UNIT I
SITE INVESTIGATION AND SELECTION OF FOUNDATION

Types of boring

1.Displacement borings

It is combined method of sampling & boring operation. Closed bottom sampler, slit cup, or piston type is forced in to the ground up to the desired depth. Then the sampler is detached from soil below it, by rotating the piston, & finally the piston is released or withdrawn. The sampler is then again forced further down & sample is taken. After withdrawal of sampler & removal of sample from sampler, the sampler is kept in closed condition & again used for another depth.

Features :

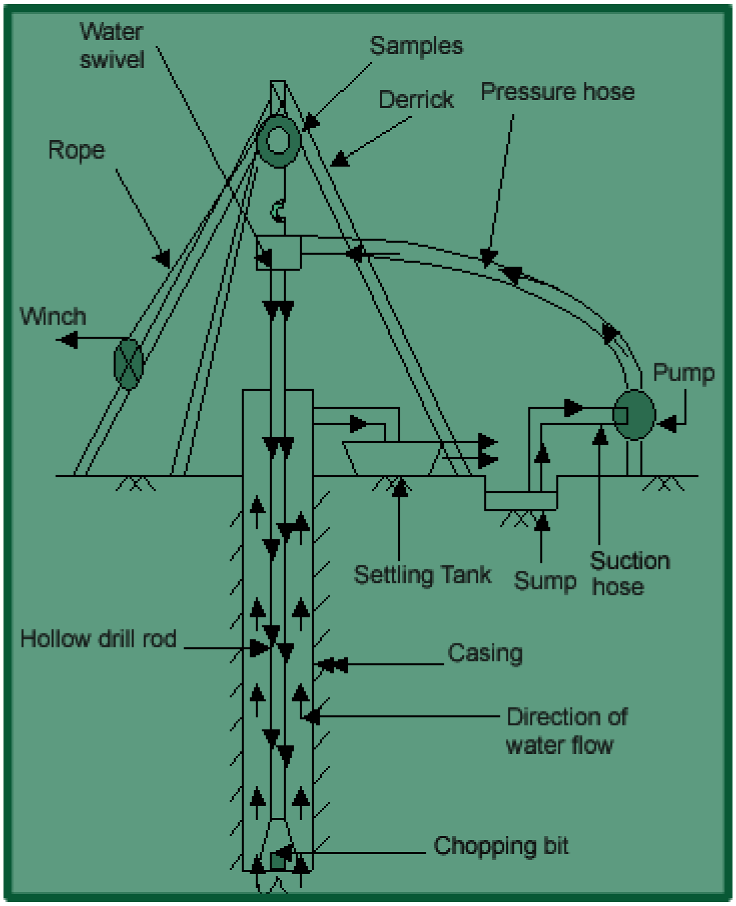
- Simple and economic method if excessive caving does not occur. Therefore not suitable for loose sand.
- Major changes of soil character can be detected by means of penetration resistance.
- These are 25mm to 75mm holes.
- It requires fairly continuous sampling in stiff and dense soil, either to protect the sampler from damage or to avoid objectionably heavy construction pit.

2.Wash boring:

It is a popular method due to the use of limited equipments. The advantage of this is the use of inexpensive and easily portable handling and drilling equipments. Here first an open hole is formed on the ground so that the soil sampling or rock drilling operation can be done below the hole. The hole is advanced by chopping and twisting action of the light bit. Cutting is done by forced water and water jet under pressure through the rods operated inside the hole.

In India the “Dheki” operation is used, i.e., a pipe of 5cm diameter is held vertically and filled with water using horizontal lever arrangement and by the process of suction and application of pressure, soil slurry comes out of the tube and pipe goes down. This can be done upto a depth of 8m –10m (excluding the depth of hole already formed beforehand)

Just by noting the change of colour of soil coming out with the change of soil character can be identified by any experienced person. It gives completely disturbed sample and is not suitable for very soft soil, fine to medium grained cohesionless soil and in cemented soil.



