = K_p \gamma H$$

$$P_p = (1/2) K_p \gamma H^2$$

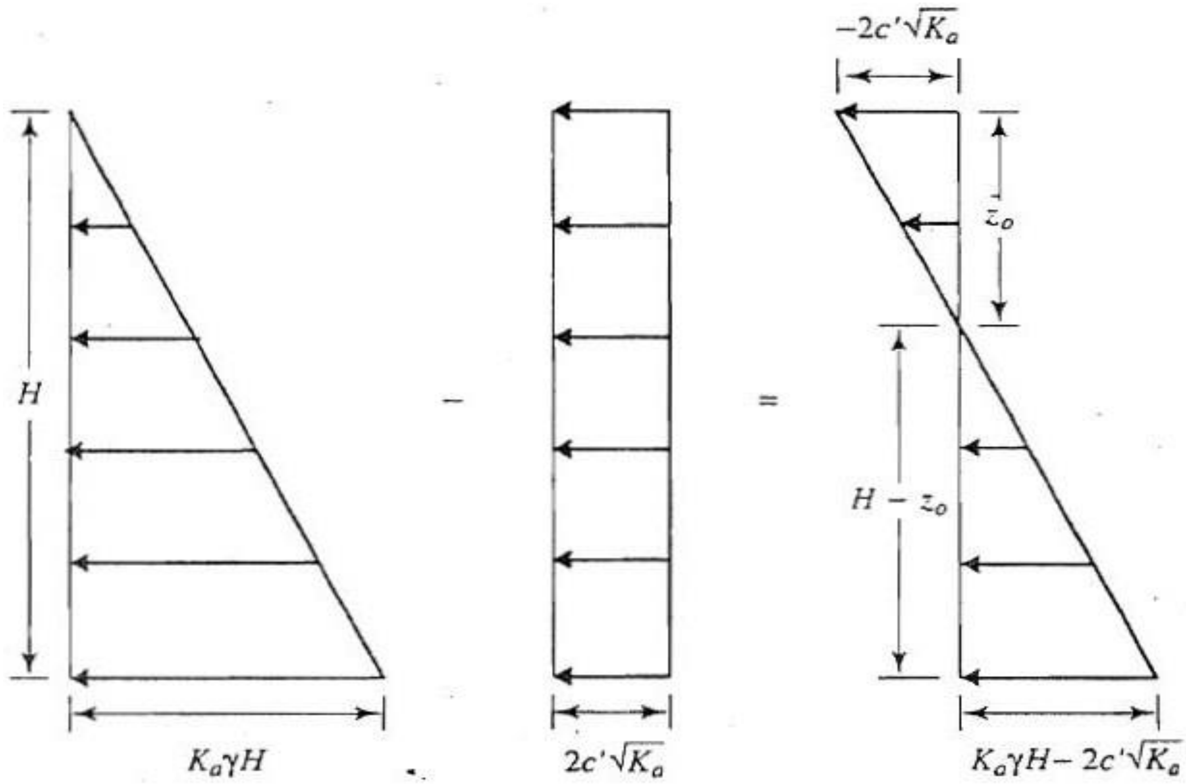
LECTURE 8

Backfill – cohesive soil with horizontal backfill.

Active Case



(a)



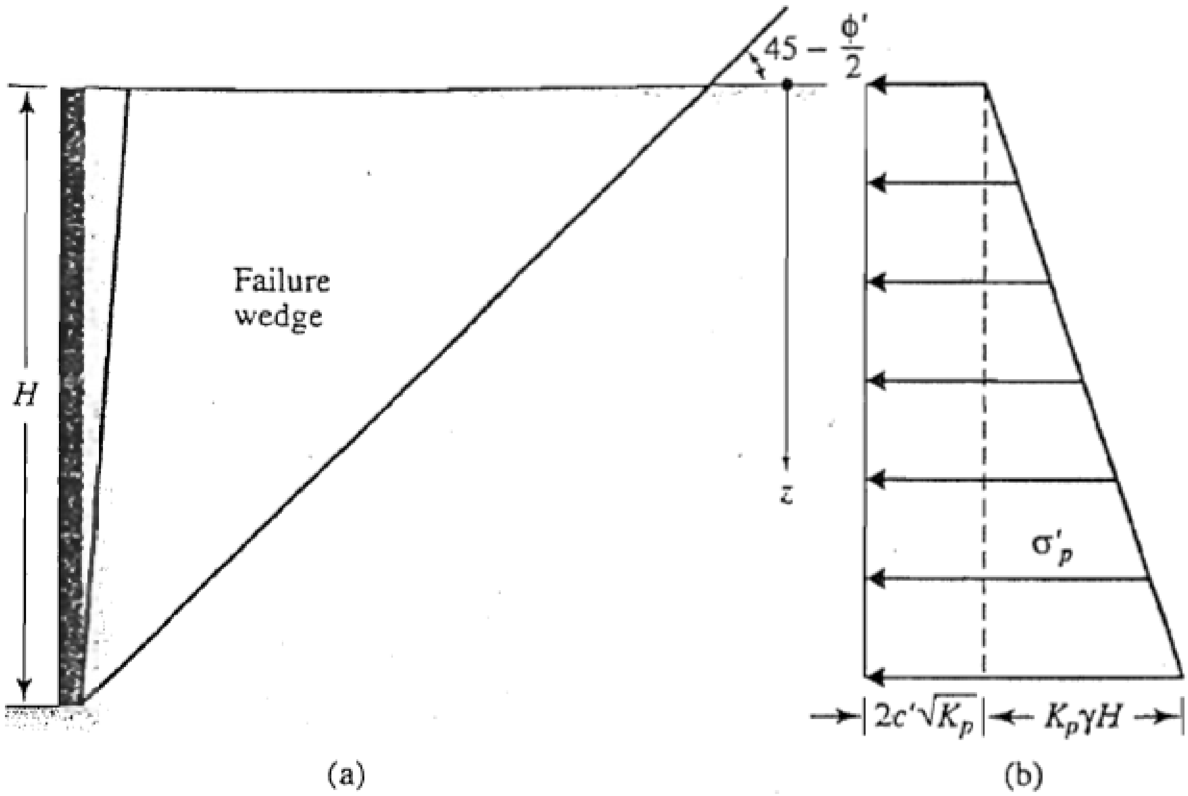
(b) (c) (d)

$$\sigma_a' = K_a \gamma z - 2\sqrt{K_a}c'$$

$$K_a \gamma z_o - 2\sqrt{K_a}c' = 0$$

$Z_0 = 2c'/(\gamma \sqrt{ka})$

Passive case

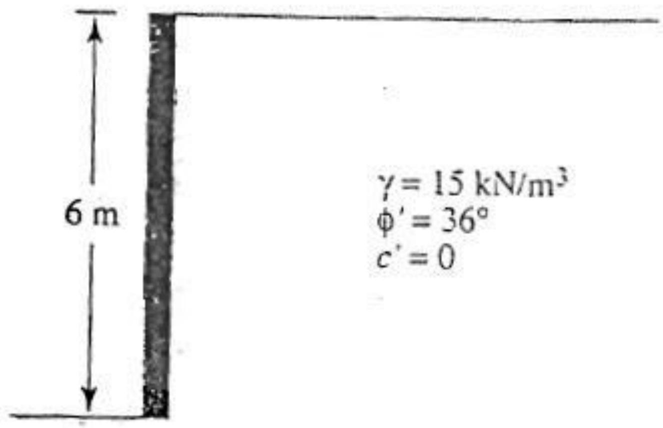


$\sigma_p' = K_p \gamma z + 2\sqrt{K_p}\, c'$

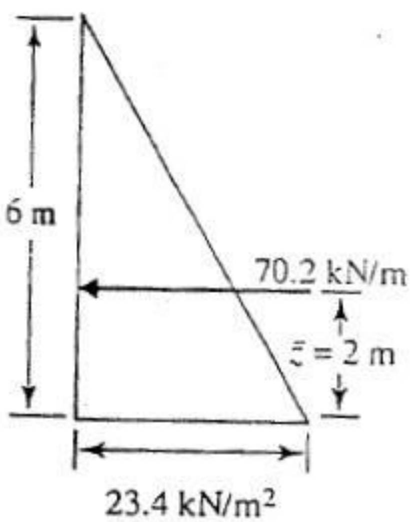
At Z = H

$\sigma_p' = K_p \gamma H + 2\sqrt{K_p}\, c'$

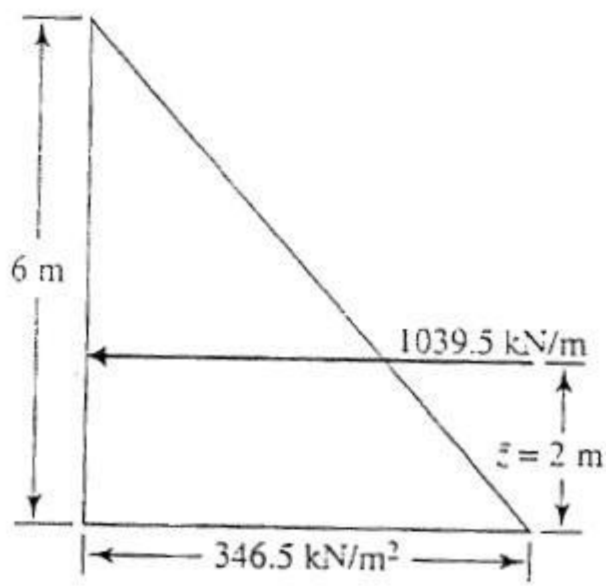
$P_p = (1/2)\, Kp\, \gamma H^2 + 2\sqrt{K_p}\, c' H$



(a)



(b)



(c)

Q.An 6m high retaining wall is shown in above figure determine.

(a) The Rankine active force per unit length of the wall and the location of the resultant.

(b) The Rankine passive force per unit length of the wall and the location of the resultant.

Solution :

As $c' = 0$, so

$\sigma_a' = K_a \sigma_0' = K_a \gamma z$

$$K_a = \frac{1 - \sin\phi'}{1 + \sin\phi'} = \frac{1 - \sin 36}{1 + \sin 36} = 0.26$$

At $Z = 0, \sigma_a' = 0; at\ z = 6m$

$$\sigma_a' = 0.26 \times 15 \times 6 = 23.4\ kN/m^2$$

$$Pa = \frac{1}{2} \times 6 \times 23.4 = 70.2 kN/m$$

Also,

$$Z = \frac{6}{3} = 2m$$

$\sigma_p' = K_p \sigma_0' = K_p \gamma z$

$$K_p = \frac{1 + \sin\phi'}{1 - \sin\phi'} = \frac{1 + \sin 36}{1 - \sin 36} = 3.85$$

At z = 0, $\sigma_p' = 0$, at z = 6m

$$\sigma_p' = 3.85 \times 15 \times 6 = 346.5 kN/m^2$$

$$P_p = \frac{1}{2} \times 6 \times 346.5 = 1039.5 kN/m$$

Also $Z = \frac{6}{3} = 2m.$

LECTURE 9

Coulomb‟s Active Pressure

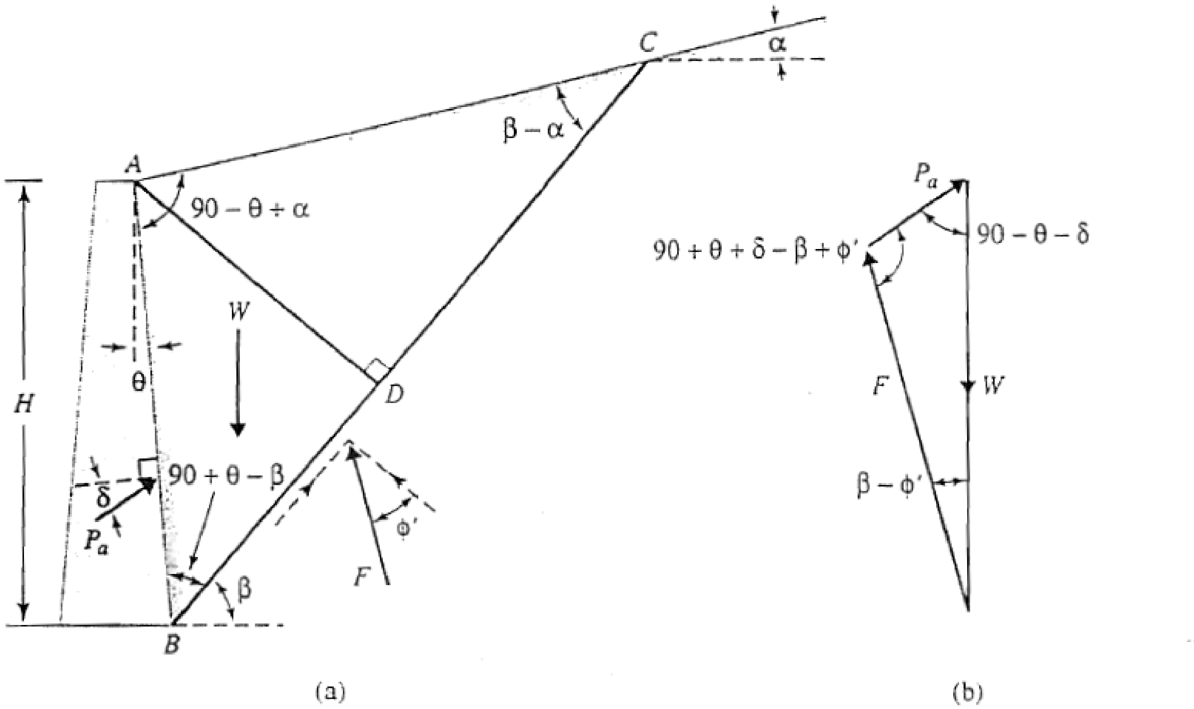


Fig. 1 Coulomb's active pressure

(a) Trial Failure wedge (b) Force polygonForces :-

1. W, the weight of the soil wedge.
2. F, the resultant of the shear and normal forces on the surface of failure BC. This is inclined at an angle of ϕ' to the normal drawn to the plane BC.
3. P_a, the active force per unit length of the wall. The direction of P_a is inclined at an angle to δ to the normal drawn to the face of the wall that supports the soil δ is the angle of friction between the soil and the wall.

The force triangle for the wedge is shown in Fig. 10.1 (b) from the law of sines, we have

$$\frac{W}{\sin(90+\theta+\delta-\beta+\phi')}=\frac{P_a}{\sin(\beta-\phi')}$$

$$P_a = (1/2)\gamma H^2 \left[\frac{\cos(\theta-\beta)\cos(\theta-\alpha)\sin(\beta-\phi')}{\cos^2\theta\,\sin(\beta-\alpha)\sin(90+\theta+\delta-\beta+\phi')} \right]$$

$$\frac{dp_a}{d\beta} = 0$$

$$P_a = (1/2)K_a\gamma H^2$$

$$K_a = \frac{\cos^2(\phi'-\theta)}{\cos^2\theta\,\cos(\delta+\theta)\left[1+\sqrt{\frac{\sin(\delta+\phi')\sin(\phi'-\alpha)}{\cos(\delta+\theta)\cos(\theta-\alpha)}}\right]^2}$$

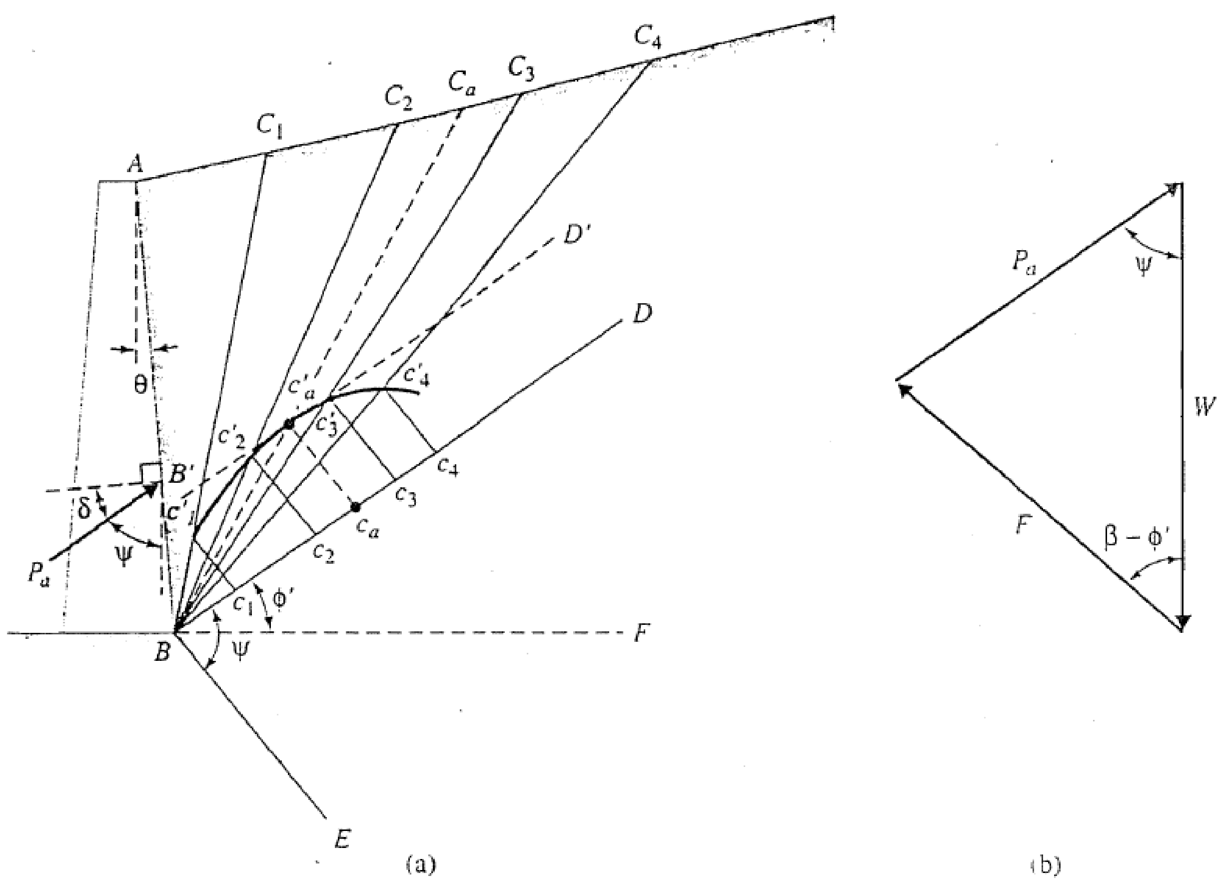


Fig. Culmann’s solution for active earth pressure.

1. Draw the features of the retaining wall and the backfill to a convenient scale.
2. Determine the value of φ (degrees) = $90-\theta-\delta$, where θ = the inclination of the back face

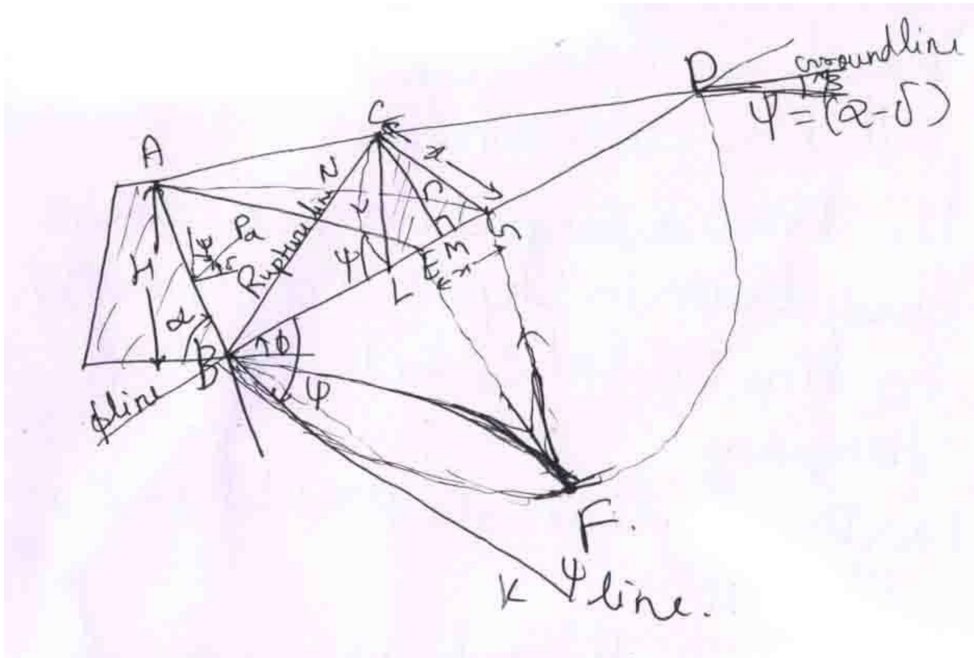
of the retaining wall with the vertical and δ = angle of wall friction.

3. Draw a line BD that makes an angle ϕ' with the horizontal.

4. Draw a line BE that makes an angle ψ with line BD.
5. To consider some trial failure wedges draw lines BC_1, BC_2, BC_3,… ....BC_n.
6. Find the areas of ABC_1, ABC_2, ABC_3..... ABC_n.
7. Determine the weight of soil, W, per unit length of the retaining wall in each of the trial failure wedges as follows :
   W_1 = (Area of ABC_1) x γ x (1)
   W_2 = (Area of ABC_2) x γ x 1
   W_3 = (Area of ABC_3) x γ x 1
   .
   .
   .
   .
   W_n = (Area of ABC_n) x γ x (1)
8. Adopt a conventional load scale and plot the weights W_1, W_2, W_3….W_n determined from step 7 on line BD.
9. Draw c_1c_1\`, $c_2$ $c_2$\`, c $_3c_3$\`… .......$C_nc_n$\` parallel to the line BE.
10. Draw a smooth curve through points c_1\`, $c_2$\`, c_3\`… .$c_n$\`. This curve is called the Culmann line.
11. Draw a tangent B\`D\` to the smooth curve drawn in step 10. B\`D\` is parallel to line BD. Let c_a\` be the point of tangency.
12. Draw a line c_aC_a\` parallel to the line BE.
13. Determine the active force per unit length of wall as

    P_a = (Length of C_aC_a\`) x (Load scale)
14. Draw a line Bc_a\`C_a. ABC_a is the desired failure wedge.

Rebhann‟s graphical method :

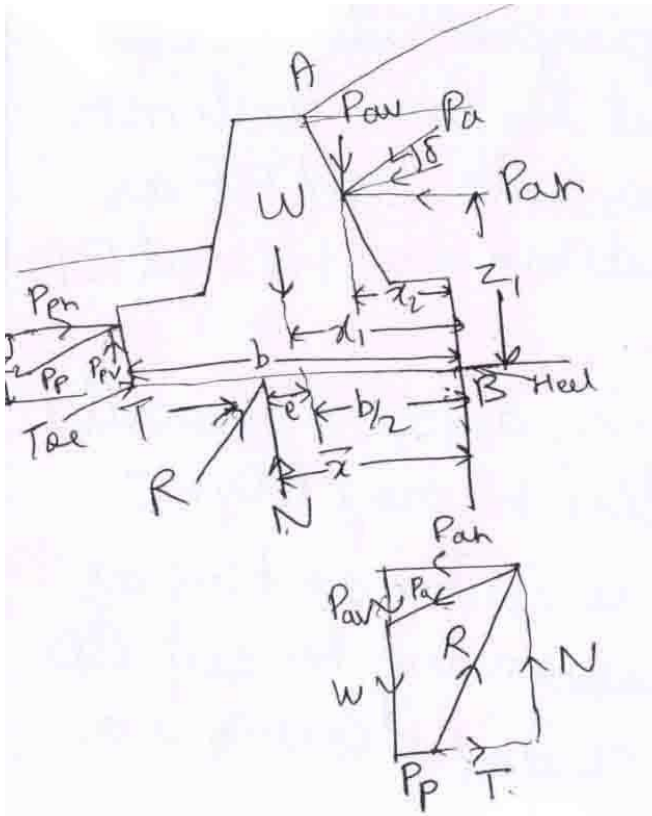


The steps involved in the graphical method are as follows :

1. Let AB represent the back face of the wall and AD the backfill surface.
2. Draw BD inclined at φ with the horizontal from the heel B of the wall to meet the backfill surface in D.
3. Draw BK inclined at $\varphi = (\alpha - \delta)$ with BD, which is the φ –line.
4. Through A, draw AE parallel to the ψ line to meet BD in E. Alternatively draw AE at $(\varphi + \delta)$ with AB to meet BD in E.
5. Describe a semicircle on BD as diameter.
6. Erect a perpendicular to BD at E to meet the semi-circle in F.
7. With B as centre and BF as radius draw an arc to meet BD in G.
8. Through G, draw a parallel line which is parallel to the ψ line to meet AD in C.
9. With G as centre and GC as radius draw an arc to cut BD in L, join CL and also draw a perpendicular CM from C on to LG. BC is the required rupture surface.

LECTURE 10

Stability considerations for gravity
Retaining walls :



$N = W + P_{av} - P_{pv}$
$T = P_{ah} - P_{ph}$

Taking moments about B.

$$N\bar{x} = Wx_1 + P_{av}x_2 + P_{ah}z_1 - P_{pv}b - P_{ph}z_2$$

$$x = \frac{(wx_1 + p_{av}x_2 + p_{ah}z_1 - p_{pv}b - p_{ph}z_2)}{N}$$

$\Rightarrow \sum M / \sum V$

*e = $\left(\bar{x} \sim \frac{b}{2}\right)$

$\sum M$ = Algebraic sum of the moments of all the actuating forces, other than that of reaction N.

$\sum V$ = Algebraic sum of all the vertical forces, other than T.

The criteria for a satisfactory design of a gravity retaining wall may be enunciated as follows :

(a) The base width of the wall must be such that the maximum pressure exerted on the foundation soil does not exceed the safe bearing capacity of the soil.
(b) Tension should not develop anywhere in the wall.
(c) The wall must be safe against sliding, that is, the factor of safety against sliding should be adequate.
(d) The wall must be safe against overturning, that is the factor of safety against overturning should be adequate.

$$\sigma_{max} = \frac{N}{b}\left(1 + \frac{6e}{b}\right)$$

$$\sigma_{min} = \frac{N}{b}\left(1 - \frac{6e}{b}\right)$$

Stability analysis of infinite slopes:-

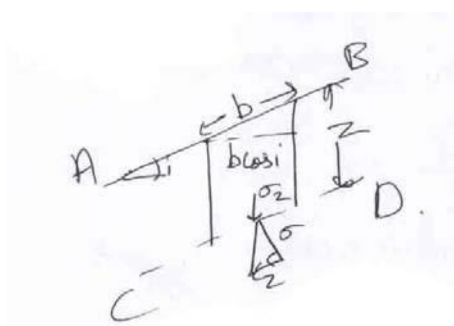


Fig. 11.2 Infinite slope

Considering a prism of soil of inclined length b‟ along the slope and depth z‟ upto the critical surface. The horizontal length of prism is *bcosi* , and its volume per unit length of prism is zbcosi.

$\therefore$ Weight of prism = w = γzb cos i

$\therefore$ Vertical stress σ_z on the surface CD is given by

$$\sigma_z = \frac{W}{b} = \gamma z \cos i$$

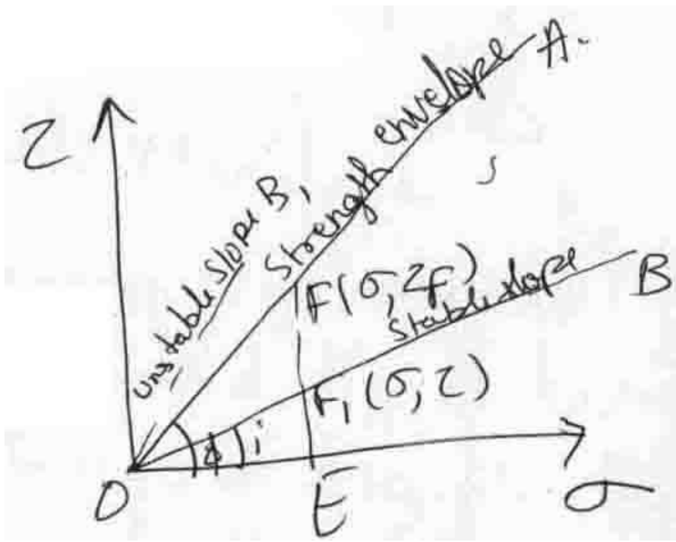
If σ and τ are the stress components normal and tangential to the surface CD, we have

$\sigma = \sigma_z \cos i = \gamma z \cos^2 i$.

and $\tau = \sigma_z \sin i = \gamma z \cos i \sin i$

$$F = \frac{\tau_F}{\tau}$$

Case (i) cohesionless soil.



$\tau_F = \sigma \tan\phi$

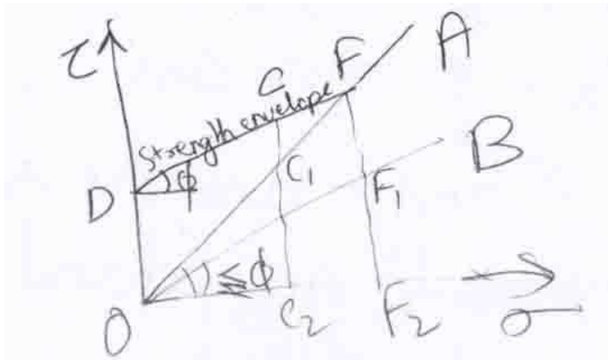
$$\frac{\sigma}{z} = \frac{\cos i}{\sin i} = \cot i = \text{constant.}$$

$\sigma = \tau \cot i$

$\tau = \sigma \tan i$

$$F = \frac{\tau_F}{\tau} = \frac{\tan\phi}{\tan i}$$

Case (ii) cohesive soil :-



$$\tau_F = c + \sigma \tan\phi$$

$$F = \frac{\tau_F}{\tau} = \frac{c + \sigma \tan\phi}{\tau}$$

Putting $\sigma = \gamma z \cos^2 i$

and $\tau = \gamma z \cos i \sin i$ we get

$$F = \frac{c + \gamma z \cos^2 i \tan\phi}{\gamma z \cos i \sin i} = \frac{C}{\gamma z \sin i \cos i} + \frac{\tan\phi}{\tan i}$$

For $z = H_c$ and $F = 1$

$\gamma H_c \cos i \sin i = c + \gamma H_c \cos^2 i \tan\phi$

$$H_c = \frac{c}{\gamma} \frac{1}{(\tan i - \tan\phi)\cos^2 i}$$

$$\frac{C}{\gamma H_c} = (\tan i - \tan\phi)\cos^2 i$$

$$S_n = \frac{C}{\gamma H_c} = \text{Stability number.}$$

If F_c represents the factor of safety with respect to cohesion and let C_m be the mobilized cohesion, at depth H, given by $\tau = \tau$

$m \quad F_c$

$$S_n = \frac{C}{\gamma H_c} = \frac{C_m}{\gamma H} = \frac{C}{F_c \gamma H}$$

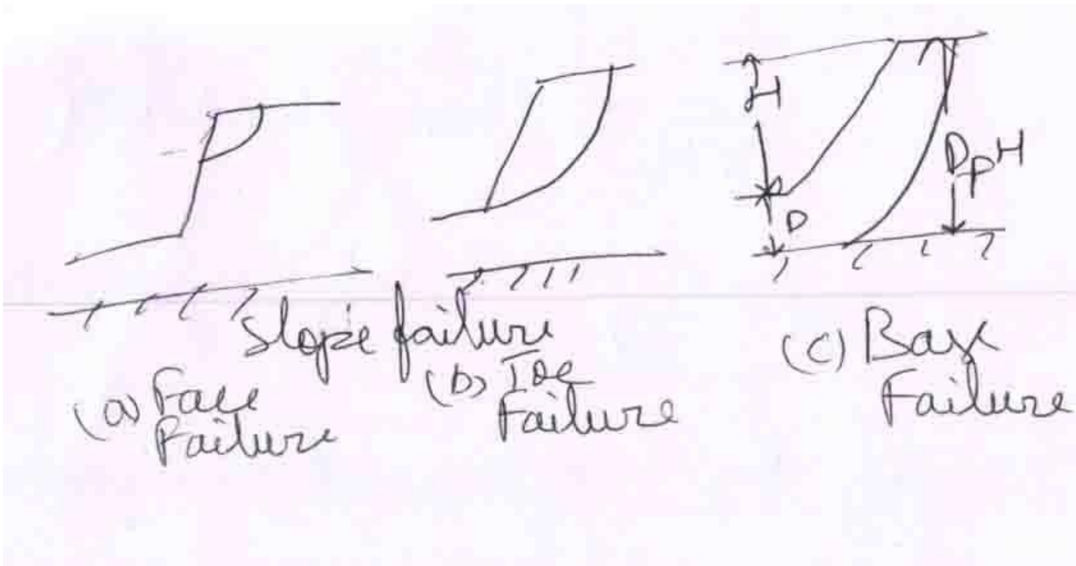
$$F_c = \frac{H_c}{H} = \text{ factor of safety with respect to cohesion.}$$

LECTURE 11

Stability analysis of finite slopes.

