Tulsiramji Gaikwad-Patil College of Engineering and Technology
Wardha Road, Nagpur-441 108 NAAC
Accredited
Approved by AICTE, New Delhi, Govt. of Maharashtra & Affiliated to RTM Nagpur University, Nagpur

Department of Civil Engineering

Name of Subject: - Geotechnical Eng- II	**Subject Code:- BECVE504T**
Unit-I : Geotechnical Exploration	**Semester: - V**
Basic Introduction: Importance and objective of field exploration , geophysical methods and its limitations, methods of subsurface exploration, methods of boring, number, location and depth of boring, types of soil samples and samplers, principles of design of samplers, collection & shipments of samples	

Course Outcome (CO):-. Students able to grace the knowledge of different soil exploration techniques to ascertain the properties of soil and clay

Learning Outcomes (LOs) :- (4 to 5 are expected and as per the COs)

- To make students learn and apply basic objectives of geological exploration .
- To understand the basic fundamentals of soil mechanics
- To Remember the basic concept of soil structures interaction
- To utilize the basic lows to avoid the failure of slope and its stability
- To learn about compression and tension behavior of soil materials

UNIT 01: GEOLOGICAL EXPLORATION

IMPORTANCE AND OBJECTIVES OF FIELD EXPLORATION

The stability of the foundation of a building, a bridge, an embankment or any other structure built on soil depends on the strength and compressibility characteristics of the subsoil. The field and laboratory investigations required to obtain the essential information on the subsoil

Soil Exploration or Soil Investigation.
Site investigations consist of determining the profile of the natural soil deposits at the site, taking the soil samples and determining the engineering properties of the soils. It also includes in-situ testing of the soils. Soil exploration is a must in the present age for the design of foundations of any project .The extent of the exploration depends upon the magnitude and

importance of the project.Soil exploration involves broadly the following:

1. Information to determine the type of foundation required such as a shallow or deep foundation.
2. Necessary information with regards to the strength and compressibility characteristics of thesubsoil to allow the Design Consultant to make recommendations on the safe bearing pressure or pileload capacity.

Soil exploration involves broadly the following:
1. Planning of a program for soil exploration.
2. To determine bearing capacity of the soil.
3. To select the type and depth of foundation for a given structure.
4. TO investigate the safety of the existing structures and to suggest the remedial measures.
5. Collection of disturbed and undisturbed soil or rock samples from the holes drilled in the field. The number and depths of holes depend upon the project.
6. Conducting all the necessary in-situ tests for obtaining the strength and compressibility characteristics of the soil or rock directly or indirectly.
7. Study of ground-water conditions and collection of water samples for chemical analysis.
8. Geophysical exploration, if required.
9. Conducting all the necessary tests on the samples of soil /rock and water collected.
10. Preparation of drawings, charts, etc.
11. Analysis of the data collected.
12. Preparation of report.

1.2 PRINCIPAL METHODS OF SUBSURFACE EXPLORATION

The various methods of the explorations may be grouped as follows:-
1. Open excavations.
2. Borings.
3. Sub-surface soundings.
4. Geophysical methods.

OPEN EXCAVATIONS

In this method of exploration, an open excavation is made to inspect the sub-strata. The methods can be divided into two categories: (1) Pits and Trenches, (2) Drifts and Shafts.

1. Pits ad Trenches:- Pits and trenches are excavated at the site to inspect the strata. IS:4453-1967 recommends a clear working space of 1.2m x 1.2m at the bottom of the pit. Shallow pits up to adepth of 3m can be made without providing any lateral support. For deeper pits, especially below the ground water table, the lateral support in the form of sheeting and bracing system Fig.1 is required .Trenches are long shallow pits. As a trench is continuous over a considerable length, it provides exposure along a line. The trenches are more suitable than pits for exploration on slopes.

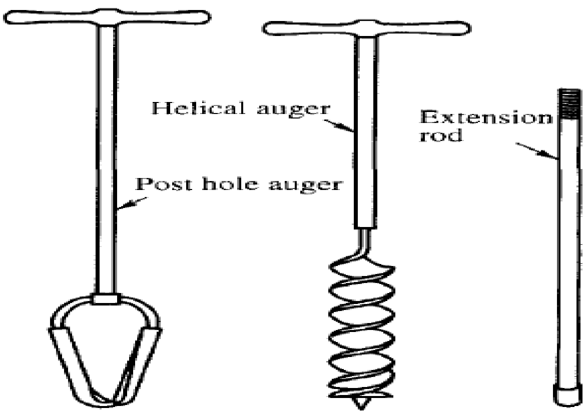
2. Drifts and Shafts:-

Drifts are horizontal tunnels made in the hill-side to determine the nature and structure of the geological formation. IS:4453-1980 recommends that a drift should have the minimum clear dimensions of 1.5m width and 2.0m height in hard rock. In soft rock, an arch roof is more advantageous than flat roof. Shafts are large size vertical holes in the geological formation. These may be rectangular or circular in section. The minimum width of a rectangular shaft is 2.4m and for a circular shaft, the minimum diameter is 2.4m.

BORINGS FOR EXPLORATION

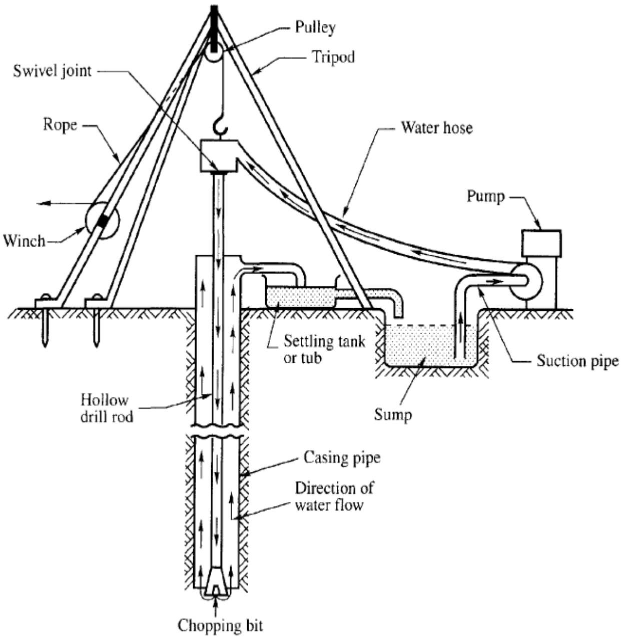
When the depth of exploration is large, borings are used for exploration. The vertical bore hole is drilled in the ground to get the information about the sub-soil strata. Samples are taken from the borehole and tested in a laboratory. Depending upon the type of soil and the purpose of boring, the following methods are used for drilling the holes.

1) Auger Boring: - Augers are used in cohesive and other soft soils above water table. Hand augers are used for depths up to about 6m. They are used in boring about 15 to 20cm in diameter. It attached to the lower end of a pipe about 18mmdiameter. The pipe is provided with a cross – arm at its top. The hole is advanced by turning the cross-arm manually and at the same time applying the rustin the downward direction. When the auger is filled with soil, it is taken out. Mechanically operate daugers are used for greater depths and they can also be used in gravely soils. Fig.2 shows a post hole auger and a helical (spiral) auger.Samples recovered from the soil brought up by augers are badly disturbed and are useful for identification purposes only. Auger boring is fairly satisfactory for highway explorations at shallow depths and for exploring borrow pits.



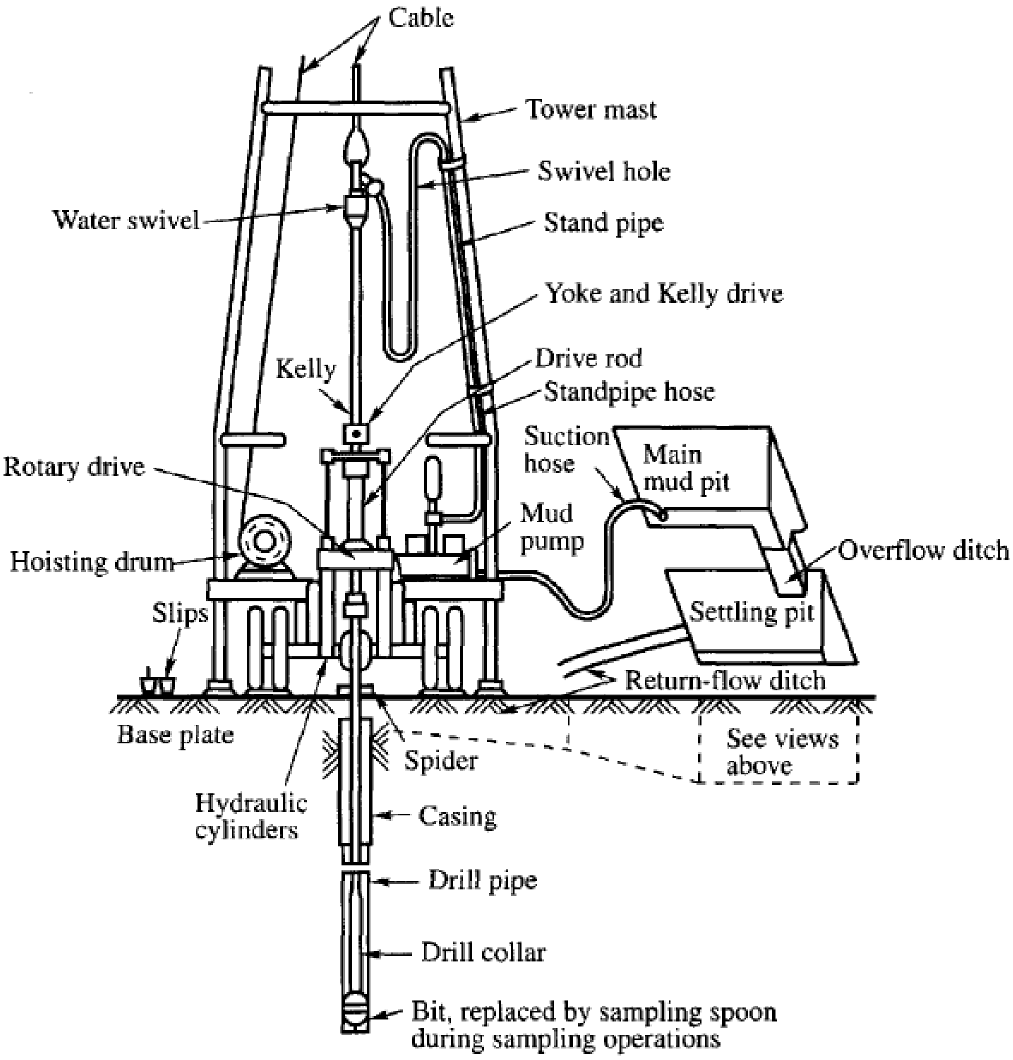
Wash boring: -

Soil exploration below the ground water table is usually very difficult to perform by means of pits or auger-holes. Wash boring in such cases is a very convenient method provided the soil is either sand, silt or clay. The method is not suitable if the soil is mixed with gravel or boulders. Fig.3 shows the assembly for a wash boring. To start with, the hole is advanced a short depth by auger and then a casing pipe is pushed to prevent the sides from caving in. The hole is then continued by the use of a chopping bit fixed at the end of a string of hollow drill rods. A stream of water under pressure is forced through the rod and the bit into the hole, which loosens the soil as the water flows up around the pipe. The loosened soil in suspension in water is discharged into a tube. The soil in suspension settles down in the tub and the clean water flows into a sump which is reused for circulation. The motive power for a wash boring is either mechanical or man power. The bit which is hollow is screwed to a string of hollow drill rods supported on a tripod by a rope or steel cable passing over a pulley and operated by a winch fixed on one of the legs of the tripod. The purpose of wash boring is to drill holes only and not to make use of the disturbed washed materials for analysis. Whenever an undisturbed sample is required at a particular depth, the boring is stopped ,and the chopping bit is replaced by a sampler. The sampler is pushed into the soil at the bottom of



Rotary Drilling: - In the rotary drilling method a cutter bit or a core barrel with a coring bit attached to the end of a string of drill rods is rotated by a power rig. The rotation of the cutting bit shears or chips the material penetrated and the material is washed out of the hole by a stream of water just as in the case of a wash boring. Rotary drilling is used primarily for penetrating the overburden between the levels of which samples are required. Coring bits, on the other hand, cut an annular hole around an intact core which enters the barrel and is retrieved. Thus the core barrel is used primarily in rocky strata to get rock samples. As the rods with the attached bit or barrel are rotated, a downward pressure is applied to the drill string to obtain penetration, and drilling fluid under pressure is introduced into the bottom of the hole through the hollow drill rods and the passages in the bit or barrel. The drilling fluid serves the dual function of cooling the bit as it enters the hole and removing the cuttings from the bottom of the hole as it returns to the surface in the annular space between the drill rods and the walls of the hole. In an uncased hole, the drilling fluid also serves to support the walls of the hole. When boring in soil, the drilling bit is removed and replaced by a sampler when sampling is required, but in rocky strata the coring bit is used to obtain continuous rock samples. The rotary drilling rig of the type given in Fig. 4 can also be used for wash

Coring Bits: - Three basic categories of bits are in use. They are diamond, carbide insert, and saw tooth. Diamond coring bits are the most versatile of all the coring bits since they produce high quality cores in rock materials ranging from soft to extremely hard. Carbide Bits are used to core soft to medium hard rock. They are less expensive than diamond bits but the rate of drilling is slower than with diamond bits. In saw-tooth bits, the cutting edge comprises a series of teeth. The teeth are faced and tipped with a hard metal alloy such as tungsten carbide to provide wear resistance and thereby increase the life of the bit. These bits are less expensive but normally used to core over burden soil and very soft rocks only.



TYPES OF SOIL SAMPLES

Soil samples are obtained during sub-surface exploration to determine the engineering properties of the soils and rocks. Soil samples are generally classified into two categories :

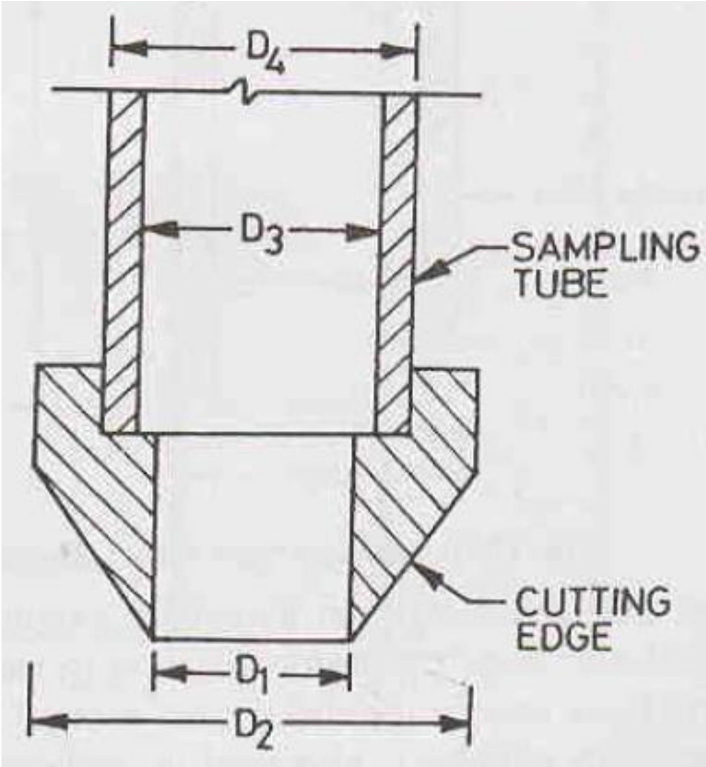
1) Disturbed samples:- These are the samples in which the natural structure of the soil gets disturbed during sampling. However, these samples represent the composition and the miner content of the soil. Disturbed samples can be used to determine the index properties of the soil ,such as grain size, plasticity characteristics, and specific gravity.