1.1Planning For Subsurface Exploration

The planning of the site exploration program involves location and depth of borings, test pits or other methods to be used, and methods of sampling and tests to be carried out. The purpose of the exploration program is to determine, within practical limits, the stratification and engineering properties of the soils underlying the site. The principal properties of interest will be the strength, deformation, and hydraulic characteristics. The program should be planned so that the maximum amount of information can be obtained at minimum cost. In the earlier stages of an investigation, the information available is often inadequate to allow a firm and detailed plan to be made. The investigation is therefore performed in the following phases:

1. Fact finding and geological survey

- Reconnaissance

1. Preliminary exploration
2. Detailed exploration

1. Fact finding and geological survey

Assemble all information on dimensions, column spacing, type and use of structure, basement requirements, and any special architectural considerations of the proposed building. Foundation regulations in the local building code should be consulted for any special requirements. For bridges the soil engineer should have access to type and span lengths as well as pier loadings. This information will indicate any settlement limitations, and can be used to estimate foundation loads.

2. Reconnaissance

This may be in the form of a field trip to the site which can reveal information on the type and behavior of adjacent sites and structures such as cracks, noticeable sags, and possibly sticking doors and windows. The type of local existing structure may influence, to a considerable extent, the exploration program and the best foundation type for the proposed adjacent structure. Since nearby existing structures must be maintained, excavations or vibrations will have to be carefully controlled. Erosion in existing cuts (or ditches) may also be observed. For highways, run off patterns , as well as soil stratification to the depth of the erosion cut , may be observed. Rock outcrops may give an indication of the presence or the depth of bedrock.

3. Auger boring

This method is fast and economical, using simple, light, flexible and inexpensive instruments for large to small holes. It is very suitable for soft to stiff cohesive soils and also can be used to determine ground water table. Soil removed by this is disturbed but it is better than wash boring, percussion or rotary drilling. It is not suitable for very hard or cemented soils, very soft soils, as then the flow into the hole can occur and also for fully saturated cohesionless soil.

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In general soil samples are categorized as shown in fig. 1.5

Fig. 1.5 Types of samples

Disturbed samples:

The structure of the soil is disturbed to the considerable degree by the action of the boring tools or the excavation equipments.

The disturbances can be classified in following basic types:

Change in the stress condition,

Change in the water content an

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Change in the stress condition,

Change in the water content and the void ratio,

Disturbance of the soil structure,

Chemical changes,

Mixing and segregation of soil constituents
The causes of the disturbances are listed below:

Method of advancing the borehole,

Mechanism used to advance the sampler,

Dimension and type of sampler,

Procedure followed in sampling and boring. **Undisturbed samples:** It retains as closely as practicable the true insitu structure and water content of the soil. For undisturbed sample the stress changes can not be avoided. The following requirements are looked for:

No change due to disturbance of the soil structure,

No change in void ratio and water content,

No change in constituents and chemical properties.

$$(C_i) = \frac{D_s - D_c}{D_c} x100\%$$

4 Requirement of good sampling process : Inside clearance ratio The soil is under great stress as it enters the sampler and has a tendency to laterally expand. The inside clearance should be large enough to allow a part of lateral expansion to take place, but it should not be so large that it permits excessive deformations and causes disturbances of the sample. For good sampling process, the inside clearance ratio should be within 0.5 to 3 %. For sands silts and clays, the ratio should be 0.5 % and for stiff and hard clays (below water table), it should be 1.5 %.

For stiff expansive type of clays, it should be 3.0 %. area ratio $(A_r) = \frac{D_w^2 - D_c^2}{D_c^2} x100\%$ Recovery ratio $(R) = \frac{L}{H} x100\%$

Where, L is the length of the sample within the tube,

H is the depth of penetration of the sampling tube.

It represents the disturbance of the soil sample. For good sampling the recovery ratio should be 96 to 98 %.

Wall friction can be reduced by suitableinside clearance, smooth finish and oiling.