Two basic types of failure of a finite slope may occur (i) slope failure

(ii) Base Failure.

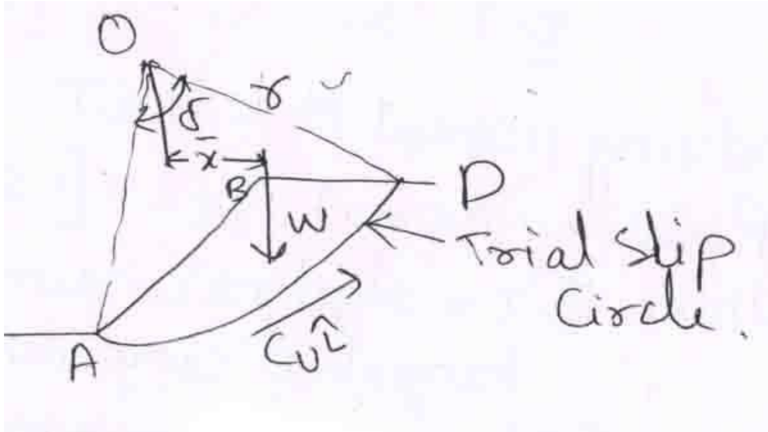


Swedish slip circle method :

(i) Analysis of purely cohesive soil (ϕ_u =0 analysis)

(ii) Analysis of a soil possessing both cohesion and friction (c-ϕ analysis)

(i) ϕ_u = 0 analysis.

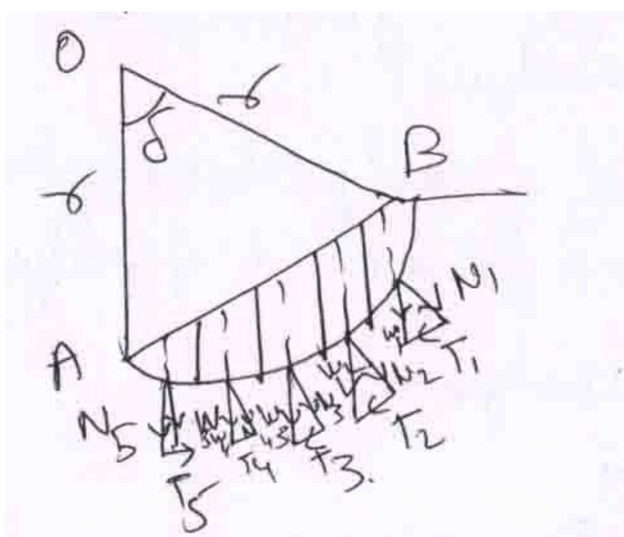


$$F = \frac{M_R}{M_D} = \frac{C_u \hat{L} r}{W\bar{x}}$$

$$W\bar{x} = c_m \hat{L} r \quad \text{or} \quad c_m = \frac{W\bar{x}}{\hat{L} r}$$

$$F = \frac{c_u}{c_m} = \frac{C_u \hat{L} r}{W\bar{x}}$$

(ii) C - ϕ analysis



Driving moment $M_D = r\sum T$

Resisting moment $M_R = r\left[c\sum \Delta L + \tan\phi \sum N\right]$

where $\sum T$ = algebraic sum of all tangential components

$\sum N$ = sum of all normal components

$\sum \Delta L = L' = \dfrac{2\pi r\delta}{360^0}$ = length AB of slip circle.

$$F = \frac{M_R}{M_D} = \frac{cL' + \tan\phi \sum N}{\sum T}$$

MODULE:2

Lecture-1

The theories are: 1. Rankine's Theory of Bearing Capacity 2. Prandtl's Theory of Bearing Capacity 3. Terzaghi's Theory of Bearing Capacity 4. Skempton's Theory of Bearing Capacity 5. Meyerhof's Theory 6. Hansen's Theory of Bearing Capacity 7. Vesic's Theory of Bearing Capacity.

1. Rankine's Theory of Bearing Capacity:

Rankine (1885) attempted to determine ultimate bearing capacity of the soil by considering the equilibrium of two elements of the soil, one below the footing and another outside the footing adjacent to the first element.

Following is the equation for ultimate bearing capacity as per Rankine's theory for cohesionless soil:

$$q_u = \gamma d_f \left[\frac{(1+\sin\phi)}{(1-\sin\phi)}\right]^2$$

or

$$q_u = K_p^2 \gamma d_f$$

where K_p is the Rankine's coefficient of passive earth pressure.

As per Rankine's theory, when the depth of foundation is zero, the ultimate bearing capacity is also zero, which is not true. As the Rankine's theory does not give reliable value of ultimate bearing capacity, it is rarely used in practice. Instead, Rankine's theory is used to determine the minimum depth of foundation as –

$$d_f = \frac{q}{\gamma}\left[\frac{(1-\sin\phi)}{(1+\sin\phi)}\right]^2$$

or

$$d_f = \frac{K_a^2 q}{\gamma}$$

where K_a is the Rankine's coefficient of active earth pressure and q the maximum pressure applied at the base of the foundation.

2. Prandtl's Theory of Bearing Capacity:

Prandtl (1920) proposed his theory of bearing capacity based on his study of penetration or punching of long hard metal puncher into a softer material. While all modern theories of bearing capacity are based on Terzaghi's theory, Terzaghi's theory itself is based on Prandtl's theory of bearing capacity.

Lecture-2

Prandtl considered a strip footing with a smooth base that sinks vertically down when placed on a ground surface. Figure 18.1 shows the failure surface of the soil as per Prandtl's theory. Prandtl has shown that when a continuous footing, with a smooth base, rests on a weightless soil possessing cohesion and friction angle, it sinks into the soil that fails by punching shear.

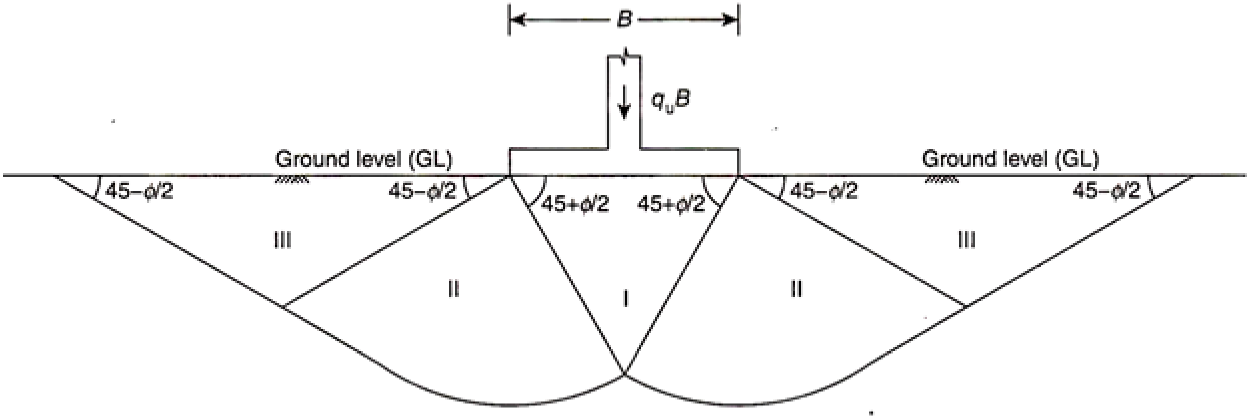


Figure18.1 Failure surface in Prandtl's theory.

When the footing sinks, the wedge-shaped zone-I below the footing, pushes zone-II in the lateral direction, which in turn pushes zone-III in upward direction. Both zone-II and zone-III are in plastic equilibrium. By considering the shape of failure surface of zone-II as logarithmic spiral, Prandtl gave the following equation for ultimate bearing capacity based on the theory of plasticity –

$$q_u = \frac{c}{\tan\phi}\left[\left\{\tan^2\left(45+\frac{\phi}{2}\right)e^{\pi\tan\phi}\right\}-1\right]$$

For a purely cohesive soil, $\phi_u = 0$. The logarithmic spiral becomes a circular arc and the ultimate bearing capacity is given by –

$q_u = (\pi + 2)c_u = 5.14c_u$ …(18.7)

Since the actual footings have a rough base, Prandtl's theory does not give accurate results.

3. Terzaghi's Theory of Bearing Capacity:

Terzaghi gave a general theory for the determination of ultimate bearing capacity of a strip footing. A strip footing is a continuous footing provided to support the wall of a load bearing structure. The footing is assumed to be continuous with length-width ratio (L/B) more than 10, so that the problem is assumed to be two-dimensional. Terzaghi's theory (1943) of bearing capacity is based on Prandtl's theory (1921).

Failure Surface in Terzaghi's Theory:

The base of the footing is assumed to be rough, unlike in Prandtl's theory, so that a wedge-shaped mass of soil abc as shown in Fig. 18.2 does not undergo any lateral displacement and sinks vertically down, when the footing is subjected to the pressure q_u, equal to the ultimate bearing capacity of the soil.

The shape of elastic zone depends on the density and cohesion of the soil. The sides ca and cb rise at an angle ϕ with horizontal. The wedge of soil abc, known as zone-I, is in a state of elastic equilibrium and behaves as if it were a part of the footing.

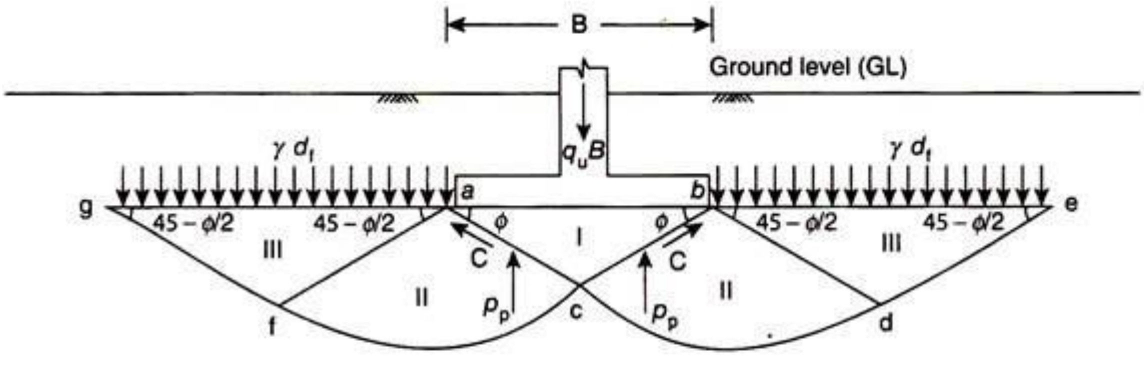


Figure 18.2 Failure surface as per Terzaghi's theory.

Figure shows the failure surface in Terzaghi's theory, which does not extend above the base of the footing. Thus, Terzaghi neglected the shear strength of the soil above the base of the footing. Terzaghi considered the effect of the soil above the base of the footing in the form of a surcharge, γd_f, acting at the level of the base of the footing, where d_f is the depth of foundation. This assumption limits the applicability of Terzaghi's theory to shallow foundations. For deep foundations, neglecting the shear strength of soil above the base of footing leads to serious errors.

Lecture-3

The failure surface in Terzaghi's theory consists of zone-II on the either side of zone-I and zone-Ill on the outer side of footing adjacent to zone-II. Zone-II is called the zone of radial shear. Sides ca and cb act as retaining walls pushing the soil in zone-II in the downward and outward direction. The bottom surface cd and cf of zone-II is a logarithmic spiral, with its center at points b and a, respectively, on the either side of the footing. Surfaces cd and cf can be defined by the equation –

$r = r_{0e\theta}{}^{\tan\phi}$ …(18.8)

where θ is the angle subtended at the center of any point on the spiral with the zero-angle radius vector cb or ca. Thus, the length of cb and ca is each equal to r_0.

Derivation of Terzaghi's Bearing Capacity Equation:

The pushing of soil by the imaginary retaining walls cb and ca through zone-II, makes the soil in zone-Ill into passive Rankine state. Sides db and de or fa and fg of zone-Ill rise at an angle (45 – ϕ/2) with the horizontal.

Terzaghi determined the ultimate bearing capacity of strip footing by considering the equilibrium of the wedge of soil abc (zone-I).

The forces acting on this wedge are as follows:

A. Upward Forces:

i. The resultant passive earth pressure, P_p, acting on the sides cb and ca at an angle ϕ with the normal to the surfaces cb and ca. Since the surfaces cb and ca are at an angle ϕ with horizontal, the direction of P_p is vertical.

ii. The cohesive force, C, acting along the sides cb and ca. The component of this cohesive force in vertical direction is equal to C sinϕ.

If c is the unit cohesion of soil, then –

C sinϕ = (c × ca sinϕ) + (c × cb sinϕ)

In Δabc (Fig. 18.3), we have ab = B and ah = B/2. So –

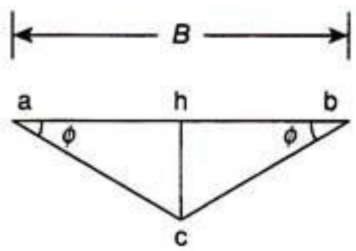


Figure 18.3 Zone-I.

$$\cos\phi = \frac{ah}{ac} \quad \Rightarrow \quad ac = \frac{ah}{\cos\phi} = \frac{B}{2\cos\phi}$$

Therefore,

$$ca = cb = \frac{B}{2\cos\phi}$$

Substituting the values of ca and cb in Eq. , we get –

$$C\sin\phi = c \times \frac{B}{2\cos\phi}\sin\phi + c \times \frac{B}{2\cos\phi}\sin\phi = Bc\tan\phi$$

A. Downward Forces:

i. The force due to the pressure q_u acting vertically downward. Its magnitude is equal to $q_u B$ per meter length of the footing.

ii. The weight of wedge of soil abc (zone-I) is –

W = γ × Area of Δabc × 1

In Δach (Fig. 18.3), ah = B/2. Hence –

$\tan\phi = ch/ah$

Therefore, ch = ah and $\tan\phi = B/(2 \tan\phi)$. So the area of Δabc will be –

$\frac{1}{2} \times ab \times ch = \frac{1}{2} \times B \times B/2 \tan\phi = B^2/4 \tan\phi$
Therefore,

$W = \gamma B^2/4 \tan\phi$
The effect of surcharge γD and the weight of soil in zone-II and zone-III is taken into account in the computation of the passive earth pressure, P_p. Equating the downward and upward forces for equilibrium of the wedge abc we have –
$q_u B + (\gamma B^2/4) \tan\phi = 2P_p + Bc \tan\phi$

Therefore –

$$q_u B = 2P_p + Bc \tan\phi - (\gamma B^2/4) \tan \phi$$

The resultant passive earth pressure P_p consists of the following three components:
a. $P_{p\gamma}$ due to the weight of soil in zone-II and zone-III, assuming the soil as cohesionless and neglecting the surcharge γD.

Lecture-4

b. P_{pc} due to the cohesion of soil in zone-II and zone-Ill assuming the soil as weightless (γ = 0) and neglecting the surcharge γD.
c. P_{pq} due to the surcharge γD, assuming the soil as cohesionless and weightless.
Terzaghi assumed that these three components of P_p can be computed independently and added, though the critical surfaces for these components are different. Thus, Terzaghi assumed that the principle of superposition is valid. Substituting the three components of P_p in Eq. , we have –

$$q_u B = 2(P_{p\gamma} + P_{pc} + P_{pq}) + Bc\tan\phi - \frac{\gamma B^2}{4}\tan\phi \qquad (18.13)$$

On rearranging, we get

$$q_u B = 2P_{pc} + Bc\tan\phi + 2P_{pq} + 2P_{p\gamma} - \frac{\gamma B^2}{4}\tan\phi \qquad (18.14)$$

Terzaghi expressed the three components P_{pc}, P_{pq}, and $P_{p\gamma}$ in terms of another set of factors N_c, N_q, and N_γ known as the bearing capacity factors as follows:

$$2P_{pc} + Bc\tan\phi = BcN_c \qquad (18.15)$$

$$2P_{pq} = B \times \gamma DN_q \qquad (18.16)$$

$$2P_{p\gamma} - \frac{\gamma B^2}{4}\tan\phi = B \times 0.5\gamma BN_\gamma \qquad (18.17)$$

where

$$N_c = \tan\phi\,(k_c + 1) \qquad (18.18)$$

$$N_q = k_q \tan\phi \qquad (18.19)$$

$$N_\gamma = 0.5\tan\phi\,(k_\gamma \tan\phi - 1) \qquad (18.20)$$

It is extremely tedious to evaluate k_c, k_q, and k_γ. Terzaghi used an approximate method to determine the ultimate bearing capacity.
Assuming $c = 0$ and surcharge $q = \gamma D = 0$ (i.e., $D = 0$), $q_u = q_\gamma = 0.5\gamma BN_\gamma$.
Assuming $\gamma = 0$ and $q = 0$, $q_u = q_c = CN_c$.
Assuming $\gamma = 0$ and $C = 0$, $q_u = q_q = qN_q$.
Substituting these terms in Eq. (18.14), we have

$$q_u B = B \times cN_c + B \times \gamma DN_q + B \times 0.5\gamma BN_\gamma \qquad (18.21)$$

Dividing throughout with B, we get

$$q_u = cN_c + \gamma DN_q + 0.5\gamma BN_\gamma \qquad (18.22)$$

Equation (18.22) is the famous Terzaghi's bearing capacity equation for ultimate bearing capacity of strip footings. It is found to be applicable for strip footing with L/B ratio > 5.

Several theories of bearing capacity were developed later by taking into account the effect of shape and depth of footing, inclination of load or ground, etc. But the same form of the Terzaghi's equation was used by all the investigators, introducing additional factors to consider these effects. In spite of several theories coming up later, Terzaghi's theory is still popular and is used as a simple solution to determine bearing capacity.

Terzaghi's Bearing Capacity Factors:

Terzaghi found that bearing capacity factors N_c, N_q, and N_γ are functions of ϕ, the angle of shearing resistance of soil. The values of N_c, N_q and N_γ as given by Terzaghi are shown in Table .

Table 18.1 Terzaghi's bearing capacity factors

ϕ	N_c	N_q	N_γ
0	5.7	1	0
5	7.3	1.6	0.5
10	9.6	2.7	1.2
15	12.9	4.4	2.5
20	17.7	7.4	5
25	25.1	12.7	9.7
30	37.2	22.5	19.7
34	52.6	36.5	35.0
35	57.8	41.4	42.4
40	95.7	81.3	100.4
45	172.3	173.3	297.5

Assumptions in Terzaghi's Theory of Bearing Capacity:
The assumptions used for deriving the bearing capacity equation may be summarized as follows:
i. The soil mass is homogeneous and isotropic.

ii. The soil mass is semi-infinite, that is, it extends infinitely below a level surface.

iii. The footing is laid at a shallow depth, that is, D < B.

Lecture-5

iv. The footing is continuous with L/B ratio > 10, so that the problem is essentially two-dimensional. Thus, Terzaghi considered plane strain condition, neglecting the effect of intermediate principal stress.

v. The base of the footing is rough.

vi. The failure surface does not extend above the base of the footing, that is, the shear strength of the soil above the base of the footing is neglected.

vii. The effect of soil above the base of the footing is considered in the form of surcharge, γD, acting at the level of the base of the footing.

viii. The load on the footing is vertical and its line of action coincides with the centroid of the footing.

ix. The shear strength of the soil is governed by Coulomb's equation.

x. The principle of superposition is valid, so that the three components of the passive earth pressure can be computed separately and then added, although their critical surfaces are different.

Ultimate Bearing Capacity for Pure Cohesive Soils:
For pure cohesive soils, $\phi_u = 0$ and Terzaghi's bearing capacity factors are $N_c = 5.7$, $N_q = 1$; $N_\gamma = 0$; $c = c_u$. By Substituting these values in Eq. , the ultimate capacity of strip footing for pure clays is given by –

$q_u = 5.7\ c_u + \gamma D$

The net ultimate bearing capacity of strip footings for clays is, therefore, given by –

$q_{nu} = 5.7\ c_u$

Effect of Water Table on Bearing Capacity:
Terzaghi's theory of bearing capacity assumes that the groundwater table is at a great depth below the base of the foundation and hence its effect is not considered. When the groundwater table is near the level of the foundation, the unit weight of soil gets reduced. The effective or submerged unit weight of the soil shall be, therefore, used in the bearing capacity equation.

The effect of water table on the bearing capacity (Fig. 18.4) can be explained with reference to the bearing capacity equation rewritten as –

$q_u = cN_c + \gamma DN_q + 0.5\gamma BN_\gamma$

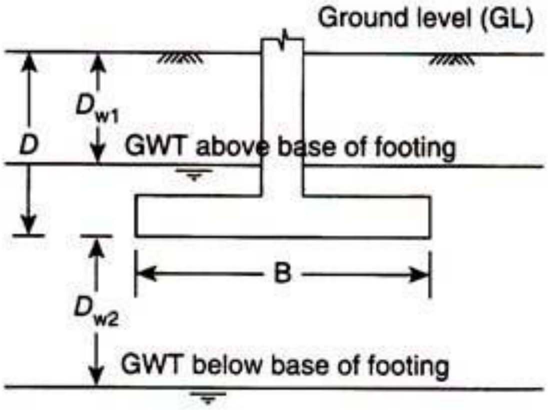


Figure 18.4 Effect of water table on bearing capacity.

The first term cN_c is not affected by the position of the water table, as it does not contain the density. However if the water table reaches the base of the footing, the unit cohesion c and the value of ϕ used to find N in the term should be determined in the laboratory under saturated conditions.

The second term γDN_q is contributed by the soil above the foundation level. Hence, this is not affected by the position of the water table, as long as the water table is at or below the base of the foundation. When the water table is at the GL, the term γDN_q gets reduced by about 50%, since for all practical purposes, the submerged density of the soil is about half of its saturated density. For any intermediate position of the water table, between the GL and the base of foundation, we may write the bearing capacity equation as –

$$q_u = cN_c + \gamma D N_q R_{w1} + 0.5\gamma B N_\gamma$$

$$R_{w1} = 0.5\left(1 + \frac{D_{w1}}{D}\right)$$

where R_{w1} is the water table correction factor, D_{w1} is the depth of water table below GL, and D is the depth of foundation.
As per Eq. when the water table is at GL, $D_{w1} = 0$ and $R_{w1} = 0.5$ and when the water table is at the base of the foundation, $D_{w1} = D$ and $R_{w1} = 1$

Lecture-6

The third term in the bearing capacity equation, that is, $0.5\ \gamma B N_\gamma$, is contributed by the soil below the base of the foundation of depth approximately equal to the width of the foundation, where the stresses due to foundation load are significant. Thus, when the water table is at a depth greater than the width of the foundation below its base the term $0.5\gamma B N_\gamma$ is not affected by the water table.
If the water table reaches the base of the foundation, the submerged density will be approximately half of the saturated density, reducing the term $0.5\gamma B N_\gamma$ by about 50%. For any intermediate position of the water table, we may write the bearing capacity equation as –

$$q_u = cN_c + \gamma D N_q R_{w1} + 0.5\gamma B N_\gamma R_{w2} \qquad (18.27)$$

$$R_{w2} = 0.5\left(1 + \frac{D_{w2}}{B}\right) \qquad (18.28)$$

where R_{w2} is the water table correction factor, D_{w2} is the depth of water table below the base of the foundation and B is the width of foundation.
When the water table is at the depth B below the base of the foundation, $D_{w2} = B$ and $R_{w2} = 1$. When the water table is at the base of the foundation, $D_{w2} = 0$ and $R_{w2} = 0.5$.
If the water table is above the base of the foundation, the soil below the base of the foundation will be under submerged condition and hence $D_{w2} = 0$ and $R_{w2} = 0.5$

Terzaghi's Bearing Capacity for Square, Circular and Rectangular Footings:

The bearing capacity equation for strip footings given by Terzaghi is modified for footings of other shapes. The ultimate bearing capacity of square footings is given as –

$q_u = 1.2cN_c + \gamma D N_q + 0.4\gamma B N_\gamma$

and the ultimate bearing capacity of circular footings will be –

$q_u = 1.2\ cN_c + \gamma D N_q + 0.3\gamma B N_\gamma$

The ultimate bearing capacity of rectangular footing will be –

$q_u = cN_cS_c + \gamma D N_q S_q + 0.5\gamma B N_\gamma S_\gamma$

where S_c, S_q , and S_γ are the shape factors defined by the following equations –

$$S_c = \left(1 + 0.2\frac{B}{L}\right) \qquad (18.32)$$

$$S_q = 1$$

$$S_\gamma = \left(1 - 0.2\frac{B}{L}\right) \qquad (18.33)$$

Lecture-7

It may be noted that for square footing, for which B = L, the shape factors are –

$S_c = 1.2$, $S_q = 1$, and $S_\gamma = 0.8$
For circular footings, the shape factors used in the equation are –

$S_c = 1.2$, $S_q = 1$, and $S_\gamma = 0.6$
Bearing Capacity for Pure Cohesive Soils for Square and Circular Footings:
For pure cohesive soils, $\phi_u = 0$ and $c = c_u$ and Terzaghi's bearing capacity factors are $N_c = 5.7$, $N_q = 1$, and $N_\gamma = 0$. Hence, the net ultimate bearing capacity of square or circular footings for pure cohesive soils, as per Terzaghi's theory is given by –
$q_{nu} = 6.84c_u$
4. Skempton's Theory of Bearing Capacity:
For saturated cohesive soils, Skempton showed that the bearing capacity factor, N_c, in Terzaghi's equation tends to increase with depth as shown in Fig. 18.9, where N_c increases with increase in D_f/B ratio.
The net ultimate bearing capacity for saturated cohesive soils under undrained conditions, as per Skempton's theory is given by –

$q_{nu} = c_uN_c$
where q_{nu} is the net ultimate bearing capacity, c_u the undrained cohesion, N_c the Skempton's bearing capacity factor given by Eqs.
For strip footing

$$N_c = 5\left(1 + 0.2\frac{D_f}{B}\right) \quad \le 7.5 \qquad (18.41)$$

For square and circular footing

$$N_c = 6\left(1 + 0.2\frac{D_f}{B}\right) \quad \le 9.0 \qquad (18.42)$$

For rectangular footing

$$N_c = 5\left(1 + 0.2\frac{D_f}{B}\right)\left(1 + 0.2\frac{B}{L}\right) \quad \text{if } \frac{D_f}{B} \le 2.5 \qquad (18.43)$$

$$N_c = 7.5\left(1 + 0.2\frac{B}{L}\right) \quad \text{if } \frac{D_f}{B} > 2.5 \qquad (18.44)$$

It may be noted that Terzaghi's value of N_c is applicable only for shallow foundations with –
$D_f/B \le 1.0$
whereas, Skempton's value of N_c can be used for all values of D_f/B.
If the shear strength of the soil for a depth of 2B/3 beneath the footing does not vary by more than about ±50% of the average value, the average value of c_u can be used in the above equation for q_{nu}.

Lecture-8

5. Meyerhof's Theory:

Meyerhof (1951) gave a general theory of bearing capacity for a strip footing at any depth. His equation is similar to that of Terzaghi, but his approach to solve the problem is different. He assumed that the logarithmic failure surface extends above the base of the foundation and as such considered the shear resistance of the soil above the base of the footing.

Figure 18.12 shows the failure surface as proposed by Meyerhof. The zone "abc" is the zone of elastic equilibrium with sides ac and be inclined at 45 + (ϕ/2) with horizontal. The zone bed is the zone of radial shear. The zone bed is the zone of mixed shear in which the shear varies from radial shear to linear shear. The surface be is known as equivalent free surface. It makes an angle β with the horizontal. The angle β increases with the depth D_f and is equal 90° for deep foundations. The resultant effect of the wedge bef of the soil is considered by the normal stress q_0 and the shear stress τ_0 on the surface be. The parameters β, q_0, and τ_0 are known as foundation depth parameters.

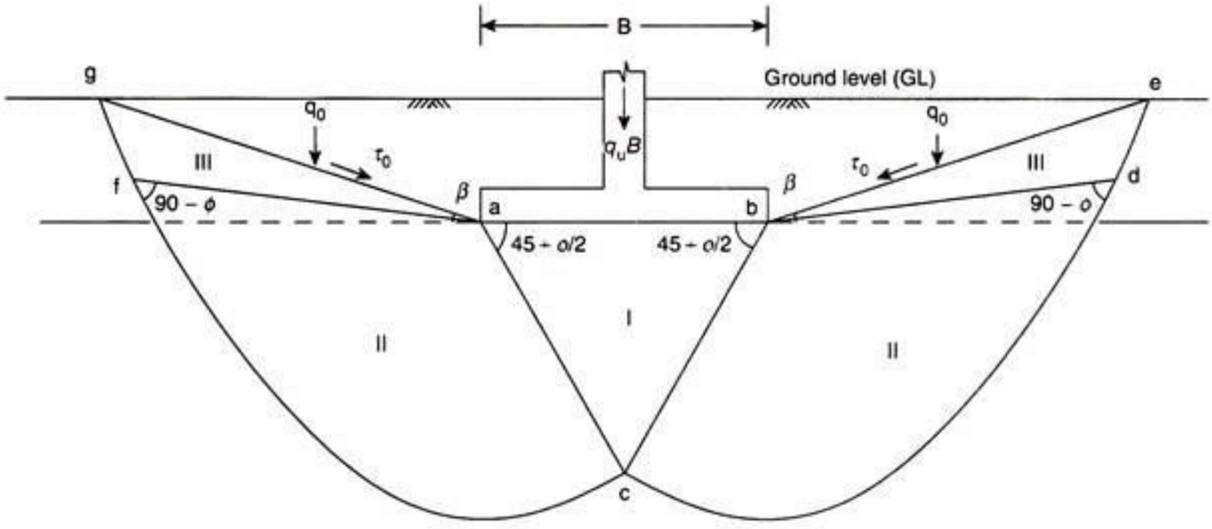


Figure 18.12 Failure surface as per Meyerhof's theory.

Meyerhof gave the following equation for ultimate bearing capacity –

$$q_u = cN_cS_cd_ci_c + \gamma D_f N_q S_q d_q i_q + 0.5\gamma B N_\gamma S_\gamma d_\gamma i_\gamma \qquad (18.45)$$

where s, d, and i are the shape, depth, and inclination factors, respectively. N_c, N_q, and N_γ are the bearing capacity factors.

$$N_q = e^{\pi \tan\phi} \tan^2\left(45 + \frac{\phi}{2}\right) \qquad (18.46)$$

$$N_c = (N_q - 1)\cot\phi \qquad (18.47)$$

$$N_\gamma = (N_q - 1)\tan 1.4\phi \qquad (18.48)$$

N_c, N_q, and N_γ depend on the roughness of the base of footing, shape of the footing in addition to D_f and ϕ. Table 18.3 gives the Meyerhof's bearing capacity factors for different values of ϕ.

Table 18.3 Meyerhof's bearing capacity factors

ϕ	N_c	N_q	N_γ
0	5.14	1.0	0.0
5	6.5	1.6	0.07
10	8.3	2.5	0.37
15	11.0	3.9	1.1
20	14.8	6.4	2.9
25	20.7	10.7	6.8
30	30.1	18.4	15.7
32	35.5	23.2	22.0
34	42.4	29.4	31.2
36	50.6	37.7	44.4
38	61.4	48.9	64.1
40	75.3	64.2	93.7
42	93.7	85.4	139.3
44	118.4	115.3	211.4

The ultimate bearing capacity, given by Meyerhof's theory, is close to the experimental values. For shallow footings, the Meyerhof's bearing capacity lies in between the general and local shear values of Terzaghi's analysis. However, for deep footings, Meyerhof's analysis gives values much greater than those given by Terzaghi's analysis. The main advantage of Meyerhof's theory is that it can also be used for deep foundations as well as for footings on slopes.

Meyerhof introduced the concept of effective width to compute bearing capacity for eccentrically loaded footings. For strip footing –

$B' = B – 2e_x$ …(18.49)

For rectangular footing –

$B' = B – 2e_x$ …(18.50)

Lecture-9

$L' = L – 2e_y$ ….(18.51)

where B' is the effective width of footing in the X-direction and L' is the effective length of footing in the Y-direction.

The shape, depth, and inclination factors in Meyerhof's bearing capacity equation are as follows –

Shape factors –

$$S_c = 1 + 0.2K_p \frac{B}{L} \quad (18.52)$$

$$S_q = S_\gamma = 1 \qquad \text{for } \phi = 0 \quad (18.53)$$

$$S_q = S_\gamma = 1 + 0.1K_p \frac{B}{L} \qquad \text{for } \phi > 0 \quad (18.54)$$

where

$$K_p = \tan^2\left(45 + \frac{\phi}{2}\right) \quad (18.55)$$

Depth factors will be

$$d_c = 1 + 0.2\sqrt{K_p}\frac{D}{B} \quad (18.56)$$

$$d_q = d_\gamma = 1 \qquad \text{for } \phi = 0 \quad (18.57)$$

$$d_q = d_\gamma = 1 + 0.1\sqrt{K_p}\frac{D}{B} \qquad \text{for } \phi > 10° \quad (18.58)$$

Inclination factors will be

$$i_c = i_q = \left[1 - \frac{\alpha}{90}\right]^2 \quad (18.59)$$

$$i_\gamma = \left[1 - \frac{\alpha}{\phi}\right]^2 \qquad \text{for } \phi > 0 \quad (18.60)$$

$$i_\gamma = 0 \qquad \text{for } \phi = 0 \quad (18.61)$$

where α is the angle of inclination of load with vertical.