2) Undisturbed samples:- These are the samples in which the natural structure of the soil and the water content are retained. However, it may be mentioned that it is impossible to get truly undisturbed sample. Some disturbance is inevitable during sampling, even when the utmost carries taken. Even the removal of the sample from the ground produces a change in the stresses and causes disturbances .Undisturbed samples are used for determining the engineering properties of the soil, such as compressibility, shear strength, and permeability. Some index properties such as shrinkage limit canalso be determined. The smaller the disturbance, the greater would be the reliability of the results.

SOIL SAMPLES AND SAMPLES

DESIGN FEATURES AFFECTING THE SAMPLE DISTURBANCE

The soil samples can be of two types: disturbed and undisturbed. A *disturbed* sample is that in which the natural structure of soils get partly or fully modified and destroyed, although with suitable precautions the natural water content may be preserved. Such a sample should, however, be *representative* of the natural soil by maintaining the original proportion of the various soil particle sintact . An *undisturbed* sample is that in which the natural structure and properties remain preserved.



SCRAPER BUCKET SAMPLER

1. If a sandy deposit contains pebbles, it is not possible to obtain samples by standard split-spoon sampler or split-spoon sampler fitted with a spring core catcher. The pebbles come in between the springs and prevent their closure. For such deposits, a scraper bucket sampler can be used .A scraper bucket sampler consists of a driving point which is attached to its bottom end. There is a vertical slit in the upper portion of the sampler. As the sampler is rotated, the scrapings of the soil enter the sampler through the slit.

3. When the sampler is filled with the scrapings, it is lifted. Although the sample is quite disturbed, it is still representative.

4. A scraper bucket sampler can also be used for obtaining the samples of cohesion less soils belowthe water table.

DEPTH OF FOUNDATION

The depth of foundation of exploration required at a particular site depends upon the degree of variation of the subsurface data in the horizontal and vertical directions. It is not possible to fix the number, disposition and depth of borings without making a few preliminary borings or soundings at the site. The depth upto which the stress increment due to superimposed loads can produce significant settlement and shear stresses is known as the significant depth. The significant depth is generally taken as the depth at which the vertical stress is 20% of the load intensity.

The depth of exploration should be about 1.5 times width of the square footing.

2) The depth of exploration should be about 3.0 times width of the strip footing.

3) If the footings are closely spaced, the whole of the loaded area acts as a raft foundation. In that case, the depth of boring should be at least 1.5 times the width of the entire loaded area.

4) In pile foundation, the depth of exploration below the tip of bearing piles is kept at least 1.5times the width of the pile group.

5) In friction piles, the depth of exploration is taken 1.5 times the width of the pile group

RECONNAISSANCE

The geotechnical engineer makes a visit to the site for a careful visual inspection in reconnaissance. The information about the following features is obtained in reconnaissance.
1) The general topography of the site, the existence of drainage, ditches and dumps of debris and sanitary fills.
2) Existence of settlement cracks in the structure already built near the site.
3) The evidence of landslides, creep of slopes and the shrinkage cracks.
4) The stratification of soils as observed from deep cuts near the site.
5) The location of high flood marks on the nearby building and bridges.
6) The depth of ground water table as observed in the well.
7) Existence of springs, swamps, etc. at the site.
8) The drainage pattern existing at the site.
9) Type of vegetation existing at the site. The type of vegetation gives a clue to the nature of the soil.

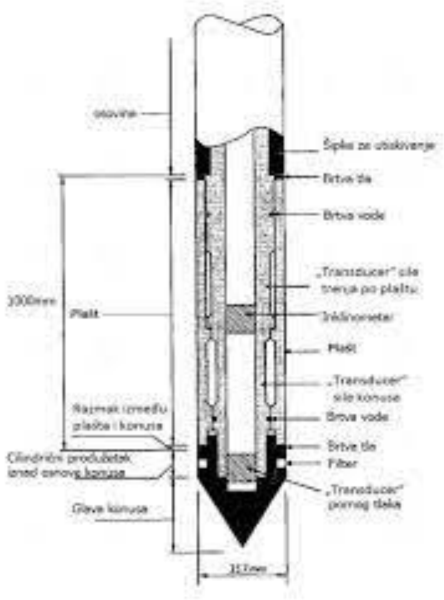
STANDARD PENETRATION TEST

1) The standard penetration test is the most commonly used in-situ test, especially for cohesionless soils which cannot be easily sampled.
2) The test is extremely useful for determining the relative density and the angle of shearing resistance of cohesionless soils.
3) It can also be used to determine the unconfined compressive strength of cohesive soils.
The test is conducted in a bore hole using a standard split-spoon sampler. When the bore hole has been drilled to the desired depth, the drilling tools are removed and the sampler is lowered to the bottom of the hole. The sampler is driven into the soil by a drop hemmer of 63.5 kg mass falling through a height of 750mm at the rate of 30 blows per minute (IS: 2131-1963). The number of hammer blows required to drive 150mm of the sample is counted. The sampler is further driven by 150mm and the number of blows recorded. Likewise, the sampler is once again further driven by 150mm and the number of blows recorded.
The number of blows recorded for the first150mm is disregarded. The number of blows recorded for the last two 150mm intervals are added to give the standard penetration number (N). In other words, the standard penetration number is equal to the number of blows required for 300mm of penetration beyond a seating drive of 150mm.If the number of blows for 150mm drive exceeds 50, it is taken as refusal and the test is discontinued. The standard penetration number is corrected for dil atancy correction and overburden
correction:-

a) ***Dilatancy Correction: -*** Silty

STATIC CONE PENETRATION TEST

The Dutch cone has an angle of 60^0 and an overall diameter of 35.7 mm, giving an end area of 10 cm^2. For obtaining the cone resistance, the cone is pushed downward at a steady rate of 10mm/sec through a depth of 35mm each time. The cone is pushed by applying thrust and not by driving. After the cone resistance has been determined, the cone is withdrawn. The sleeve is pushe don to the cone and both are driven together into the soil and the combined resistance is also determined.



IN – SITU VANE SHEAR TEST

1) In – situ vane – shear test is conducted to determine the shear strength of a cohesive soil in its natural condition.
2) The apparatus is similar to one used in laboratory. It consists of four blades, 100 mm (or 150mm or 200 mm long), attached at right angles to a steel rod.
3) The steel rod has a torque – measuring device at its top.
4) The height – diameter ratio (H/D) of the apparatus is generally equal to 2.
5) For conducting the test, the shear – vane is pushed into the ground at the bottom of the borehole. When a torque is applied through the handle at the top of the rod, the soil is sheared along a cylindrical surface.
6) The torque required to

GEOPHYSICAL METHODS

A number of geophysical methods are used in preliminary investigations of sub – soil strata. The methods can be used for the location of different strata and for a rapid evaluation of the subsoilcharacteristics. The geophysical methods can be broadly divided into the two categories: Seismic methods and Electrical resistivity methods.

1) Seismic Method: - The seismic methods are based on the principle that the elastic shock waves have different velocities in different materials. Seismic methods of subsurface explorations generally utilize the refracted waves.
i) The shock wave is created by a hammer blow or by a small explosive charge at a point P.
. The shock wave travels through the top layer of the soil (or rock) with a velocity V_1, depending upon the type of material in layer – I. The observation of the first arrival of the waves is recorded by geophones located at various points, such as A, B, C.
ii) The geophones convert the ground vibration into electrical impulses and transmit them to a recording apparatus.
iii) It is assumed that $V_3 > V_2 > V_1$ in Fig.16. At geophones located close to the point of impact,
such as point A, the direct waves with velocity V_1 reach first.

Limitation of the seismic methods: -

1) The methods cannot be used if a hard layer with a greater seismic velocity overlies a softer layer with a smaller seismic velocity.
2) The methods cannot be used for the areas covered by concrete, asphalt pavements or any

other artificial hard crust, having a high seismic velocity.
3) If the area contains some underground features, such as buried conduits, irregularly dipping strata, and irregular water table, the interpretation of the results becomes very difficult.
4) If the surface layer is frozen, the method cannot be successfully used, as it corresponds to a case of harder layer overlying a softer layer.
5) The methods require sophisticated and costly equipment.
6) For proper interpretations of the seismic survey results, the services of an expert are required.
Electrical resistivity methods:- The electrical resistivity (ρ) of a conductor is expressed aswhere R = electrical resistance (ohms), A = area of cross-section of the conductor (cm^2),L = length of conductor (cm), ρ = electrical resistivity (ohm-centimeter).

The resistivity of a material depends upon the type of material, its water content and the concentration of dissolved ions and many other factors. Rocks and dry soils have a greater resistivity than saturated clays.The method is also known as the resistivity mapping method. Four electrodes are used at a
constant spacing a. To conduct the test, four electrodes, which are usually in the form of metal spikes, are driven into the ground. The two outer electrodes are known as current electrodes. The two inner electrodes are called potential electrodes. The mean resistivity of the strata is determined by applying a D.C. current to the outer electrodes and by measuring the voltage drop between the inner electrodes. A current of 50 to 100 milliamp is usually supplied.

Summery

Site investigations consist of determining the profile of the natural soil deposits at the site, taking the soil samples and determining the engineering properties of the soils. It also includes in-situ testing of the soils. Soil exploration is a must in the present age for the design of foundations of any Project
The extent of the exploration depends upon the magnitude and importance of the project. Soil exploration involves broadly the following:
The depth of foundation of exploration required at a particular site depends upon the degree of variation of the subsurface data in the horizontal and vertical directions. It is not possible to fix the number, disposition and depth of borings without making a few preliminary borings or soundings at the site. The depth up to which the stress increment due to superimposed loads can produce significant settlement and shear stresses is known as the significant depth. The significant depth is generally taken as the depth at which the vertical stress is 20% of the load intensity.

Exercise
1 Explain the objective of Geotechnical exploration
2 Explain the various types of Boring
3 What is the working principle of Electric resistivity method
4 Define the profile of the soil

Name of Subject: - Geotechnical Engg-II	**Subject Code:- BECVE504T**
Unit-II : Stability of Slopes	**Semester: - V**
Stability of Slopes : Causes and types of slope failure, stability analysis of infinite slopes, Taylor's stability numbers & stability charts, stability analysis of finite slope for purely C- soils and C - ϕ soils, center of critical slip circle, (Swedish circle method), slices method for homogeneous C - ϕ soil slopes with pore pressure consideration, Friction circle method, method of improving stability of slopes;	

Course Outcome (CO):-. Students able to understand the basic objective of stability of soil

Learning Outcomes (LOs) :- (4 to 5 are expected and as per the COs)

- To understand the basic norms of stability of slope
- To learn and apply the remedies of various causes of slope failure
- To Remember the basic concept of cracks
- To utilize the basic lows to avoid the failure of slope and its stability
- To learn about friction circle method

Stability of Slopes

Earth embankments are commonly required for railways, roadways, earth dam, levees and river training works. The Stability of these embankments or slopes as they are commonly called should be very thoroughly analyzed since their failure may lead to loss of human life as well as colossal economic loss.

The failure of a mass of soil located beneath slopes is called a slide. It involves a downward and outward movement of the entire mass of soil that participates in the failure. The failure of slopes takes place mainly due to *(i)* the gravitational forces, and (ii) seepage forces within the soil. They may also fail due to excavation or undercutting of its foot, or due to gradual disintegration of the structure of the soil. Slides may occur in almost every conceivable manner, slowly or suddenly, and with or without any apparent provocation.

An analysis of stability of slope consists of two parts:

1) The determination of the most severely stressed internal surface and the magnitude of the shearing stress to which it is subjected
2) The determination of the shearing strength along this surface.

The shearing stress to which any slopes can be subjected depends upon the unit weight of the material and the geometry of the slopes. While shearing strength which can be mobilized to resist the shearing stress depends on the character of the soil, its density and drainage condition.

Slopes may be two types: infinite slopes and finite slopes.

CAUSES AND TYPES OF SLOPE FAILURE

Slope Stability: Slope stability is an extremely important consideration in the design and construction of earth dams. The stability of a natural slope is also important. The results of a slope failure can often be catastrophic, involving the loss of considerable property and many lives.

Causes of Failure of Slopes:

The important factors that cause instability in a slope and lead to failure are:-

1. Gravitational force
2. Force due to seepage water
3. Erosion of the surface of slopes due to flowing water
4. The sudden lowering of water adjacent to a slope
5. Forces due to earthquakes.

The effect of all the forces listed above is to cause movement of soil from high points to low points. The most important of such forces is the component of gravity that acts in the direction of probable motion

TYPES OF SLOPE FAILURE

A Slope may have any one of the following types of failures.

1) Rotational failure:-

This type of failure occurs by rotation along a slip surface by downward and outward movement of the soil mass. The slip surface is generally circular for homogeneous soil conditions and non-circular in case of non-homogeneous conditions. Rotational slips are further divided into 3 types.

i) Toe failure, in which the failure occurs along the surface that passes through the toe.

ii) Slope failure, in which the failure occurs along a surface that intersects the slope above the toe.

iii) Base failure, in which the failure surface passes below the toe.

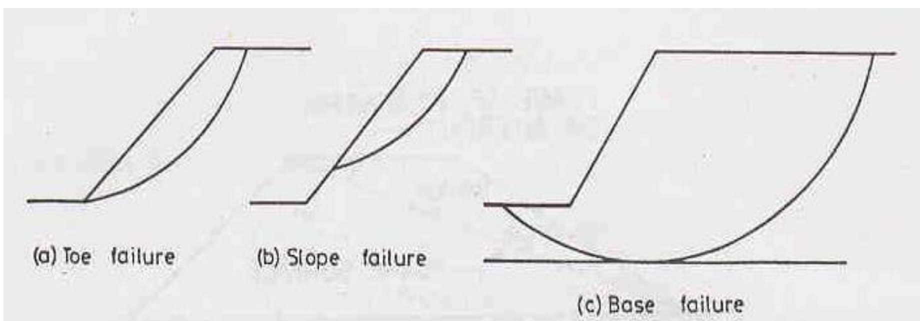


Figure.2 Rotational failure.

2) Translational failure:-

A constant slope of unlimited extent and having uniform soil properties at the same depth below the free surface is known as an infinite slope. It occurs in an infinite slope along a long failure surface parallel to the slope. Translational failures may occur along slopes of layered materials.