The non-returned wall should have large orifice to allow air and water to escape. **In-situ tests General** The in situ tests in the field have the advantage of testing the soils in their natural, undisturbed condition. Laboratory tests, on the other hand, make use of small size samples obtained from boreholes through samplers and therefore the reliability of these depends on the quality of the so called 'undisturbed' samples. Further, obtaining undisturbed samples from non-cohesive, granular soils is not easy, if not impossible. Therefore, it is common practice to rely more on laboratory tests where cohesive soils are concerned. Further, in such soils, the field tests being short duration tests, fail to yield meaningful consolidation settlement data in any case. Where the subsoil strata are essentially non-cohesive in character, the bias is most definitely towards field tests. The data from field tests is used in empirical, but time-tested correlations to predict settlement of foundations. The field tests commonly used in subsurface investigation are:

Penetrometer test

Pressuremeter test

Vane shear testPlate load test

Geophysical methods

Penetrometer Tests :

Standard penetration test (SPT)

Static cone penetration test (CPT)

Dynamic cone penetration test (DCPT) Standard penetration test

The standard penetration test is carried out in a borehole, while the DCPT and SCPT are carried out without a borehole. All the three tests measure the resistance of the soil strata to penetration by a penetrometer. Useful empirical correlations between penetration resistance and soil properties are available for use in foundation design.

This is the most extensively used penetrometer test and employs a split-spoon sampler, which consists of a driving shoe, a split-barrel of circular cross-section which is longitudinally split into two parts and a coupling. IS: 2131-1981 gives the standard for carrying out the test.

Procedure

1. The borehole is advanced to the required depth and the bottom cleaned.
2. The split-spoon sampler, attached to standard drill rods of required length is lowered into the borehole and rested at the bottom
3. The split-spoon sampler is driven into the soil for a distance of 450mm by blows of a drop hammer (monkey) of 65 kg falling vertically and freely from a height of 750 mm. The number of blows required to penetrate every 150 mm is recorded while driving the sampler. The number of blows required for the last 300 mm of penetration is added together and recorded as the N value at that particular depth of the borehole. The number of blows required to effect the first 150mm of penetration, called the seating drive, is disregarded. The split-spoon sampler is then withdrawn and is detached from the drill rods. The split-barrel is disconnected from the cutting shoe and the coupling. The soil sample collected inside the split barrel is carefully collected so as to

preserve the natural moisture content and transported to the laboratory for tests. Sometimes, a thin liner is inserted within the split-barrel so that at the end of the SPT, the liner containing the soil sample is sealed with molten wax at both its ends before it is taken away to the laboratory. The SPT is carried out at every 0.75 m vertical intervals in a borehole. This can be increased to 1.50 m if the depth of borehole is large. Due to the presence of boulders or rocks, it may not be possible to drive the sampler to a distance of 450 mm. In such a case, the N value can be recorded for the first 300 mm penetration. The boring log shows refusal and the test is halted if

50 blows are required for any 150mm penetration

100 blows are required for 300m penetration

10 successive blows produce no advance.

- **Precautions**

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

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

The drop hammer must be of the right weight and the fall should be free, frictionless and vertical. The SPT is carried out at every 0.75 m vertical intervals in a borehole. This can be increased to 1.50 m if the depth of borehole is large. Due to the presence of boulders or rocks, it may not be possible to drive the sampler to a distance of 450 mm. In such a case, the N value can be recorded for the first 300 mm penetration. The boring log shows refusal and the test is halted if

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

The drop hammer must be of the right weight and the fall should be free, frictionless and vertical. The height of fall must be exactly 750 mm. Any change from this will seriously affect the N value.

The bottom of the borehole must be properly cleaned before the test is carried out. If this is not done, the test gets carried out in the loose, disturbed soil and not in the undisturbed soil. When a casing is used in borehole, it should be ensured that the casing is driven just short of the level at which the SPT is to be carried out. Otherwise, the test gets carried out in a soil plug enclosed at the bottom of the casing.