Meyerhof recommended the use of ϕ_r in place of ϕ for strip footings and rectangular footings under a plane strain condition.

where ϕ_r is the angle of shearing resistance under a plane strain condition to be used for computation of bearing capacity and ϕ the angle of shearing resistance obtained from triaxial compression tests.

Lecture-10

6. Hansen's Theory of Bearing Capacity:

Brinch Hansen (1970) extended Meyerhof's theory to determine the bearing capacity for footings with an inclined base and for footings with sloping ground surface. Hansen, Vesic, and Prandtl computed bearing capacity assuming the base of the footing as smooth.

Hansen introduced the base inclination and ground surface inclination factors in Meyerhof's bearing capacity equation and proposed the following equation for bearing capacity –

$$q_u = cN_cS_cd_ci_cb_cg_c + \gamma DN_qS_qd_qi_qb_qg_q + 0.5\gamma BN_\gamma S_\gamma d_\gamma i_\gamma d_\gamma g_\gamma \quad (18.63)$$

The factors $S_cd_ci_cb_cg_c$, $S_qd_qi_qb_qg_q$, and $S_\gamma d_\gamma i_\gamma d_\gamma g_\gamma$ are called the *A* factors. The factors N_c, N_q, and N_γ are called the *N* factors. So

$$N_q = K_p e^{\pi \tan\phi} \quad (18.64)$$

$$N_c = (N_q - 1)\cot\phi \quad (18.65)$$

$$N_\gamma = 1.5(N_q + 1)\tan\phi \quad (18.66)$$

where

$$K_p = \tan^2\left(45 + \frac{\phi}{2}\right)$$

Table 18.4 gives the bearing capacity factors proposed by Hansen. It may be noted that N_c and N_q factors given by Hansen are the same as those given by Meyerhof. The shape, depth, and inclination factors proposed by Hansen are functions of not only B/L and D/B but also of other factors such as N_c, N_q, ϕ, and many others. Computation of bearing capacity by Hansen's theory is more complicated than that by other theories.

Table 18.4 Hansen bearing capacity factors

ϕ	N_c	N_q	N_γ
0	5.14	1.0	0.0
6	6.81	1.23	0.11
10	8.3	2.5	0.39
16	11.63	4.34	1.43
20	14.8	6.4	2.95
26	22.25	11.85	7.94
30	30.1	18.4	15.07
32	35.5	23.2	20.79
34	42.4	29.4	28.77
36	50.6	37.7	40.05
38	61.4	48.9	56.17
40	75.3	64.2	79.54
42	93.7	85.4	113.95
44	118.4	115.3	165.48

It was observed that Terzaghi's theory gives conservative values of bearing capacity for cohesionless soils, while the values obtained for cohesive soils from the equations are more than those given by experimental results. Bearing capacity values as obtained by Hansen's theory for cohesive soils are in better agreement with experimental results on model footings.

7. Vesic's Theory of Bearing Capacity:

Vesic (1973) used the failure surface similar to that used in Terzaghi's theory, except that the slope of elastic wedge (zone-I) is assumed to be (45 + ϕ/2) with horizontal. He used the same form of equation as used by Hansen but modified some of the A factors, while adopting the other A factors from Hansen's theory, as shown below –

$$q_u = cN_cS_cd_ci_c + \gamma D_fN_qS_qd_qi_q + 0.5\gamma BN_\gamma S_\gamma d_\gamma i_\gamma$$

The bearing capacity factors N_q and N_c suggested by Vesic are the same as those given by Hansen. Vesic gave a slightly modified equation for N_y, that is –

$$N_q = K_pe^{\pi \tan\phi}$$

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

$$N_\gamma = 2(N_q + 1)\tan\phi$$

where

$$K_p = \tan^2\left(45 + \frac{\phi}{2}\right)$$

Table 18.5 Vesic's bearing capacity factors

ϕ	N_c	N_q	N_γ
0	5.14	1.0	0.0
5	6.5	1.6	0.5
10	8.3	2.5	1.2
15	11.0	3.9	2.6
20	14.8	6.4	5.4
25	20.7	10.7	10.8
30	30.1	18.4	22.4
32	35.5	23.2	30.2
34	42.4	29.4	41.1
36	50.6	37.7	56.3
38	61.4	48.9	78.0
40	75.3	64.2	109.4
42	93.7	85.4	155.6
44	118.4	115.3	224.6

Table 18.5 gives the bearing capacity factors proposed by Vesic.

The value of N_c and N_q computed by different methods will be more or less same, and the difference between different theories of bearing capacity lies only in the value of N_γ as –

$N_\gamma = (N_q - 1) \tan (1.4\phi)$ Meyerhof

$N_\gamma = 1.5(N_q - 1) \tan\phi$ Hansen

$N_\gamma = 2(N_q - 1) \tan\phi$ Vesic

Lecture-11

The settlement of a shallow foundation can be divided into two majorcategories:

(a) elastic, or immediate settlement and
(b) consolidation settlement.

Immediate, or elastic settlement

Immediate, or elastic settlement of a foundation takes place during orimmediately after the construction of the structure.

Consolidation settlement

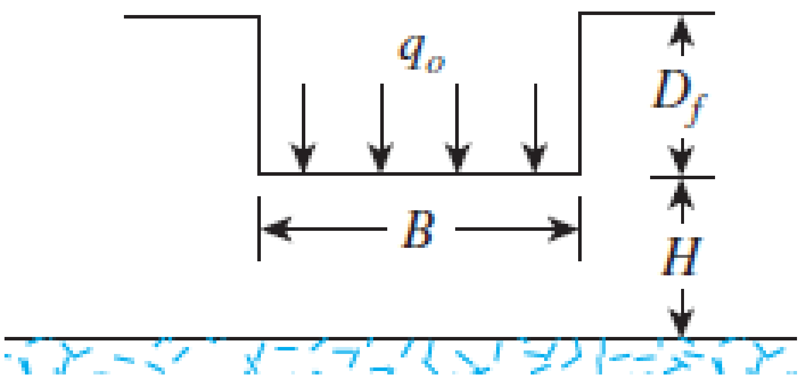
Consolidation settlement occurs over time. Pore water is extruded from thevoid spaces of saturated clayey soils submerged in water. The total settlement of a foundation is the sum of the elastic settlement and the consolidation settlement.

Consolidation settlement comprises two phases: *primary* and *secondary*. The fundamentals of primary consolidation settlement were explained in detail in Chapter 2.

Secondary consolidation settlement occurs after the completion of primaryconsolidation caused by *slippage* and *reorientation* of soil particles under a sustained load. Primary consolidation settlement is more significant thansecondary settlement in inorganic clays and silty soils. However, in organicsoils, secondary consolidation settlement is more significant.

Elastic Settlement of Shallow Foundation on Saturated Clay(μ_S = 0.5)

The average settlement of ***flexible*** foundations on saturated clay soils (Poisson's ratio, μ_S = 0.5).



$$S_e = A_1 A_2 \frac{q_o B}{E_s}$$

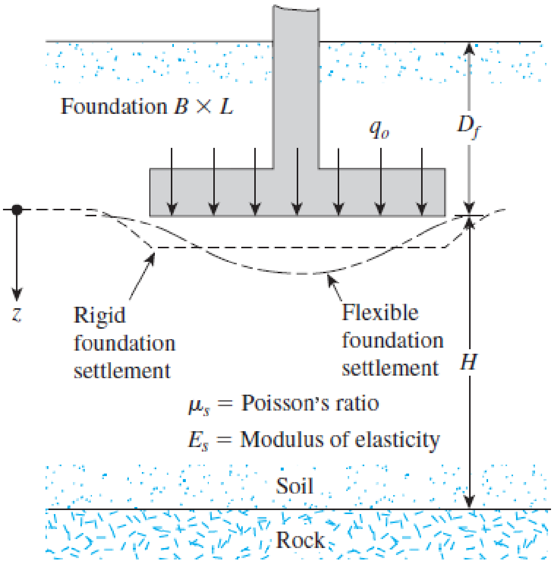
$$E_s = \beta c_u$$

$A_1 = f(H/B, L/B)$
$A_2 = f(D_f/B)$
L = length of the foundation
B = width of the foundation
D_f = depth of the foundation
H = depth of the bottom of the foundation to a rigid layer
q_o = net load per unit area of the foundation

Lecture-12

Elastic Settlement in Granular Soil

Settlement Based on the Theory of Elasticity



$$S_e = q_o(\alpha B')\frac{1-\mu_s^2}{E_s} I_s I_f$$

where

q_o = net applied pressure on the foundation
μ_s = Poisson's ratio of soil
E_s = average modulus of elasticity of the soil under the foundation, measured from $z = 0$ to about $z = 5B$
$B' = B/2$ for center of foundation
$= B$ for corner of foundation
I_s = shape factor (Steinbrenner, 1934)

$$= F_1 + \frac{1-2\mu_s}{1-\mu_s} F_2$$

$$I_f = \text{depth factor (Fox, 1948)} = f\left(\frac{D_f}{B}, \mu_s, \text{and } \frac{L}{B}\right)$$

Table 7.2 Variation of F_1 with m' and n'

n'	m' 1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	3.5	4.0
0.25	0.014	0.013	0.012	0.011	0.011	0.011	0.010	0.010	0.010	0.010
0.50	0.049	0.046	0.044	0.042	0.041	0.040	0.038	0.038	0.037	0.037
0.75	0.095	0.090	0.087	0.084	0.082	0.080	0.077	0.076	0.074	0.074
1.00	0.142	0.138	0.134	0.130	0.127	0.125	0.121	0.118	0.116	0.115
1.25	0.186	0.183	0.179	0.176	0.173	0.170	0.165	0.161	0.158	0.157
1.50	0.224	0.224	0.222	0.219	0.216	0.213	0.207	0.203	0.199	0.197
1.75	0.257	0.259	0.259	0.258	0.255	0.253	0.247	0.242	0.238	0.235
2.00	0.285	0.290	0.292	0.292	0.291	0.289	0.284	0.279	0.275	0.271
2.25	0.309	0.317	0.321	0.323	0.323	0.322	0.317	0.313	0.308	0.305
2.50	0.330	0.341	0.347	0.350	0.351	0.351	0.348	0.344	0.340	0.336
2.75	0.348	0.361	0.369	0.374	0.377	0.378	0.377	0.373	0.369	0.365
3.00	0.363	0.379	0.389	0.396	0.400	0.402	0.402	0.400	0.396	0.392
3.25	0.376	0.394	0.406	0.415	0.420	0.423	0.426	0.424	0.421	0.418
3.50	0.388	0.408	0.422	0.431	0.438	0.442	0.447	0.447	0.444	0.441
3.75	0.399	0.420	0.436	0.447	0.454	0.460	0.467	0.458	0.466	0.464
4.00	0.408	0.431	0.448	0.460	0.469	0.476	0.484	0.487	0.486	0.484
4.25	0.417	0.440	0.458	0.472	0.481	0.484	0.495	0.514	0.515	0.515
4.50	0.424	0.450	0.469	0.484	0.495	0.503	0.516	0.521	0.522	0.522
4.75	0.431	0.458	0.478	0.494	0.506	0.515	0.530	0.536	0.539	0.539
5.00	0.437	0.465	0.487	0.503	0.516	0.526	0.543	0.551	0.554	0.554

Table 7.3 Variation of F_2 with m' and n'

n'	m' 1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	3.5	4.0
0.25	0.049	0.050	0.051	0.051	0.051	0.052	0.052	0.052	0.052	0.052
0.50	0.074	0.077	0.080	0.081	0.083	0.084	0.086	0.086	0.0878	0.087
0.75	0.083	0.089	0.093	0.097	0.099	0.101	0.104	0.106	0.107	0.108
1.00	0.083	0.091	0.098	0.102	0.106	0.109	0.114	0.117	0.119	0.120
1.25	0.080	0.089	0.096	0.102	0.107	0.111	0.118	0.122	0.125	0.127
1.50	0.075	0.084	0.093	0.099	0.105	0.110	0.118	0.124	0.128	0.130
1.75	0.069	0.079	0.088	0.095	0.101	0.107	0.117	0.123	0.128	0.131
2.00	0.064	0.074	0.083	0.090	0.097	0.102	0.114	0.121	0.127	0.131
2.25	0.059	0.069	0.077	0.085	0.092	0.098	0.110	0.119	0.125	0.130
2.50	0.055	0.064	0.073	0.080	0.087	0.093	0.106	0.115	0.122	0.127
2.75	0.051	0.060	0.068	0.076	0.082	0.089	0.102	0.111	0.119	0.125
3.00	0.048	0.056	0.064	0.071	0.078	0.084	0.097	0.108	0.116	0.122
3.25	0.045	0.053	0.060	0.067	0.074	0.080	0.093	0.104	0.112	0.119
3.50	0.042	0.050	0.057	0.064	0.070	0.076	0.089	0.100	0.109	0.116
3.75	0.040	0.047	0.054	0.060	0.067	0.073	0.086	0.096	0.105	0.113
4.00	0.037	0.044	0.051	0.057	0.063	0.069	0.082	0.093	0.102	0.110
4.25	0.036	0.042	0.049	0.055	0.061	0.066	0.079	0.090	0.099	0.107
4.50	0.034	0.040	0.046	0.052	0.058	0.063	0.076	0.086	0.096	0.104
4.75	0.032	0.038	0.044	0.050	0.055	0.061	0.073	0.083	0.093	0.101
5.00	0.031	0.036	0.042	0.048	0.053	0.058	0.070	0.080	0.090	0.098

Table 7.4 Variation of I_f with D_f/B, B/L, and μ_s

		B/L		
μ_s	D_f/B	0.2	0.5	1.0
0.3	0.2	0.95	0.93	0.90
	0.4	0.90	0.86	0.81
	0.6	0.85	0.80	0.74
	1.0	0.78	0.71	0.65
0.4	0.2	0.97	0.96	0.93
	0.4	0.93	0.89	0.85
	0.6	0.89	0.84	0.78
	1.0	0.82	0.75	0.69
0.5	0.2	0.99	0.98	0.96
	0.4	0.95	0.93	0.89
	0.6	0.92	0.87	0.82
	1.0	0.85	0.79	0.72

To calculate settlement at the **centre** of the foundation, use:

$$\alpha = 4$$

$$m' = \frac{L}{B} \qquad n' = \frac{H}{\left(\frac{B}{2}\right)}$$

Lecture-13

To calculate settlement at a **corner** of the foundation, use:

$$\alpha = 1$$

$$m' = \frac{L}{B} \qquad n' = \frac{H}{B}$$

The elastic settlement of a **rigid** foundation can be estimated as:

$$S_{e(\text{rigid})} \approx 0.93 S_{e(\text{flexible, center})}$$

Due to the nonhomogeneous nature of soil deposits, the magnitude of **Es** may vary with depth. For that reason, Bowles (1987) recommended usinga weighted average of **Es**:

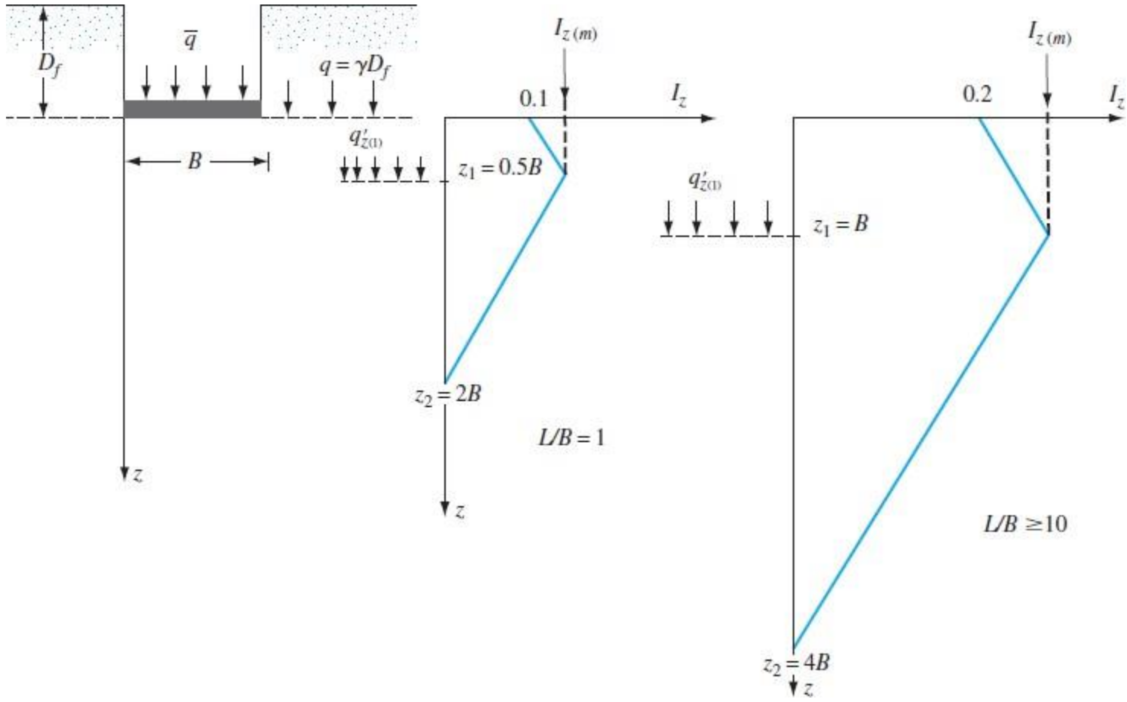
$$E_s = \frac{\Sigma E_{s(i)} \Delta z}{\bar{z}}$$

where

$E_{s(i)}$ = soil modulus of elasticity within a depth Δz

$\bar{z}$ = H or $5B$, whichever is smaller

Settlement of Sandy Soil: Use of Strain Influence Factor



$$S_e = C_1 C_2(\bar{q} - q)\sum_{0}^{z_2}\frac{I_z}{E_s}\Delta z$$

where

I_z = strain influence factor
C_1 = a correction factor for the depth of foundation embedment = $1 - 0.5\,[q/(\bar{q} - q)]$
C_2 = a correction factor to account for creep in soil
= $1 + 0.2 \log$ (time in years/0.1)
$\bar{q}$ = stress at the level of the foundation
$q = \gamma D_f$ = effective stress at the base of the foundation
E_s = modulus of elasticity of soil

The recommended variation of the strain influence factor Iz for square(L/B =1) or circular foundations and for foundations with L/B = 10.

I_z at $z = 0$

$$I_z = 0.1 + 0.0111\left(\frac{L}{B} - 1\right) \leq 0.2$$

Variation of z_1/B for $I_{z(m)}$

$$\frac{z_1}{B} = 0.5 + 0.0555\left(\frac{L}{B} - 1\right) \leq 1$$

Variation of z_2/B

$$\frac{z_2}{B} = 2 + 0.222\left(\frac{L}{B} - 1\right) \leq 4$$

Note that the maximum value of Iz [that is, Iz(m)] occurs at $z = z_1$ and then reduces to zero at $z = z_2$. The maximum value of Iz can be calculatedas:

$$I_{z(m)} = 0.5 + 0.1\sqrt{\frac{\bar{q} - q}{q'_{z(1)}}}$$

$$E_s = 2.5q_c \text{ (for square foundation)}$$

$$E_s = 3.5q_c \text{ (for } L/B \geq 10\text{)}$$

where qc = cone penetration resistance

$$E_{s(\text{rectangle})} = \left(1 + 0.4 \log\frac{L}{B}\right)E_{s(\text{square})}$$

Lecture-14

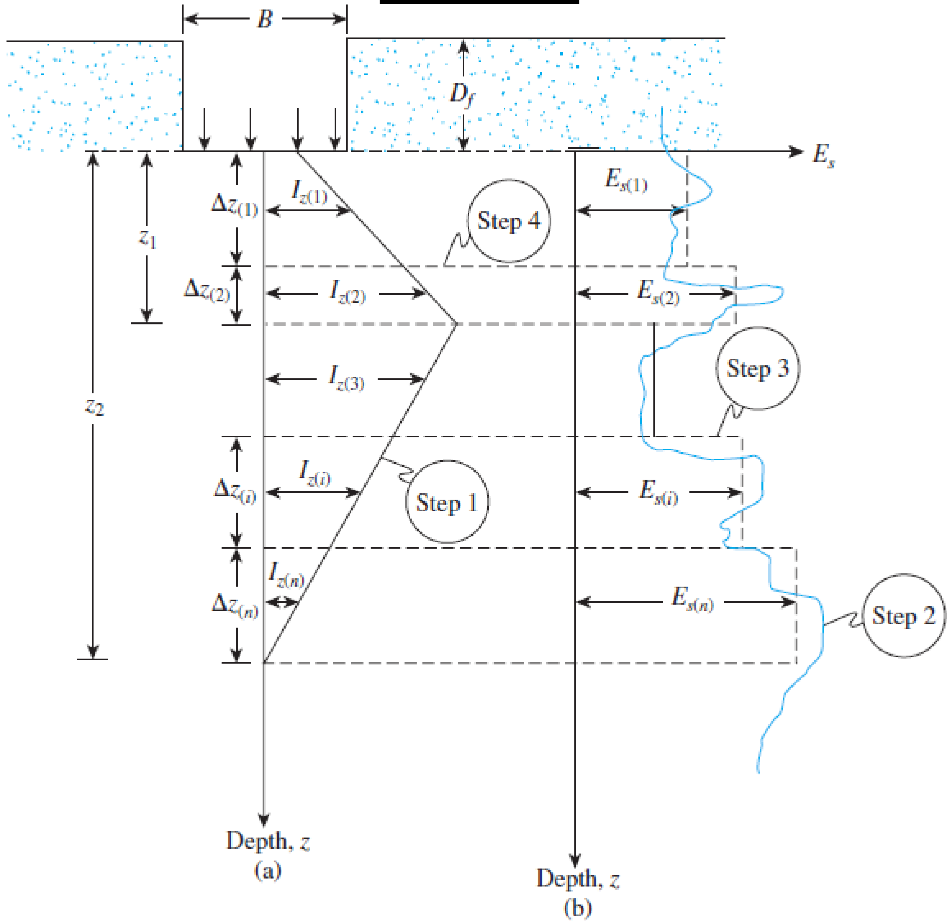


Table 7.5 Calculation of $\Sigma \frac{I_z}{E_s}\Delta z$

Layer no.	Δz	E_s	I_z at the middle of the layer	$\frac{I_z}{E_s}\Delta z$
1	$\Delta z_{(1)}$	$E_{s(1)}$	$I_{z(1)}$	$\frac{I_{z(1)}}{E_{s(1)}}\Delta z_1$
2	$\Delta z_{(2)}$	$E_{s(2)}$	$I_{z(2)}$	
⋮	⋮	⋮	⋮	
i	$\Delta z_{(i)}$	$E_{s(i)}$	$I_{z(i)}$	$\frac{I_{z(i)}}{E_{s(i)}}\Delta z_i$
⋮	⋮	⋮	⋮	⋮
n	$\Delta z_{(n)}$	$E_{s(n)}$	$I_{z(n)}$	$\frac{I_{z(n)}}{E_{s(n)}}\Delta z_n$
				$\Sigma \frac{I_z}{E_s}\Delta z$

$$C_1 = 1 - 0.5 \times \frac{q}{\bar{q} - q}$$

$$C_2 = 1 + 0.2 \log\left(\frac{\text{Time in years}}{0.1}\right)$$

$\bar{q} - q$ = **net stress** applied at the base of the foundation, so if you are given the **net load** $\rightarrow \bar{q} - q = \frac{\text{Net Load}}{\text{Foundation Area}}$

Substitute in the equation of Se in order to get the elastic settlement of sandy soil.

Settlement of Foundation on Sand Based on Standard PenetrationResistance (Meyerhof Method)

Meyerhof proposed a correlation for the net bearing pressure for foundations with the standard penetration resistance, N_{60}. The net pressurehas been defined as:

$$q_{\text{net}} = \bar{q} - \gamma D_f$$

According to Meyerhof's theory, for **25 mm (1 in.)** of estimated maximumsettlement:

In English units:

$$q_{\text{net}}(\text{kip/ft}^2) = \frac{N_{60}}{2.5} F_d S_e \quad (\text{for } B \leq 4 \text{ ft})$$

and

$$q_{\text{net}}(\text{kip/ft}^2) = \frac{N_{60}}{4}\left(\frac{B+1}{B}\right)^2 F_d S_e \quad (\text{for } B > 4 \text{ ft})$$

where

F_d = depth factor = $1 + 0.33(D_f/B)$
B = foundation width, in feet
S_e = settlement, in inches

Lecture-16

In SI units:

$$q_{\text{net}}(\text{kN/m}^2) = \frac{N_{60}}{0.05} F_d \left(\frac{S_e}{25}\right) \quad (\text{for } B \le 1.22 \text{ m})$$

and

$$q_{\text{net}}(\text{kN/m}^2) = \frac{N_{60}}{0.08} \left(\frac{B + 0.3}{B}\right)^2 F_d \left(\frac{S_e}{25}\right) \quad (\text{for } B > 1.22 \text{ m})$$

where

Se = settlement, in mm.

N_{60} = the standard penetration resistance between the bottom of thefoundation and 2B below the bottom.

Later, Meyerhof (1965) suggested that the net allowable bearing pressure should be increased by about 50%. Bowles (1977) proposed that the modified form of the bearing equations be expressed as:

$$q_{\text{net}}(\text{kip/ft}^2) = \frac{N_{60}}{2.5} F_d S_e \quad (\text{for } B \le 4 \text{ ft})$$

and

$$q_{\text{net}}(\text{kip/ft}^2) = \frac{N_{60}}{4} \left(\frac{B + 1}{B}\right)^2 F_d S_e \quad (\text{for } B > 4 \text{ ft})$$

where

F_d = depth factor = $1 + 0.33(D_f/B)$
B = foundation width, in feet
S_e = settlement, in inches

In SI units:

$$q_{\text{net}}(\text{kN/m}^2) = \frac{N_{60}}{0.05} F_d \left(\frac{S_e}{25}\right) \quad (\text{for } B \le 1.22 \text{ m})$$

and

$$q_{\text{net}}(\text{kN/m}^2) = \frac{N_{60}}{0.08} \left(\frac{B + 0.3}{B}\right)^2 F_d \left(\frac{S_e}{25}\right) \quad (\text{for } B > 1.22 \text{ m})$$

where B is in meters and Se is in mm.

Effect of the Rise of Water Table on Elastic Settlement

Terzaghi suggested that the submergence of soil mass reduces the soil stiffness by about half, which in turn doubles the settlement.

In most cases of foundation design, it is considered that, if the ground watertable is located **1.5B** to **2B** below the bottom of the foundation, it will not have any effect on the settlement.

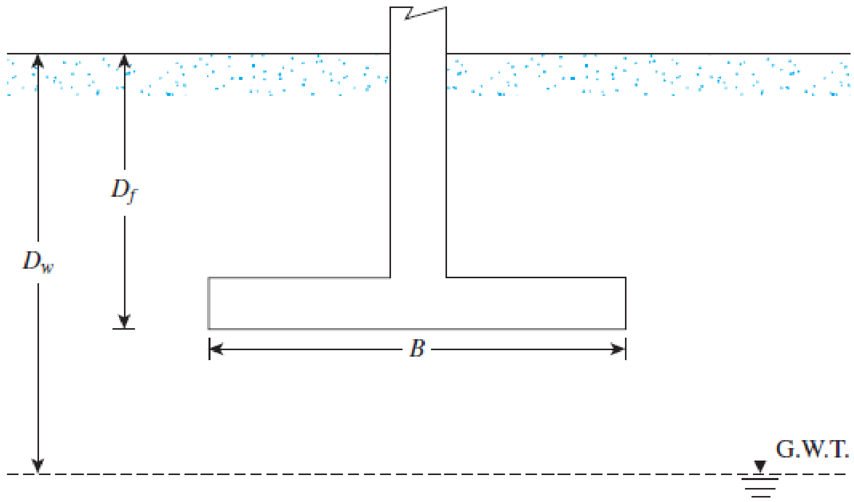


Figure 7.19 Effect of rise of ground water table on elastic settlement in granular soil

The total elastic settlement (S'_e) due to the rise of the ground water table can be given as:

$$S'_e = S_e C_w$$

where

S_e = elastic settlement before the rise of ground water table
C_w = water correction factor

The following are some empirical relationships for C_W:

- Peck, Hansen, and Thornburn (1974):

$$C_w = \frac{1}{0.5 + 0.5\left(\frac{D_w}{D_f + B}\right)} \geq 1$$

- Teng (1982):

$$C_w = \frac{1}{0.5 + 0.5\left(\frac{D_w - D_f}{B}\right)} \leq 2 \qquad \left(\text{for water table below the base of the foundation}\right)$$

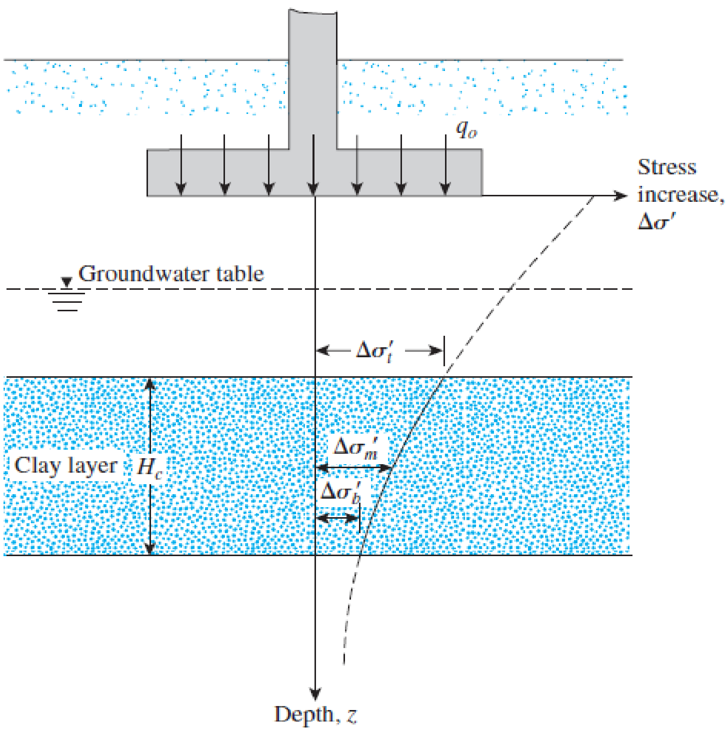
- Bowles (1977):

$$C_w = 2 - \left(\frac{D_w}{D_f + B}\right)$$

Consolidation Settlement

Primary Consolidation Settlement

Consolidation settlement occurs over time in saturated clayey soilssubjected to an increased load caused by construction of the foundation.



On the basis of the one-dimensional consolidation settlement equationsare:

$$S_{c(p)} = \int \varepsilon_z dz$$

where

ε_z = vertical strain

$= \dfrac{\Delta e}{1 + e_o}$

Δe = change of void ratio

$= f(\sigma'_o, \sigma'_c, \text{ and } \Delta\sigma')$

So,

$$S_{c(p)} = \frac{C_c H_c}{1 + e_o} \log \frac{\sigma'_o + \Delta\sigma'_{av}}{\sigma'_o}$$ (for normally consolidated clays) $\sigma'_o + \Delta\sigma'_{av} > \sigma'_c$ [Eq. (2.65)]

$$S_{c(p)} = \frac{C_s H_c}{1 + e_o} \log \frac{\sigma'_o + \Delta\sigma'_{av}}{\sigma'_o}$$ (for overconsolidated clays with $\sigma'_o + \Delta\sigma'_{av} < \sigma'_c$) [Eq. (2.67)]

$$S_{c(p)} = \frac{C_s H_c}{1 + e_o} \log \frac{\sigma'_c}{\sigma'_o} + \frac{C_c H_c}{1 + e_o} \log \frac{\sigma'_o + \Delta\sigma'_{av}}{\sigma'_c}$$ (for overconsolidated clays with $\sigma'_o < \sigma'_c < \sigma'_o + \Delta\sigma'_{av}$) [Eq. (2.69)]

where

σ'_o = average effective pressure on the clay layer before the construction of the foundation

$\Delta\sigma'_{av}$ = average increase in effective pressure on the clay layer caused by the construction of the foundation

σ'_c = preconsolidation pressure

e_o = initial void ratio of the clay layer

C_c = compression index

C_s = swelling index

H_c = thickness of the clay layer

$$\Delta\sigma'_{av} = \tfrac{1}{6}(\Delta\sigma'_t + 4\Delta\sigma'_m + \Delta\sigma'_b)$$

where $\Delta\sigma'_t$, $\Delta\sigma'_m$, and $\Delta\sigma'_b$ are, respectively, the effective pressure increases at the *top, middle,* and *bottom* of the clay layer that are caused by the construction of the foundation.