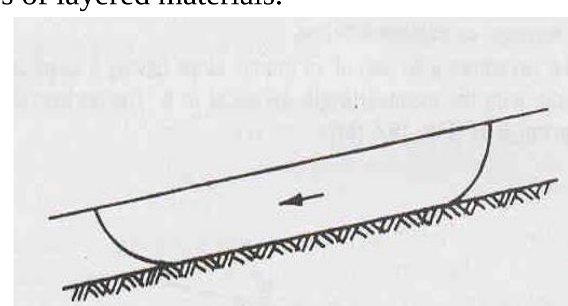


Figure.3 Translational failure.

Compound failure:-

A compound failure is a combination of the rotational slips and the translational slip. A compound failure surface is curved at the two ends and plane in the middle portion. A compound failure generally occurs when a hard stratum exists at considerable depth below the toe.

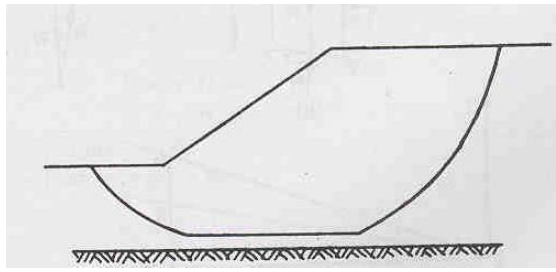


Figure.4 Compound failure.

3) Wedge failure:-

A failure along an inclined plane is known as plane failure or wedge failure or block failure. It occurs when distinct blocks and wedges of the soil mass become separated.

FRICTION-CIRCLE METHOD

Physical Concept of the Method

The principle of the method is explained with reference to the section through shown in below fig A trial circle with center of rotation O is shown in the figure. With center O and radius sin Φ’, where R is the radius of the trial circle, a circle is drawn.

The friction circle method of slope analysis is a convenient approach for both graphical and mathematical solutions. It is given this name because the characteristic assumption of the method refers to the Φ-circle.

The forces considered in the analysis are

1. The total weight W of the mass above the trial circle acting through the center of mass. The center of mass may be determined by any one of the known methods.

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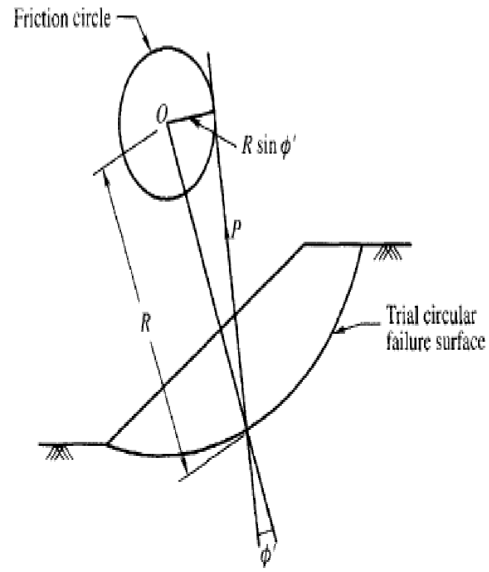


Figure. Principle of friction circle method

IMPROVING STABILITY OF SLOPES

The slopes which are susceptible to failure by sliding can be improved and made usable and safe. Various methods are used to stabilize the slopes. The methods generally involve one or more of the following measures, which either reduces the mass which may cause sliding or improve the shear strength of the soil in the failure zone.

1) Slope flattening reduces the weight of the mass tending to slide. It can be used wherever possible.
2) Providing a beam below the toe of the slope increases the resistance to movement. It is specially useful when there is a possibility of a base failure.
3) Drainage helps in reducing the seepage forces and hence increases the stability. The zone of subsurface water is lowered and infiltration of the surface water is prevented.

Densification by use of explosives, vibroflotation, or tera probe helps in increasing the shear strength of cohesionless soils and thus increasing the stability.

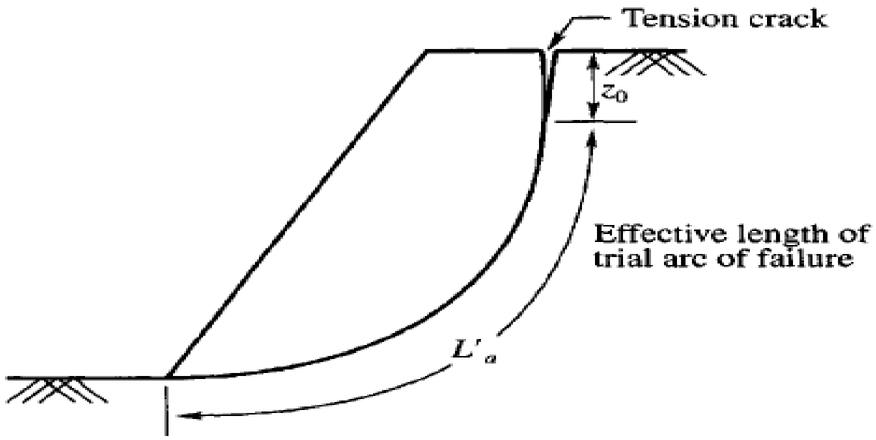
TENSION CRACKS

If a dam is built of cohesive soil, tension cracks are usually present at the crest. The depth of such cracks may be computed from the equation

$$z_0 = \frac{2c'}{\gamma}$$

where z_0 = depth of crack, c' = unit cohesion, y = unit weight of soil.

The effective length of any trial arc of failure is the difference between the total length of arc minus the depth of crack as shown in figure



Summery

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Densification by use of explosives, vibroflotation, or tera probe helps in increasing the shear strength of cohesion less soils and thus increasing the stability.

Exercise

1. How drainage helps in reducing the seepage
2. Define the tension crack
3. How stability of slope can be improved
4. Define the compound failure

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Department of Civil Engineering

Name of Subject: - Geotechnical Engg –II	**Subject Code:- BECVE504T**
Unit-III : Earth Pressure	**Semester: - V**
Earth Pressure : Earth pressure at rest, active and passive pressure; general & local states of plastic equilibrium in soil. Rankine's and Coulomb's theories of earth pressure. Effects of surcharge & submergence. Determination of Active earth pressure through graphical construction; Rebhann's and Culman's method	

Course Outcome (CO):-. Students able to understand the various failure

Learning Outcomes (LOs) :- (4 to 5 are expected and as per the COs)

- To understand the basic concept and techniques of soil failure
- To learn and apply the local states of plastic equilibrium
- To remember the effect of lateral earth pressure
- To learn about submerged backfill

LATERAL EARTH PRESSURE: Earth pressure at rest, active & passive pressure, General & local states of plastic equilibrium in soil. Rankines and Coulomb‟s theories for earth pressure. Effects of surcharge, submergence. Rebhann‟s criteria for active earth pressure. Graphical construction by Poncelet and Culman for simple cases of wall-soil system for active pressure condition

INTRODUCTION This is required in designs of various earth retaining structures like: -
i) Retaining walls
ii) Sheeting & bracings in cuts / excavations
iii) Bulkheads
iv) Bridge abutments, tunnels, cofferdams etc.

Lateral earth pressure depends on:-
i) Type of soil.

ii) Type of wall movement:-

a) Translatory

b) Rotational

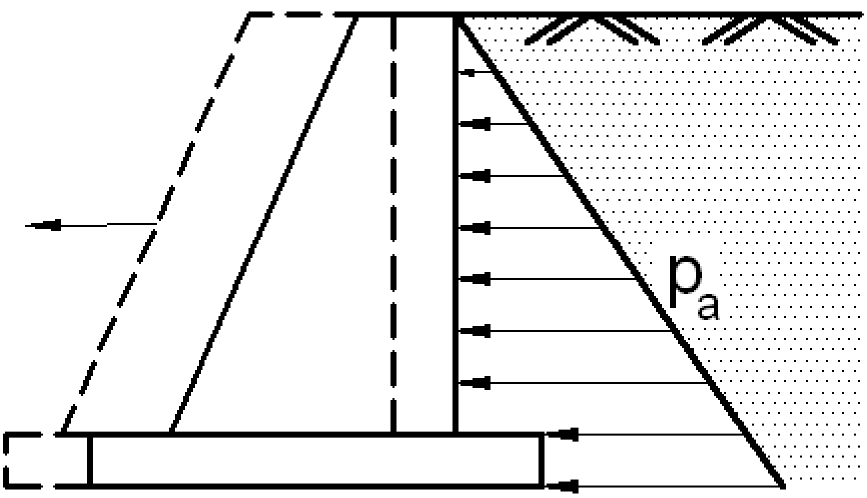
iii) Soil-structure interaction.

A retaining wall or retaining structure is used for maintaining the ground surface at different elevations on either side of it. The material retained or supported by the structure is called backfill which may have its top surface horizontal or inclined. The position of the backfill lying above a horizontal plane at the elevation of the top of a wall is called the surcharge, and its inclination to the horizontal is called surcharge angle β.

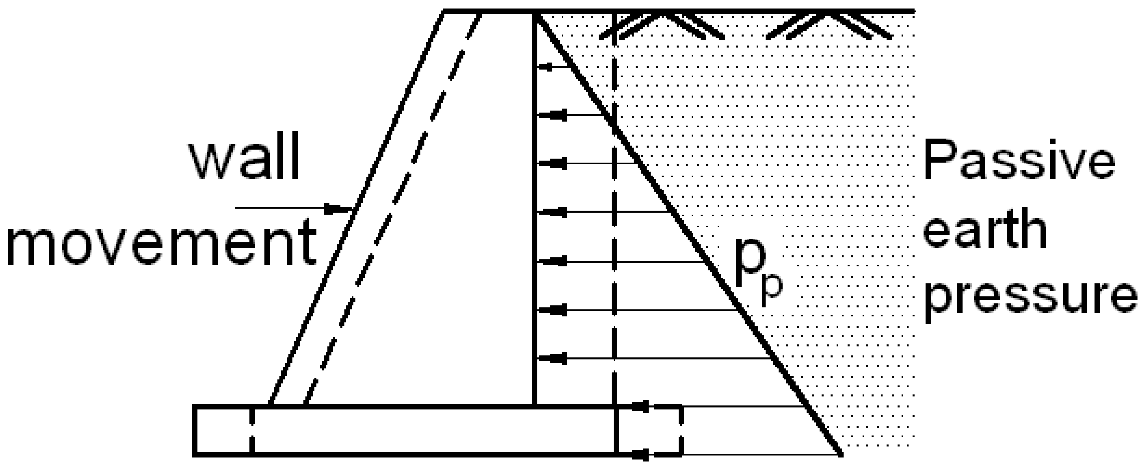
Lateral earth pressure can be grouped into 3 categories, depending upon the movement of the retaining wall with respect to the soil retained. The soil retained is also known as the backfill.
1) At-rest pressure: -

The lateral earth pressure is called at-rest pressure when the soil mass is not subjected to any lateral yielding or movement. This case occurs when the retaining wall is firmly fixed at its top and is not allowed to rotate or move laterally. Fig.1(a) shows the basement retaining walls which are restrained against the movement by the basement slab provided at their tops. Another example of the at-rest pressure is that of a bridge abutment wall which is restrained at its top by the bridge slab. The at-rest condition is also known as the elastic equilibrium, as no part of soil mass has failed and attained the plastic equilibrium.

Active earth pressure: - A state of active pressure occurs when the soil mass yields in such a way that it tends to stretch horizontally. It is a state of plastic equilibrium as the entire soil mass is on the verge of failure. A retaining wall when moves away from the backfill, there is a stretching of the soil mass and the active state of earth pressure exists. In Fig.2, the active pressure develops on the rigid hand side when the wall moves towards left.



Passive earth pressure: - A state of active pressure occurs when the soil mass yields in such a way that it tends to stretch horizontally. It is another extreme of the limiting equilibrium condition. In Fig.3, the passive pressure develops on the left-side of the wall below the ground level, as the soil in this zone is compressed when the movement of the wall is towards left. Another example of the passive earth pressure is the pressure acting on an anchor block.



EARTH PRESSURE AT REST

The earth pressure at rest, exerted on the back of a rigid, unyielding retaining structure, can be calculated using theory of elasticity, assuming the soil to the semi-infinite, homogeneous, elastic and isotropic. Consider an element of soil at a depth 'z' being acted upon by vertical stress σ_v and horizontal stress σ_h. There will be no shear stress. The lateral strain ϵ_h in the horizontal direction is given by:

$$\epsilon_h = \frac{1}{E}\,[\sigma_h - \mu\,(\sigma_h - \sigma_v)]$$

The earth pressure at rest corresponding to the condition of zero lateral strain ($\epsilon_h = 0$). Hence

$$\sigma_h = \mu\,(\sigma_h + \sigma_v)$$

$$\frac{\sigma_h}{\sigma_v} = K_0' = \frac{\mu}{1-\mu}$$

where K_v is the coefficient of the earth pressure at rest. Designating the lateral earth pressure (σ_h) at rest by p_0 and substituting $\sigma v = \gamma$ z, we have,

$$p_0 = K_0 \gamma z$$

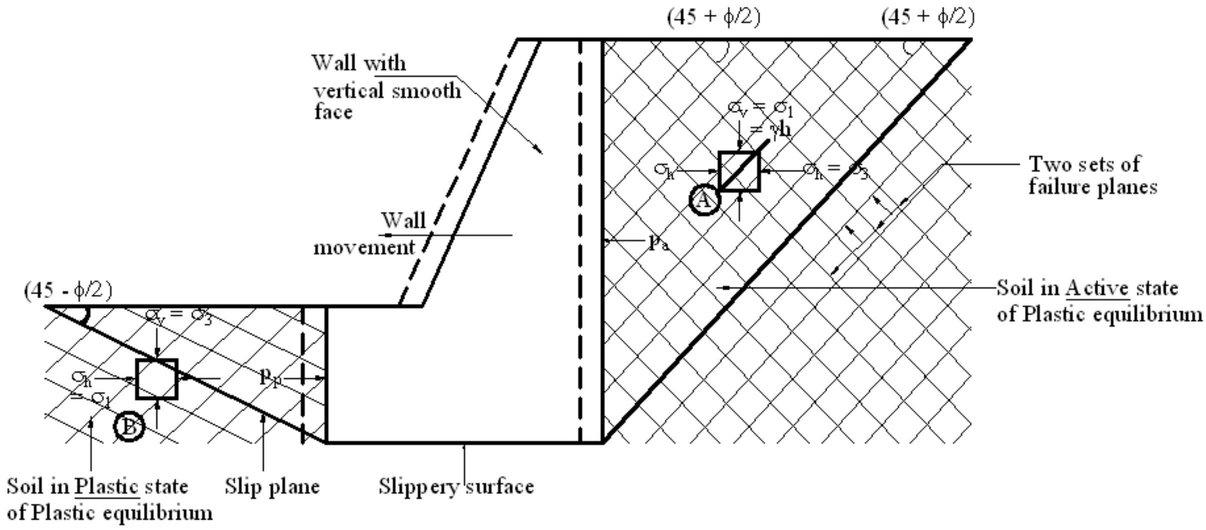
The pressure distribution diagram is thus triangular with zero intensity at z = 0 and an intensity of $K_0\gamma H$ at the base of the wall, where z = H. The total pressure P0 per unit length for the vertical height H is given by

$$P_0 = \int_0^H K_0 \gamma z.d_z = \frac{1}{2} K_0 \gamma H^2$$

The behavior of soil is not in accordance with the elastic theory and do not have a well-defined value of the Poisson's ratio.