When the test is carried out in a sandy soil below the water table, it must be ensured that the water level in the borehole is always maintained slightly above the ground water level. If the water level in the borehole is lower than the ground water level, ‘quick' condition may develop in the soil and very low N values may be recorded. In spite of all these imperfections, SPT is still extensively used because the test is simple and relatively economical.

it is the only test that provides representative soil samples both for visual inspection in the field and for natural moisture content and classification tests in the laboratory. SPT values obtained in the field for sand have to be corrected before they are used in empirical correlations and design charts. IS: 2131-1981 recommends that the field value of N be corrected for two effects, namely, (a) effect of overburden pressure, and (b) effect of dilatancy. (a) Correction for overburden pressure

Several investigators have found that the penetration resistance or the N value in a granular soil is influenced by the overburden pressure. Of two granular soils possessing the same relative density but having different confining pressures, the one with a higher confining pressure gives a higher N value. Since the confining pressure (which is directly proportional to the overburden pressure) increases with depth, the N values at shallow depths are underestimated and the N values at larger depths are overestimated. To allow for this, N values recorded from field tests at different effective overburden pressures are corrected to a standard effective overburden pressure.

Static cone penetration test At field SCPT is widely used of recording variation in the in-situ penetration resistance of soil in cases where in-situ density is disturbed by boring method & SPT is unreliable below water table. The test is very useful for soft clays, soft silts, medium sands & fine sands.

Procedure

By this test basically by pushing the standard cone at the rate of 10 to 20 mm/sec in to the soil and noting the friction, the strength is determined.

After installing the equipment as per IS-4968, part III the sounding rod is pushed in to the soil and the driving is operated at the steady rate of 10 mm/sec approximately so as to advance the cone only by external loading to the depth which a cone assembly available.

For finding combine cone friction resistance, the shearing strength of the soil q_s , and tip resistance q_c is noted in gauge & added to get the total strength

LimitationsThis test is unsuitable for gravelly soil & soil for having SPT N value greater than 50. Also in dense sand anchorage becomes to cumbersome & expensive & for such cases Dynamic SPT can be used. This test is also unsuitable for field operation since erroneous value obtained due to presence of brick bats, loose stones etc.

Geophysical exploration General Overview Geophysical exploration may be used with advantage to locate boundaries between different elements of the subsoil as these procedures are based on the fact that the gravitational, magnetic, electrical, radioactive or elastic properties of the different elements of the subsoil may be different. Differences in the gravitational, magnetic and radioactive properties of deposits near the surface of the earth are seldom large enough to permit the use of these properties in exploration work for civil engineering projects. However, the resistivity method based on the electrical properties and the seismic refraction method based on the elastic properties of the deposits have been used widely in large civil engineering projects.

Different methods of geophysical explorations 1 Electrical resistivity method:

Electrical resistivity method is based on the difference in the electrical conductivity or the electrical resistivity of different soils. Resistivity is defined as resistance in ohms between the opposite phases of a unit cube of a material.

$\rho = \left(\frac{RA}{L} \right)$

ρ is resistivity in ohm-cm,

R is resistance in ohms,

A is the cross sectional area (cm^2),

L is length of the conductor (cm).

The resistivity values of the different soils are listed in table 1.4

Material	Resistivity (Ω -cm)
Massive rock	> 400
Shale and clay	1.0
Seawater	0.3
Wet to moist clayey soils	1.5 - 3.0

Table 1.4 : Resistivity of different materials

- **Procedure**

The set up for the test is given in figure 1.13. In this method, the electrodes are driven approximately 20cms in to the ground and a dc or a very low frequency ac current of known magnitude is passed between the outer (current) electrodes, thereby producing within the soil an electrical field and the boundary conditions. The electrical potential at point C is V_c and at point D is V_d which is measured by means of the inner (potential) electrodes respectively.

$$V_c = \frac{I\rho}{2\Pi}\left(\frac{1}{r_1} - \frac{1}{r_2}\right) \text{---------(1.1.1)} \qquad V_D = \frac{I\rho}{2\Pi}\left(\frac{1}{r_3} - \frac{1}{r_4}\right) \text{---------(1.1.2)}$$

where,

ρ is resistivity,

I is current,

r_1, r_2, r_3 and r_4 are the distances between the various electrodes as shown in fig. 1.13.