Lecture-17

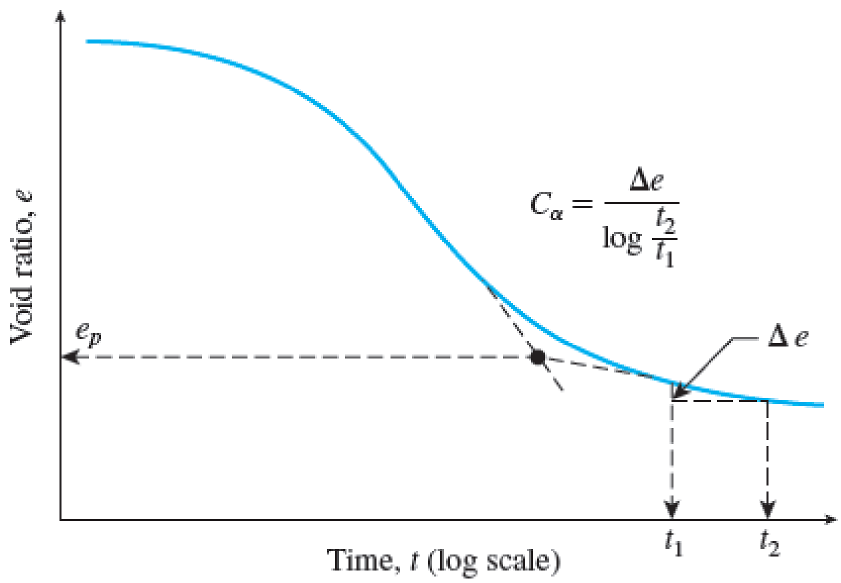
Secondary Consolidation Settlement

At the end of primary consolidation (i.e., after the complete dissipation ofexcess pore water pressure) some settlement is observed that is due to the plastic adjustment of soil fabrics. This stage of consolidation is called <u>secondary consolidation</u>.

A plot of deformation against the logarithm of time during secondary consolidation is practically linear as shown in the figure below.

The magnitude of the secondary consolidation can be calculated as:

$$S_{c(s)} = C'_\alpha H_c \log(t_2/t_1)$$



where

$C'_\alpha = C\alpha / (1+ e_p)$ (varies between 0.0005 to 0.001)e_p = void ratio at the end of primary consolidation
H_c = thickness of clay layer

From the figure, the secondary compression index can be defined as:

$$C_\alpha = \frac{\Delta e}{\log t_2 - \log t_1} = \frac{\Delta e}{\log (t_2/t_1)}$$

where

$C\alpha$ = secondary compression index

Δe = change of void ratiot_1,

t_2 = time

Secondary consolidation settlement is more important in the case of all organic and highly compressible inorganic soils. In over-consolidatedinorganic clays, the secondary compression index is very small and of lesspractical significance.

Field Load Test (Plate Load Test)

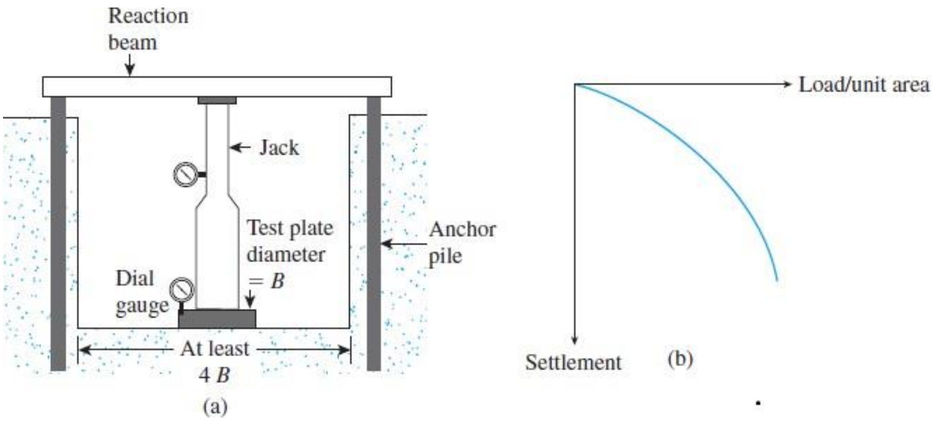
The ultimate load-bearing capacity of a foundation, as well as the allowable bearing capacity based on tolerable settlement considerations, can beeffectively determined from the field load test, generally referred to as theplate load test.

The plates that are used for tests in the field are usually made of steel and are 25 mm (1 in.) thick and occasionally, square plates that are 305 mm X305 mm (12 in. X 12 in.) are used.

To conduct a plate load test, a hole is excavated with a minimum diameterof 4B (B is the diameter of the test plate) to a depth of Df, the depth of theproposed foundation.

The plate is placed at the centre of the hole, and a load that is about 1/4 to1/5 of the estimated ultimate load is applied to the plate in steps by meansof a jack.

During each step of the application of the load, the settlement of the plate is observed on dial gauges. At least one hour is allowed to elapse betweeneach application. The test should be conducted until failure, or at least untilthe plate has gone through 25 mm (1 in.) of settlement.



For tests in clay:

$$q_{u(F)} = q_{u(P)}$$

where

$q_{u(Fd)}$ = ultimate bearing capacity of the proposed foundation

$q_{u(Pd)}$ = ultimate bearing capacity of the test plate

For tests in sandy soils:

$$q_{u(F)} = q_{u(P)} \frac{B_F}{B_P}$$

where

B_F = width of the foundation

B_P = width of the test plate

The allowable bearing capacity of a foundation, based on settlement considerations and for a given intensity of load, **qo**, is:

$$S_F = S_P \frac{B_F}{B_P} \quad \text{(for clayey soil)}$$

and

$$S_F = S_P \left(\frac{2B_F}{B_F + B_P} \right)^2 \quad \text{(for sandy soil)}$$

Module III

Lecture-1

DEEP FOUNDATION:

- If the depth of footing greater or equal to the Width of footing, it is known as the deep Foundation. A deep foundation is a type of foundation which is placed at a greater depth below the ground surface and transfers structure loads to the earth at depth.
- Deep Foundation is used Where the bearing capacity of the soil is very low. The load coming from the superstructure is further transmitted vertically to the soil.
- Deep foundations are founded too deeply below the finished ground surface for their base bearing capacity to be affected by surface conditions, this is usually at depths >3 m below finished ground level.

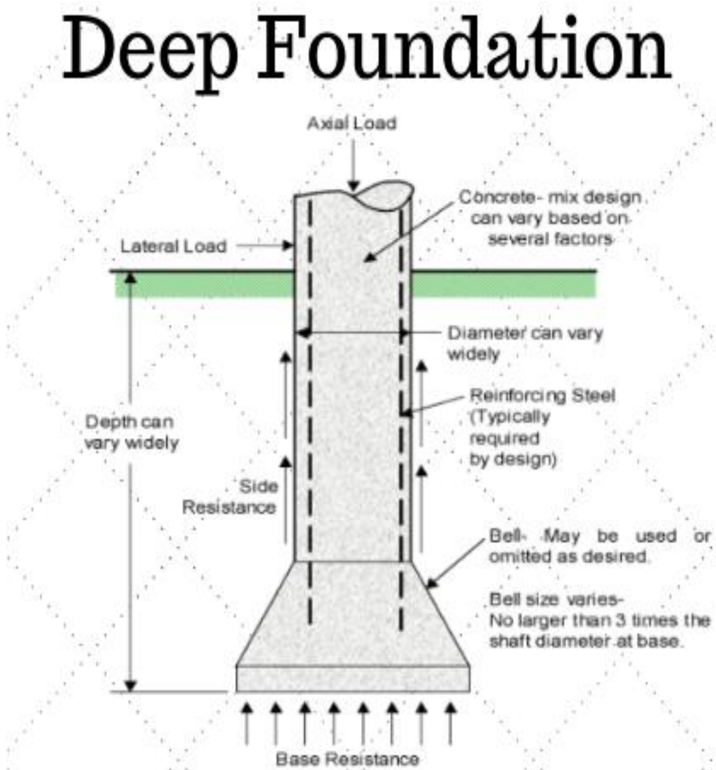


Figure: Deep Foundation

Difference between shallow and deep foundations:

Sl. No.	Detail	Shallow Foundation	Deep Foundation

1	Meaning	Shallow means having little depth.	Deep means extending far away from a point of reference, especially downwards. Extending far down from the top or surface
2	Definition	Foundation which is placed near the surface of the earth or transfers the loads at shallow depth is called the shallow foundation.	Foundation which is placed at a greater depth or transfers the loads to deep strata is called the deep foundation.
3	Cost	A shallow foundation is cheaper.	Deep foundations are generally more expensive than shallow foundations.
4	Types of Foundations	Shallow foundations include strip footing, isolated footings, combined footings, mat foundations, and grade beams.	Deep foundations include pile caps, piles, drilled piers, and caissons.
5	Depth of the Foundation	Shallow foundations can be made in depths of as little as 3ft (1m)	Deep foundations can be made at depths of 60 – 200ft (20 – 65m).
6	Shape of Footing	A shallow foundation is a rectangular or square shape.	A deep foundation is a bit circular and cylindrical shape.
7	Feasibility	Shallow foundations are easier to construct.	A deep foundation is more complex.
8	Mechanism of Load Transfer	Shallow foundations transfer loads mostly by end bearing.	Deep foundations rely both on end bearing and skin friction, with few exceptions like end-bearing pile.

9	Advantages	Construction materials are available, less labour is needed, construction procedure is simple at an affordable cost, etc.	Deep foundation can be provided at a greater depth, Provide lateral support and resist uplift, effective when foundation at shallow depth is not possible, can carry a huge load, etc.
10	Disadvantages	Possibility of a settlement, usually applicable for lightweight structure, weak against lateral loads, etc.	More expensive needs skilled labours, complex construction procedures, can be time-consuming and some types of deep foundations are not very flexible, etc.

Types of Deep Foundations:
A deep foundation is the following types.

1. Pile foundation
2. pier foundation
3. Caisson or well foundation

1. Pile foundation
The foundation in which load transfers to a low level by means of vertical members known as piles. A pile a slender structural member made of steel, concrete, or wood. A pile either drive into the piles may be of timber, concrete, or steel.

This type of foundation generally adopted whenever hard strata are available at great depth and bedding is uneven or the topsoil has a poor bearing capacity or there are large fluctuations is subsoil water level, or the topsoil is of expansive nature. Pile foundation transfers the load through friction as well as bearing.

Necessity of Pile Foundations:
Piles foundations are used in the following conditions:
1) When the strata at or just below the ground surface is highly compressible and very weak to support the load transmitted by the structure.
2) When the plan of the structure is irregular relative to its outline and load distribution. It would cause non-uniform settlement if a shallow foundation is constructed. A pile foundation is required to reduce differential settlement.
3) Pile foundations are required for the transmission of structural loads through deep water to a firm stratum.
4) Pile foundations are used to resist horizontal forces in addition to support the vertical loads in earth retaining structures and tall structures that are subjected to horizontal forces due to wind and earthquake.
5) Piles are required when the soil conditions are such that a washout, erosion and scour of soil may occur from underneath a shallow foundation.

Classification of pile foundation

Piles foundation are two types

- According to material
  - Concrete piles
  - Timber piles
  - Steel piles
  - Composite piles
- According to function
  - Bearing piles
  - Friction piles
  - Sheet piles

According to material Piles foundation:

Concrete piles

- ✓ Concrete piles make up of concrete. These are precast as well as cast in situ piles.
- ✓ Precast concrete piles manufacture in a factory and then driven into the ground at the place required.
- ✓ Precast concrete piles generally reinforced with steel wires. These are generally 30cm to 50cm in cross-section and up to 20m in length. A cast steel shoe is provided at the bottom of the pile.

Lecture-2

Timber piles

- ✓ Timber piles prepare from seasoned wood. These are generally circular in shape and the diameter varies from 20cm to 50cm.
- ✓ The length is generally 20 timber the dimmable. The bottom is sharpened. There is an iron shoe at the bottom and at the top, there is a cap.
- ✓ These piles are below the permanent water table, otherwise, they decay due to fungi and insects. These piles are uneconomical nowadays.

Steel piles

- ✓ Steel piles may be of 1 or hollow pipe section. These are very easy to drive because of their small sectional area.
- ✓ The piles driven with open ends. The soil within the pipe drives out by compressed air. Then, those piles filled with concrete.
- ✓ Steel piles mostly use as bearing piles. Steel pipe cannot use as a frictional pile as it has the less available surface area and smoothness of the surface.

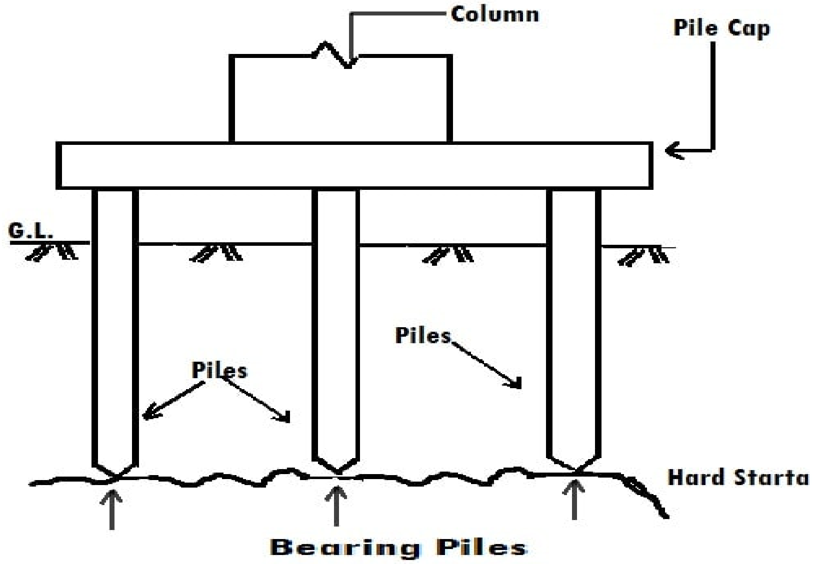
Composite piles

- ✓ Composite piles are the composites of wood and concrete. Wooden piles are more durable underwater.
- ✓ Pile consisting of a wooden section uses for the lower portion and a concrete section for its upper portion.
- ✓ The joint between the wood and concrete section design to withstand forces coming on it when adjacent piles are driving.

According to function Piles foundation

Bearing piles

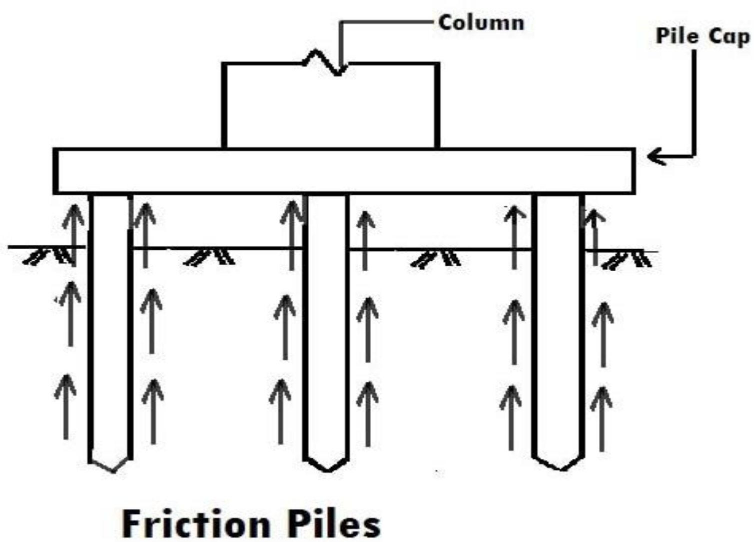
These piles are driven up to the hard stratum. They transfer the load of the structure to the hard stratum below. These piles virtually act as columns. These piles used to bear vertical loads on their ends.



Friction piles

When the soil very loose or soft to a considerable depth, when friction piles used. The friction develops along the surface of the pile from the surrounding soil. The load of the structure balance by the friction developed.

It is not necessary to drive such piles up to hard beds. The surface of the pile makes rough to increase the skin friction.



Lecture-3

Sheet piles

Sheet piles are generally thin piles. They make plates of concrete, timber, and steel. These piles are driven into the ground for either separating members or for stopping the seepage of water, they are not meant for carrying the vertical load.

Classification of sheet pile Foundation

i. Concrete sheet piles
ii. Steel sheet piles
iii. Timber sheet piles

Concrete sheet piles

These Sheet piles are always precast and reinforcement is used as pre-design. The piles square or rectangular and driven side by side so as to form a continuous wall. The width of each unit varies from 50 cm to 60 cm and thickness varies from 2 cm to 6 cm. The reinforcement is in the form of vertical bars and hoops.

Steel Sheet Pile Foundation

Steel sheet piles are most commonly used. Their piles are available in several sales, under different trade names. They generally made from steel sheets 20 cm to 30 cm wide and also 4 m to 5 m long with suitable interlocking arrangements. Therefore they form fairly watertight joints.

Timber Sheet pile Foundation

Timber sheet piles used only for temporary works like cofferdams. They generally make of wooden boars 8 cm to 15 cm thick, 20 cm also wide and 2 m and 4 m long. They may be jointed by either butt or V-joints. The bottom chamfered so as to form a cutting edge. If necessary, the top and bottom are provided with a suitable iron fitting.

2. Pier foundation

A pier is a vertical column of a relatively larger cross-section than a pile. A pier is installed in an a dry area by excavating a cylindrical hole of a large diameter to the desired depth and then back filling it with concrete.

A distinction between a cast-in-situ pile and a pier is rather arbitrary. A cast-in-situ pile greater than 0.6 m diameter has generally termed a pier.

The difference between the pile foundation and pier foundation lies in the method of construction. Though pile foundations transfer the load through friction and bearing, pier foundations transfer the load only through the bearing.

Generally, the pier foundation is shallower in-depth than the pile foundation. Pier foundation is preferred in a location where the top strata consist of decomposed rock overlying strata of sound rocks. In such a condition, it becomes difficult to drive the bearing piles through decomposed rock.

In the case of stiff clays, which offer large resistance to the driving of a bearing pile, a pier foundation can be conveniently constructed.

Types of pier foundation

Pier foundation may be of the following types

- Masonry or concrete pier
- Drilled caisson

When a good bearing stratum exists up to 5m below ground level, brick masonry or concrete foundation piers in excavated pits may be used. The size and spacing of the pier are dependent upon the depth of the hard bed, nature of overlying soil and super-imposed load.

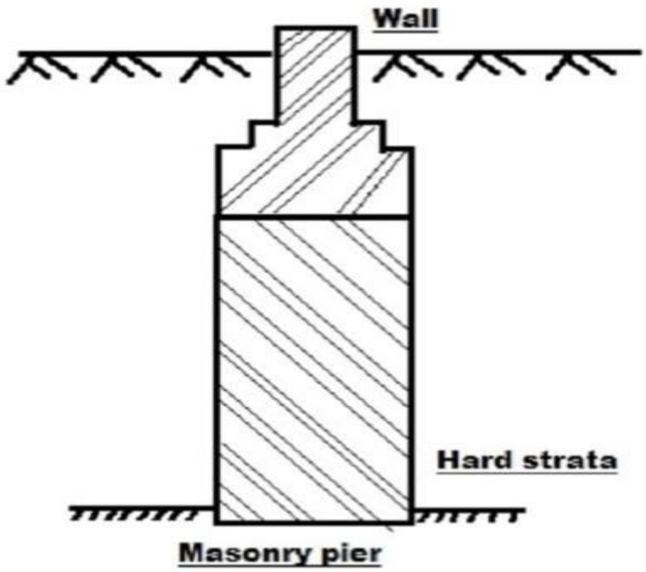


Figure: Masonry pier foundation

Lecture-4

Concrete pier or a masonry pier

- ✓ The concrete pier is made up of concrete. These are precast, as well as cast in situ pier Precast concrete pier, are manufactured in a factor and then driven into the ground at the place required. The precast concrete pier generally reinforces with steel wires.
- ✓ There is generally 30 cm to 50 cm in cross-section and up to 20 m in length. A cast steel shoe is provided at the bottom of the pier.

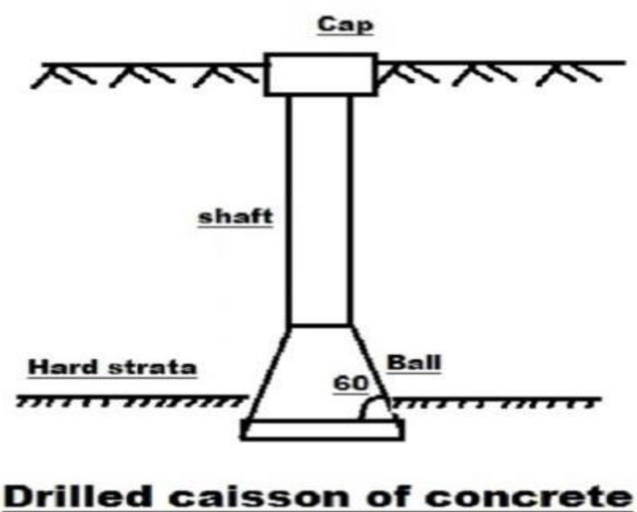


Figure: Drilled caisson pier foundation

Drilling caisson

- ✓ The terms drilled caisson, foundation pier, or sub pier and interchangeably used by engineers to denote a cylindrical foundation. A drilled caisson large compressed member subjected to an axial load at the top and reaction at the bottom.
- ✓ Drilled cassion generally drill with mechanical means. Drilled caisson may be of three types
- ❖ Concrete caisson with the enlarged bottom.
- ❖ The caisson of steel pipe with concrete filled in the pipe.
- ❖ Caisson with concrete and steel core in steel pipe.

<u>Pier foundation and brick arch foundation</u>

- ✓ The construction of arches is old technology. Such type of foundation is of much use where the bearing capacity of the soil is good and there exist some loose-filled up soil pockets in between.
- ✓ The arches can build by avoiding the pressure on such loose pockets and transfer the load to the isolated footings built to support the arches. For the construction of such a foundation, the use of variable material like brick or concrete blocks can be made.
- ✓ In order to resist the lateral forces, buttresses at the corner or at the end build. With the use of such a foundation, there is a considerable saving in the masonry and concrete between the two footings.
- L-section
- Cross-section
- ✓ Piers foundation for a wall carrying heavy loads. Piers dug at regular intervals and filled with cement concrete.
- ✓ The piers may rest on good bearing strata. These piers connected by concrete or masonry arch, over which the wall may be constructed.
- ✓ If required, a concrete beam may provide over the arch if the arch construct of masonry. When the arches construct with a gap above the ground level.
- ✓ This gap would permit the free vehicle movement of soil during swelling and shrinkage operations.

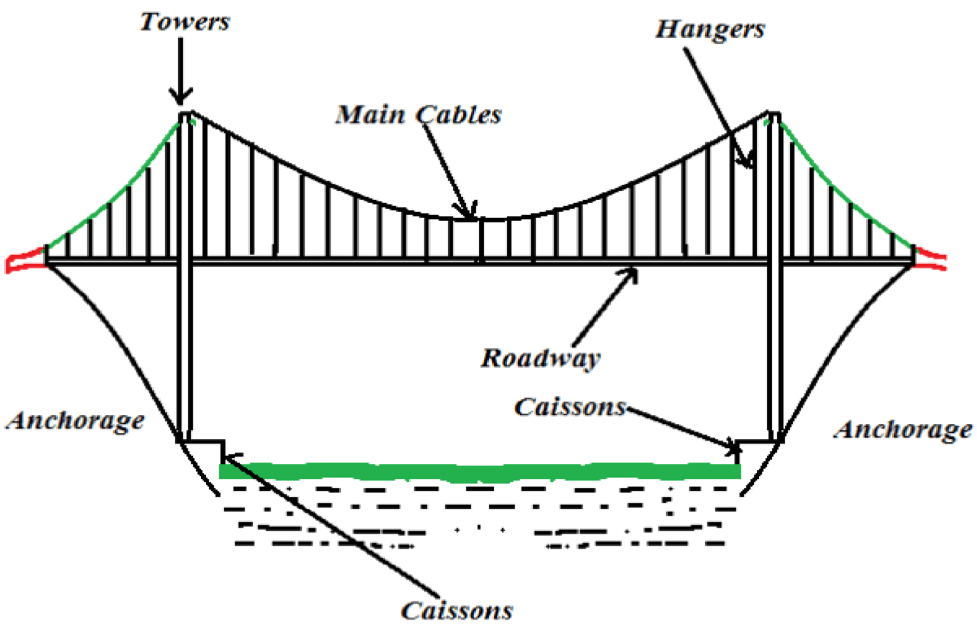
<u>Advantages of pier foundation</u>

- It has a wide range of variety when it comes to design. There are varied materials we can here to increase the aesthetic view and also it remains in our budget.
- Bearing capacity can increase by under-reaming the bottom.
- Pier foundation saves money and time as it does not need extensive excavation a lot of concrete.

3. <u>Caisson foundation</u>

The Term caisson is derived from the French word ‘Caisse’ meaning a box. In civil engineering, a caisson is defined as a type of foundation in the shape of a hollow prismatic box, which is constructed above the ground level and then sunk to the desired depth.

This is part of the Well foundation. Its watertight chamber use for laying foundations underwater, as in rivers, harbors lakes, etc.



Types of caisson foundation

The caisson can be a divide in the following three types-

- ❖ Open caissons
- ❖ Pneumatic caisson
- ❖ Floating caisson

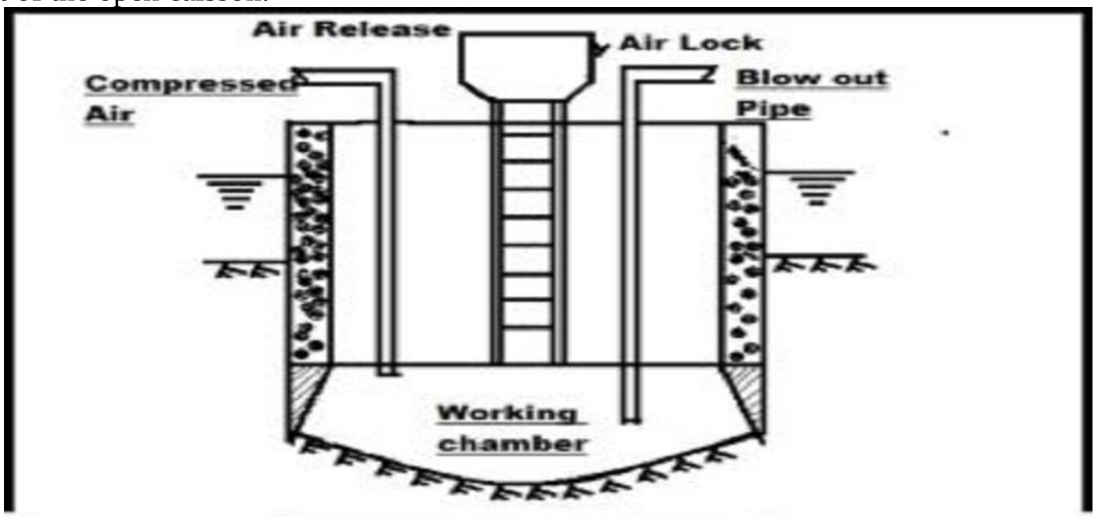
Open caisson foundation

- ✓ The open caissons are open both at the top and at the bottom and these caissons are hollow chambers. The bottom of the caisson has a cutting edge.
- ✓ The caissons sunk into place by removing the soil from the inside of the shaft (chamber) until the bearing stratum reach.
- ✓ An open caisson sinking generally does by penetrating it in the dry or from dewatered, construction area, or from an artificial island.
- ✓ An artificial island of the sand construct for the purpose of raising the ground surface above the water level.
- ✓ Thus a dry area obtains for sinking the caisson. The size of the sand island should be sufficient to provide a working area around the caisson.
- ✓ In case it not possible to sink the caisson in dry. It is built-in spillways or barges and transported to its final position by floating.
- ✓ False bottoms use for this purpose. Guide piles generally need to sink the initial few lifts of the caisson. Sinking is done through open water and then penetrating it into the soil.
- ✓ An open caisson generally makes of timber, metal, reinforced concrete, or for building and bridges foundation. Open caissons call wells.
- ✓ Foundations form the most common type of deep foundations for bridges in India.

Pneumatic caisson Foundation

- ✓ Pneumatic caisson requires when the soil enclosed in an open caisson can not excavate satisfactorily through. Its shaft during sinking operation.
- ✓ A pneumatic caisson also employs when there is a great influx of water or where difficult obstructions anticipate during the sinking. Pneumatic caissons closed at the top but open at the bottom in construction.

- ✓ Pneumatic caisson has a working chamber at its bottom in. Which compressed air maintain at the required pressure to prevent the entry of water and mud into the chamber.
- ✓ The method of construction of pneumatic caisson is similar to that for open caissons except that the working chamber keeps airtight. The ultimate load carrying capacity and cutting edge are also similar to that of the open caisson.



Pneumatic Caisson

Floating caisson

- ✓ Floating caisson is large hollow boxes open at the top and closed at the bottom. These float to the place where these caissons to finally installed. These sunk at that place by filling them with sand, ballast, dry concrete, and gravel.
- ✓ Unlike open and pneumatic caissons, a floating caisson does no penetrated the soil, it rests on a leveled bearing surface. The load-carrying capacity realizes on the base resistance because there is no side friction.
- ✓ After the caisson sunk to its final position, it completely fills with sand or gravel. A concrete cap constructs at the top of the caisson to bear structural loads. A rip-rap is placing around the base to prevent scour underneath.
- ✓ Generally, these caissons construct of R.C.C or steel. The shape of caisson in the plan may be circular, square, rectangular, or elliptical. It usually contains a number of cells formed by diaphragm walls. A caisson design as a ship. When it has to be floated in rough waters and provided with suitable internal strutting.

Uses of caisson foundation

- ✓ Caisson is used in building bridge piers as it stays in water almost all the time.
- ✓ It is also used for the pump house which is subjected to huge vertical as well as horizontal forces.
- ✓ It is also used for large multi-stories building. But sometimes is use.
- ✓ Pneumatic caisson is used in railway bridges, garbage pits, water supply, sewage facilities etc.
- ✓ Caisson provides access to the deep shaft or a tunnel.
- ✓ Caissons have also been used in the installation of hydraulic electors where a single-stage ram is installed below the ground level.

Advantages of caissons foundation

The advantages of caisson

- ✓ Excavation bottom and pouring of concrete are done in dry conditions.

- ✓ As there is access to the bottom of the caisson, obstruction such as boulders or logs can be easily removed.
- ✓ The verticality of pneumatic caisson is easier to check and control than open caissons.
- ✓ Since concrete placed is dry, good and reliable quality work can be obtained.
- ✓ Soil bearing capacity can easily be determined by conducting in-situ tests in the working chamber.

Disadvantages of caissons foundation

The disadvantages of caissons

- ✓ The cost of construction is high. Pneumatic caissons only use when open caissons are not feasible.
- ✓ The depth of penetration below the water level limit to 35 m.
- ✓ There a lot of inconveniences caused to the workmen who work under high pressure. The workers may develop caisson disease. Hence, proper health controls are necessary for workers.
- ✓ In pneumatic caissons, a large number of manual work needs that increases the cost.
- ✓ The high degree of skill required in sinking.

PILE FOUNDATIONS:

Pile foundation is one type of deep foundation which is used when the bearing capacity of soil is low, in under water construction, load of structure is high, location where shallow foundation is not possible, etc.

When and Where Pile Foundation is Used?

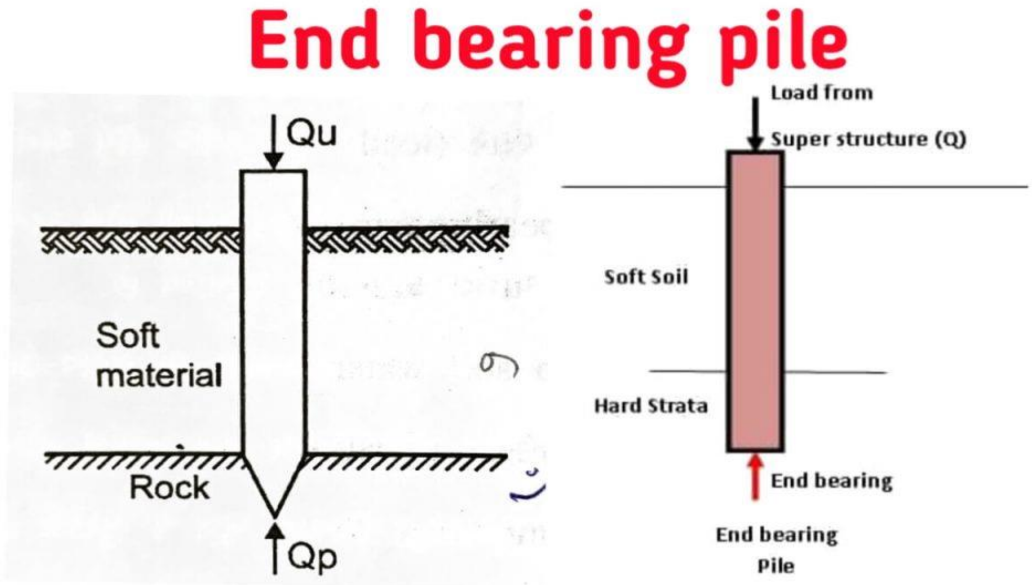
- ✓ Load coming from the structure (load) is too high and its expansion over the soil is not uniform.
- ✓ The bearing capacity of the soil is low and the required holding capacity is obtained at a greater depth.
- ✓ Fluctuates Sub-soil water level.
- ✓ Construction of a raft or grillage foundation may be too expensive or not feasible.
- ✓ In the area, where future pipelines for water, sewerage, gas etc. are to be laid.
- ✓ If the structure is located on the beach or on the bank of a river and there is a possibility of erosion of the foundation due to water logging.
- ✓ Pile foundation is useful for the foundation of the structure in the sea.
- ✓ Anchor piles are used to provide grip against upward pressure and lateral pressure on the structure.
- ✓ In marine constructions like dock, pier, fender piles are used to protect the construction from the impact of ships.