S.No.	**Soil Type**	**K0**
1	Loose sand	0.4
2	Dense sand	0.6
3	Sand compacted in layers	0.8
4	Soft clay	0.6
5	Hard clay	0.5

PLASTIC EQUILIBRIUM CONDITIONS IN SOIL (ACTIVE & PASSIVE CASES): -



ACTIVE EARTH PRESSURE: RANKINES THEORY Rankine's theory of lateral earth pressure is applied to uniform cohesionless soil only. Following are the assumptions of the Rankine theory: -

1) The soil mass is semi-infinite, homogeneous, dry and cohesionless.

2) The ground surface is a plane which may be horizontal or inclined.

3) The back of the wall is vertical and smooth.

4) The wall yields about the base and thus satisfies the deformation condition for plastic deformation.

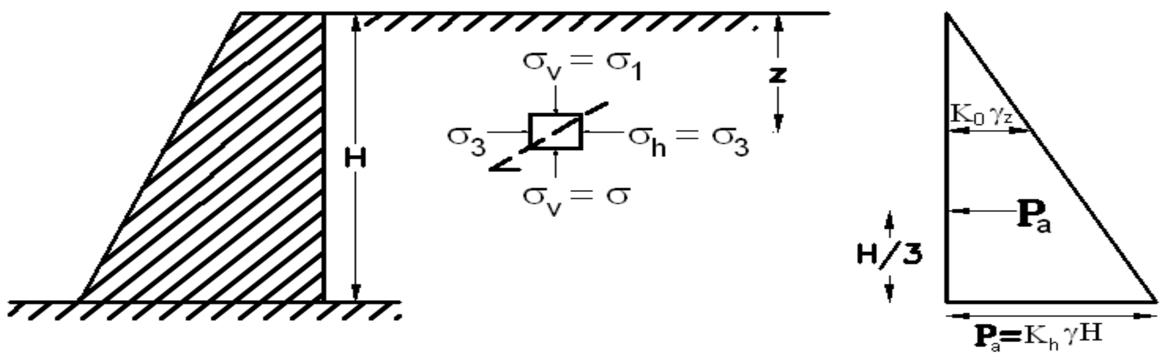
The following cases of cohesionless backfill will now be considered: -

1) Dry or moist backfill with no surcharge.

2) Submerged backfill.

3) Backfill with uniform surcharge.

4) Inclined back and surcharge.



(a)

(b)

Dry or moist backfill with no surcharge: -

Consider an element at a depth 'z' below the ground surface. When the wall is at the point of moving outwards (i.e., away from the fill), the active state of plastic equilibrium is established. The horizontal pressure σ_h is then the minimum principal stress σ_3 and the vertical pressure σ_v is the major principal stressσ_1. From the stress relationship, we have,

$$\sigma_1 = \sigma_3 \tan^2\left(45^0 + {}^{\emptyset}/_{2}\right)$$

$$\frac{\sigma_3}{\sigma_1} = \frac{\sigma_h}{\sigma_v} = \frac{1}{\tan^2\left(45^0 + {}^{\emptyset}/_{2}\right)}$$

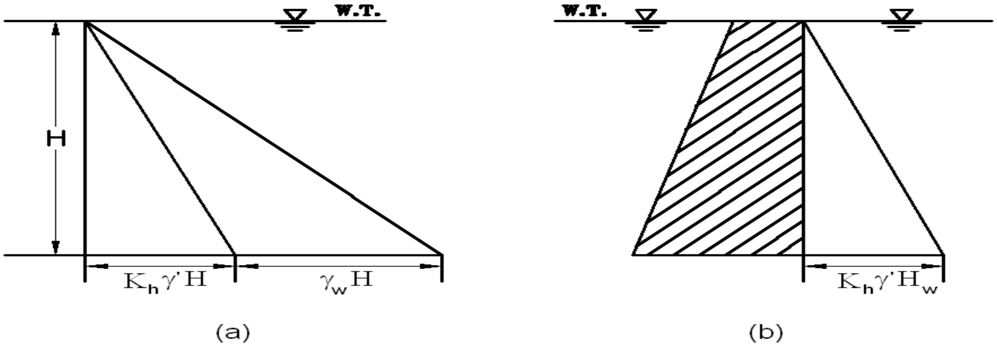
Submerged backfill: -

In this case, the sand fill behind the retaining wall is saturated with water. The lateral pressure is made up of two components:

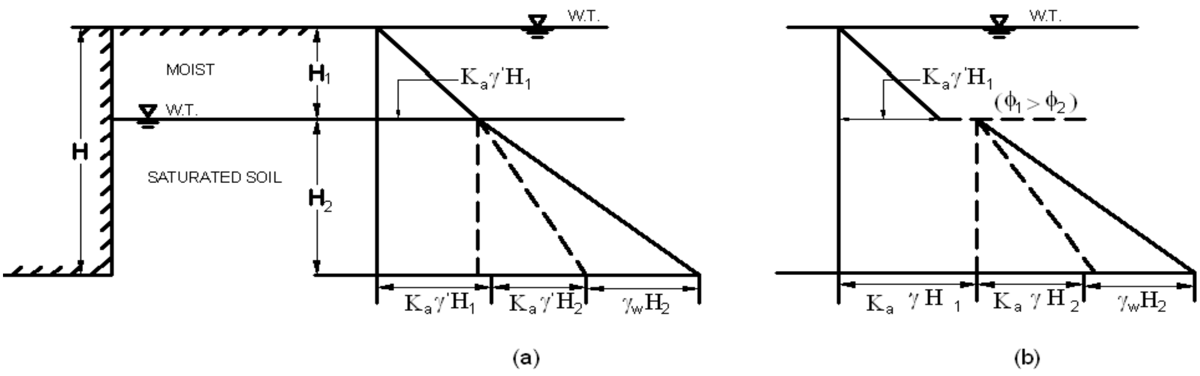
i) Lateral pressure due to submerged weight γ' of the soil, and

ii) Lateral pressure due to water. Thus, at any depth 'z' below the surface,

$$P_a = K_a\gamma'z + \gamma_w z$$



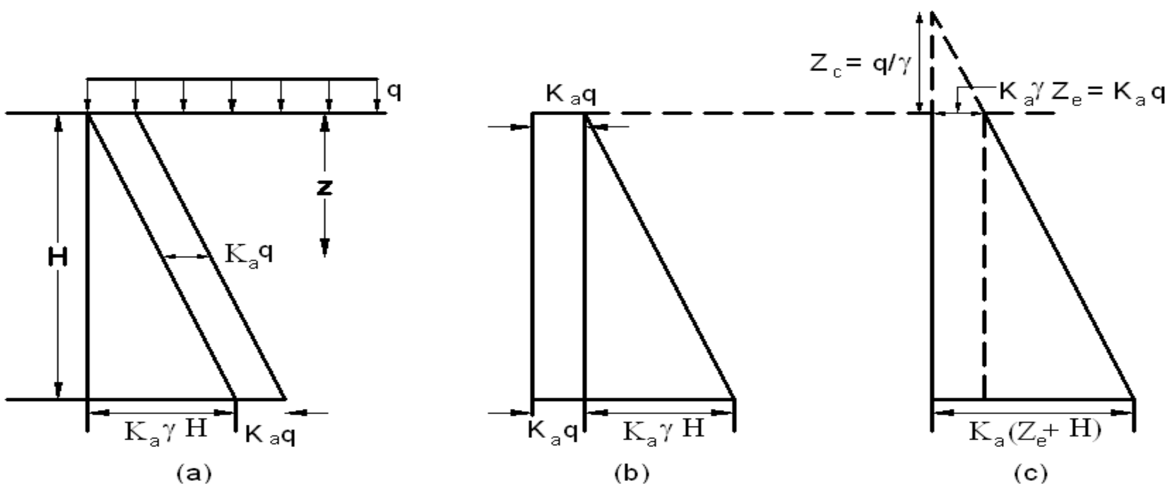
The pressure at the base of the retaining wall (z = H) is given by $P_a = K_a\gamma'H + \gamma_w H$ If the free water stands to both sides of the wall the water pressure need not be considered, and the net lateral pressure is given by, $P_a = K_a\gamma'H$ If the backfill is partly submerged, the lateral pressure intensity at the base of the wall is given by, $P_a = K_a\gamma H_1 + K_a\gamma' H_2 + \gamma_w H_2$



The lateral intensity at the base of wall is given by; $P_a = K_{a2}\gamma H_1 + \gamma' K_a \gamma' H_2 + \gamma_w H_2$

3) Backfill with uniform surcharge: -

If the backfill is horizontal and carries a surcharge of uniform intensity 'q' per unit area, the vertical pressure increment, at any depth 'z' will increase by 'q'. The increase in the lateral pressure due to this will be $K_a q$. Hence the lateral pressure at any depth 'z' is given by, $P_a = K_a\gamma z + K_a q$ At the base of the wall, the pressure intensity is $P_a = K_a\gamma z + K_a q$



The height of fill z_e, equivalent to the uniform surcharge intensity is given by the relation, $K_a\gamma z_e = K_a q$

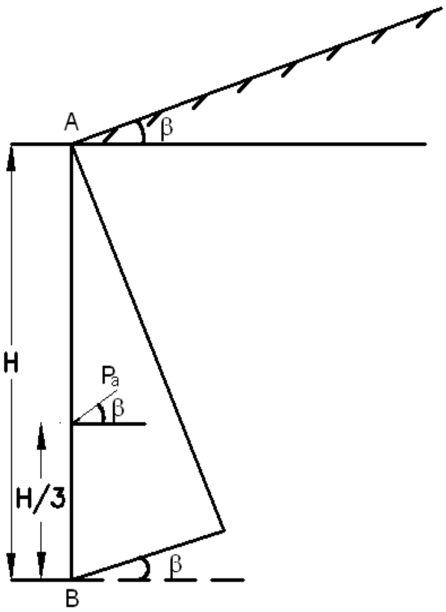
Backfill with sloping surface: -

The sloping surface behind the wall is inclined at an angle β with the horizontal; β is called the surcharge angle. The total active earth pressure P_a for the wall of height H is given by; $P_a = ½ K_a \gamma H^2$ The resultant acts H/3 above the base in direction parallel to the surface, as shown in fig.4

Fig.4

5) Inclined Back and surcharge: -

Fig.5 shows a retaining wall with an inclined back supporting a backfill with horizontal ground surface. The total active earth pressure P1 is first calculated on a vertical plane BC passing through the heel BC. The total pressure P is the resultant of the horizontal pressure P1 and the weight W of the wedge ABC:
where P_1



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where $P_1 = ½ K_a \gamma H_2$

$$P = \sqrt{P_1^2 + W^2}$$

RANKINE'S EARTH PRESSURE WHEN THE SURFACE IS INCLINED

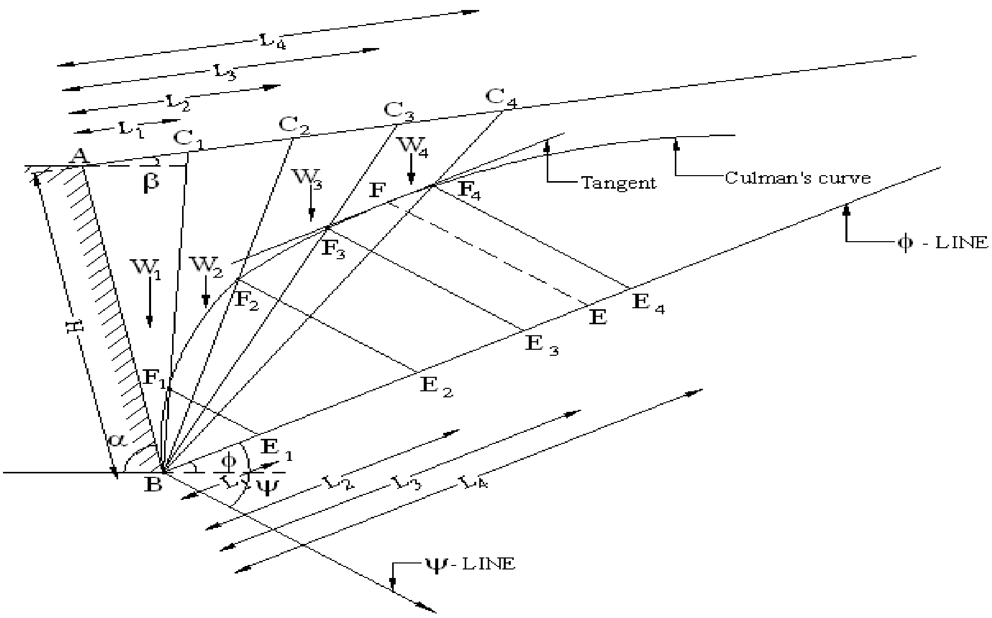
Two stresses are called conjugate stresses when the direction of one stress is parallel to the plane on which the other stress acts. Rankine assumed that the vertical stress on an element of the soil within the inclined backfill and the lateral stress on the vertical plane of the element are conjugate stresses. In other words, he assumed that the lateral stress is parallel to the inclined backfill.

Let us consider an element of soil at depth 'Z' below the soil surface inclined at angle 'i' to horizontal. The angle 'i' is known as the angle of surcharge. The intensity of vertical stress (□v) on the element is given by

The other conjugate stress is the lateral stress (□x). It may be mentioned that the vertical stress □v is not the principal stress, as a shear stress also exists on the inclined plane at the top of the element. Likewise, the lateral stress □x is also not a principal stress. A relationship between the lateral pressure and the vertical stress can be obtained for the active and passive cases as given below. **Active earth pressure:** Fig.6 shows the Mohr circle corresponding to the active limiting conditions

CULMANN'S CONSTUCTION FOR ACTIVE PRESSURE

Culmann (1866) developed a method which is more general than Rehbann's method. It can be used to determine Coulomb's earth pressure for ground surface of any configuration, for various types of surcharge loads and for layered back fills. Culmann's construction is, in fact the method of construction of the force triangle in a rotated oriented. The procedure consists of following steps:



Draw GL, □ □ □ □ line and ψ line.

2) Take a trial slip plane BC_1 and calculate the weight of the wedge ABC_1. Show it as BE_1 (to certain scale) on □ □ □ □ line.

3) Through E1 draw E_1F_1 parallel to the ψ – line to cut the slip plane at F_1.

4) Similarly, second trial plane BC_2. Repeat as above.

5) Take number of slip planes BC_3, BC_4 etc., plot the weight of the corresponding wedges on the ψ – line and obtain points F_1, F_2, F_3, F_4 etc.

COULOMB'S WEDGE THEORY Coulomb (1776) developed a method for the determination of the earth pressure in which he considered the equilibrium of the sliding wedge which is formed when the movement of the retaining wall takes place. The following assumptions are made: -
1) The backfill is dry, cohesionless, homogeneous, isotropic and ideally plastic material.

2) The slip surface is a plane surface which passes through the heel of the wall.

3) The wall surface is rough. The resultant earth pressure on the wall is inclined at an angle δ is the angle of the friction between the wall and the backfill.

4) The sliding wedge itself acts as a rigid body.