Potential difference between C and D = $V_{CD} = V_C - V_D = \frac{I\rho}{2\Pi}\left[\left(\frac{1}{r_1} - \frac{1}{r_2}\right) - \left(\frac{1}{r_3} - \frac{1}{r_4}\right)\right]$--------- (1.1.3)

$$\rho = \frac{2\Pi V_{CD}}{I}\left[\frac{1}{\left(\frac{1}{r_1} - \frac{1}{r_2}\right) - \left(\frac{1}{r_3} - \frac{1}{r_4}\right)}\right]$$ --------- (1.1.4) If $r_1 = r_4 = (r_2/2) = (r_3/2)$ then resistivity is given as, $\rho = \frac{2\Pi R r_1}{I}$ ---------(1.1.5)

where ,

Resistance $R = (V_{CD}/I)$

Thus, the apparent resistivity of the soil to a depth

approximately equal to the spacing r_1 of the electrode can be computed. The resistivity unit is often so designed that the apparent resistivity can be read directly on the potentiometer.

In "resistivity mapping" or "transverse profiling" the electrodes are moved from place to place without changing their spacing, and the apparent resistivity and any anomalies within a depth equal to the spacing of the electrodes can thereby be determined for a number of points.

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Seismic refraction method **General** This method is based on the fact that seismic waves have different velocities in different types of soils (or rock) and besides the wave refract when they cross boundaries between different types of soils. In this method, an artificial impulse are produced either by detonation of explosive or mechanical blow with a heavy hammer at ground surface or at the shallow depth within a hole. These shocks generate three types of waves. Longitudinal or compressive wave or primary (p) wave, Transverse or shear waves or secondary (s) wave, Surface waves.

It is primarily the velocity of longitudinal or the compression waves which is utilized in this method. The equation for the velocity of the p-waves (V_c) and s-waves (V_s) is given as,

$$V_c = \sqrt{\frac{E(1-\mu)}{(1+\mu)(1-2\mu)\rho}}$$ ------- (1.2.1) $$V_s = \sqrt{\frac{E}{2\rho(1+\mu)}}$$ ------- (1.2.2)

Where,

E is the dynamic modulus of the soil,

μ is the Poisson's ratio,

ρ is density and,

G is the dynamic shear modulus.

v These waves are classified as direct, reflected and refracted waves. The direct wave travel in approximately straight line from the source of impulse. The reflected and refracted wave undergoes a change in direction when they encounter a boundary separating media of different seismic velocities (Refer fig. 1.19). This method is more suited to the shallow explorations for civil engineering purpose. The time required for the impulse to travel from the shot point to various points on the ground surface is determined by means of geophones which transform the vibrations into electrical currents and transmit them to a recording unit or oscillograph, equipped with a timing mechanism. Assumptionshyj

METHODS OF ANALYSIS

LIMIT EQUILIBRIUM

The so-called limit equilibrium method has traditionally being used to obtain approximate solutions for the stability problems in soil mechanics. The method entails a assumed failure surface of various simple shapes—plane, circular, log spiral. With this assumption, each of the stability problems is reduced to one of finding the most dangerous position of the failure or slip surface of the shape chosen which may not be particularly well founded, but quite often gives acceptable results. In this method it is also necessary to make certain assumptions regarding the stress distribution along the failure surface such that the overall equation of equilibrium, in terms of stress resultants, may be written for a given problem. Therefore, this simplified method is used to solve various problems by simple statics.

Although the limit equilibrium technique utilizes the basic concept of upper-bound rules.

Of Limit Analysis, that is, a failure surface is assumed and a least answer is sought, it does not meet the precise requirements of upper bound rules, so it is not a upper bound. The method basically gives no consideration to soil kinematics, and equilibrium conditions are satisfied in a limited sense. It is clear then that a solution obtained using limit equilibrium method is not necessarily upper or lower bound. However, any upper-bound limit analysis solution will be obviously limit equilibrium solution.