Types of pile foundation based on function or use:

1. End bearing pile
2. Friction pile
3. Compaction pile
4. Tension pile
5. Anchor pile
6. Fender pile
7. Better pile
8. Sheet pile

Lecture-5

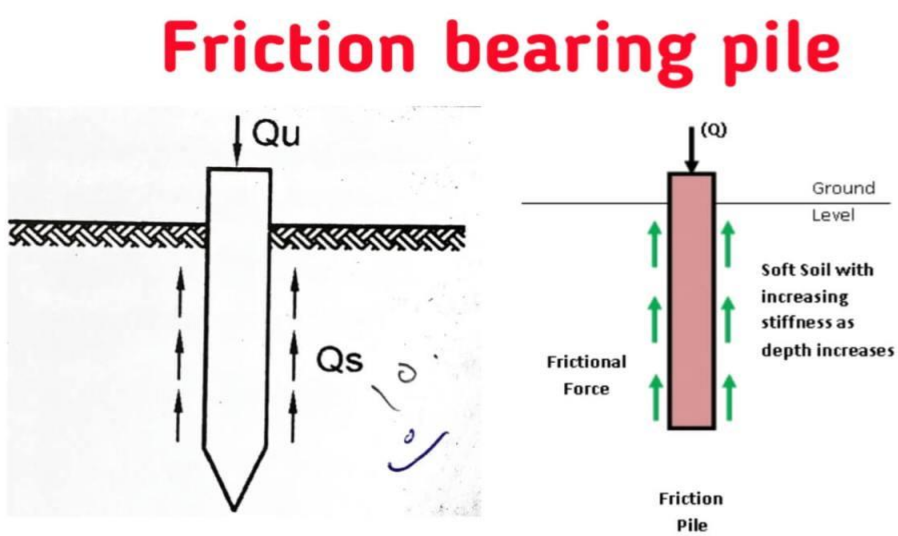
1. End bearing pile:



This type of pile is used to transfer load from water or soft soil to the hard rock below. The load is transferred to the lower end of the pile.

Qu = Qp is used for this pile.

2. Friction pile:



The surface of such pile is kept rough so that the load is carried by the friction (skin friction) generated between the surrounding soil and the surface of the pile.

For a friction pile, Q = Qs

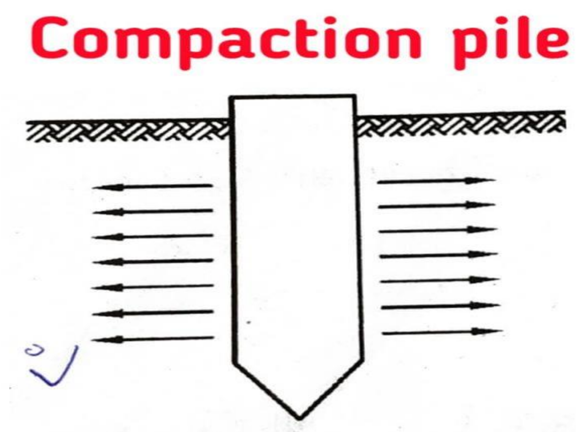
Qu = Ultimate load on pile Qs = Skin friction

The carrying capacity of the friction pile can be increased as follows:

- By increasing the diameter of the pile
- By insert pile to a greater depth.
- By make roughing surface of the pile.
- By keeping group of pile.

3. Compaction pile:

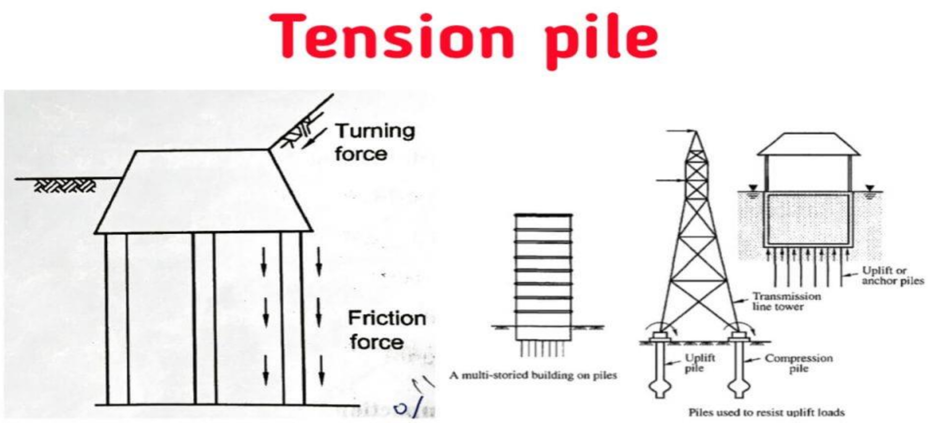
When a pile is insert into granular soil to increase the bearing capacity of the soil, it is called a compaction pile.



Pile itself does not carry any load, so it is made of weak material. Sand piles are commonly used as compaction piles. In which a pipe is insert into the ground, so that the soil around the pipe undergoes lateral movement and compaction of the soil occur, then the pipe is slowly pulled out and filled with sand. This is how the sand pile is prepared.

4. Tension pile:

Lecture-6



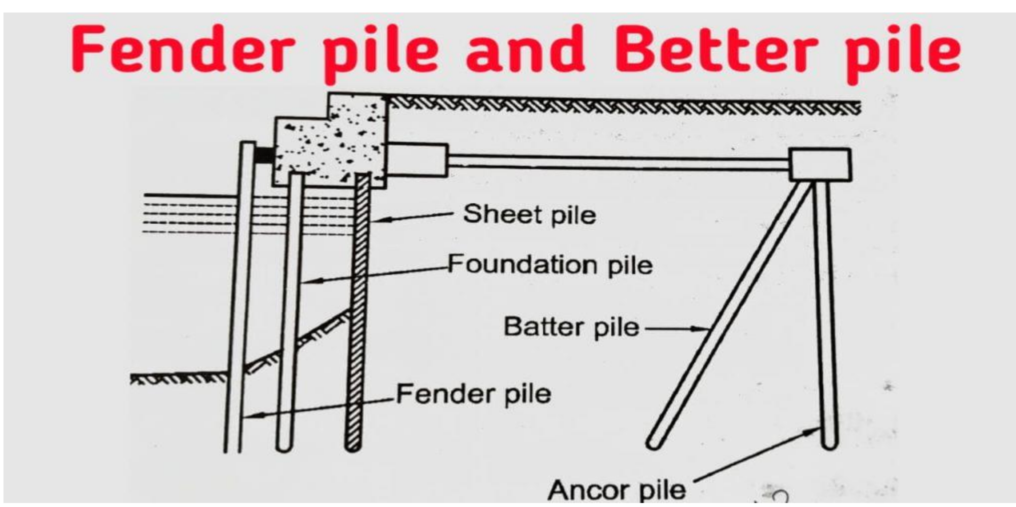
When a structure feels uplifted due to ground water pressure or due to overturning moment, tension pile is used to keep the structure attached to the ground. In such a pile tensile strength is generated.

The resistance to tensile force is due to the force of friction with the soil on the surface of the pile.

5. Anchor pile:

When a pile is used to provide anchorage against the shear pull of a sheet pile or other type of pulling force, it is called an anchor pile.

6. Fender pile:



A pile that is used to protect a structure in the sea from damage caused by abrasion or collision of a ship is called a fender pile. Such piles are usually made of wood.

7. **Better pile:**

When the pile is insert into the ground to prevent horizontal force or transverse force, is called Better pile.

8. **Sheet pile:**

The main function of sheet pile is to separate two members under the ground. Such piles are not designed to withstand any vertical loads.

The sheet pile is used for the following purposes.

- To separate the base from the lateral soil.
- To stop the movement of ground water, which is very necessary for cofferdam.
- To prevent the vibrations of the machine from reaching the side structure.
- To build maintenance wall for marine works like docks, wharfs.
- To prevent erosion of river banks.
- To support the sides of the base trench.
- To build cut off wall below the dam.
- To increase the holding capacity of soil by keeping it confined.
- To prevent erosion of the foundation of the river by river or sea water.

Types of pile foundation based on materials:

(1) Concrete piles (2) Timber piles (3) Steel piles (4) Composite piles

1. **Concrete piles**

✓ Concrete piles make up of concrete. These are precast as well as cast in situ piles.
✓ Precast concrete piles manufacture in a factory and then driven into the ground at the place required.
✓ Precast concrete piles generally reinforced with steel wires. These are generally 30cm to 50cm in cross-section and up to 20m in length. A cast steel shoe is provided at the bottom of the pile.

2. **Timber piles**

✓ Timber piles prepare from seasoned wood. These are generally circular in shape and the diameter varies from 20cm to 50cm.
✓ The length is generally 20 timber the dimmable. The bottom is sharpened. There is an iron shoe at the bottom and at the top, there is a cap.

- ✓ These piles are below the permanent water table, otherwise, they decay due to fungi and insects. These piles are uneconomical nowadays.

3. **Steel piles**

- ✓ Steel piles may be of 1 or hollow pipe section. These are very easy to drive because of their small sectional area.
- ✓ The piles driven with open ends. The soil within the pipe drives out by compressed air. Then, those piles filled with concrete.
- ✓ Steel piles mostly use as bearing piles. Steel pipe cannot use as a frictional pile as it has the less available surface area and smoothness of the surface.

4. **Composite piles**

- ✓ Composite piles are the composites of wood and concrete. Wooden piles are more durable underwater.
- ✓ Pile consisting of a wooden section uses for the lower portion and a concrete section for its upper portion.
- ✓ The joint between the wood and concrete section design to withstand forces coming on it when adjacent piles are driving.

PILE DRIVING:

Piles are driven into the ground by means of hammers or by using a vibratory driver. Such piles are called driven piles. In some special cases, piles are installed by jetting or partial augering.

The following methods are commonly used.

(1) Hammer Driving: Fig. 25.1 shows a pile driving rig. It consists of a hoist mechanism, a guiding frame and a hammer device. The hammers used for pile driving are of the following types:

(i) Drop hammer: A drop hammer is raised by a winch and allowed to drop on the top of the pile under gravity from a certain height. During the driving operation, a cap is fixed to the top of the pile and a cushion is generally provided between the pile and the cap. Another cushion, known as hammer cushion, is placed on the pile cap on which the hammer causes the impact. The drop hammer is the oldest type of hammer used for pile driving. It is rarely used these days because of very slow rate of hammer blows.

(ii) Single-acting hammer: In a single-acting hammer, the ram is raised by air (or steam) pressure to the required height. It is then allowed to drop under gravity on the pile cap provided with a hammer cushion.

(iii) Double-acting hammer: In a double-acting hammer, air (or steam) pressure is used to raise the hammer. When the hammer has been raised to the required height, air (or steam) pressure is applied to the other side of the piston and the hammer is pushed downward under pressure. This increases the impact energy of the hammer.

(iv) Diesel hammer: A diesel hammer consists of a ram and a fuel injection system. It is also provided with an anvil block at its lower end. The ram is first raised manually and the fuel is injected near the anvil. As soon as the hammer is released, it drops on the anvil and compresses the air-fuel mixture and ignition takes place. The pressure so developed pushes the pile downward and raises the ram. The fuel is again injected and the process is repeated.

The ram lifts automatically. It has to be manually raised only once at the beginning.

Diesel hammers are not suitable for driving piles in soft soils. In such soils, the downward movement of the pile is excessive and the upward movement of the ram after impact is small. The height achieved after the upward movement of the hammer may not be sufficient to ignite the air-fuel mixture.

Diesel hammers are self-contained and self-activated.

(2) Vibratory Pile Driver: A vibratory pile driver consists of two weights, called exciters, which rotate in opposite directions. The horizontal components of the centrifugal force generated by exciters cancel each other but the vertical components add. Thus a sinusoidal dynamic vertical force is applied to the pile, which ropes the pile downward. The frequency of vibration is kept equal to the natural frequency of pile-soil system for better results.

A vibratory pile driver is useful only for sandy and gravelly soils. The speed of penetration is good. The method is used where vibrations and noise of conventional driving methods cannot be permitted.

(3) Jetting Techniques: When the pile is to penetrate a thin hard layer of sand or gravel overlying a softer soil layer, the pile can be driven through the hard layer by jetting techniques. Water under pressure is discharged at the pile bottom point by means of a pipe to wash and loosen the hard layer.

(4) Partial Augering Method: Batter piles (inclined piles) are usually advanced by partial augering. In this method, a power auger is used to drill the hole for a part of the depth. The pile is then inserted in the hole and driven with hammers to the required depth.

Lecture-7

LOAD CARRYING CAPACITY OF PILES :- STATIC AND DYNAMIC FORMULAE

Static Formulae for Estimating the Load Capacity of Piles:

Static formulae give the static resistance offered by the soil/rock at the base and along the surface of the pile to the loads applied on the pile.

Basic Principle:

The unit point-bearing resistance of a pile may be given by –

$f_p = cNc + \sigma' Nq + 0.5\gamma BN\gamma$ …(20.2)

where c is the unit cohesion of the soil at the pile tip; σ', the effective overburden pressure at the base of the pile; γ, the density of the soil at the pile tip; B, the width or diameter of the pile; and Nc, Nq, and Nγ, the bearing capacity factors.

The magnitude of the third term, $0.5\gamma BN\gamma$, in Eq. (20.2) is very small for deep foundations compared with the second term, $\sigma' Nq$, and, hence, is neglected. Therefore,

$f_p = cNc + \sigma' Nq$ …(20.3)

The total bearing resistance of pile is given by –

$Q_p = f_p A_p$ …(20.4)

The total skin friction resistance is given by –

$Q_s = f_s A_s$ …(20.5)

The ultimate load capacity of the pile is given by –

$Q_u = f_p A_p + f_s A_s$ …(20.6)

Piles in Sands:

For pure sands –

$Qu = \sigma' Nq + fsAs$ …(20.7)

Thus, the point-bearing resistance of piles in granular soils increases proportionately with the increase in the length of the pile. However, when the embedded pile length is more than a critical depth, the point-bearing resistance does not increase further with the increase in the pile length. This is due to the arching action in granular soils. For driven piles in granular soils, the critical depth is found to be equal to 15 D for loose- to medium-dense sands and 20 D for dense sands. The maximum value of unit point-bearing resistance is limited to 11000 kN/m2 for silica sand and 5000 kN/m2 for calcareous sand. The unit skin friction resistance is given by –

fs = σh tan δ – K$\bar{\sigma}$ tan δ …(20.8)

where σh is the average horizontal pressure over the pile length, acting normal to the pile surface, K is the lateral earth pressure coefficient, and 8 is the angle of friction between the pile and the soil. The total skin friction resistance is given by –

Qs = fsAs …(20.9)

As per IS – 2911 (Part I), δ = ϕ, K = 1 – 3 for loose- to medium-dense sand.

The value of σh and, hence, the average pressure, cr increases with an increase in depth from the ground level. However, for depth greater than 15-20 times the pile diameter, the value of σh is restricted to the maximum value corresponding to the depth equal to 15-20 times the pile diameter. The maximum value of the unit skin friction resistance is about 100 kN/m2 for silica and 20 kN/m2 for calcareous sand.

Piles in Clay:

Piles in clays or cohesive soils carry most of the load by skin friction resistance of the pile shaft. The load-carrying capacity using static formula is computed on the basis of total stress approach taking ϕu = 0, assuming undrained conditions. The load capacity is a function of the reduction factor, α. The value of α depends on the undrained shear strength of the soil. The value of α is close to 1 for soft clays with low undrained strength. It decreases with the increase in the stiffness of the clay, and for very stiff clays, it may be as low as 0.3.

However, the skin friction resistance will be more for stiff clays due to higher shear strength. In the case of piles driven in clay, the soil loses some of its shear strength due to the sensitivity by remolding. As time elapses, most of the lost strength is regained by thixotropy. A time gap of minimum 30 days should be maintained from the driving of piles in clay and loading the pile. For the same reason, pile load tests in the case of driven piles in clays should be performed at least 30 days after piles are driven. Static formulae should be taken only as a guide for estimation of load capacity of the piles. It should always be supplemented by pile load tests.

Static Formula as per IS Code for Piles in Sand:

As per IS – 2911 (Part I)-1979, the ultimate load capacity of a pile in granular soil is given by –

$$Q_u = A_p(0.5\gamma' d N_\gamma + \sigma'_p N_q) + \sum_{i=1}^{n} K\sigma_{si} \tan\delta A_{si} \qquad (20.10)$$

where Ap is the cross-sectional area of pile at the bottom; d the diameter of the stem; γ’ the effective unit weight of soil at pile tip at bottom; σ’p the effective stress at the pile toe; K the coefficient of earth pressure; σ’si the average effective stress for the layer; Asi the surface area of pile for the ith soil layer; Nγ the bearing capacity factor for general shear as per IS – 6403-1981; and Nq the bearing capacity factor.

The bearing capacity factor, Nq, depends on the method of installation of the pile, that is, driven or bored pile, and on the angle of internal friction of the soil, ϕ

$$\phi = \frac{(\phi_i + 40)}{2} \qquad (20.11)$$

where ϕi is the in-situ angle of shearing resistance.

Figure 20.12 shows the value of Nq as a function of ϕ, applicable for driven piles as recommended by IS:2911 (Part 1/ Sec I and Sec III)-1979.

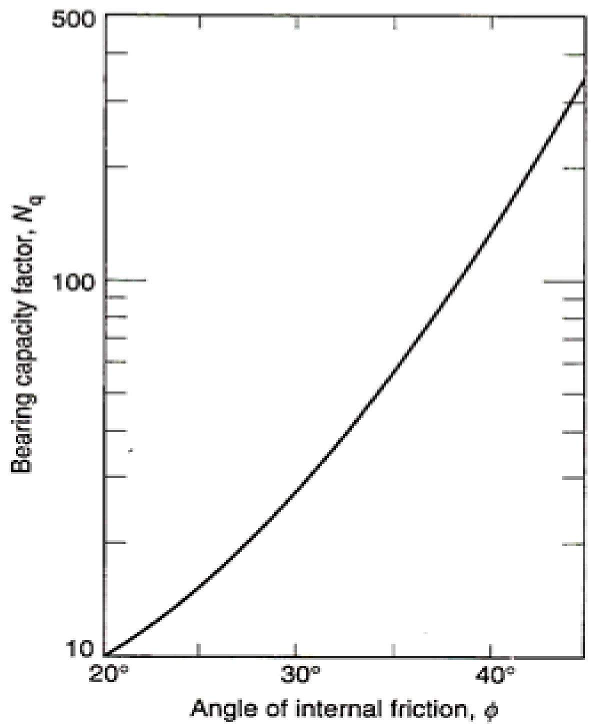


Figure 20.12 Bearing capacity factor N_q for driven piles [as per IS:2911 (Part I/Sec I)–1979].

Figure 20.13 shows the value of Nq as a function of 0 applicable for bored piles as recommended by IS – 2911 (Part 1/ Sec II and Sec IV)-1979.

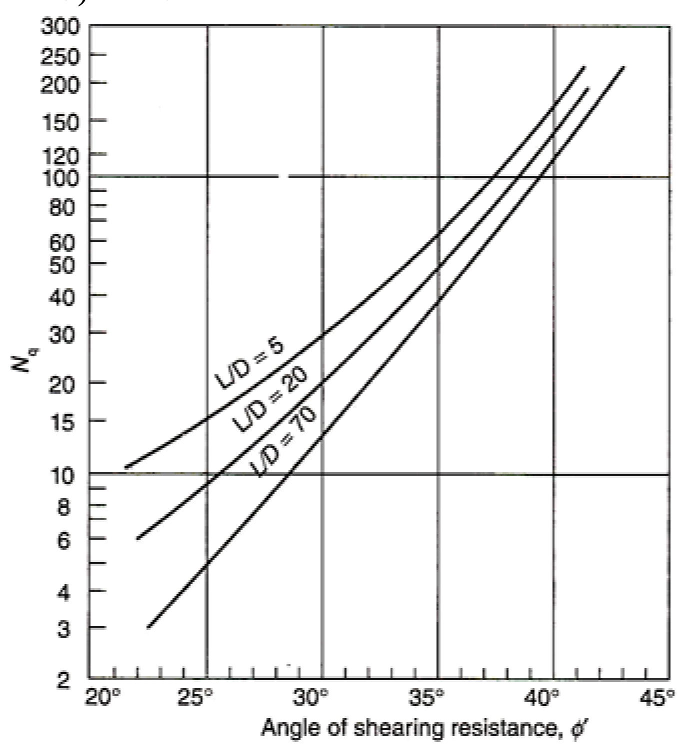


Figure 20.13 Bearing capacity factor N_q for bored piles [as per IS:2911 (Part I/Sec II and Sec IV)–1979].

Both the charts of Nq are based on Berezantsev's curves for d/B of 20 up to $\phi = 35°$ and Vesic's curves for $\phi > 35°$. Berezantsev's curves for Nq are shown in Fig. 20.14.

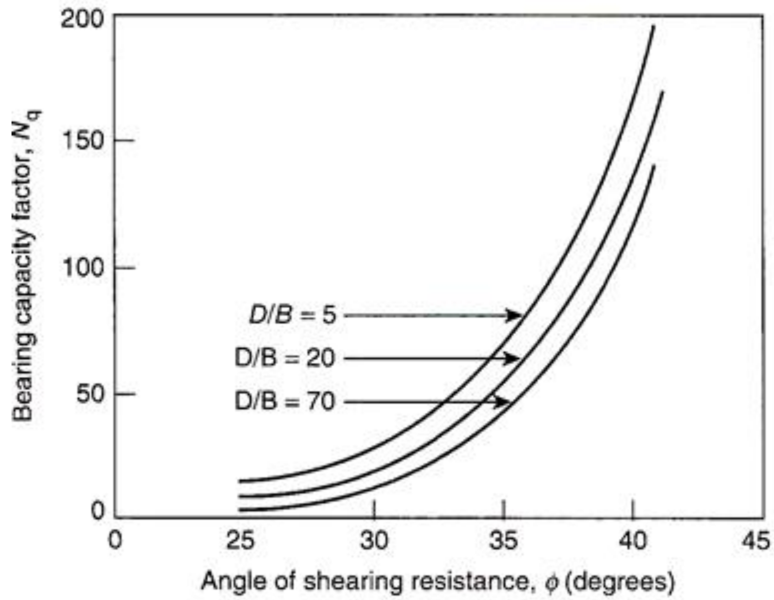


Figure 20.14 Berezantsev's N_q for piles in sands.

Lecture-7

The value of lateral pressure efficient K depends upon the method of construction of the driven pile or the bored pile, as shown in Table 20.4.

Table 20.4 Lateral pressure coefficient for piles

S. No.	*Type of Pile*	*K*
1.	Bored pile	1 to 2
2.	Driven pile	1 to 3

The above static formula is applicable for driven cast in-situ piles. In the case of bored piles, the same formula can be used, but the value of ϕ should be reduced by 3° to account for the loosening of the sand due to drilling of the hole. Also $K = 1 - \sin\phi$ for bored piles. This value varies from 0.3 to 0.75 for bored piles.

The unit point-bearing resistance of bored piles is generally about half to one-third of that of driven piles. Bored cast in-situ piles with an enlarged base show a point-bearing resistance of about 1.5-2 times that of a pile without the enlargement.

Static Formula as per IS Code for Piles in Clay:

As per IS – 2911 (Part I)-1979, the ultimate load capacity of a pile in cohesive soil (clay) is given by –

$Qu = cNcAp + \alpha\bar{c}As$ …(20.12)

where c is the cohesion at the pile tip in kgf/cm2; Nc the bearing capacity factor, equal to 9 for piles; Ap the cross- sectional area of pile toe in cm2; α the reduction factor also called shear mobilization factor or adhesion factor; $\bar{c}$ the average cohesion over the pile length in kg/cm2; and As the surface area of pile shaft in cm2. The value of α as recommended by IS:2911 (Part I)-1979 is given in Table 20.5.

Table 20.5 Value of adhesion factor

S. No.	SPT N Value	Value of α	
		Bored Pile	Driven Pile
1.	< 4	0.7	1
2.	4–8	0.5	0.7
3.	8–15	0.4	0.4
4.	>15	0.3	0.3

Meyerhof's Method for the Load Capacity of Pile:

Meyerhof suggested the following methods for estimation of load capacity of the piles for cohesionless soils and cohesive soils.

For piles in cohesionless soils, the point-bearing resistance of a pile increases with the depth in sands and reaches its maximum value at the critical embedment ratio. The critical embedment ratio $(L/d)_{cr}$ typically ranges from 15 d for loose- to medium-dense sands to 20 d for dense sands. The point-bearing capacity is given by –

$Q_p = A_p\, q\, N_q$ …(20.13)

The correlation of the point-bearing resistance with the SPT N value of cohesionless soils is given by –

$Q_p = A_p \times 0.4N(L/d)$ …(20.14)

where N is the average SPT N value over the depth 10 d above and 4 d below the pile toe.

For saturated clays, the total point-bearing resistance as per Meyerhof's method is given by –

$Q_p = A_p C_u N_c$ …(20.15)

where $N_c = 9$.

Janbu's Method:

As per Janbu's method, the point-bearing capacity of a pile is given by –

$$Q_p = A_p(cN_c + qN_q) \qquad (20.16)$$

$$N_c = (N_q - 1)\cot\phi \qquad (20.17)$$

for sands

$$N_q = [\tan\phi + \sqrt{(1+\tan^2\phi)}]^2 e^{\pi\tan\phi} \qquad (20.18)$$

for clays

$$N_q = [\tan\phi + \sqrt{(1+\tan^2\phi)}]^2 e^{(2\pi/3)\tan\phi} \qquad (20.19)$$

Goodman Method for Point Load Capacity of Piles Resting on Rock:

Goodman (1980) gave the following equation to compute the load capacity of piles resting on rock –

$$Q_p = A_p q_u (N+1) \qquad (20.20)$$

$$N_\phi = \tan^2\left(45 + \frac{\phi}{2}\right) \qquad (20.21)$$

where q_u is the unconfined compressive strength of rock and ϕ is the effective friction angle of rock.

The design value of q_u shall be obtained by dividing the lab q_u by a factor of 5 to account for distributed fractures in the rock, which are not reflected by compression tests on small samples –

$$q_{u(design)} = \frac{q_{u(lab)}}{5} \qquad (20.22)$$

Typical values of qu and ϕ for different types of rock are given in Table 20.6.

Table 20.6 Typical values of q_u and ϕ

S. No.	*Type of Rock*	*$qu \times 10^3$ kN/m²*	*ϕ (deg)*
1.	Sandstone	70–140	27–45
2.	Lime stone	105–210	30–0
3.	Shale	35–70	10–20
4.	Granite	140–210	40–50
5.	Marble	60–70	25–30

Dynamic Formulae for Estimating the Load Capacity of Piles:

Dynamic formulae have been developed as a means to estimate the load capacity of driven piles based on the resistance offered by the soil/rock during pile driving.

i. Ultimate Load Capacity:

It is the maximum load which a pile or pile shaft can carry before failure of ground, when the soil fails by shear as evidenced from the load-settlement curves, or failure of the pile.

ii. Working Load:

It is the load assigned to a pile according to design.

Principle of Dynamic Formulae:

Dynamic formulae are based on the law governing the impact of elastic bodies. Dynamic formulae used for the estimation of ultimate load capacity of driven piles are based on the simple principle that the energy imparted on the pile during driving is equal to the work done in causing penetration of the pile per blow. Thus,

$Wh = Q_uS$ …(20.23)

where W is the weight of the hammer, h is the height of fall of the hammer, Qu is the ultimate load capacity of the pile, which is actually the ultimate resistance offered by the soil supporting the pile, and S is the penetration of the pile per blow, also known as set (from the word "permanent set" in the stress strain theory).

Thus, the load-carrying capacity of driven piles can be estimated on the basis of data obtained during the driving of the pile. The formulae used are, therefore, known as dynamic formulae. As dynamic formulae use the data obtained during the driving of the pile for the estimation of load capacity, they are applicable or useful only for driven piles.

The penetration of the pile during driving under each blow of the hammer depends on the load resistance capacity of the soil into which the pile is driven. The greater is the penetration of the pile per blow, the lesser will be the load resistance capacity of the soil.

Dynamic formulae have been developed on the basis of this principle, considering additional factors such as:

1. Elastic compression of the pile.
2. Additional pressure used for driving the pile as in the case of a double-acting steam hammer.

As the input energy is used to estimate the load capacity based on the penetration of the pile per blow, the loss of energy in applying each blow should be subtracted from the total input energy of Eq. (20.23). Otherwise, dynamic formulae would overestimate the load capacity. The loss of energy in each blow can be due to the inefficient hammer or hammer blow. Also, only that part of input energy which causes penetration of the pile should be used to estimate the load capacity. For example, part of the input energy used for elastic compression of the pile should be deducted before equating it to the work done.

Lecture-8

Types of Dynamic Formulae:

The following are some important dynamic formulae:

1. Engineering News formula.
2. Hiley's formula.
3. Danish formula.

Engineering News formula is the simplest and most popular dynamic formula for the estimation of load capacity. Hiley's formula has been developed later to overcome some of the limitations of Engineering News formula.

1. Engineering News Formula:

This formula was first published in 1888 in Engineering News record and, hence, got its name. As per Engineering News formula, the ultimate load capacity of driven piles is given by –

$$Q_u = \frac{Wh\eta_h}{S+C} \qquad (20.24a)$$

where W is the weight of the hammer, h is the height of fall of the hammer in cm, % is the efficiency of the hammer, S is the set or penetration of the pile per blow, usually taken as the average penetration of the pile for the last 5-10 blows, C is the empirical constant to account for reduction in theoretical set due to energy loss, that is, 2.5 cm for drop hammer and 0.25 cm for stream hammer.

The value of ηh is given in Table 20.7 for different types of hammers. A factor of safety of 6 is used to determine the safe load. The high value of factor of safety reflects the degree of uncertainty associated with the formula.

Table 20.7 Efficiency of hammer

S. No.	Type of Hammer	η_h
1.	Drop hammer	0.7–0.9
2.	Steam hammer	0.75–0.85
3.	Diesel hammer	0.8–0.9

Modified Engineering News Formula:

The Engineering News formula has been modified to consider the energy losses in the hammer blow due to the impact as given by –

$$Q_u = \frac{Wh\eta_h}{S+C}\left(\frac{W+e^2P}{W+eP}\right) \qquad (20.24b)$$

where e is the coefficient of restitution of the pile and P is the weight of the pile.

2. Hiley's Formula:

The energy losses in the application of a hammer blow are not completely considered in the Engineering News formula. Hiley's formula is developed to compute the ultimate load capacity of driven piles, considering various energy losses. Hiley's formula is recommended by IS – 2911 (Part I)-1984 for the determination of ultimate load capacity of piles. As per this code, the modified Hiley's formula is given by –

$$Q_u = \frac{Wh\eta}{S+(C/2)} \qquad (20.25)$$

$$\eta = \left(\frac{W+e^2P}{W+P}\right) \qquad \text{if } W \ge eP \qquad (20.26a)$$

$$\eta = \left(\frac{W+e^2P}{W+P}\right)-\left(\frac{W-eP}{W+P}\right)^2 \qquad \text{if } W < eP \qquad (20.26b)$$

$$C = C_1 + C_2 + C_3 \qquad (20.27)$$

when driving without a dolly or helmet and a cushion of 2.5-cm thickness –

c = 1.77Qu /A …(20.28b)

when driving with a short dolly or helmet and a cushion of up to 7.5-cm thickness –

$$C_1 = \frac{9.05Q_u}{A} \tag{20.28b}$$

$$C_2 = 0.657\frac{Q_u L}{A} \tag{20.29}$$

$$C_3 = 3.55\frac{Q_u L}{A} \tag{20.30}$$

where Qu is the ultimate load capacity of the driven pile in t; W is the weight of the hammer or ram in t; h is the height of free fall of the hammer or ram in cm; η is the efficiency of the hammer blow; S is the final set or penetration of the pile per blow in cm; C is the temporary elastic compressing of (a) dolly and packing (C_1) and (b) pile (C_2) and ground (C_3); P is the weight of the pile, anvil, helmet, and follower in t; e is the coefficient of restitution between the pile and the hammer or ram; 1 is the length of the pile in m; and A is the cross-sectional area of the pile in cm^2.