Summary

A retaining wall or retaining structure is used for maintaining the ground surface at different elevations on either side of it. The material retained or supported by the structure is called backfill which may have its top surface horizontal or inclined. The position of the backfill lying above a horizontal plane at the elevation of the top of a wall is called the surcharge, and its inclination to the horizontal is called surcharge angle β.

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Exercise
1 Explain passive and active earth pressure
2 Explain Rankings earth pressure theory
3 Explain moist backfill surcharge
4 Explain plastic equilibrium

Department of Civil Engineering

Name of Subject: - Geotechnical Engg –II	**Subject Code:- BECVE504T**
Unit-IV: Ground water improvement	**Semester: - V**
Need of ground improvement, ground improvement techniques, stabilization using lime, cement & fly ash; pre aloading concept, vibro compaction/flotation, concept of sand drains, stone columns, encased stone column, concept of NPVD (natural prefabricated vertical drain) and PPVD (polymer prefabricated vertical drain). Basic concept of reinforced soil, different types of Geo synthetics,	

Course Outcome (CO):-. Students able to understand the various technique of ground water improvement

Learning Outcomes (LOs) :- (4 to 5 are expected and as per the COs)

- To understand the basic concept and techniques of ground water Improvement
- To learn and apply the local states of plastic equilibrium
- To remember the coulomb theory of earth pressure
- To learn about submerged backfill

INTRODUCTION Improve or modify the properties of the soil either by excavating the poor quality soil and replacing it with soil having better engineering properties, or by in situ treatment without excavation – these processes are known as ground improvement. Ground improvement refer to the improvement in or modification to the engineering properties of a soil that are carried out at a site where the soil in its natural state does not possess properties that are acceptable to us for the proposed Civil Engineering activity.

What is to be improved? Engineers often ask for the following improvements in soil behavior:- 1) An increase in the bearing capacity. 2) A reduction in the amount of settlement and in the time, in which it occurs, 3) An increase in the capacity to retard seepage, 4) An acceleration in the rate at which drainage occurs, 5) Elimination of the possibility of liquefaction, 6) An increase in the stability of a slope or of a vertical cut or of an underground opening. To achieve the desired soil behavior, we have to alter soil properties, e.g. increase the shear strength, reduce the compressibility and increase or decrease the permeability.

METHOD OF SOIL STABILISATION Stabilisation in a broad sense incorporates the various methods employed for modifying the properties of a soil to improve its engineering performance. Stabilisation is used for a variety of engineering works, the most common application being in the construction of road and air – field pavements. Methods of stabilization may be grouped in two main types :- 1) Modification or improvement of a soil property of the existing soil without any admixture for e.g. Compaction and drainage, and, 2) Modification of the properties with the help of admixtures for e.g. mechanical stabilization. **USE OF ADMIXTURES (LIME, CEMENT, FLYASH) IN STABILISATION** The physical properties of soils can often economically be improved by the use of admixtures. Some of the more widely used admixtures include lime, portland cement and asphalt. The process of soil stabilization first involves mixing with the soil a suitable additive which changes its property and then compacting the admixture suitably. This method is applicable only for soils in shallow foundations or the base courses of roads, airfield pavements, etc.

1) Lime stabilization :-

The type of limes commonly used to stabilize fine – grained soils are hydrated high calcium lime [$Ca(OH)_2$], calcium quicklime (CaO), monohydrated dolomitic lime [$Ca(OH)_2.MgO$], and dolomitic quicklime. Lime stabilization in the field can be done in three ways:-

1) The in – situ material can be mixed with the proper amount of lime at the site and then compacted after the addition of moisture.

2) The soil can be mixed with the proper amount of lime and water at a plant and then hauled

Cement stabilization :-

Soil-cement is the reaction product of an intimate mixture of pulverized soil and measured amounts of portland cement and water, compacted to high density. As the cement hydrates, the mixture becomes a hard, durable structural material. Hardened soil-cement has the capacity to bridge over local weak points in a subgrade.

When properly made, it does not soften when exposed to wetting and drying, or freezing and thawing cycles. Portland cement and soil mixed at the proper moisture content has been used increasingly in recent years to stabilize soils in special situations. Probably the main use has been to build stabilized bases under concrete pavements for highways and airfields. Soil cement mixtures are also used to provide wave protection on earth dams. There are three categories of soil-cement they are: 1. Normal soil-cement usually contains 5 to 14 percent cement by weight and is used generally for stabilizing low plasticity soils and sandy soils. 2. Plastic soil-cement has enough water to produce a wet consistency similar to mortar. This material is suitable for use as water proof canal linings and for erosion protection on steep slopes where road building equipment may not be used. 3. Cement-modified soil is a mix that generally contains less than 5 percent cement by volume. This forms a less rigid system than either of the other types, but improves the engineering properties of the soil and reduces the ability of the soil to expand by drawing in water. The cement requirement depends on the gradation of the soil. A well graded soil containing gravel, coarse sand and fine sand with or without small amounts of silt or clay will require 5 percent or less cement by weight. Poorly graded sands with minimal amount of silt will require about 9 percent by weight. The remaining sandy soils will generally require 7 percent. Non-plastic or moderately plastic silty soils generally require about 10 percent, and plastic clay soils require 13 percent or more.

3) Cement

BASIC CONCEPT OF REIFORCED EARTH Reinforced earth consists of a compacted soil mass within reinforcing elements or membranes, usually in the form of horizontal strips of metal (such as galvanized steel, stainless steel or aluminium alloys), rods of metals, wire grids, fibre glass strips/rods, bamboos or geotextiles, are embedded. The main application of the reinforced earth technique is the reinforced wall. A reinforced earth wall consists of three components :-

i) Wall facing elements:- The wall facing elements are provided at the free boundary of a reinforced earth structure, to provide some form of barrier so that the soil mass is contained. These elements may either be flexible, or stiff, it should be strong enough to hold back the soil and should allow fastening to attach reinforcing elements. These are made from steel, aluminium, plastic, fibre, or reinforced concrete.

ii) Reinforcing elements/membranes:- The reinforcing elements consists of any or the following:-

(i) Galvanised steel strips

(ii) Rods of galvanized steel

(iii)Strips or rods of other metal such as stainless steel, aluminium

(iv) Galvanised iron grids,

(v) Fibre glass strips

(vi) Glass-Fibre reinforced plastic (GRP),

(vii) Geosynthetic reinforcements such as geotextiles, geomembranes, geogrids, geostrips, geocomposits, etc.

(viii) Bamboos.

iii) Compacted backfill:- The soil for the backfill should be predominantly coarse-grained and it has been proposed that not more than 10% of the particles should pass the 63micron sieve. The first layer of reinforcement strips is placed at the level ground surface and backingfilling is done with granular soil, compacting it in the processes of laying.

ADVANTAGED OF REINFORCED EARTH STRUCTURES The following are advantages of reinforced earth structures:-

(i) Reinforced earth structures are quite flexible. Hence these can withstand foundation deformations/settlements.

(ii) Reinforced earth structure being flexible, can withstand earth-quake forces more efficiently than conventional rigid structures.

(iii) Reinforced earth structures are much more economical in comparison to the conventional structures of masonry or concrete.

(iv) Reinforced structural elements can be transported easily. Hence these can be constructed speedily.

(v) Reinforced earth structures can also be constructed in stages.

(vi) The reinforcing elements used for such structures are easily available in various sizes and shapes. They can be easily stored, handled and placed during construction.

Geotextiles are permeable or porous fabrics, made from synthetic materials that are used with geotechnical material (such as soil or rock) as an integral part of a man made product, structure or system. As per ASTM, geotextiles are permeable sheets of synthetic fibres like polyester, polypropylene, polamide (nylon), viscose etc. where geomembranes are imperbeable sheets or films made from a polymer and which may be reinforced with textile. **Salient Features of geo-synthetics:-**

i) Geotextiles and geostrips:- These include wovwn and non-woven geotextiles used for drainage, stabilization and reinforcement function. Geostrips are in the form of cut fabric or long strips of geotextiles. They are produced from polypropylene and high density polyethylene. They can be connected with anchors at the ends.

ii) Geogrids:- These include extruded, woven, flexible and stiff types of geogrids used for stabilization and reinforcement.

iii) Geomembranes:- These include HDPE, PVC, CSPE, PP liners etc, and are basically impervious.

iv) Geonets:- These include LDPE ans HDPE nets and have functions similar to geogrids.

v) Geocells or Geoweb members:- These include multicoloured HDPE cells of varying heights, used for stabilization applications. They are made from prefabricated polymetric systems.

vi) Geofoam :- These include polystyrene sheets of varying dimensions used for light weight fills and other applications.

vii) Geosynthetic clay liners:- These include geotextile/clay/geotextile composites.

Applications of Geotextiles:-

1) Geotextiles are made either from the natural fibres or from synthetic materials. The bio-degradable nature of natural fibres have restricted their use to some specific applications whereas synthetic materials have made their way for wide applications.

2) Another advantage of using these fibres for geotextiles is related to their ability to be engineered chemically, physically to suit particular geotechnical engineering applications.

3) Ease and speed of construction.

4) Ability ti withstand differential settlement.

5) Suitability for phased construction at restricted sites.

6) Ability to provide viable solution to exceptionally difficult, otherwise, intractable construction problems.

FUNCTIONS OF VARIOUS GEOSYNTHETIC MATERIALS

1) Fluid transmission and drainage function:- A geotextile provides fluid transmission when it collects a liquid or gas and conveys it within its own plane, towards an outlet.Drainage is related to the role of filtration and is a function of the permeability of a geotextile and its pore opening size or porometry. The permeability involved in the fluid transmission function is the permeability in the plane of the geotextile.

2) Filtration function:- A geotextile acts as a filter when it allows liquid to pass normal to its own plane while preventing most soil particles from being carried away by liquid current. Two cases can be considered :- (i) Ageotextile, placed across a flow pf liquid carrying fine particles, stops most of the particles (where they accumulate on the filter) while allowing water to pass through, and (ii) a geotextile, placed in contact with a soil, allows water seeping from the soil to pass through, while preventing any movement of soil particles. The two important properties required in a geotextile to work as a filter are () permeability across its own plane and (ii) porometry.

3) Separation function:- A geotextile acts as a separator when placed between a fine soil and a coarse material such as gravel or stone ballast. It prevents the fine soil and the coarse material from inter-mixing under the action of reapeated applied loads. The key factors for a geotextile to satisfy this function are (i)porometry, (ii)toughness and(iii)strength.

4) Protection function:- A geotextile protects a material when it distributes stresses and strains transmitted to the protected material. Two cases can be considered:-

(a) Surface protection:- A geotextile, placed on the soil, prevents one of being damaged by such action as weather, light traffic, surface water flow. Etc.

(b) Interface protection:- A geotextile, placed between two materials prevents one of the materials from being damaged by concentrated stresses applied by (or strains imposed by) the other material.

5) Reinforcement function:- Reinforced earth:- Since the tensile strength of soil is practically nil, geotextiles which are good in tensile strength, can contribute to the load carrying capacity of soil.A geotextile can work as reinforcement in two ways:-

(a) As tensile member:- A geotextile acts as a tensile member when it provides tensile modulus and tensile strength to a soil with which it is interacting through interface shear strength, i.e. friction, cohesion-adhesion and/or interlocking between geotextile and soil.

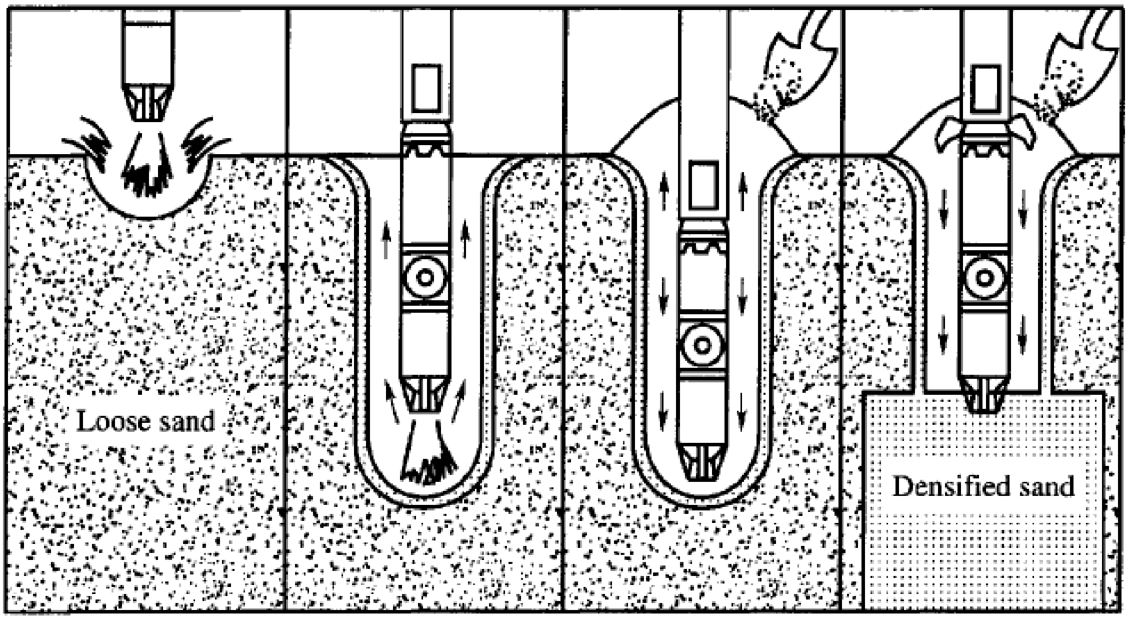
(b) As tensioned membrane:- A geotextile function as a tensioned membrane when it is placed between two materials two materials having different pressures, and its tension balances the pressure difference between two materials, thus strengthening the structure.

VIBROFLOTATION Vibro-compaction, sometimes referred to as Vibrofloation, is the rearrangement of soil particles into a denser configuration by the use of powerful depth vibration**.** Vibrocompaction is a ground improvement process for densifying loose sands to create stable foundation soils. The principle behind vibrocompaction is simple. The combined action of vibration and water saturation by jetting rearranges loose sand grains into a more compact state. Vibrocompaction is performed with specially-designed vibrating probes. Both horizontal and vertical modes of vibration have been

used in the past. The vibrators used by TerraSystems consist of torpedo-shaped probes 12 to 16 inches in diameter which vibrates at frequencies typically in the range of 30 to 50 Hz. The probe is first inserted into the ground by both jetting and vibration. After the probe reaches the required depth of compaction, granular material, usually sand, is added from the ground surface to fill the void space created by the vibrator. A compacted radial zone of granular material is created

APPLICATIONS:

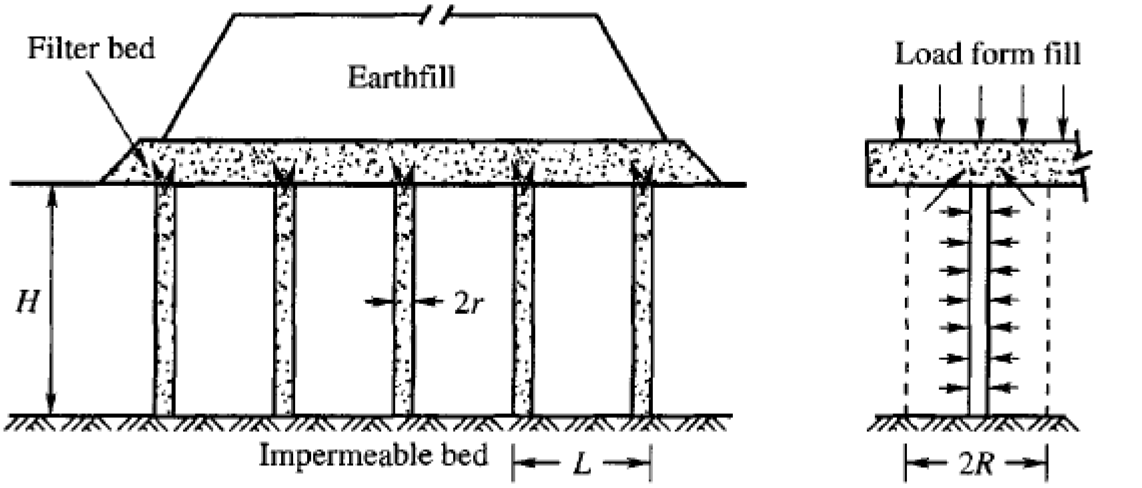
- Reduction of foundation settlements.
- Reduction of risk of liquefaction due to seismic activity.
- Permit construction on granular fills.