Dolly is a cushion of hard wood or other material placed on the top of the casing to receive the blows of the hammer. Helmet is a temporary steel cap placed on the top of the pile to distribute the blow over the cross section of the pile and prevent the head of the pile from damage. The upper portion of the helmet is known as dolly and is designed to hold in position a pad, block, or packing or other resilient material for preventing or absorbing shock from the hammer blow. Follower is an extension piece used to transmit the hammer blows on to the pile head. Follower is used when the pile is driven below the pile frame leaders out of reach of the hammer. Follower is also known as a long dolly.

Table 20.8 Coefficient of restitution

S. No.	*Type of Hammer/Ram*	*e*
1.	Steel ram of double-acting steam hammer striking on steel anvil	0.5
2.	Cast iron ram of single-acting steam hammer or drop hammer striking on the head of RCC pile	0.4
3.	Single-acting stem hammer or drop hammer striking a helmet with hardwood dolly striking on RCC pile or directly on the head of timber pile	0.25
4.	Deteriorated pile head or dolly	0

When the pile finds refusal during driving, P should be substituted by 0.5 P in Eqs. (20.27) and (20.28). The values of e for RCC piles as recommended by IS – 2911 (Part I/Sec I and III)-1979 (R 1997) are given in Table 20.8. As it may be observed, Hiley's formula contains the unknown Qu on both sides of the equation and has to be solved for Qu by trial and error. A factor of safety of 2.5 is used on the ultimate load to compute the allowable load.

Modified Hiley's formula is superior to the Engineering News formula, as it takes into account the energy losses during pile driving. The efficiency of the hammer is usually provided by the manufacturer. The usual value of efficiency is given in Table 20.9 for different types of hammers.

Table 20.9 Efficiency of hammer

S. No.	Type of Hammer	Efficiency of Hammer (%)
1.	Trigger-operated drop hammer	100
2.	Winch-operated drop hammer	80
3.	Single-acting steam hammer	90
4.	McKiernan–Terry-type double-acting steam hammer	90

3. Danish Formula:

The ultimate load capacity of the pile as per Danish formula is given by –

$$Q_u = \frac{Wh\eta_h}{S+(C/2)} \quad (20.31)$$

where W is the weight of hammer; h the height of fall of the hammer; ηh the efficiency of hammer; S the final set per blow; and C the elastic compression of the pile given by –

$$C = \sqrt{\frac{2\eta_h Whl}{AE}} \quad (20.32)$$

Where I is the length of the pile; A the cross-sectional area of the pile; and E the modulus of elasticity of pile material. A factor of safety of 3 to 4 is used to determine the allowable load from the ultimate load.

Limitations of Dynamic Formulae:

Following are the limitations of the dynamic formic formulae:

- Ultimate load computed from dynamic formulae represents the resistance of the ground to pile driving but not the static load capacity of the pile. When piles are driven through saturated fine sand, the pore pressure developed reduces the load capacity of the pile by as much as 44% in the Engineering News formula. Thus, dynamic formulae are suitable only for coarse sands, where pore water drains out without development of pore pressure.
- When piles are driven through cohesive soils, the skin friction resistance is reduced and the end-bearing resistance is increased. Thus, dynamic formulae do not represent static load capacity for cohesive soils and, hence, are not suitable for such soils.
- There is uncertainty over the relationship between the dynamic and the static resistance of the soil.
- The law of impact used in dynamic formulae for the computation of load capacity is not strictly valid for piles subjected to the restraining influence of the soil.
- The group action and reduced efficiency of the pile group, compared with the sum of individual load capacity of the piles in the group, are not accounted for in dynamic formulae.
- In the Engineering News formula, the weight of the pile and, hence, its inertia effect are not considered.
- It is difficult to estimate the energy losses due to vibrations and damage to the dolly or packing accurately and, consequently, it results in errors in the computed load capacity of the pile.

PILE LOAD TEST:

- The Pile Load Test is the most reliable method of determining the load carrying of a pile. This test can be performed either on a working pile that forms the foundation of the structure or on a test pile.

- Pile loading test is one of the most common methods for testing the actual in-situ load capacity of any pile. The test method involves the direct measurement of pile head displacement in response to a physically applied load. In this test, piles can be tested for compression, tension, or lateral load

Loads Acting on Piles:

Following are the loads which are to be taken into account while designing a pile.

1) Direct vertical load coming from the superstructure.
2) Impact stresses developed during the process of pile driving.
3) Stresses developed during handling operations.
4) Bending stress developed due to the curvature of a pile.
5) Bending stresses developed due to the eccentricity of loads coming on the pile.
6) Lateral forces due to the wind, waves, currents of water, etc.
7) Impact forces due to the ice sheets or bergs.
8) Impact forces due to ships, in case of marine structures.
9) Force due to the uplift pressure.
10) Earthquake forces.

Lecture-9

Pile Load Test Procedure

The following the procedure of pile load test,

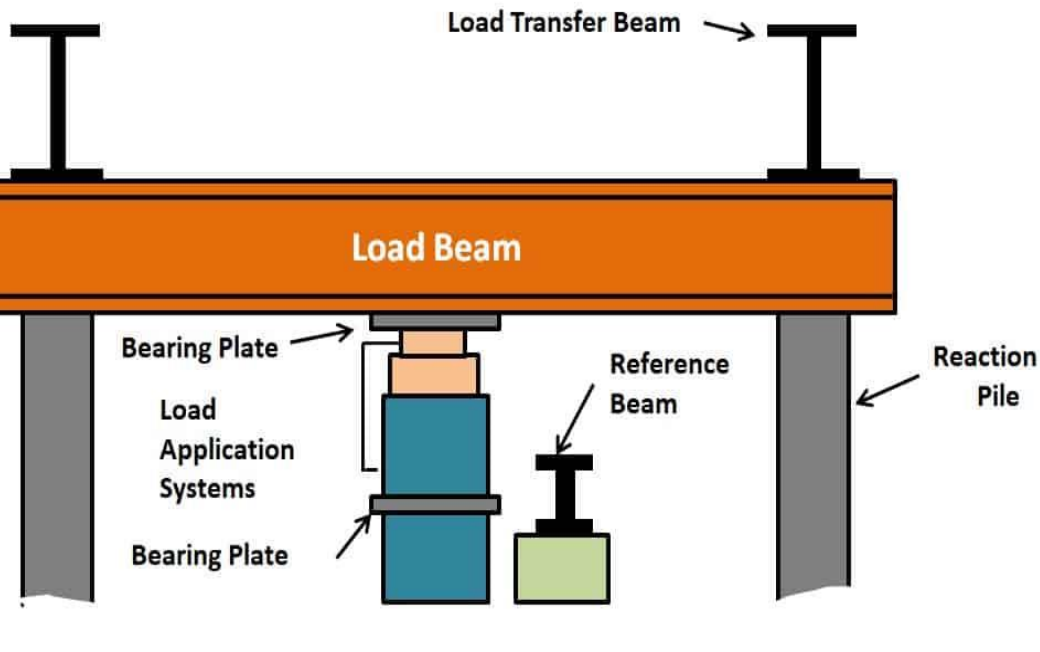


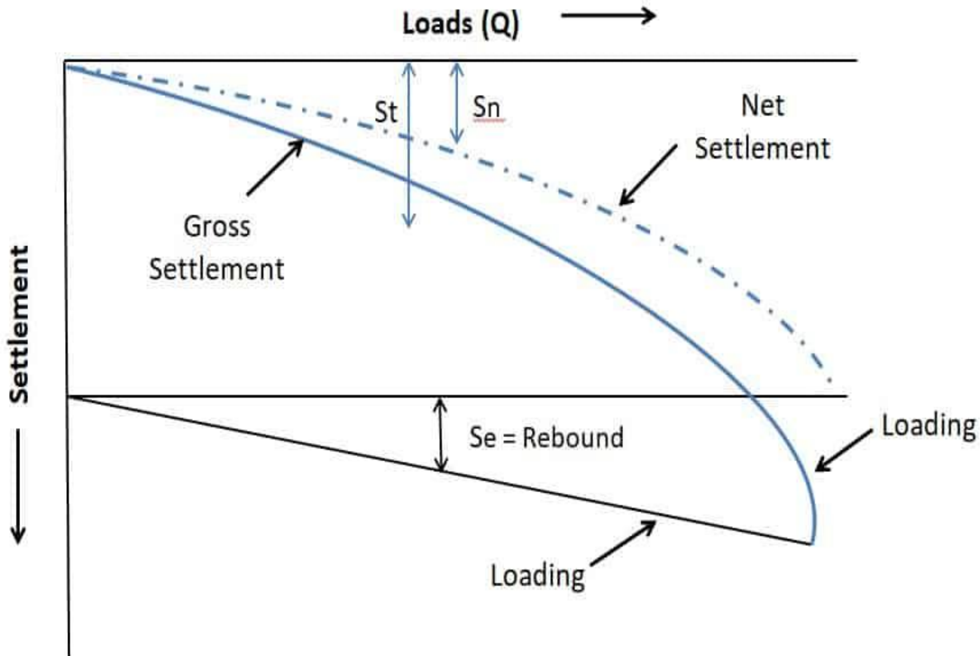
Figure: Pile Load Test

- The sets up for the load test on a pile consist of two anchor piles provided with an anchor girder or a reaction girder at their top as shown in Fig.
- The test pile is generally installed between two anchor piles in such a manner in which the foundation piles are to be installed.

- The test pit should be at least 3B or 2.5 m clear from the anchor piles.
- The toad is applied through a hydraulic jack resting on the reaction girder. The measurements of the settlement of the pile are recorded with the help of three dial gauges, with respect to a fixed reference mark.
- The test is conducted after a period of 3 dales after installation of the test pile in sandy soils, and after a period of one month after the installation of the test pile in silts and soft days.
- This is because by driving the test pile the soil properties are altered and with the passage of time much of the original properties are restored.

Pile Load Test

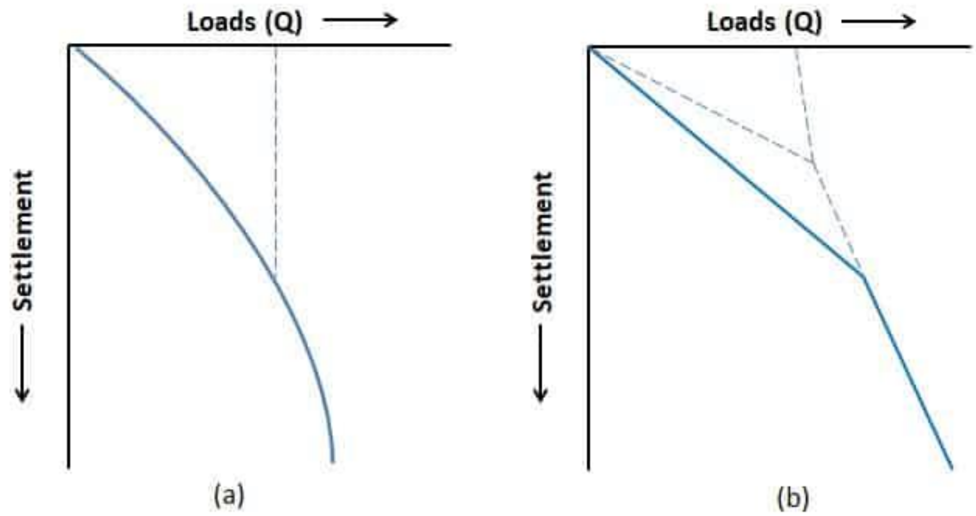
- The load is generally applied in an equal amount of increment and that is about 20 % of the allowable load. Settlements should be recorded with three dial gauges.
- Each load increment is maintained till the rate of movement of the pile is not more than 0.1 mm per hour in sandy soils and 0.02 mm per hour in layer soils or a maximum of two hours (IS: 2911 — 1979).
- For each load inclement settlements are observed at 0.5, 1, 2, 4, 8, 12, 16, 20, 60 minutes. The loading should he continued up to twice the safe load or the load at which the total settlement reaches a specified value.
- The load is removed in the same decrements at 1 hour interval and the final rebound is recorded after 24 hours after the entire load has been removed.



Load-Settlement Curve

- The measured values of the settlement are plotted against the corresponding values of Load to obtain the load settlement curve. Fig. shows a typical load settlement curve (firm line) for loading as well as unloading obtained from a pile load test.

For given load, the net settlement (Sn) is given by,
Sn = St – Se
Where, Sn = Net Settlement
St = Total Settlement or Gross Settlement
Se = Elastic Settlement (rebound)
Fig. shows two loads-settlement curves obtained from a pile load tests on two different soils. The ultimate load Qu may be determined as the abscissa of the point where the load settlement curve changes to a steep straight line.
Alternatively, the ultimate load Qu is the abscissa of the point of intersection of initial and final tangents of the load settlement curve. The allowable load is usually taken as one-half of the ultimate load.



Determination of Ultimate Load from Load Settlement Curve for Pile

According to IS: 2911 a 1974 (Parta4), the allowable load may be taken as one of the following whichever is less.

- 50 % of the load at which the total settlement is 10 % of the diameter of the pile.
- Two—thirds of the final load at which total settlement is 12 mm.
- Two–thirds of the load which causes a net settlement of 6 mm.

The limiting settlement criteria are also sometimes specified. Under the load twice the allowable load, the net settlement should not exceed 20 mm or the gross settlement should not exceed 25 mm.

GROUP ACTION OF PILES, SETTLEMENT OF PILE GROUPS IN CLAY AND IN SAND:

The pile spacing mainly controls the behavior of pile groups. The spacing should not be too small so that upheaval of ground surface takes place during driving into dense or in-compressible material. On the other hand, if spacing is too large, uneconomic pile cap may result. When driving piles in sand and gravels, it is advisable to start driving at the center of the group and then to work outward, in order to avoid difficulty with "tightening up" of the ground.

In the group the following minimum spacing is recommended.

Types of Pile	Minimum Spacing
Friction	Perimeter of the pile
End Bearing	Twice the least width

Screw piles	3/2 the diameter of screw blades

Settlement of Pile Groups in Clay

The settlement of a group of piles in clay cannot be predicted from the results of loading test on a single pile because of time effects, remolding of the soil owing to the pile driving and scale effects are quite different for the single test pile and the pile group. To compute settlement the mode of load transform is to be decided first. The following assumptions have been used.

- An equivalent raft at two third the pile length over the area enclosed by the piles at that depth.
- An equivalent raft at two third the pile length over a large area because of the side friction on the group of piles. A spread of one horizontal to four verticals may be reasonable. Figure 1 shows this situation.

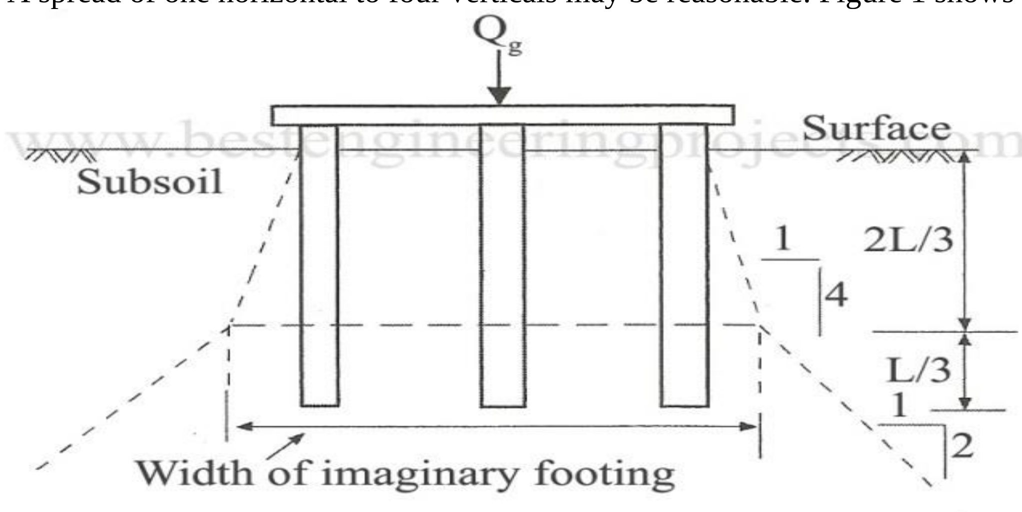


Fig 1 Settlement of Pile Groups in Clay

- An equivalent raft at the base of the length over the area enclosed by the piles at that depth. This is applicable to point bearing piles and is shown in Fig.2.

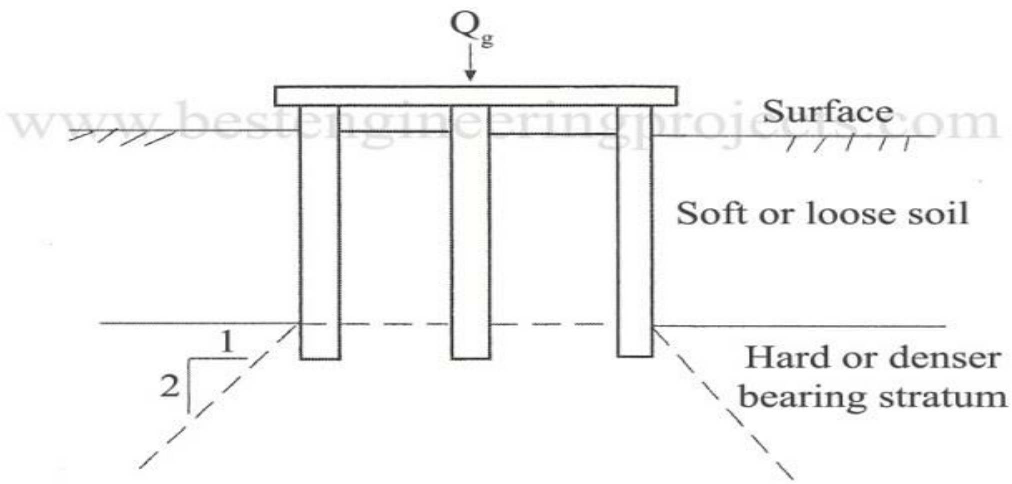


Fig 2 Settlement of Pile Group in Hard Bearing Stratum

Settlement of Pile Groups in Sand

The settlement of pile groups in granular soils can be calculated in a manner similar to that employed for pile groups in clay. In this case a virtual raft is assumed at the base of the pile. The settlement is calculated using the results of Dutch cone or Standard penetration tests.

Generally the above procedure is not adopted. The settlement is calculated using the results of the load tests on individual piles. Skempton in 1953 has compared the settlements of a numbers of pile groups with a settlement of corresponding individual piles and has proposed the following relationship between the settlement sb of a pile group of width B; and observed settlement ss of a single pile of the same loading intensity.

$$\frac{s_b}{s_s} = [\frac{4B+3}{B+4}]^2$$

Care should be taken when the piles are driven into sand and gravels which are underlain by clay, if the stresses transferred to the clay from the pile group may result over-stressing or excessive consolidation. The factor of safety against the bearing capacity failure in the clay can be assumed by assuming a spread of load onto the surface of the clay in the manner as shown in Fig.3 below.

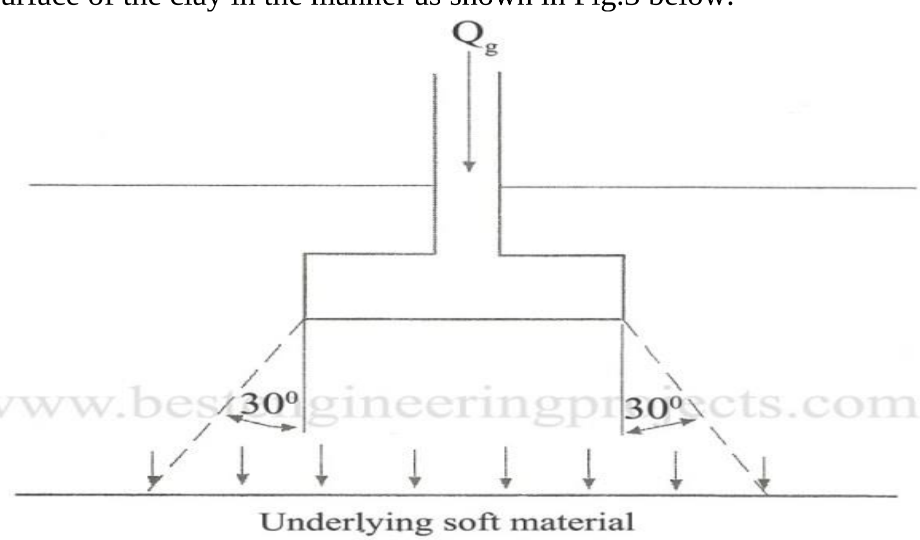


Fig 3 Stress Distribution in Underlying Soft Layer

The settlement in the underlying layer can be computed in the normal way by first determining the distribution of stress throughout the clay layer using the Fig.3.

Lecture-10

NEGATIVE SKIN FRICTION

- When the soil layer surrounding a portion of the pile shaft settles more than the pile, a downward drag occurs on the pile. The drag is known as negative skin friction.
- Negative skin friction develops when a soft or loose soil surrounding the pile settles after the pile has been installed. The negative skin friction occurs in the soil zone which moves downward relative to the pile.The negative friction imposes an extra downward load on the pile. The magnitude of the negative skin friction is computed using the same method as discussed in the preceding sections for the (positive) frictional resistance. However, the direction is downwards.

The net ultimate load-carrying capacity of the pile is given by the equation(fig. 25.10)

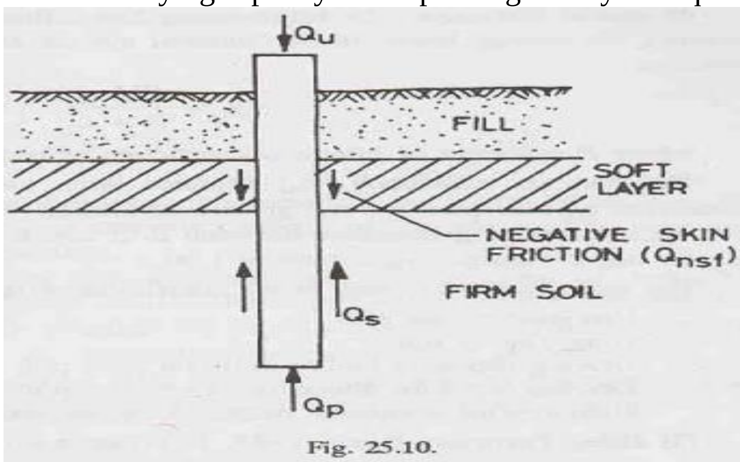


Fig. 25.10.

$$Q_u' = Q_u - Q_{nsf} \qquad ...(25.19)$$

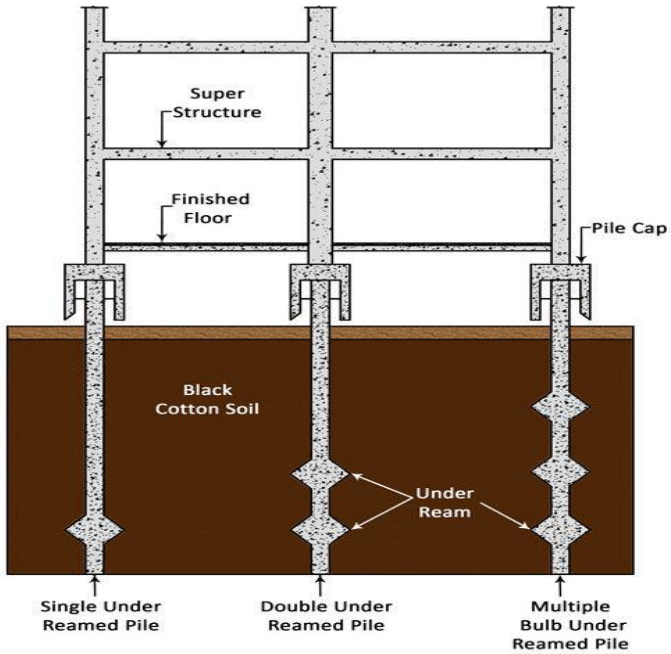
where Q_{nsf} = negative skin friction,

Q_u' = net ultimate load.

Where it is anticipated that negative skin friction would impose undesirable, large downward drag on a pile, it can be eliminated by providing a protective sleeve or a coating for the section which is surrounded by the settling soil.

UNDER-REAMED PILES FOUNDATION

- Under-Reamed Piles Foundation is an answer in area where black cotton soil could cause structural instability. Many times, during, soils undergo volumetric changes due to moisture variation underneath the ground surface. This expansion and shrinkage can cause distress which is very dangerous and critical as far as bearing of the foundation is concerned. The fact is that Under Reamed Piles are considered as most safe and economical foundation for such black cotton soils or expansive soils.
- An Under-Reamed Pile is a cast-in-situ concrete pile, having one or more bulb in its lower portion. This bulb is called an under ream.
- When only one bulb is provided at the bottom of the pile, it is known as single Under-Reamed Pile foundation. When two or more bulbs are provided at the bottom of the pile, it is known as multiple bulbs Under-Reamed Pile foundation.



Uses of Under-Reamed Piles

Under-Reamed Piles are widely used for different types of soils such as sandy soils, clayey soils and also expansive soils. Under-Reamed Piles are required to be taken down to a certain depth because of the following considerations:

- ❖ To avoid the undesirable effect of seasonal moisture changes in expansive soils such as black cotton soils.
- ❖ To reach hard strata.
- ❖ To obtain adequate capacity for downward, upward, lateral loads and moments.
- ❖ To take the foundations below the scour level.
- ❖ They have also been found useful for factory buildings and machine foundations.
- ❖ Under-Reamed Piles are also used under situations, where the vibration and noise caused during construction of piles, are to be avoided.

Lecture-11

WELL FOUNDATION

Well foundation is being used for 100 of years as a deep foundation which is generally provided below the water level for bridges/heavily loaded structures since Roman and Mughal periods.
Well foundation is beneficial for bridge piers, particularly in case of scouring river beds. The various improvised technique of well construction has emerged with the availability of modern equipment which has enabled a reduction in construction time.

Types of Well Foundation:

Types of Well Foundation Depending on Shapes:

Wells can be of different types depending on their cross-sectional shapes which are as follows.

- ❖ Circular
- ❖ Double D
- ❖ Double octagonal
- ❖ Twin Circular
- ❖ Single/ double/ multiple cell rectangular Box well: In this type of well top is open but the bottom is close during construction. This type of well can be used when the load is not very heavy.

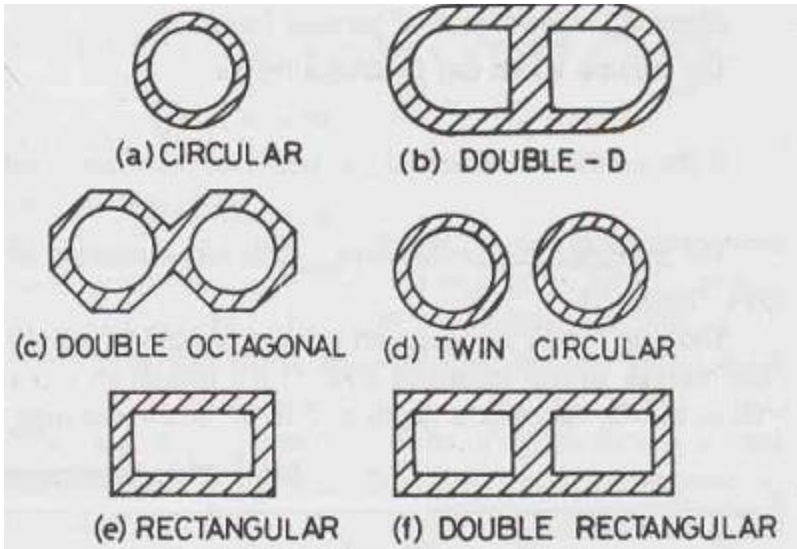


Fig. 27.2. Diferent Shapes of Wells.

<u>Advantages of Circular Well:</u>
Circular type is by far the most commonly used type of well foundation because:

- Simplicity in construction
- least tilt & shift
- weight per sq. meter of the surface area is the highest not required high sinking effort
- Uniform moment of resistance in all directions, etc.

<u>Types of Well Foundation Depending on Construction Method:</u>
Depending on site condition and construction methodology, the well foundation can be classified in following types:

- **Open caisson or open well:**

The processes of sinking are continued till the well reaches the required depths/founding level(RL). During the construction top and bottom portion are opened, once the well is reached at the ground surface or the required depth. The bottom part is sealed with concrete which is known as the bottom plug, and the dredge hole is filled with sand.

- **Land Well:**

Most conventional type, easy for construction, adopted when natural land is available during the construction period.

- **Island Well:**

Similar to land wells, but in deep water where an artificial island can be constructed and maintained during the entire construction period. Generally possible where the depth of water is up to 6m and the velocity of water is within 1m/sec.

- **Floating Caisson:**

It is a special type of well foundation, and where it is not possible to construct and maintain the island during the entire construction period.
The disadvantage of this open well:
If the boulder deposit/Rock is there in the ground surface or the bottom of the water body, then it isn’t easy to in possess or progress the construction of this type of well.

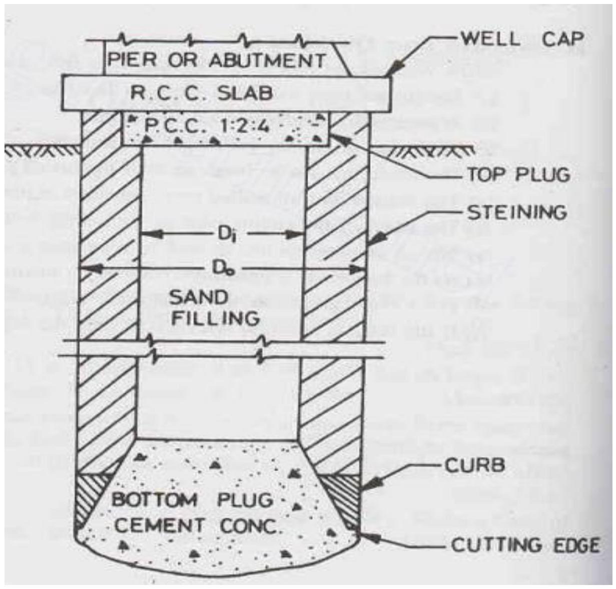
- **Pneumatic Caisson:**

It is closed at the top and opens at the bottom. Complete water is pushed out from the caisson by compressed air. Working chamber and shaft are made air-tight for the construction worker to carry out excavation work underneath. Maximum depth may be 33.00m due to risk to the lives of the worker and team.

Disadvantages:

- In this type of well foundation, construction cost is relatively high.
- The depth of penetration below water level is limited to about thirty-five meter.
- Higher pressure under the working chamber is beyond the endurance of the human body.

Components of Well Foundation:



Cutting Edge:

It is the lowest part of the foundation well, and fabricated in the yard as per approved drawing. Cutting edge shall be fabricated in a number of segments depending upon circumference and ease of handling and transport. Cutting edge helps to cuts the soil during the sinking operation.

Well Curb:

Well Curb is a RCC ring beam-type component of well having a steel cutting edge below. Vertical cross-section of the well curb is wedge-shaped, which helps during the well sinking. The curb supports well steining, and the steining is kept slightly projected outer side to reduce the skin friction during well sinking.

Well Steining:

Steining is a wall or shall type of structure made up of RCC, and which helps to transfer the load to the curb. It serves as an enclosure for excavating the soil for the penetration of the foundation well.

Bottom Plug:

After completion of sinking, the bottom of the foundation well is plugged with the help of concrete. The well curb confines the bottom plug, and functions like a raft against soil pressure from below.

Back-filling:

The well is dewatered after setting of bottom plug concrete, and it is backfilled by sand or excavated approved material. Sometimes water can be used instead of sand/excavated material.

Top plug:

It is a concrete layer of thickness around 500mm, as mentioned in design and drawing. Top plug provided over the filling and inside the well dredge hole.

Well cap:

A Well cap is a thick concrete mat that rests on the top of well stening. It is a part of the foundation and used to distribute loads of superstructure over the steining.

MODULE-4

Lecture-1

Soil Exploration :-

Primary objectives

(i) Determination of the nature of the deposits of soil.
(ii) Determination of the depth and thickness of the various soil strata and their extent in the horizontal direction.
(iii) The location of ground water and fluctuations in GWT.
(iv) Obtaining soil and rock samples from the various strata.

Soil Exploration:- The subsoil exploration should enable the engineer to draw the soil profile indicating the sequence of the strata and the properties of the soils involved.
In general the methods available for soil exploration may be classified as follows:
1. Direct methods – Test pits, trial pits or Trenches.
2. Semi direct methods…… Borings.
3. Indirect methods…… Soundings or penetration tests
and geophysical methods.

Test Pits:
Test pits or trenches are open type or accessible exploratory methods. Soils can be inspected in their natural condition. The necessary soil samples may be obtained by sampling techniques and used for ascertaining strength and other engineering properties by appropriate laboratory tests.

Lecture-2

Boring:-
Making or drilling boreholes into the ground with a view to obtain soil or rock samples from specified or known depths is called boring.
The common methods of advancing bore holes are:-
1. Auger boring.
2. Auger and shell boring.
3. Wash boring
4. Percussion drilling.
5. Rotary drilling.

Spacing of Borings:

Sl no	Nature of the project	Spacing of borings (meters)
1	Highway (subgrade survey)	300 to 600
2	Earth dam	30 to 60
3	Borrow pits	30 to 120
4	Multistory buildings	15 to 30
5	Single story factories	30 to 90

Depth of Borings:- In order to furnish adequate information for settlement predictions, the borings should penetrate all strata that could consolidate significantly under the load of the structure. This necessarily means that, for important and heavy structures such as bridges and tall buildings, the borings should extend to rock. Recommended depths of borings for building are about 3.5 m and 6.5 m for single and two story buildings.

Soil sampling:- Soil sampling is the process of obtaining samples of soil from the desired depth at the desired location in a natural soil deposit, with a view to assess the engineering properties of the soil for ensuring a proper design of the foundation.

Types of samples :- Samples of soil taken out of natural deposits for testing may be classified as disturbed samples and undisturbed samples, depending upon the degree of disturbance caused during sampling operations.

Sounding and penetration tests :- Methods of sounding normally consist of driving or pushing a standard sampling tube or a cone. If a sampling tube is used to penetrate the soil, the test is referred to as the Standard Penetration Test (SPT). If a cone is used to penetrate the soil, the test is called a cone penetration test. Static and dynamic cone penetration tests are used depending upon the mode of penetrationstatic or dynamic.

Standard Penetration Test (SPT) :- SPT is widely used to determine the parameters of the soil in-situ. The test is especially suited for cohessionless soils as a correlation has been established between the SPT value and the angle of internal friction of the soil.

1. Reference can be made to IS 2131 ± 1981 for details on Standard Penetration Test.
2. It is a field test to estimate the penetration resistance of soil.
3. It consists of a split spoon sampler 50.8 mm OD, 35 mm ID, min 600 mm long and 63.5 kg hammer

freely dropped from a height of 750 mm.
4. Test is performed on a clean hole 50 mm to 150 mm in diameter.
5. Split spoon sampler is placed vertically in the hole, allowed to freely settle under its own weight or with blows for first 150 mm which is called seating drive.
6. The number of blows required for the next 300 mm penetration into the ground is the standard penetration number N
7. Apply the desired corrections (such as corrections for overburden pressure, saturated fine silt and energy).
8. N is correlated with most properties of soil such as friction angle, undrained cohesion, density etc.

Advantages of Standard Penetration Test are

1. Relatively quick & simple to perform
2. Equipment & expertise for test is widely available
3. Provides representative soil sample
4. Provides useful index for relative strength & compressibility of soil
5. Able to penetrate dense & stiff layers
6. Results reflect soil density, fabric, stress strain behavior
7. Numerous case histories available

Disadvantages of Standard Penetration Test are

1. Requires the preparation of bore hole.
2. Dynamic effort is related to mostly static performance
3. If hard stone is encountered, difficult to obtain reliable result.
4. Test procedure is tedious and requires heavy equipment.
5. Not possible to obtain properties continuously with depth.

The number of blows for a penetration of 300 mm is designated as the standard penetration value or number N. Corrections for the observed N values :-

1. Due to overburden
2. Due to dilatancy.

Lecture-3

Cone Penetration Test

Cone Penetration Test can either be Static Cone Penetration Test or Dynamic Cone Penetration Test. Continuous record of penetration resistance with depth is achieved. Consists of a cone 36 mm dia (1000 mm 2) and 60° vertex angle. Cone is carried at the lower end of steel rod that passes through steel tube of 36 mm dia. Either the cone, or the tube or both can be forced in to the soil by jacks. Cone is pushed 80 mm in to the ground and resistance is recorded, steel tube is recorded, steel tube is pushed up to the cone and resistance is recorded .Further ,both cone and tube are penetrated 200 mm and resistance is recorded. Total resistance (qc) gives the CPT value expressed in kPa.

Cone resistance represents bearing resistance at the base and tube resistance gives the skin frictional resistance. Total resistance can be correlated with strength properties, density and deformation characteristics of soil. Correction for overburden pressure is applied. Approximately, N = 10qc (kPa).

Advantages of SCPT are

1. Continuous resistance with depth is recorded.
2. Static resistance is more appropriate to determine static properties of soil.
3. Can be correlated with most properties of soil.

Disadvantages of SCPT are

1. Not very popular in India.
2. If a small rock piece is encountered, resistance shown is erratic & incorrect.
3. Involves handling heavy equipment.

Static Cone Penetration Test:-

The static cone test is most successful in soft or loose soils like silty sands, loose sands, layered deposits of sands, silts and clays as well as in clayey deposits. Basically the test procedure for determining the static cone and frictional resistances consists of pushing the cone alone through the soil strata to be tested, then the cone and the friction jacket, and finally the entire assembly in sequence and noting the respective resistance in the first two cases. The process is repeated at predetermined intervals. After reaching the deepest point of investigation the entire assembly should be extracted out of the soil.

Dynamic cone penetration test:-

The cone shall be driven into the soil by allowing the hammer to fall freely through 750 mm each time.

In-situ vane shear test:-

The vane is pushed with a moderately steady force up to a depth of four times the diameter of the borehole or 50 cm, whichever is more, below the bottom. No torque shall be applied during the thrust. The torque applicator is tightened to the frame properly. After about 5 minutes, the gear handle is turned so that the vane is rotated at the rate of 0.10 /s.

Seismic Refraction :-

Here velocity v = $C\sqrt{\frac{Eg}{\gamma}}$

where, v = velocity of the shock wave,

E = modulus of elasticity of the soil.

g = acceleration due to gravity.

γ= density of the soil

c = a dimensionless constant involving Poisson's ratio.

Electrical Resistivity:-

Resistivity is usually defined as the resistance between opposite faces of a unit cube of the material. Each soil has its own resistivity depending upon the water content, compaction and composition for example, the resistivity is high for loose dry gravel or solid rock and is low for saturated silt.

$$\rho = 2\pi D\frac{E}{I}$$

Where, D = distance between electrodes (m) E = potential drop between the inner electrodes (volts) I = current following between the outer electrodes (Amperes) and ρ = mean resistivity (ohmm).

Lecture-4

SUBSURFACE EXPLORATION

Earthwork forms the largest activity of a Civil Engineer. It is well understood that irrespective of the type of civil engineering structure on earth –

• It has to be rested either in soil (e.g., foundations)
• Rested on soil (e.g., pavements) or
• The structure is itself constructed making use of soil (e.g., Earthen dams).

This implies that a better knowledge of the spatial variation of the soils encountered is essential. Therefore, before construction of any civil engineering work a thorough investigation of the site is essential. Site investigations constitute an essential and important engineering program which, while guiding in assessing the general suitability of the site for the proposed works, enables the engineer to prepare an adequate and economic design and to foresee and provide against difficulties that may arise during the construction phase. Site investigations are equally necessary in reporting upon the safety or causes of failures of existing works or in examining the suitability and availability of construction materials. Site investigation refers to the methodology of determining surface and subsurface features of the proposed area.

Information on surface conditions is necessary for planning the accessibility of site ,for deciding the disposal of removed material (particularly in urban areas), for removal of surface water in water logged areas, for movement of construction equipment, and other factors that could affect construction procedures.

Information on subsurface conditions is more critical requirement in planning and designing the foundations of structures, dewatering systems, shoring or bracing of excavations, the materials of construction and site improvement methods. Soil Exploration The knowledge of subsoil conditions at a site is a prerequisite for safe and economical design of substructure elements. The field and laboratory studies carried out for obtaining the necessary information about the surface and subsurface features of the proposed area including the position of the ground water table, are termed as soil exploration or site investigation.

Objectives of soil exploration program

The information from soil investigations will enable a Civil engineer to plan, decide, design, and execute a construction project. Soil investigations are done to obtain the information that is useful for one or more of the following purposes.

1. To know the geological condition of rock and soil formation.
2. To establish the groundwater levels and determine the properties of water.
3. To select the type and depth of foundation for proposed structure
4. To determine the bearing capacity of the site.
5. To estimate the probable maximum and differential settlements.
6. To predict the lateral earth pressure against retaining walls and abutments.

Lecture-5

7. To predict the lateral earth pressure against retaining walls and abutments.
8. To select suitable construction techniques
9. To predict and to solve potential foundation problems
10. To ascertain the suitability of the soil as a construction material.
11. To determine soil properties required for design
11. Establish procedures for soil improvement to suit design purpose
12. To investigate the safety of existing structures and to suggest the remedial measures.
13. To observe the soil performance after construction.
14. To locate suitable transportation routes.

The objectives of soil investigations from various requirements point of view is summarized in Table 1.1

Table 16.1 Objectives of soil investigations

Design requirements	• define stratigraphy/geology. • to determine soil properties required for design. • aid material selection. • to determine the type and depth of foundation
Construction requirements	• to select suitable construction techniques • define equipment and techniques needed • to locate suitable transportation routes
Auditing	• checking a site prior to sale/purchase • to establish procedures for soil improvement to suit design purpose
Monitoring	• to observe the soil performance after construction • determine reasons for poor behaviour • document performance for future reference

Scope of soil investigation

The scope of a soil investigation depends on the type, size, and importance of the structure, the client, the engineer's familiarity with the soil at the site, and local building codes. Structures that are sensitive to settlement such as machine foundations and high-use buildings usually require a thorough soil investigation compared to a foundation for a house. A client may wish to take agreater risk than normal to save money and set limits on the type and extent of the site investigation. If the geotechnical engineer is familiar with a site, he/she may undertake a very simple soil investigation to confirm his/her experience. Some local building codes have provisions that set out the extent of a site investigation. It is mandatory that a visit be made to the proposed site.
In the early stages of a project, the available information is often inadequate to allow a detailed plan to be made. A site investigation must be developed in phases.

Lecture-6

Phases of a Soils Investigation

The soil investigation is conducted in phases. Each preceding phase affects the extent of the next phase. The various phases of a soil investigation are given below:

Phase I. Collection of available information such as a site plan, type, size, and importance of the structure, loading conditions, previous geotechnical reports, topographic maps, air photographs, geologic maps, hydrological information and newspaper clippings.

Phase II. Preliminary reconnaissance or a site visit to provide a general picture of the topography and geology of the site. It is necessary that you take with you on the site visit all the information gathered in Phase I to compare with the current conditions of the site. Here visual inspection is done to gather information on topography, soil stratification, vegetation, water marks, ground water level, and type of construction nearby.

Phase II. Detailed soils exploration. Here we make a detailed planning for soil exploration in the form, trial pits or borings, their spacing and depth. Accordingly, the soil exploration is carried out. The details of the soils encountered, the type of field tests adopted and the type of sampling done, presence of water table if met with are recorded in the form of bore log. The soil samples are properly labeled and sent to laboratory for evaluation of their physical and engineering properties.

Phase IV. Write a report. The report must contain a clear description of the soils at the site, methods of exploration, soil profile, test methods and results, and the location of the groundwater. This should include information and/or explanations of any unusual soil, water bearing stratum, and soil and groundwater condition that may be troublesome during construction.

Soil Exploration Methods

1) Trial pits or test pits
2) Boring
3) probes (in situ test) and geophysical methods

Specific recommendations are made by Indian standards regarding the type, extent and details of subsurface explorations and the number, depth and spacing of boreholes for the following civil engineering works. Following is the list of various codes specified for the said purpose:

- Foundations of Multi-storeyed Buildings (IS: 1892, 1979)
- Earth and rockfill Dams (IS: 6955, 1973)
- Power House Sites (IS: 10060, 1981)
- Canals and Cross Drainage Works (IS: 11385, 1985)
- Ports and Harbours (IS: 4651 – Part 1, 1974)

Trial pits or test pits

- Applicable to all types of soils
- Provide for visual examination in their natural condition
- Disturbed and undisturbed soil samples can be conveniently obtained at different depths

- Depth of investigation: limited to 3 to 3.5 m

Advantages

i) Cost effective
ii) Provide detailed information of stratigraphy
iii) Large quantities of disturbed soils are available for testing
iv) Large blocks of undisturbed samples can be carved out from the pits
v) Field tests can be conducted at the bottom of the pits

Disadvantages

i) Depth limited to about 6m
ii) Deep pits uneconomical
iii) Excavation below groundwater and into rock difficult and costly
iv) Too many pits may scar site and require backfill soils.

Limitations

i) Undisturbed sampling is difficult
ii) Collapse in granular soils or below ground water table

Exploratory borings

Boring is carried out in the relatively soft and uncemented ground which isnormally found close to ground surface. The techniques used vary widely across the world.

Location, spacing and depth of borings

It depends on:
i) Type of structure
ii) Size of the structure
iii) Weight coming

General guidelines for location and depth of bore holes

Boreholes are generally located at

- The building corners
- The centre of the site
- Where heavily loaded columns or machinery pads are proposed.
- At least one boring should be taken to a deeper stratum, probably up to the bedrock if practicable
- Other borings may be taken at least to significant stress level.

LECTURE 7

Spacing of Bore Holes – Codal Recommendations

According to IS 1892 (1979) Code of practice for subsurface investigation:

- For a small building one bore hole or test pit at the centre can give necessary data
- For a building covering not more than 4000 sq.m, one bore hole or test pit at each corner and one at centre is adequate.
- For a large project, the number will depend on its geological features and variation of strata. Generally a grid of 50 m spacing should be used with a combination of bore holes and sounding tests.

Borehole Spacing- Guidelines

Table 17.1 gives the general guidelines for the spacing of boreholes

Type of project	**Spacing (m)**	**Spacing (ft)**
Multi-storey building	10-30	30-100
Industrial plant	20-60	60-200
Highway	250-500	800-1600
Residential subdivision	250-500	800-1600
Dams and dikes	40-80	130-260

Depth of Investigation

The depth of investigation depends on

- The size and type of proposed structure
- Sequence of proposed strata.

The depths of boreholes should cover the zone of soil that will be affected by the structural loads. There is no fixed rule to follow. In most cases, the depths of boreholes are governed by experience based on the geological character of the ground, the importance of the structure, the structural loads, and the availability of equipment.

Guidelines for depth of investigation:

1. At least one boring should be taken to deeper stratum, probably up to the bedrock if practicable.

2. Borings should penetrate at least 3 m into rock.
3. Other borings may be taken at least to significant stress level.

4. In compressible soils such as clays, the borings should penetrate at least between 1 and 3 times the width of the proposed foundation or until the stress increment due to the heaviest foundation load is less than 10%, whichever is greater.

5. In very stiff clays, borings should penetrate 5-7 m to prove that the thickness of the stratum is adequate.

6. Borings must penetrate below any fills or very soft deposits below the proposed structure.

7. The minimum depth of boreholes should be 6 m unless bedrock or very dense material is encountered.

Significant depth

The investigation shall be carried out to the point at which the vertical stress due to proposed structure is equal to or less than 10% of original effective stress at the point before the structure is constructed – **significant depth**

Methods of borings

i) Auger boring – preferred for shallow depths , low ground water table
ii) Wash boring: high water table, deeper soil deposit
iii) Rotary drilling: high quality boring, also for rock drilling
iv) Percussion drilling: fast drilling, not taking samples, gravel

Hand Auger

Enables quick assessment of the soils present in the top few metres of the profile. It is limited by depth of water table in sandy soils and the presence of strong layer.

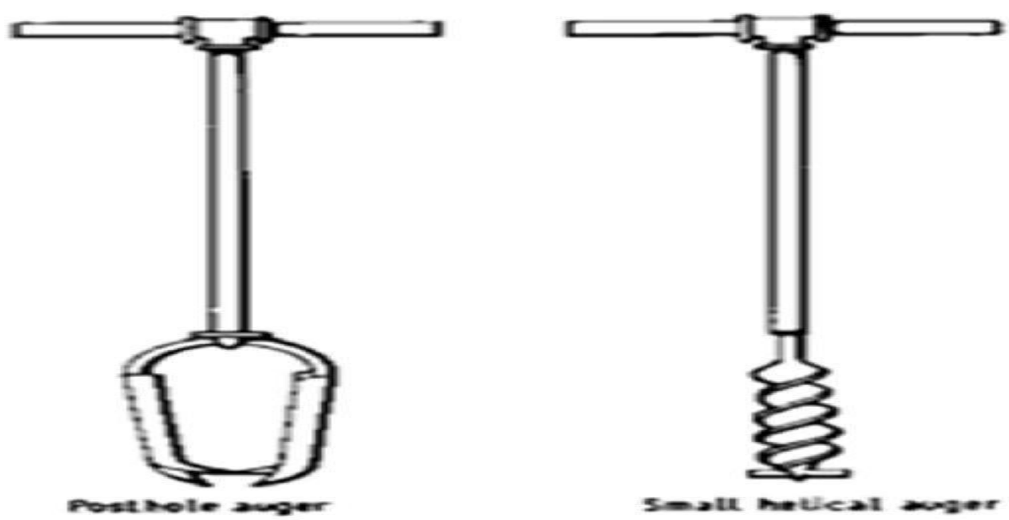


Fig. Augers

Manual boring

It is suitable up to depths of 6m to 8 m. The soil samples obtained from auger borings are highly disturbed. In some non-cohesive soils or soils having low cohesion, the wall of the bore holes will not stand unsupported. In such circumstance, a metal pipe is used as a casing to prevent the soil from caving in .

Mechanical boring

Fig. **Mechanical boring**

LECTURE 8

Wash Boring

Wash boring relies on relatively little drilling action and can form a hole primarily by jetting. This can be undertaken with light equipment without the need for a drilling rig.

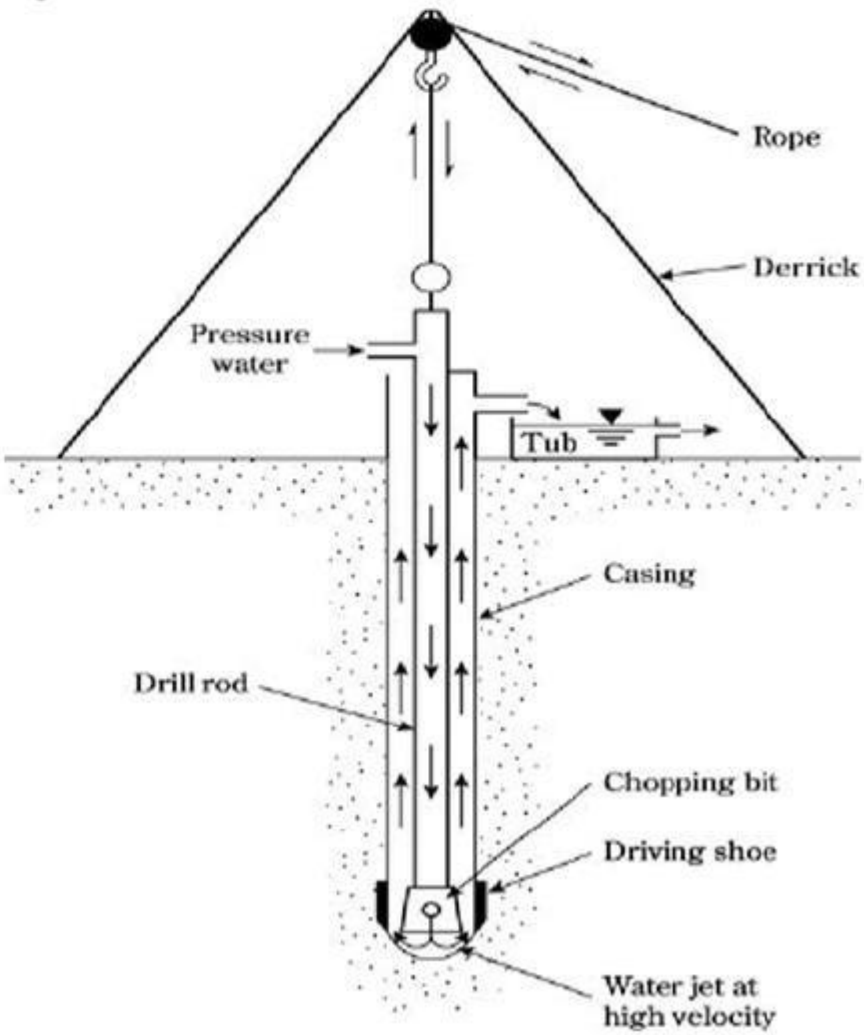


Fig. Wash Boring set up

Soil Sampling

Need for sampling

Sampling is carried out in order that soil and rock description, and laboratory testing can be carried out.

Laboratory tests typically consist of:

i). Index tests (for example, specific gravity, water content)

ii). Classification tests (for example, Atterberg limit tests on clays); and
iii) Tests to determine engineering design parameters (for example strength, compressibility, and permeability).

Factors to be considered while sampling soil

i) Samples should be representative of the ground from which they are taken.
ii) They should be large enough to contain representative particle sizes, fabric, and fissuring and fracturing.

iii) They should be taken in such a way that they have not lost fractions of the *situ soil (for example, coarse or fine particles) and, where strength and compressibility tests are planned ,they should be subject to as little a disturbance as possible.*

Type of soil samples

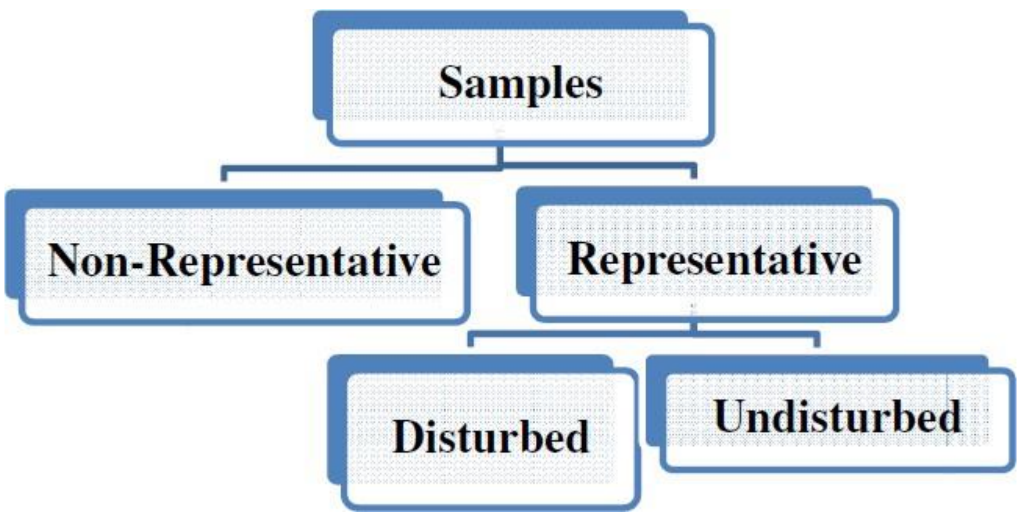


Fig. Types of soil samples

Non-Representative Samples

Non-Representative soil samples are those in which neither the in-situ soil structure, moisture content nor the soil particles are preserved.
• They are not representative
• They cannot be used for any tests as the soil particles either gets mixed up or some particles may be lost.
• e.g., Samples that are obtained through wash boring or percussion drilling.

Disturbed soil samples

Disturbed soil samples are those in which the in-situ soil structure and moisture content are lost, but the soil particles are intact.

• They are representative

• They can be used for grain size analysis, liquid and plastic limit, specific gravity, compaction tests, moisture content, organic content determination and soil classification test performed in the lab

• e.g., obtained through cuttings while auguring, split spoon (SPT), etc.

Undisturbed soil samples

Undisturbed soil samples are those in which the in-situ soil structure and moisture content are preserved.

• They are representative and also intact

• These are used for **consolidation, permeability or shear strengths test (Engineering properties)**

• More complex jobs or where **clay** exist

• In **sand** is very difficult to obtain undisturbed sample

• Obtained by using Shelby tube (thin wall), piston sampler, surface (box), vacuum, freezing, etc.,

Causes of Soil disturbances

- Friction between the soil and the sampling tube
- The wall thickness of the sampling tube
- The sharpness of the cutting edge
- Care and handling during transportation of the sample tube
- To minimize friction

The sampling tube should be pushed instead of driven into the ground Sampling tube that are in common use have been designed to minimize sampling disturbances.

Design Features affecting the sample disturbance

- Area ratio
- Inside Clearance
- Outside Clearance
- Recovery Ratio
- Inside wall friction
- Design of non-return value
- Method of applying force
- sizes of sampling tubes

Area ratio

$$\text{Area ratio } A_r = \frac{Max.\,Cross\ sectional\ area\ of\ the\ cutting\ edge}{Area\ of\ the\ soil\ sample}$$

$$A_r = \frac{{D_2}^2 - {D_1}^2}{{D_1}^2} x\, 100$$

Where, D_1 = inner diameter of the cutting edge.
D_2 = outer diameter of the cutting edge.

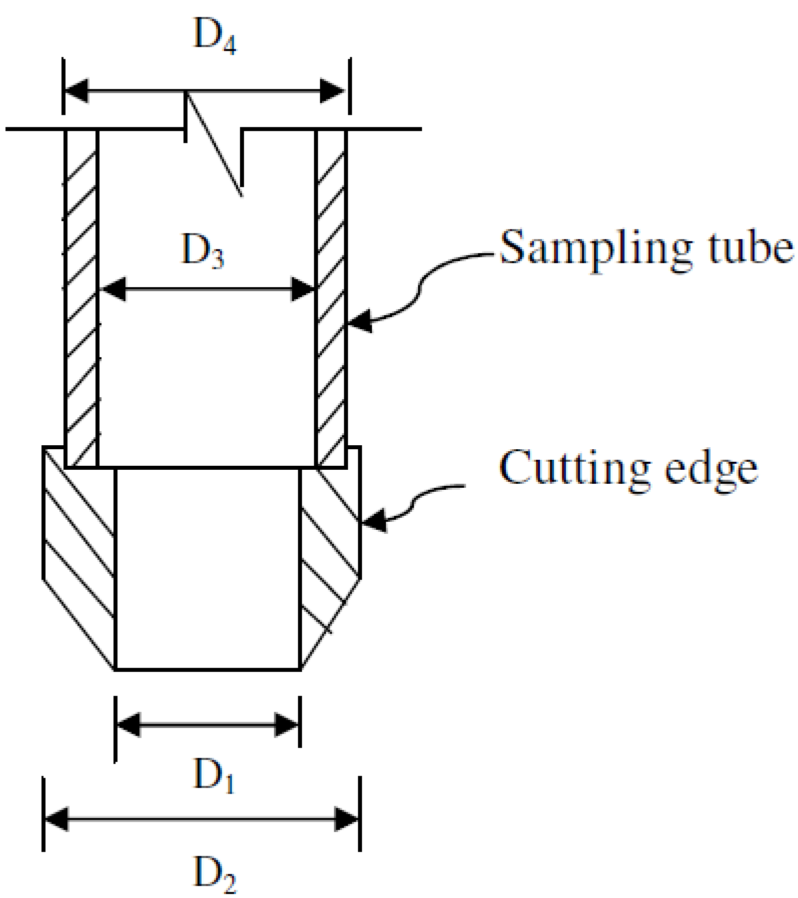


Fig. 18.3 Typical Sampling tube

- For obtaining good quality undisturbed samples, the area ratio should be less than or equal to 10%.

- It may be high as 110% for thick wall sampler like split spoon sampler and may be as low as 6 to 9% for thin wall samples like Shelby tube.

Inside Clearance

$$Ci = \frac{D_3 - D_1}{D_1} \; x\, 100$$

Where D_3 = inner diameter of the sample tube

- The inside clearance allows **elastic expansion** of the sample when it enters the sampling tube.
- It helps in reducing the **frictional drag** on the sample, and also helps **to retain the core.**
- For an undisturbed sample, the inside clearance should be between **0.5 and 3%.**

LECTURE 9

Outside Clearance

$$Co = \frac{D_2 - D_4}{D_4} \; x\, 100$$

Where D_4 = outer diameter of the sample tube

- Outside clearance facilitates the **withdrawal** of the sample from the ground.
- For **reducing the driving force**, the outside clearance should be as small as possible.
- Normally, it lies between zero and 2%.
- C_o Should not be more than C_i

Recovery Ratio

$$R_r = \frac{L}{H}$$

Where
L = length of the sample within the tube and
H = Depth of penetration of the sampling tube

- R_r = 96 – 98 % for getting a satisfactory undisturbed sample

Inside wall friction

- The friction on the inside wall of the sampling tube causes disturbances of the sample.
- Therefore the inside surface of the sampler should be as smooth as possible.
- It is usually smeared with oil before use to reduce friction.

Design of non-return valve

- The non – return valve provided on the sampler should be of proper design.
- It should have an orifice of large area to allow air, water or slurry to escape quickly when the sampler is driven.
- It should close when the sample is withdrawn.

Method of applying force

- The degree of disturbance depends upon the method of applying force during sampling and depends upon the rate of penetration of the sample.
- For obtaining **undisturbed samples, the sampler should be pushed and not driven**

Type of Soil Samplers

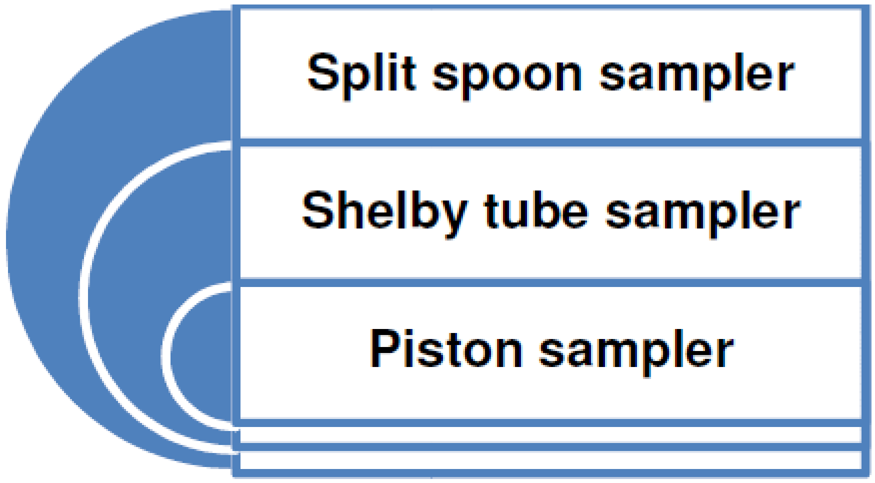


Fig. Type of samplers

Split Spoon Sampler

- Has an inside diameter of 35mm and an outside diameter of 50mm.
- Has a split barrel which is held together
  and a cap at the upper end.
- The thicker wall of the standard sampler permits higher driving stresses than the Shelby tube but does so at the expense of higher levels of soil disturbances.
- Split spoon samples are highly disturbed.
- They are used for visual examination and for classification tests.

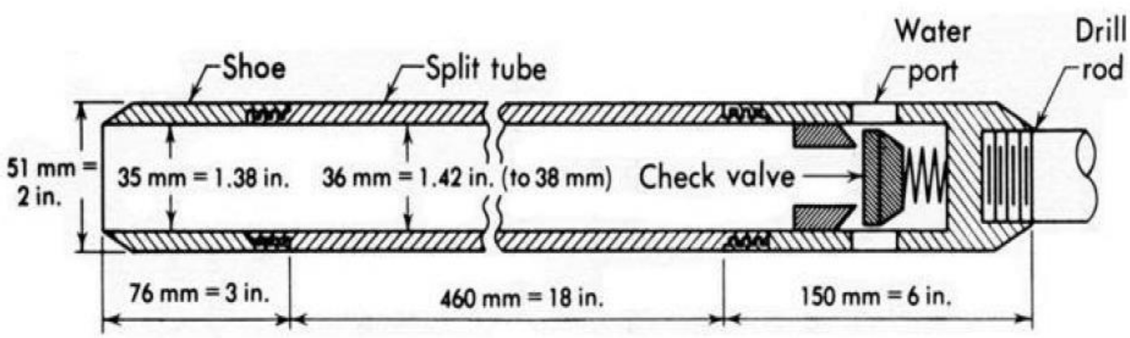


Fig. Split spoon sampler

Shelby Tube Sampler

- Thin-walled seamless steel tube of diameter 50mm or 75mm
- The bottom end of the tube is sharpened.
- The tubes can be attached to drilling rods.
- The drilling rod with the sampler attached is lowered to the bottom of the borehole and the sampler is pushed into the soil.
- The soil sample inside the tube is then pulled out.
- The two ends of the sampler are sealed and sent to the lab.
- The samples can be used for consolidation or shear tests.