SAND DRAIN INSTALLATION

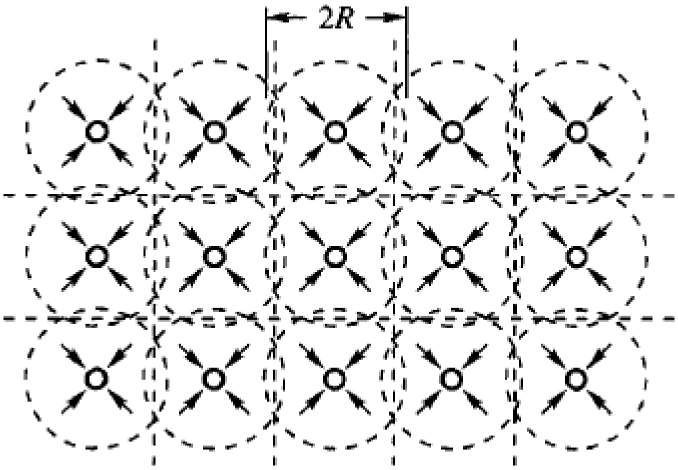
The main application of the radial consolidation is in the design of sand drains used to increase the rate of drainage in the embankment. Sand drains are constructed by driving a casing (or a hollow mandrel) into the embankment and making vertical bore holes. The holes are backfilled with a suitably graded sand. The casing is withdrawn after the sand has been filled. The drains are generally laid either in a square pattern or a triangular pattern. The spacing (S) of the drains is kept smaller than the thickness of the embankment (2H) in order to reduce the length of the radial

PRE – LOADING Preloading is a technique that can successfully be used to densify soft to very soft cohesive soils. Large-scale construction sites composed of weak silts and clays or organic materials (particularly marine deposits), sanitary landfills , and other compressible soils may often be stabilized effectively and economically by preloading. Preloading compresses the soil. Compression takes place when the water in the pores of the soil is removed which amounts to artificial consolidation of soil in the field. In order to remove the water squeezed out of the pores and hasten the period of consolidation, horizontal and vertical drains are required to be provided in the mass. The preload is generally in the form of an imposed earth fill which must be left in place long enough to induce consolidation. The process of consolidation can be checked by providing suitable settlement plates and piezometers. The greater the surcharge load, shorter the time for consolidation. This is a case of three-dimension consolidation. Two types of vertical drains considered are 1. Cylindrical sand drains 2. Wick (prefabricated vertical) drains.



(a) Vertical section

(b) Section of single drain



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(a) Surface protection:- A geotextile, placed on the soil, prevents one of being damaged by such action as weather, light traffic, surface water flow. Etc.

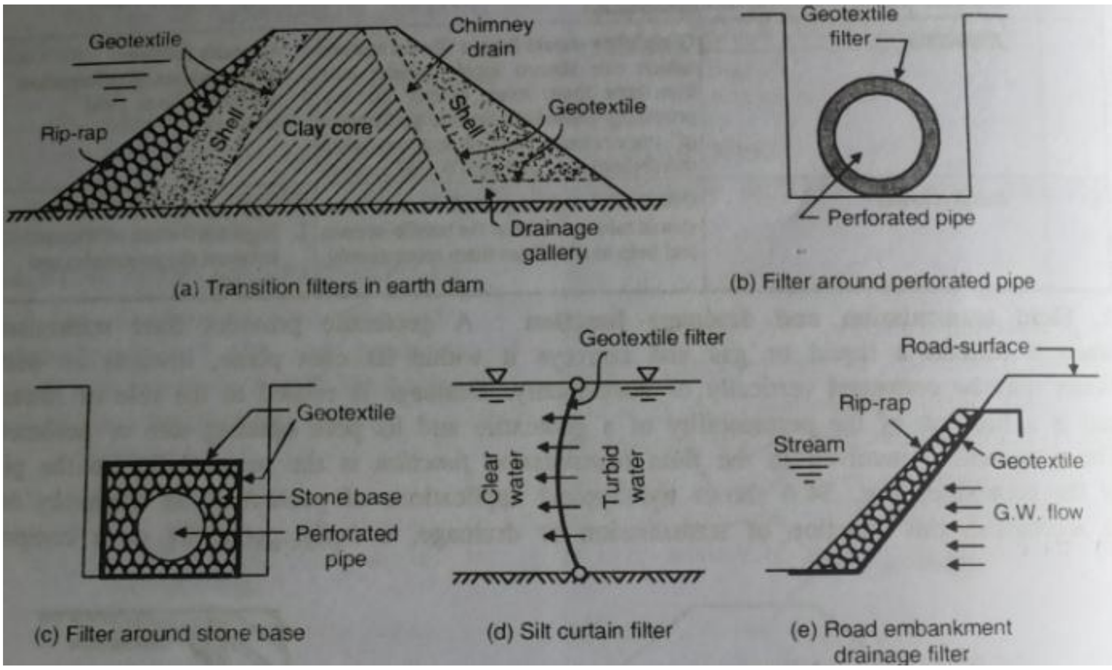
(b) Interface protection:- A geotextile, placed between two materials prevents one of the materials from being damaged by concentrated stresses applied by (or strains imposed by) the other material.

5) Reinforcement function:- Reinforced earth:- Since the tensile strength of soil is practically nil, geotextiles which are good in tensile strength, can contribute to the load carrying capacity of soil.A geotextile can work as reinforcement in two ways:-

(a) As tensile member:- A geotextile acts as a tensile member when it provides tensile modulus and tensile strength to a soil with which it is interacting through interface shear strength, i.e. friction, cohesion-adhesion and/or interlocking between geotextile and soil.

(b) As tensioned membrane:- A geotextile function as a tensioned membrane when it is placed between two materials two materials having different pressures, and its tension balances the pressure difference between two materials, thus strengthening the structure.

Fluid transmission and drainage function: A geotextile acts as a filter when it allows liquid to pass normal to its own plane while preventing most soil particles from being carried away by liquid current. Two cases can be considered: (i) where they accumulate on the filter.) while allowing water to pass through, and (ii) a geotextile, placed in contact with a soil, allows water seeping from the soil to pass through, while preventing any movement of soil particles. The two important properties required in a geotextile to work as a filter are (i) permeability across its own plane, (ii) porometry.



(a) Transition filters in earth dam
(b) Filter around perforated pipe
(c) Filter around stone base
(d) Silt curtain filter
(e) Road embankment drainage filter

Summary

Soil-cement is the reaction product of an intimate mixture of pulverized soil and measured amounts of portland cement and water, compacted to high density. As the cement hydrates, the mixture becomes a hard, durable structural material. Hardened soil-cement has the capacity to bridge over local weak points in a subgrade.
When properly made, it does not soften when exposed to wetting and drying, or freezing and thawing cycles. Portland cement and soil mixed at the proper moisture content has been used increasingly in recent years to stabilize soils in special situations. Probably the main use has been to build stabilized bases under concrete pavements for highways and airfields. Soil cement mixtures are also used to provide wave protection on earth dams. There are three categories of soil-cement they are: 1. Normal soil-cement usually contains 5 to 14 percent cement by weight

and is used generally for stabilizing low plasticity soils and sandy soils. 2. Plastic soil-cement has enough water to produce a wet consistency similar to mortar

Exercise

1 Define the separation function

2 Explain the vibrofloation

3 What the various technique of ground water improvement

4   Write a short note on Pre loading

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Department of Civil Engineering

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Name of Subject: - Geotechnical Engg –II	Subject Code:- BECVE504T
Unit-V: Shallow foundation	**Semester: - V**
Bearing capacity of soil: Factor affecting bearing capacity, Terzaghis theory, its validity and limitation, types of shear failure in foundation soil, effect of water table on bearing capacity, (introduction to IS method, factor affecting bearing capacity, field determination of bearing capacity through plate load test and standard penetration test,)	

Course Outcome (CO):-. Students able to understand the various application of shallow foundation

Learning Outcomes (LOs) :- (4 to 5 are expected and as per the COs)

- To understand the basic concept and Application of shallow foundation
- To learn and apply the local states of plastic equilibrium
- To remember the design concept of shallow foundation
- To learn about various bearing capacity

Unit-05:- Foundation

SHALLOW FOUNDATIONS: Bearing capacity of soils : Terzagi"s theory , its validity and limitations , bearing capacity factors , types of shear failure in foundation soil , effect of water table on bearing capacity factors , types of shear failure in foundation soil , effect of water table on bearing capacity , correction factors for shape and depth of footings. Bearing capacity estimation from N-value , factors affecting bearing capacity , presumptive bearing capacity. **Settlement of shallow foundation :** causes of settlement , elastic and consolidation settlement , differential settlement , control of excessive settlement. Proportioning the footing for equal settlement . Plate load test : Procedure , interpretation for bearing capacity and settlement prediction**. (8)**

INRODUCTION It is the customary practice to regard a foundation as shallow if the depth of the foundation is less than or equal to the width of the foundation. A foundation is an integral part of a structure. The stability of a structure depends upon the stability of the supporting soil. Two important factors that are to be considered are:- 1. The foundation must be stable against shear failure of the supporting soil. 2. The foundation must not settle beyond a tolerable limit to avoid damage to the structure.

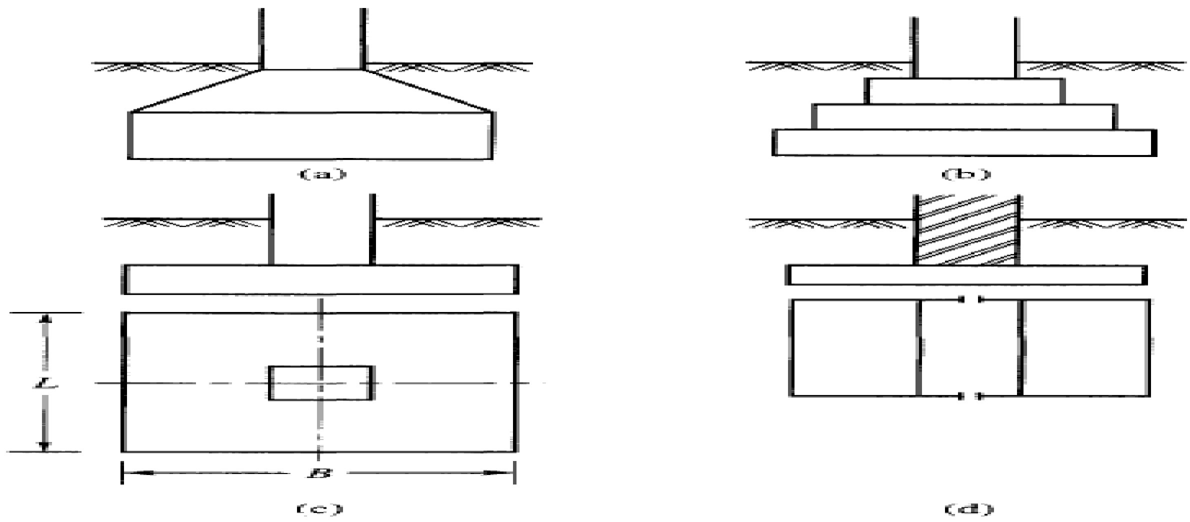


Figure.1 Types of shallow foundations: (a) plain concrete foundation, (b) stepped reinforced concrete foundation, (c) reinforced concrete rectangular foundation, and (d) reinforced concrete wall foundation.

DEFINITIONS

1) Footing:- A footing is a portion of the foundation of a structure that transmits loads directly to the soil.

2) Foundation: - A foundation is that part of he structure which is in direct contact with and transmits loads to the ground.

3) Foundation soil: - It is the upper part of the earth mass carrying the load of the structure.

Bearing capacity: - The supporting power of a soil or rock is referred to as its bearing capacity.

5) Gross pressure intensity (*q*):- The gross pressure intensity q is the total pressure at the base of the footing due to the weight of the superstructure, self-weight of the footing and the weight of the earth fill, if any.

6) Net pressure intensity (*qn*):- It is defined as the excess pressure, or the difference in intensities of the gross pressure after the construction of the structure and the original overburden pressure. Thus, if D is the depth of the footing

7) Ultimate bearing capacity (q_f):- The ultimate bearing capacity is defined as the minimum gross pressure intensity at the base of the foundation at which the soil fails in shear.

8) Net ultimate nearing capacity (q_{nf}):- It is minimum net pressure intensity causing shear failure of soil. The ultimate bearing capacity q_f and the net ultimate capacity are evidently connected by the following relation:
Where, σ is the effective surcharge at the base level of the foundation.
9) Net safe bearing capacity (q_{ns}):- The net safe bearing capacity is the net ultimate bearing capacity divided by a factor of safety F

10) Safe bearing capacity (q_s):- The maximum pressure which the soil can carry safely without risk of shear failure is called the safe bearing capacity.

11) Allowable bearing capacity or pressure (qa):- It is the net loading intensity at which neither the soil fails in shear nor there is excessive settlement detrimental to the structure.

BEARING CAPACITY OF SOILS: ***Terzagi's theory, its validity and limitations*** Terzaghi (1943) used the same form of equation as proposed by Prandtl (1921) and extended his theory

BEARING CAPACITY OF SOILS: ***Terzagi's theory, its validity and limitations*** Terzaghi (1943) used the same form of equation as proposed by Prandtl (1921) and extended his theory to take into account the weight of soil and the effect of soil above the base of the foundation on the bearing capacity of soil. Terzaghi made the following assumptions for developing an equation for determining q_u for a c-Φ soil. (1) The soil is semi-infinite, homogeneous and isotropic, (2) the problem is two-dimensional, (3) the base of the footing is rough, (4) the failure is by general shear, (5) the load is vertical and symmetrical, (6) the ground surface is horizontal,
(7) the overburden pressure at foundation level is equivalent to a surcharge load $q'_0 = \gamma D_f$ where γ is the effective unit weight of soil, and D the depth of foundation less than the width B of the foundation,

(8) the principle of superposition is valid, and (9) Coulomb's law is strictly valid, that is, $\sigma = c + \sigma \tan \Phi$. **Limitations:-**
(1) As the soil compresses, Φ changes, slight downward movement of footing may not develop the plastic zones.