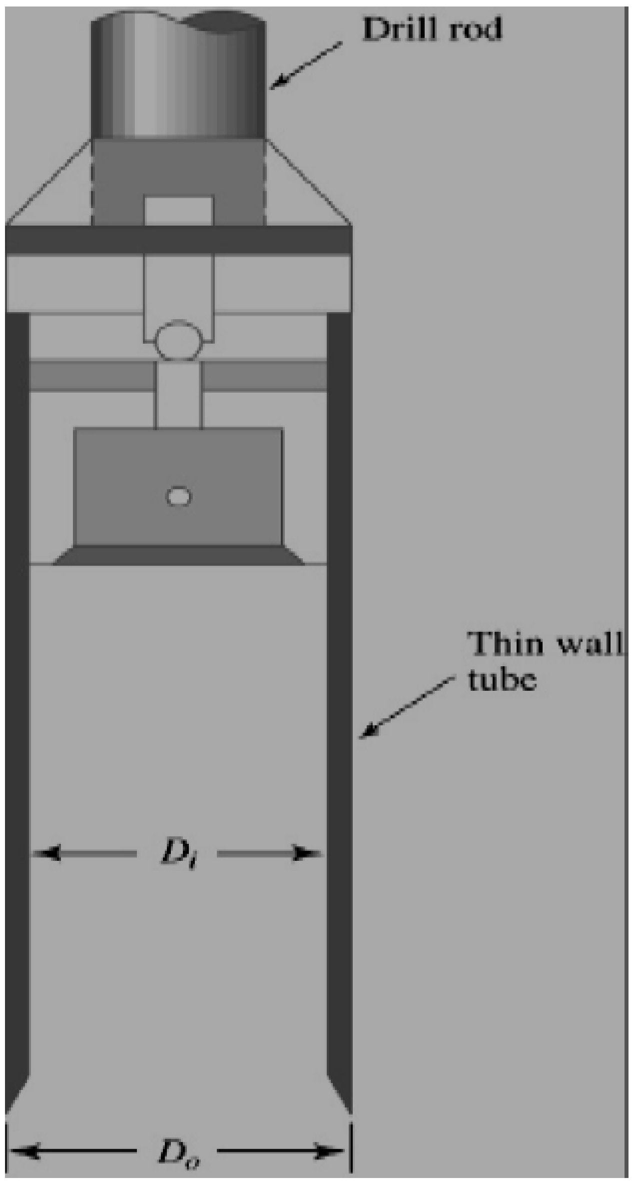


Fig. Shelby tube sampler

Plate Shelby tube sampler

LECTURE 10

Piston Sampler

- For sampling very soft or larger than 76.2mm in diameter to get high quality undisturbed samples, they tend to fall out of the sampler. Then piston samplers are used.
- They consist of a thin wall tube with a piston.
- Initially, the piston closes the end of the thin wall tube. The sampler is lowered to the bottom of the borehole and then the thin wall tube is pushed into the soil hydraulically past the piston.
- Later the pressure is released through a hole in the piston rod. To a large extent, the presence of the piston prevents distortion in the sample by not letting the soil squeeze into the sampling tube very fast and by not admitting excess soil.
- Consequently, samples obtained in this manner are less disturbed than those obtained by Shelby tubes.

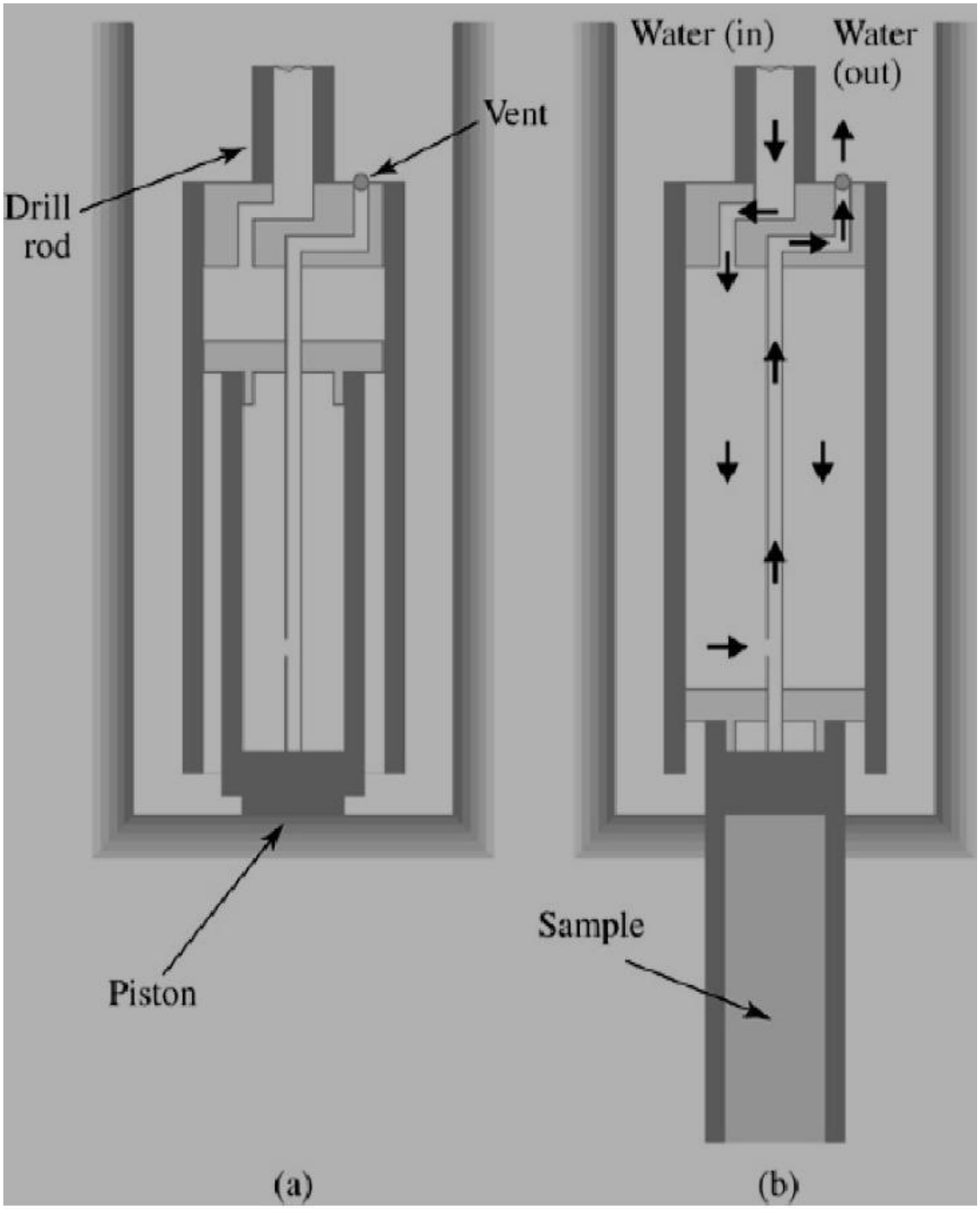


Fig. Piston sampler

Bore Hole Support or Stabilization of Bore Hole

This necessary to prevent cohesion less soils against caving while drilling bore hole.
Either of the following is used for bore hole support

• **Steel casing –** hydraulically pushed
• **Drilling mud** – Circulation bentonite slurry

Stabilization of bore holes using drilling mud

Bentonite mud or Drilling mud" is a thin mixture of water and bentonite clay, which can be mixed in powder form to the drilling water to create higher density suspension.

Advantages

• It is advantages over water.
• Firstly, it is more viscous and can therefore lift cuttings adequately at a lower velocity.
• Secondly it will cake the edges of the borehole, and the outside of the core, and will largely eliminate the seepage of water out of the borehole, thus reducing problems of loss of return.
• Hence, smaller volumes of flush fluid will be required and the fluid may he recirculated via a settling tank (where the cuttings are allowed to drop out of suspension).
• The cake formed on the outside of the borehole has the effect of considerably improving Borehole stability and the prevention of softening of weak rock cores.

Disadvantages

• The bentonite mud-soil cakes are difficult to dispose of, at the end of drilling a borehole. The mud cannot simply be tipped on the site, and it cannot be discharged into nearby sewers.
• Bentonite mud must be properly mixed, using appropriate equipment, in order to ensure that it is of the correct consistency and does not contain unmixed dry bentonite lumps, capable of clogging flush ports in the core barrel.

In Situ Testing

- There is a wide variety of different tests that can be used for evaluating the properties of the ground.
- It is often preferable to do an *in situ test in an attempt to* measure a particular parameter, rather than obtain a sample and do a laboratory test.
    a. sampling results in disturbance (reduces strength and stiffness)
    *b.*sometimes only best (strongest) material is recovered, and is not representative of overall *in situ material*

Parameters obtained from In Situ Testing

- Typical parameters that may be obtained either directly, or indirectly from *in situ tests:*

1. strength
2. stiffness
3. permeability
4. relative density

In-situ Tests

In situ testing is a division of field testing corresponding to the cases where the ground is tested in-place by instruments that are inserted in or penetrate the ground. In-situ tests are normally associated with tests for which a borehole either is unnecessary or is only an incidental part of the overall test procedure, required only to permit insertion of the testing tool or equipment.The role of specialized in-situ testing for site characterization and the research and development of in-situ techniques have received considerable attention over the last 15 years or so. The use of specialized in-situ testing in geotechnical engineering practice is rapidly gaining increased popularity. In Europe, specialized in-situ testing has been commonly used for more than 25 years. Improvements in apparatus, instrumentation, and technique of deployment, data acquisition and analysis procedure have been significant. The rapid increase in the number, diversity and capability of in-situ tests has made it difficult for practicing engineers to keep abreast of specialized in-situ testing and to fully understand their benefits and limitations. Table below summarizes the primary advantages and disadvantages of in-situ testing

advantages and disadvantages of in-situ testing

Advantages	Disadvantages
• Tests are carried out in place in the natural environment without sampling disturbance, which can cause detrimental effects and modifications to stresses, strains, drainage, fabric and particle arrangement. • Continuous profiles of stratigraphy and engineering properties/characteristics can be obtained. • Detection of planes of weakness and defects are more likely and practical. • Methods are usually fast, repeatable, produce large amounts of information and are cost effective Tests can be carried out in soils that are either impossible or difficult to sample without the use of expensive specialized methods. • A large volume of soil may be tested than is normally practicable for laboratory testing. This may be more representative of the soil mass.	• Samples are not obtained; the soil tested cannot be positively identified. The exception to this is the SPT in which a sample, although disturbed, is obtained. • The fundamental behaviour of soils during testing is not well understood. • Drainage conditions during testing are not known. • Consistent, rational interpretation is often difficult and uncertain. • The stress path imposed during testing may bear no resemblance to the stress path induced by full-scale engineering structure. • Most push-in devices are not suitable for a wide range of ground conditions. • Some disturbance is imparted to the ground by the insertion or installation of the instrument. • There is usually no direct measurement of engineering properties.

LECTURE 11

The in-situ tests that are most commonly used in practice are:

(i) Standard penetration test (SPT)
(ii) Cone – penetration test (CPT)
(iii) Piezo-cone penetration test (CPTU)
(iv) Field vane shear test (FVT)
(v) Pressure meter test (PMT)
(vi) Dilatometer test (DMT)
(vii) Becker Penetration Test (BPT) and
(viii) Iowa Bore hole shear test (BHST)
(ix) Plate load test

Standard Penetration Test (SPT):
One of the most common in-situ tests is the standard penetration test or SPT. This test which was originally developed in the late 1920s, is currently the most popular and economical means to obtain subsurface information (both inland and offshore). It offers the advantage of low cost, applicability to many soil types, samples are obtained (although disturbed) and a large database from which many useful correlations have been developed.

Procedure:
The standard penetration test is conducted in a borehole using a standard split-spoon sampler.

(i) When the borehole (55 to 150 mm in dia) has been drilled to the desired depth, the drilling tools are removed and the split-spoon sampler, attached to standard drill rods of required length is lowered to the bottom of the borehole and rested at the bottom .

(ii) The split-spoon sampler is then driven into the soil for a distance of 450 mm in three stages of 150 mm each by blows of a drop hammer of 63.5 kg mass falling vertically and freely through a height of 750 mm at the rate of 30 blows per minute (IS 2131 – 1981). The number of blows required to penetrate every 150-mm is recorded while driving the sampler. If full penetration is obtained, the blows for the first 150 mm is retained for reference purposes, but not used to compute the SPT value because the bottom of the boring is likely to be disturbed by the drilling process and may be covered with loose soil that may fall from the sides of the boring. The number of blows required for the next 300 mm of penetration is recorded as the SPT value. The number of blows is designated as the “Standard Penetration Value” or “Number” N.

(iii) The slit-spoon sampler is then withdrawn and is detached from the drill rods. The split barrel is disconnected from the cutting shoe and the coupling. The soil sample collected inside the split barrel is carefully collected so as to preserve the natural moisture content and transported to the laboratory for tests. Sometimes, a thin liner is inserted within the split-barrel so that at the end of the SPT, the liner containing the soil sample is sealed with molten wax at both its ends before it is taken away to the laboratory. Usually SPT is carried out at every 0.75-m vertical interval or at the change of stratum in a borehole. This can be increased to 1.5 m if the depth of borehole is large. Due to the presence of boulders or rocks, it may not be possible to drive the sampler to a distance of 450 mm. In such a case, the N value can be recorded for the first 300-mm penetration.

The boring log shows refusal and the test is halted if:
(i) 50 blows are required for any 150 mm penetration
(ii) 100 blows are required for 300 mm penetration
(iii) 10 successive blows produce no advance

Plate Standard Penetration testing

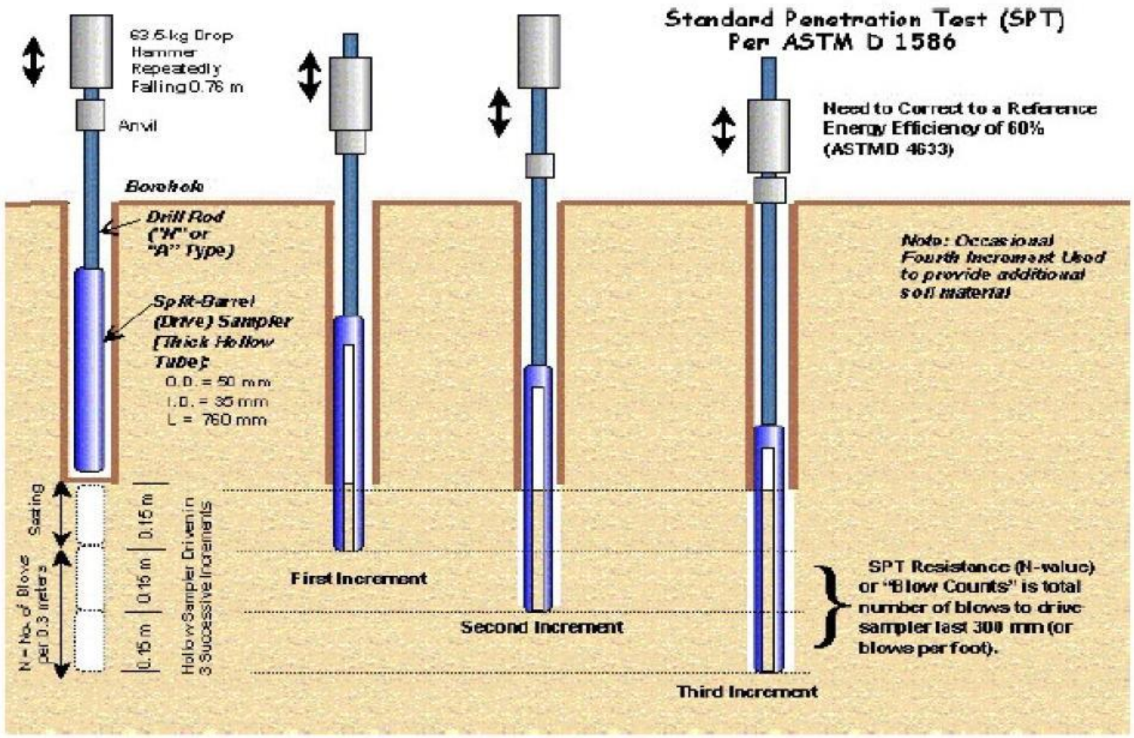
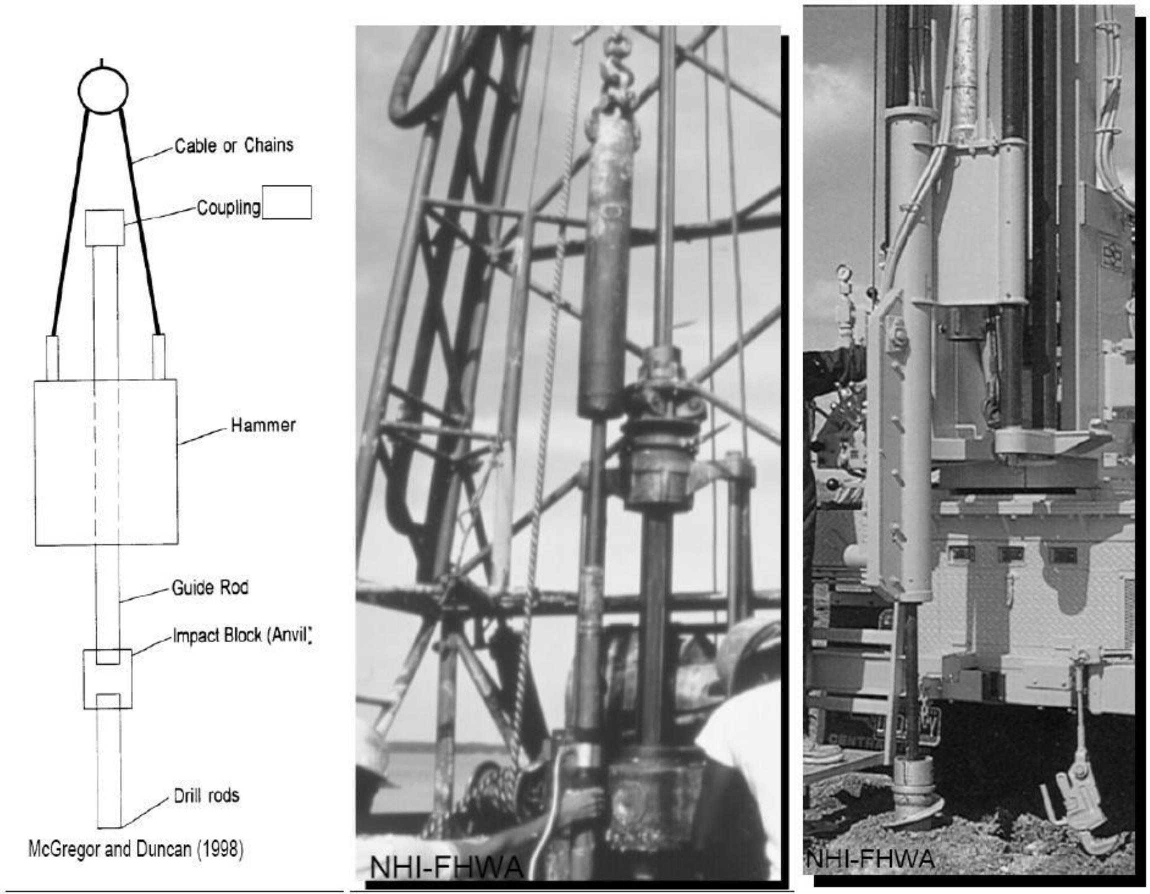


Fig. Stages of Standard Penetration testing



a) Donut hammer **b)Safety Hammer** **c) Automatic hammer**

Fig. Types of SPT hammers

Precautions:

Some of the precautions to be observed to avoid some of the pitfalls of the test are as follows:

(i) The drill rods should be of standard specification and should not be in bent condition.

(ii) The split spoon sampler must be in good condition and the cutting shoe must be free from wear and tear.

(iii) The drop hammer must be of right weight and the fall should be free, frictionless and vertical.

(iv) The height of fall must be exactly 750 mm. Any change in this will seriously affect the N value.

(v) The bottom of the borehole must be properly cleaned before the test is carried out. If this is not done, the test gets carried out in the loose, disturbed soil and not in the undisturbed soil.

(vi) When a casing is used in borehole, it should be ensured that the casing is driven just short of the level at which the SPT is to be carried out. Otherwise, the test gets carried out in a soil plug enclosed at the bottom of the casing.

(vii) If the water level in the borehole is lower than the ground water level, „quick" condition may develop in the soil and very low N values may be recorded.
In spite of all these imperfections, SPT is still extensively used because the test is simple and relatively economical. It is the only test that provides representative soil samples both for visual inspection in the field and for natural moisture content and classification tests in the laboratory. Because of its wide usage, a number of time-tested correlations between N value and soil parametersare available, mainly for cohesionless soils. Even design charts for shallow foundations resting on cohesionless soils have been developed on the basis of N values. The use of N values for cohesive soils is limited, since the compressibility of such soils is not reflected by N values.

SPT values obtained in the field for sand have to be corrected before they are used in empirical correlations and design charts. IS: 2131 – 1981 recommends that the field value of N corrected for two effects, namely,

(a) effect of overburden pressure, and (b) effect of dilatancy

(a) Correction for overburden pressure:
Several investigators have found that the overburden pressure influences the penetration resistance or the N value in a granular soil. If two granular soils possessing the same relative density but having different confining pressures are tested, the one with a higher confining pressure gives a higher N value. Since the confining pressure (which is directly proportional to the overburden pressure) increases with depth, the N values at shallow depths are underestimated and the N valuesat larger depths are overestimated. Hence, if no correction is applied to recorded N values, the relative densities at shallow depths will be underestimated and at higher depths, they will beoverestimated. To account for this, N values recorded (N_R) from field tests at different effective overburden pressures are corrected to a standard effective overburden pressure.
The corrected N value is given by

$$N_C = C_N N_R$$

Where, N_C = corrected value of observed N value
C_N = correction factor for overburden pressure
N_R = Recorded or observed N value in the field
The correction proposed by Peck, Hanson and Thornburn (1974) is given by the equation:

$$C_N = 0.77 \log_{10} \frac{2000}{\sigma'}$$

Where, σ ′ = Effective overburden pressure at the depth at which N value is recorded, in kPa .

LECTURE 12

(b) Correction for dilatancy:

Dilatancy correction is to be applied when N_c obtained after overburden correction, exceeds 15 in saturated fine sands and silts. IS: 2131 – 1981 incorporates the Terzaghi and Peck recommended dilatancy correction (when $N_c > 15$) using the equation .

$$\mathbf{N'c = 15 + 0.5\,(N' - 15)}$$

Where N'_c = final corrected value to be used in design charts.

$N_c > 15$ is an indication of a dense sand, based on the assumption that critical void ratio occurs at approximately $N_c = 15$. The fast rate of application of shear through the blows of a drop hammer is likely to induce negative pore water pressure in saturated fine sand under undrained condition of loading. Consequently, a transient increase in shear resistance will occur, leading to a SPT value higher than the actual one.

Note: The overburden correction is applied first. This value is used as observed N value and then the dilatancy correction is applied.

Correlation of 'N' with engineering properties:

The value of standard Penetration number depends upon the relative density of the cohesionless soil and the UCC strength of the cohesive soil.
The angle of shearing resistance (φ) of the cohesionless soil depends upon the number N. In general, greater the N-value, greater is the angle of shearing resistance. Table below gives the average values of φ for different ranges of N.

Table: Correlation between N value and angle of shearing resistance

N	Denseness	φ
0-14	Very loose	$25^o - 32^o$
4-10	Loose	$27^o - 35^o$
10-30	Medium	$30^o - 40^o$
30-50	Dense	$35^o - 45^o$
>50	Very dense	$> 45^o$

The consistency and the UCC strength of the cohesive soils can be approximately determined from the SPT number N. Table **22.2** gives the approximate values of UCC strength for different ranges of N.

Table: correlation between N value and UCC strength

N	Consistency	q_u (kN/m^2)
0-2	Very soft	< 25
2-4	Soft	25 – 50
4-8	Medium	50 – 100
8-15	Stiff Very	100 – 200
15-30	stiff	200 – 400
>30	Hard	> 400

It can also be determined from the following relation
$q_u = 12.5$ N
Where, q_u = UCC strength (kN/m^2)

Cone Penetration Test (CPT)

(a) Dynamic Cone Penetration Test (DCPT)

In this test, a cone, which has an apex angle of 60^o and attached to drill rods is driven into the soil by blows of a hammer of 63.5 kg, falling freely from a height of 750 mm. The blow count for every 100-mm penetration of the cone is continuously recorded. The cone is driven till refusal or upto the required depth and the drill rods are withdrawn, leaving the cone behind in the ground. The number of blows required for 300-mm penetration is noted as the dynamic cone resistance, N_{cd}. The test gives a continuous record of N_{cd} with depth. No sample, however, can be obtained in this test.

Dynamic cone penetration tests are performed either by using a 50 mm diameter cone without bentonite slurry (IS: 4968 – Part I – 1976) or a 65 mm diameter cone with bentonite slurry (IS: 4968 – Part II – 1976). When bentonite slurry is used, the set-up has an arrangement for the circulation of slurry so that friction on the drill rod is eliminated. The dynamic cone test is a quick test and helps to cover a large area under investigation rather economically. It helps in identifying the uniformity or the variability of the subsoil profile at the site and reveals local soft pockets, if any. It can also establish the position of rock stratum, when required. The test is much less expensive and much quicker than the SPT. If the tests are carried out close to a few boreholes, the data from DCPT canbe compared with the SPT data and correlation between the two established for the particular site conditions. The correlation can then be used to obtain N values from N_{cd} values.

Some approximate correlations between N_{cd} and N, applicable for medium to fine sands are given below:

When a 50 mm diameter cone is used,
N_{cd} = 1.5 N for depths upto 3 m
N_{cd} = 1.75 N for depths from 3 m to 6 m
N_{cd} = 2.0 N for depths greater than 6 m

(b) Static cone penetration test (CPT)

The static cone penetration test, simply called the cone penetration test (CPT), is a simple test that is presently widely used in place of SPT, particularly for soft clays and fine to medium sand deposits. The test was developed in Holland and is, therefore, also known as the Dutch cone test. The test assembly is shown in Fig **22.2** The penetrometer that is commonly used is a cone with an apex angle of 60^o and a base area of 10 cm^2.

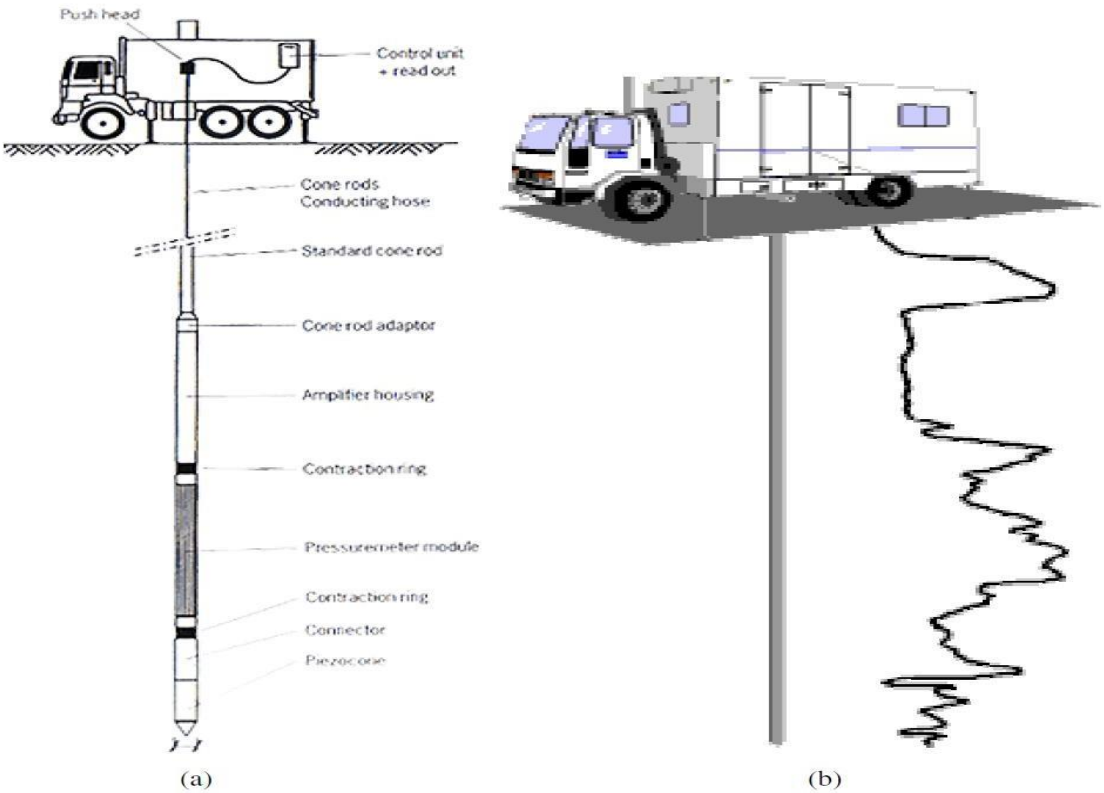


Fig. Cone penetration set up

The sequence of operations of the penetrometer is as follows:
1. *Position 1:* The cone and the friction jacket is in a stationary position.
2. *Position 2*: The cone is pushed into the soil by the inner drill rod/sounding rod to a depth „**a**", at a steady rate of 20 mm/s, till a collar engages the cone. The tip resistance q_c called the cone or point resistance, can be calculated by the force Q_c read on a pressure gauge.
The tip resistance, $q_c = Q_c / A_c$
Where A_c is the base area
Normally the value of **a = 40 mm**

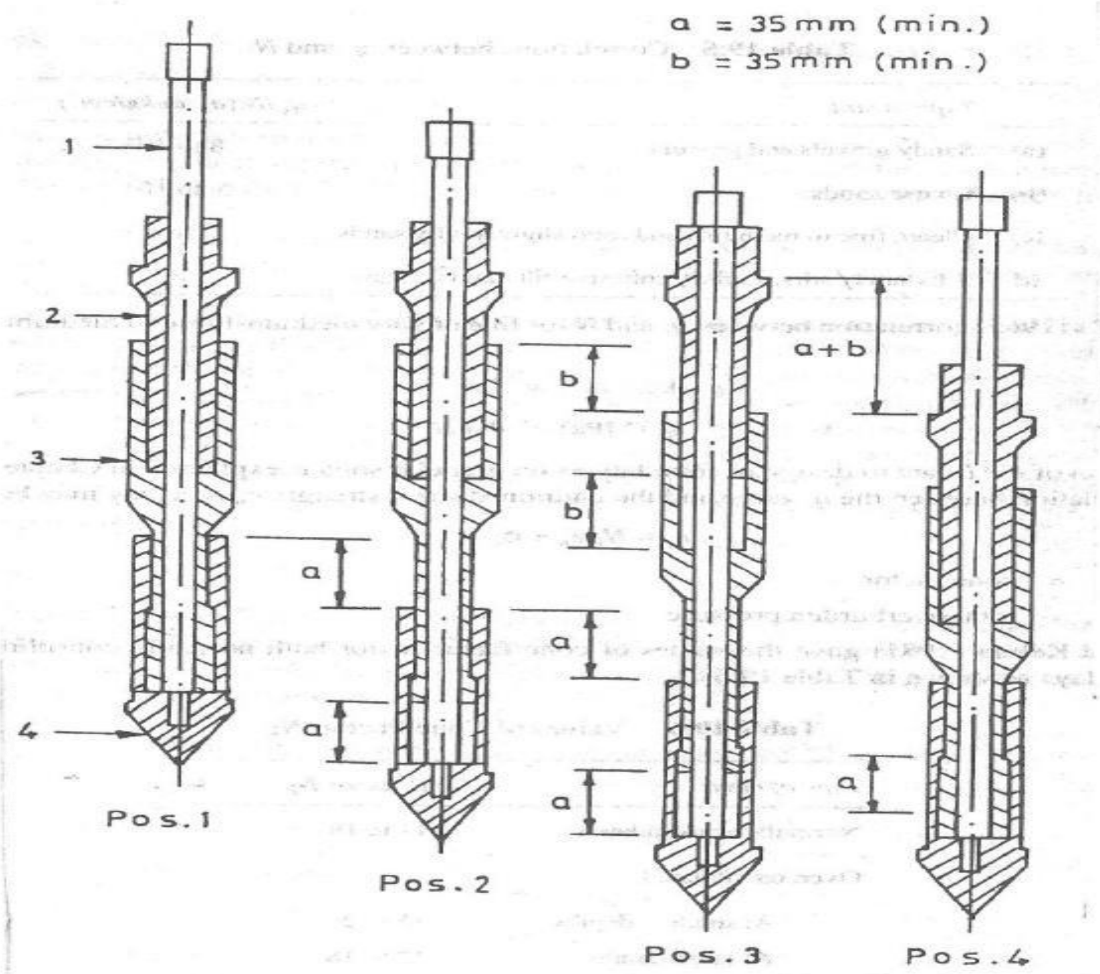


Fig. Sequence of operations of CPT