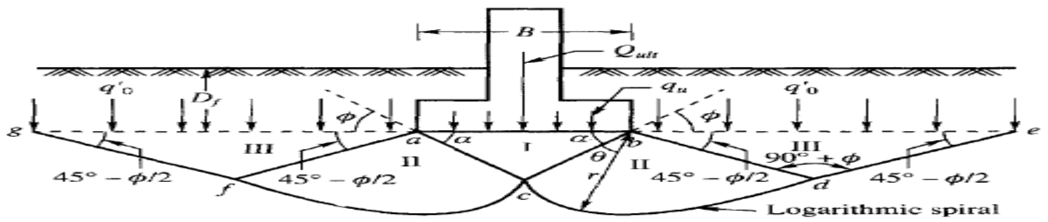
(2) Error due to separate calculation of three component of Pp, and then their addition, although their critical surfaces are not identical, is small and on the safe side.

(3) Error due to assumption that failure zones do not extend above horizontal plane through the base of footing, increase with the depth of foundation, and hence the theory is suitable for shallow foundation only.

Mechanism of Failure The shapes of the failure surfaces under ultimate loading conditions are

Mechanism of Failure The shapes of the failure surfaces under ultimate loading conditions are given in Fig. 2. The zones of plastic equilibrium represented in this figure by the area *gedcf* may be subdivided into 1. Zone I of elastic equilibrium 2. Zones II of radial shear state

3. Zones III of Rankine passive state When load *qu* per unit area acting on the base of the footing of width B with a rough base is transmitted into the soil, the tendency of the soil located within zone I is to spread but this is counteracted by friction and adhesion between the soil and the base of the footing. Due to the existence of this resistance against lateral spreading, the soil located immediately beneath the base remains permanently in a state of elastic equilibrium, and the soil located within this central Zone I behaves as if it were a part of the footing and sinks with the footing under the superimposed load. The depth of this wedge shaped body of soil *abc* remains practically unchanged, yet the footing sinks. This process is only conceivable if the soil located just below point *c* moves vertically downwards. This type of movement requires that the surface of sliding *cd* (Fig. 2) through point *c* should start from a vertical tangent. The boundary *be* of the zone of radial shear bed (Zone II) is also the surface of sliding. As per the theory of plasticity, the potential surfaces of sliding in an ideal plastic material intersect each other in every point of the zone of plastic equilibrium at an angle (90° - 0). Therefore the boundary be must rise at an angle Φ to the horizontal provided the friction and adhesion between the soil and the base of the footing suffice to prevent a sliding motion at the base



TYPES OF SHEAR FAILURE

1) GENERAL SHEAR FAILURE: -

Fig.a shows a strip footing resting on the surface of a dense sand or a stiff clay. The figure also shows the load settlement curve for the footing, where ‘q’ is the load per unit area and ‘s’ is the settlement. At a certain load intensity equal to qu, the settlement increases suddenly. A shear failure occurs in the soil at that load and the failure surfaces extend to the ground surface. This type of failure is known as general shear failure. A heave on the sides is always observed in general shear failure.

2) LOCAL SHEAR FAILURE: -

Fig.b shows a strip footing resting on a medium dense sand or on a clay of medium consistency. The figure also shows the load – settlement curve. When the load is equal to a certain value . The foundation movement is accompanied by sudden jerks. The failure surfaces gradually extend outwards from the foundation, as shown. However, a considerable movement of the foundation is required for the failure surfaces to extend to the ground surface (shown dotted). The load at which this happens is equal to qu, beyond this point, an increase of load is accompanied by a large increase in settlement. This type of failure is known as local shear failure. A heave is observed only when there is substantial vertical settlement.

3) PUNCHING

PUNCHING SHEAR FAILURE: -

Fig.c shows a strip footing resting on a loose sand or soft clay. In this case, the failure surfaces do not extend up to the ground surface. There are jerks in foundation at a load of . The footing fails at a load qu at which stage the load – settlement curve becomes steep and practically linear. This type of failure is called the punching shear failure. No heave is observed. There is only vertical movement of footing.

EFFECT OF WATER TABLE ON BEARING CAPACITY When the water table is above the base of the footing, the submerged weight γ' should be used for the soil below the water table for computing the effective pressure or the surcharge. When the water table is located somewhat below the base of the footing, the elastic wedge is partly of moist soil and partly of submerged soil and a suitable reduction factor should be used with the wedge term $\frac{1}{2}\gamma B N_\gamma$, since it uses effective unit weight.

SETTLEMENT OF FOUNDATION

(a) Settlement under loads

Foundation settlement under loads can be classified into 3 types.

(1) Immediate or elastic settlement (Si): - Immediate or elastic settlement takes place during or immediately after the construction of the structure. It is also known as the distortion settlement as it is due to distortions (and not the volume change) within the foundation soil. Although the settlement is not truly elastic, it is computed using elastic theory, especially for cohesive soils.

(2) Consolidation settlement (Sc): - This component of the settlement occurs due to gradual expulsion of water from the voids of the soil. This component is determined using Terzaghi's theory of consolidation.

(3) Secondary Consolidation Settlement (Ss): - This component of the settlement is due to secondary consolidation. This settlement occurs after completion of the primary

Settlement due to other causes

In addition to settlement under loads, the settlement may also occur to a number of other causes.

1) Underground erosion: - Underground erosion may cause formation of cavities in the subsoil which when collapse cause settlement.

2) Structural collapse of soil: - Structural collapse of some soils, such as saline, non-cohesive soils, gypsum, silts and clays and loess, may occur due to dissolution of materials responsible for intergranular bond of grains.

3) Thermal changes: - Temperature change cause shrinkage in expansive soils due to which settlement occurs.

4) Frost heave: - Frost heave occurs if the structure is not founded below the depth of frost penetration.

5) Vibration and Shocks: - Vibrations and shock cause large settlements, especially in loose, cohesionless soils.

6) Mining subsidence: - Subsidence of ground may occur due to removal of minerals and other materials from mines below.

7) Land slides: - If land slides occur on unstable slopes, there may be serious settlement problems.

8) Creep: - The settlement may also occur due to creep on clay slopes.

9) Changes in the vicinity: - If there are changes due to construction of a new building near the existing foundation, the settlement may occur due to increase in the stresses.

Suitable measures are taken to reduce the settlements due to all above causes.

PLATE LOAD TEST :

Plate load test is a field test to determine the ultimate bearing capacity of soil, and the probable settlement under a given loading. The test essentially consists in loading a rigid plate at the foundation level, and determining the settlements corresponding to each load increment. The ultimate bearing capacity is then taken as the load at which the plate starts sinking at a rapid rate. The method assumes that down to the depth of influence of stresses, the soil strata is reasonably uniform.

The bearing plate is square, of minimum recommended size 30 cm square and maximum size 75 cm square. The plate is machined on sides and edges, and should have a thickness sufficient to withstand effectively and bending stresses that would be caused by maximum anticipated load. The thickness of steel plate should not be less than 25 mm. The test pit width is made five times the width of the plate Bp. At the centre of the pit, a small square hole is dug whose size is equal to the size of the plate and the bottom level of which correspond to the level of the actual foundation. The depth Dp of the hole should be such that

The loading to the test plate may be applied with the help of a hydraulic jack. The reaction of the hydraulic jack may be bores by either of the following two methods:

a) Gravity loading platform method.

b) Reaction truss method.

In the case of gravity loading method, a platform is constructed over a vertical column resting on the plate, and the loading is done with the help of sand bags, stones or concrete blocks. When load is applied to the plate, it sinks or settles. The settlement of the plate is measured with the help of sensitive dial gauges. For square plate, two dial gauges are used. The dial gauges are

mounted on independently supported datum bar. As the plate settles, the ram of the dial guage moves down and settlement is reconsidered. The load is indicated on the load – guage of the hydraulic jack.

Summary

It is the customary practice to regard a foundation as shallow if the depth of the foundation is less than or equal to the width of the foundation. A foundation is an integral part of a structure. The stability of a structure depends upon the stability of the supporting soil. Two important factors that are to be considered are:- 1. The foundation must be stable against shear failure of the supporting soil. 2. The foundation must not settle beyond a tolerable limit to avoid damage to the structure. Terzagi‟s theory , its validity and limitations , bearing capacity factors , types of shear failure in foundation soil , effect of water table on bearing capacity factors , types of shear failure in foundation soil , effect of water table on bearing capacity , correction factors for shape and depth of footings. Bearing capacity estimation from N-value , factors affecting bearing capacity

Exercise

1 Explain shallow foundation and its type

2 Write a note on plate load test

3 Define landslides

4 Explain the effect of water table on bearing capacity of soil

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Department of Civil Engineering

Name of Subject: - Geotechnical Engg –II	Subject Code:- BECVE504T
Unit-VI: Pile foundation	**Semester: - V**
Bearing capacity of soil: Factor affecting bearing capacity, Terzaghis theory, its validity and limitation, types of shear failure in foundation soil, effect of water table on bearing capacity, (introduction to IS method, factor affecting bearing capacity, field determination of bearing capacity through plate load test and standard penetration test,)	

Course Outcome (CO):-. Students able to understand the various application of pile foundation

Learning Outcomes (LOs) :- (4 to 5 are expected and as per the COs)

- To understand the basic concept and Application of pile foundation
- To learn and apply the local states of plastic equilibrium
- To remember the design concept of pile foundation
- To learn about various bearing capacity and settlement due to pile foundation

INTRODUCTION Shallow foundations are normally used where the soil close to the ground surface and up to the zone of *significant stress* possesses sufficient bearing strength to carry the superstructure load without causing distress to the superstructure due to settlement. However, where the top soil is either loose or soft or of a swelling type the load from the structure has to be transferred to deeper firm strata. The structural loads may be transferred to deeper firm strata by means of piles. Piles are long slender columns either driven, bored or *cast-in-situ.* Driven piles are made of a variety of materials such as concrete, steel, timber etc., whereas *cast-in-situ* piles are concrete piles. They may be subjected to vertical or lateral loads or a combination of vertical and lateral loads. If the diameter of a *bored-cast-in-situ* pile is greater than about 0.75 m, it is sometimes called a drilled pier, drilled caisson or drilled shaft. The distinction made between a small diameter bored *cast-in-situ* pile (less than 0.75 m) and a larger one is just for the sake of design considerations.

CLASSIFICATION OF PILES Piles may be classified as long or short in accordance with the Lid ratio of the pile (where L = length, d = diameter of pile). A short pile behaves as a rigid body and rotates as a unit under lateral loads. The load transferred to the tip of the pile bears a significant proportion of the total vertical load on the top. In the case of a long pile, the length beyond a particular depth loses its significance under lateral loads, but when subjected to vertical load, the frictional load on the sides of the pile bears a significant part to the total load. Piles may further be classified as vertical piles or inclined piles. Vertical piles are normally used to carry mainly vertical loads and very little lateral load. When piles are inclined at an angle to the vertical, they are called batter piles or raker piles. Batter piles are quite effective for taking lateral loads, but when used in groups, they also can take vertical loads.

TYPES OF PILES ACCORDING TO THEIR COMPOSITION Piles may be classified according to their composition as 1. Timber Piles, 2. Concrete Piles, 3. Steel Piles. **Timber Piles:** Timber piles are made of tree trunks with the branches trimmed off. Such piles shall be of sound quality and free of defects. The length of the pile may be 15 m or more. The diameter of the piles at the butt end may vary from 30 to 40 cm. The diameter at the tip end should not be less than 15 cm. Piles entirely submerged in water last long without decay provided marine borers are not present. After being driven to final depth, all pile heads, treated or untreated, should be sawed square to sound undamaged wood to receive the pile cap. But before concrete for the pile cap is poured, the head of the treated piles should be protected by a zinc coat, lead paint or by wrapping the pile heads with fabric upon which hot pitch is applied. Driving of timber piles usually results in the crushing of the fibers on the head (or brooming) which can be somewhat controlled by using a driving cap, or ring around the butt.

The usual maximum design load per pile does not exceed 250 kN. Timber piles are usually less expensive in places where timber is plentiful. Refer Fig.1.

Concrete Piles: Concrete piles are either precast or cast-in-situ piles. Precast concrete piles are cast and cured in a casting yard and then transported to the site of work for driving. Precast piles may be made of uniform sections with pointed tips. Tapered piles may be manufactured when greater bearing resistance is required. Normally piles of square or octagonal sections are manufactured since these shapes are easy to cast in horizontal position. Necessary reinforcement is provided to take care of handling stresses. Piles may also be prestressed. Maximum load on a prestressed concrete pile is approximately 2000 kN and on precast piles 1000 kN. The optimum load range is 400 to 600 kN.

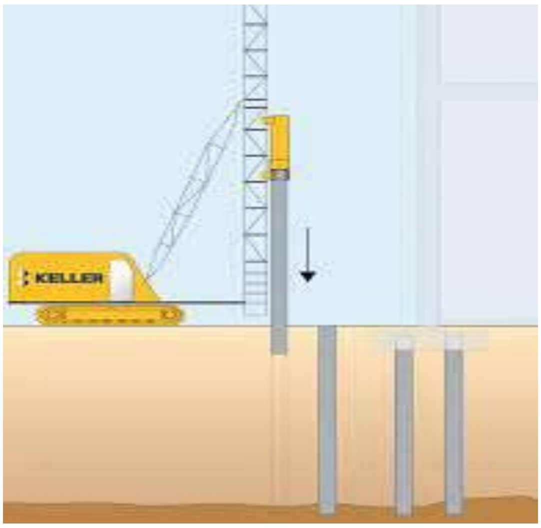
Steel Piles: Steel piles are usually rolled H shapes or pipe piles, H-piles are proportioned to withstand large impact stresses during hard driving. Pipe piles are either welded or seamless steel pipes which may be driven either open-end or closed-end. Pipe piles are often filled with concrete after driving, although in some cases this is not necessary. The optimum load range on steel piles is 400 to 1200kN.

TYPES OF PILES ACCORDING TO THE METHOD OF INSTALLATION According to the method of construction, there are three types of piles. They are 1. Driven piles, 2. Cast-in-situ piles and 3. Driven and cast-in-situ piles.

Driven Piles: Piles may be of timber, steel or concrete. When the piles are of concrete, they are to be precast. They may be driven either vertically or at an angle to the vertical. Piles are driven using a pile hammer. When a pile is driven into granular soil, the soil so displaced, equal to the volume of the driven pile, compacts the soil around the sides since the displaced soil particles enter the soil spaces of the adjacent mass which leads to densification of the mass. The pile that compacts the soil adjacent to it is sometimes called a compaction pile. The compaction of the soil mass around a pile increases its bearing capacity. If a pile is driven into saturated silty or cohesive soil, the soil around the pile cannot be densified because of its poor drainage qualities. The soil adjacent to the piles is remolded and loses to a certain extent its structural strength. The immediate effect of driving a pile in a soil with poor drainage qualities is, therefore, to decrease its bearing strength. However, with the passage of time, the remolded soil regains part of its lost strength due to the reorientation of the disturbed particles (which is termed thixotrophy} and due to consolidation of the mass. The advantages and disadvantages of driven piles are:

Advantages 1. Piles can be precast to the required specifications. 2. Piles of any size, length and shape can be made in advance and used at the site. As a result, the progress of the work will be rapid. 3. A pile driven into granular soil compacts the adjacent soil mass and as a result the bearing capacity of the pile is increased. 4. The work is neat and clean. The supervision of work at the site can be reduced to a minimum. The storage space required is very much less. 5. Driven piles may conveniently be used in places where it is advisable not to drill holes for fear of meeting ground water under pressure. 6. Drivens pile are the most favored for works over water such as piles in wharf structures or jetties

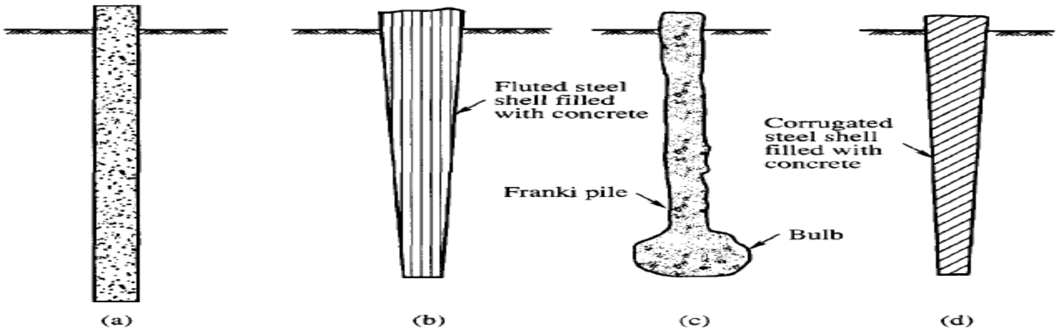
Disadvantages 1. Precast or prestressed concrete piles must be properly reinforced to withstand handling stresses during transportation and driving. 2. Advance planning is required for handling and driving. 3. Requires heavy equipment for handling and driving. 4. Since the exact length required at the site cannot be determined in advance, the method involves cutting off extra lengths or adding more lengths. This increases the cost of the project. 5. Driven piles are not suitable in soils of poor drainage qualities. If the driving of piles is not properly phased and arranged, there is every possibility of heaving of the soil or the lifting of the driven piles during the driving of a new pile. 6. Where the foundations of adjacent structures are likely to be affected due to the vibrations generated by the driving of piles, driven piles should not be used.



Cast-in-situ Piles Cast-in-situ piles are concrete piles. These piles are distinguished from drilled piers as small diameter piles. They are constructed by making holes in the ground to the required depth and then filling the hole with concrete. Straight bored piles or piles with one or more bulbs at intervals may be cast at the site. The latter types are called under-reamed piles. Reinforcement may be used as per the requirements. Cast-in-situ piles have advantages as well as disadvantages.

Advantages 1. Piles of any size and length may be constructed at the site. 2. Damage due to driving and handling that is common in precast piles is eliminated in this case. 3. These piles are ideally suited in places where vibrations of any type are required to be avoided to preserve the safety of the adjoining structure

Disadvantages 1. Installation of cast-in-situ piles requires careful supervision and quality control of all the materials used in the construction. 2. The method is quite cumbersome. It needs sufficient storage space for all the materials used in the construction. 3. The advantage of increased bearing capacity due to compaction in granular soil that could be obtained by a driven pile is not produced by a cast-in-situ pile. 4. Construction of piles in holes where there is heavy current of ground water flow or artesian pressure is very difficult.



Driven and Cast-in-situ Piles This type has the advantages and disadvantages of both the driven and the cast-in-situ piles. The procedure of installing a driven and cast-in-situ pile is as follows: A steel shell is driven into the ground with the aid of a mandrel inserted into the shell. The mandrel is withdrawn and concrete is placed in the shell. The shell is made of corrugated and reinforced thin sheet steel (mono-tube piles) or pipes (Armco welded pipes or common seamless pipes). The piles of this type are called a shell type. The shell-less type is formed by withdrawing the shell while the concrete is being placed. In both the types of piles the bottom of the shell is closed with a conical tip which can be separated from the shell

enlarged bulb may be formed in both the types of piles. Franki piles are of this type. The common types of driven and cast-in-situ piles are given in . In some cases the shell will be left in place and the tube is concreted. This type of pile is very much used in piling over water

USES OF PILES The major uses of piles are: 1. To carry vertical compression load. 2. To resist uplift load. 3. To resist horizontal or inclined loads.

SELECTION OF PILE The selection of the type, length and capacity is usually made from estimation based on the soil conditions and the magnitude of the load. The factors that govern the selection of piles are: 1. Length of pile in relation to the load and type of soil 2. Character of structure 3. Availability of materials 4. Type of loading 5. Factors causing deterioration.

PILE DRIVING Piles are commonly driven by means of a hammer supported by a crane or by a special device known as a pile driver. The hammer is guided between two parallel steel members known as leads. The leads are carried on a frame in such a way that they can be supported in a vertical position or an inclined position. Hammers are of the following types:

(1) Drop Hammer:- If a hammer (ram or monkey) is raised by winch and allowed to fall by gravity on the top of a pile, it is called a drop hammer.

(2) Single acting hammer:- If the hammer is raised by steam, compressed air or internal combustion, but is allowed to fall by gravity alone, it is called a single acting hammer.

(3) Double acting hammer:- The double acting hammer employs steam or air for lifting the ram and for accelerating the downward stroke. It operates with succession of rapid blows.

(4) Diesel hammer:- The diesel hammer is a small, light weight self-contained and self-acting type, using gasoline for fuel. The total driving energy is the sum of the impact of the ram plus the energy delivered by explosion.

(5) Vibratory hammer:- The driving unit vibrated at high frequency.

LOAD TRANSFER MECHANISM OF AXIALLY LOADED PILES Consider the pile shown in Fig. 4(b) is loaded to failure by gradually increasing the load on the top. If settlement of the top of the pile is measured at every stage of loading after equilibrium condition is attained, a load settlement curve as shown in Fig. 4(c) can be obtained. If the pile is instrumented, the load distribution along the pile can be determined at different stages of loading and plotted as shown in Fig. 4(b).

When a load Q_1 acts on the pile head, the axial load at ground level is also Q_1 but at level A_1 (Fig. 4(b)), the axial load is zero. The total load Q_1 is distributed as friction load within a length of pile L_1. The lower section A_1B of pile will not be affected by this load. As the load at the top is increased to Q_2, the axial load at the bottom of the pile is just zero. The total load Q_2 is distributed as friction load along the whole length of pile L. The friction load distribution curves along the pile shaft may be as shown in the figure. If the load put on the pile is greater than Q_2, a part of this load is transferred to the soil at the base as point load and the rest is transferred to the soil surrounding the pile. With the increase of load Q on the top, both the friction and point loads continue to increase. The friction load attains an ultimate value Q_f at a particular load level, say Q_m, at the top, and any further increment of load added to Q_m will not increase the value of Q_f. However, the point load, Q_p still goes on increasing till the soil fails by punching shear failure. It has been determined by Van Wheele (1957) that the point load Q increases linearly with the elastic compression of the soil at the base. The relative proportions of the loads carried by skin friction and base resistance depend on the shear strength and elasticity of the soil. Generally the vertical movement of the pile which is required to mobilize full end resistance is much greater than that required to mobilize full skin friction. Experience indicates that in bored cast-in-situ piles full frictional load is normally mobilized at a settlement equal to 0.5 to 1 percent of pile diameter and the full base load Q_b at 10 to 20 percent of the diameter. But, if this ultimate load criterion is applied to piles of large diameter in clay, the settlement at the working load (with a factor of safety of 2 on the ultimate load) may be excessive. A typical load-settlement relationship of friction load and base load is shown in Fig. 4(d) (Tomlinson, 1986) for a large diameter bored and cast-in-situ pile in clay. It may be seen from this figure that the full shaft resistance is mobilized at a settlement of only 15 mm whereas the full base resistance, and the ultimate resistance of the entire pile, is mobilized at a settlement of 120 mm. The shaft load at a settlement of 15 mm is only 1000 kN which is about 25 percent of the base resistance. If a working load of 2000 kN at a settlement of 15 mm is used for the design, at this working load, the full shaft resistance will have been mobilized whereas only about 50 percent of the base resistance has been mobilized. This means if piles are designed to carry a working load equal to 1/3 to 1/2 the total failure load, there is every likelihood of the shaft resistance being fully mobilized at the working load. This has an important bearing on the design.

When a load Q_1 acts on the pile head, the axial load at ground level is also Q_1 but at level A_1 (Fig. 4(b)), the axial load is zero. The total load Q_1 is distributed as friction load within a length of pile

L_1. The lower section A_1B of pile will not be affected by this load. As the load at the top is increased to Q_2, the axial load at the bottom of the pile is just zero. The total load Q_2 is distributed as friction load along the whole length of pile L. The friction load distribution curves along the pile shaft may be as shown in the figure. If the load put on the pile is greater than Q_2, a part of this load is transferred to the soil at the base as point load and the rest is transferred to the soil surrounding the pile. With the increase of load Q on the top, both the friction and point loads continue to increase. The friction load attains an ultimate value Q_f at a particular load level, say Q_m, at the top, and any further increment of load added to Q_m will not increase the value of Q_f. However, the point load, Q_p still goes on increasing till the soil fails by punching shear failure. It has been determined by Van Wheele (1957) that the point load Q increases linearly with the elastic compression of the soil at the base. The relative proportions of the loads carried by skin friction and base resistance depend on the shear strength and elasticity of the soil. Generally the vertical movement of the pile which is required to mobilize full end resistance is much greater than that required to mobilize full skin friction. Experience indicates that in bored cast-in-situ piles full frictional load is normally mobilized at a settlement equal to 0.5 to 1 percent of pile diameter and the full base load Q_b at 10 to 20 percent of the diameter. But, if this ultimate load criterion is applied to piles of large diameter in clay, the settlement at the working load (with a factor of safety of 2 on the ultimate load) may be excessive. A typical load-settlement relationship of friction load and base load is shown in Fig. 4(d) (Tomlinson, 1986) for a large diameter bored and cast-in-situ pile in clay. It may be seen from this figure that the full shaft resistance is mobilized at a settlement of only 15 mm whereas the full base resistance, and the ultimate resistance of the entire pile, is mobilized at a settlement of 120 mm. The shaft load at a settlement of 15 mm is only 1000 kN which is about 25 percent of the base resistance. If a working load of 2000 kN at a settlement of 15 mm is used for the design, at this working load, the full shaft resistance will have been mobilized whereas only about 50 percent of the base resistance has been mobilized. This means if piles are designed to carry a working load equal to 1/3 to 1/2 the total failure load, there is every likelihood of the shaft resistance being fully mobilized at the working load. This has an important bearing on the design.

PILE CAPACITY BY STATIC FORMULA & DYNAMIC FORMULA The ultimate load carrying capacity, or ultimate bearing capacity, or the ultimate bearing resistance, Q_f of a pile is defined as the maximum load which can be carried by a pile, and at which the pile continues to sink without further increase of load. The load carrying capacity of a pile can be determined by the following methods:-

i) DYNAMIC FORMULAE

When a pile hammer hits the pile, the total driving energy is equal to the weight of hammer times the height of drop or stroke. Following are some commonly used dynamic formulae:

a) **Engineering news formula:** The engineering news formula was proposed by A.M. Wellington (1818) in the following general form:

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

Where, Qa = allowable load W = weight of hammer H = height of fall F= factor of safety = 6 S = final set (penetration) per blow, usually taken as average penetration, cm per blow for last 5 blows of a drop hammer, or 20 blows of a steam hammer. C = empirical constant Denoting W in kg, Hin cm, S in cm, and C = 2.5 cm for drop hammer and = o.25 cm for single and double acting hammers, the above formula reduces to the following forms :-

Hiley's formula: Indian standard IS : 2911 (Part I) 1964 gives the following based of original expression of Hiley:

$$Q_f = \frac{\eta h W H \eta b}{(S+C/2)}$$

Q_f = ultimate load of pile W= weight of hammer, in kg H = height of drop of hammer, in cm S = penetration or set, in cm, per blow C = total elastic compression = $C_1 + C_2 + C_3$ C_1, C_2, C_3 = temporary elastic compression of dolly and packing, pile and soil respectively

NEGATIVE SKIN FRICTION When the soil layer surrounding a portion of the pile shaft settles more than the piles, 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 and extra downward load on the pile. The net ultimate load – carrying of the pile is given by the equation

PILE LOAD TEST

The pile load test can be performed either on a working pile which forms the foundation of the structure or on a test pile. The test load is applied with the help of calibrated jack placed over a rigid circular or square plate which in turn is placed on the head of the pile projecting above ground level. The reaction of the jack is borne by a truss or platform which may have gravity loading (in the form of sand bags etc.) or alternatively, the truss can be anchored to the ground with the help of anchor piles. In the later case, under-reamed piles or soil anchors may be used for anchoring the truss. The load is applied in equal increments of about one-fifth of the estimated allowable load. The settlements are recorded with the help of three dial gauges of sensitivity 0.02mm, symmetrically arranged over the plate, and fixed to an independent datum the test is increased to a value 2 ½ times the estimated load or to a load which causes a settlement equal to one-tenth of the pile diameter, whichever occurs earlier. The results are plotted in the form of load – settlement curve. The ultimate load is clearly indicated by load settlement curve approaching vertical

GROUP ACTION OF PILES

A pile is not used singularly beneath a column or a wall, because it is extremely difficult to drive the pile absolutely vertical and to place the foundation exactly over its centre line. If eccentric loading results, the connection between the pile and the column may break or the pile may fail structurally because of bending stresses. In actual, structural loads are supported by several piles in a triangular pattern are used. The loads are usually transferred to the pile group through a reinforced concrete slab, structurally tied to the pile tops such that the piles act as one unit. The slab is known as pile cap. The load acts on pile cap which distributes the load to the piles. The load carrying capacity of a pile group is not necessarily equal to the sum of the capacity of the individual piles. Estimation of the load – carrying capacity of a pile group is a complicated problem. When the piles are spaced a sufficient distance apart, the group capacity may approach the sum of the individual capacities

stresses transmitted by the piles to the soil may overlap, and this may reduce the load – carrying capacity of the piles. For such case, the capacity is limited by the group action.
The efficiency (η_g) of a group of piles is defined as the ratio of the ultimate load of the group to the sum of individual ultimate loads.

Summary

Piles are long slender columns either driven, bored or *cast-in-situ.* Driven piles are made of a variety of materials such as concrete, steel, timber etc., whereas *cast-in-situ* piles are concrete piles. They may be subjected to vertical or lateral loads or a combination of vertical and lateral loads. If the diameter of a *bored-cast-in-situ* pile is greater than about 0.75 m, it is sometimes called a drilled pier, drilled caisson or drilled shaft. The distinction made between a small diameter bored *cast-in-situ* pile (less than 0.75 m) and a larger one is just for the sake of design considerations.

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Exercise

1 **Explain the concrete pile**
2 **Define Negative skin friction**
3 **Explain group action of pile**
4 **Define pile structures**