INTRODUCTION

Partly for the simplicity in practice and partly because of the historical development of deformable of solids, the problems of soil mechanics are often divided into two distinct groups – the stability problems and elasticity problems. The stability problems deal with the conditions of ultimate failure of mass of soil. Problems of earth pressure, bearing capacity, and stability of slopes most often are considered in this category. The most important feature of such problems is the determination of the loads which will cause the failure of the soil mass. Solutions of these problems are done using the *theory of perfect elasticity***.** The elasticity problems on the other hand deal with the stress or deformation of the soil where no failure of soil mass is involved. Stresses at points in a soil mass under the footing, or behind a retaining wall, deformation around tunnels or excavations, and all settlement problems belong to this category. Solutions to these problems are obtained by using the *theory of linear elasticity*.

Intermediate between the elasticity and stability problems are the problems mentioned above are the problems known as *progressive failure*. Progressive failure problems deal with the elastic- plastic transition from the initial linear elastic state to the ultimate failure state of the soil by plastic flow. The following section describes some of the methods of analysis which are unique with respect to each other.

DIFFERENT METHODS OF ANALYSIS

There are basically four methods of analysis:

- Limit Equilibrium.
- Limit Analysis.
- Method of Characteristics.
- Finite Element / Discrete Element Method. **THEOREMS**

There are two theorems which are used for the various analyses. Some follow one theorem while some methods of analysis follow the other. They are the upper bound and the lower bound theorems.

In the **Upper bound theorem** , loads are determined by equating the external work to the internal work in an assumed deformation mode that satisfies:

Boundary deformation pattern.

Strain and velocity compatibility conditions.

These are kinematically admissible solutions. This analysis gives the maximum value for a particular parameter.

In the **Lower bound theorem** , loads are determined from the stress distribution that satisfies:

Stress equilibrium conditions.

Stress boundary conditions.

Nowhere it violates the yield condition.

These are statically admissible solutions. This analysis gives the minimum value for a particular parameter.

However by assuming different failure surfaces the difference between the values obtained the upper and lower bound theorems can be minimized.

Rankine earth pressure $\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{xy}}{\partial y} = X = \gamma$ ----------(3) where γ is the unit weight of the soil $\frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau_{xy}}{\partial x} = Y = 0$ ----------(4)

$\partial \sigma_y, \partial \tau_{xy} = 0$ along a horizontal plane.

at a depth *x*, integrating equation (3) and (4),

$$\sigma_x = \gamma x + C$$

$$\tau_{xy} = D$$

Boundary conditions:

if there is no surcharge, C=0, D=0 at x=0.

$$\tau_{xy} = \sigma \sin \phi \sin 2\theta = 0$$

$$\sigma \neq 0, \phi = 0, \sin 2\theta = 0.$$

Hence $\theta = 0$ (active conditions) or $\theta = \frac{\pi}{2}$ (passive conditions)

This implies that in passive case, $\theta = \frac{\pi}{2}$ and in active case $\theta = 0$ .where θ is the inclination of the major principle stress with the x direction.

Determination of earth pressure coefficients

$$\sigma_x = \gamma x = \sigma(1 + \sin \phi \cos 2\theta)$$

$\sigma = \frac{\gamma x}{1 + \sin \phi}$ (for active case, $\theta = 0$)

$$\sigma_y = \sigma(1 - \sin \phi \cos 2\theta)$$

$= \frac{\gamma x}{1 + \sin \phi}(1 + \sin \phi)$ ---------(5)

$\sigma_y = K_a \gamma x$ ---------(6) from eqn(5) and (6), coefficient of active earth pressure $K_a = \dfrac{1-\sin\phi}{1+\sin\phi}$ similarly, in the passive case , $\theta = \dfrac{\pi}{2}$

$\sigma_x = \sigma(1-\sin\phi)$ ----------(7)

$\sigma_y = \sigma(1+\sin\phi)$ ----------(8)

from eqn(7) and (8), coefficient of passive earth

pressure $K_p = \dfrac{1+\sin\phi}{1-\sin\phi}$ **Inclination of failure plane**

The failure planes at particular plane will make an angle of $\pm\mu$ with the direction of major principal stress.

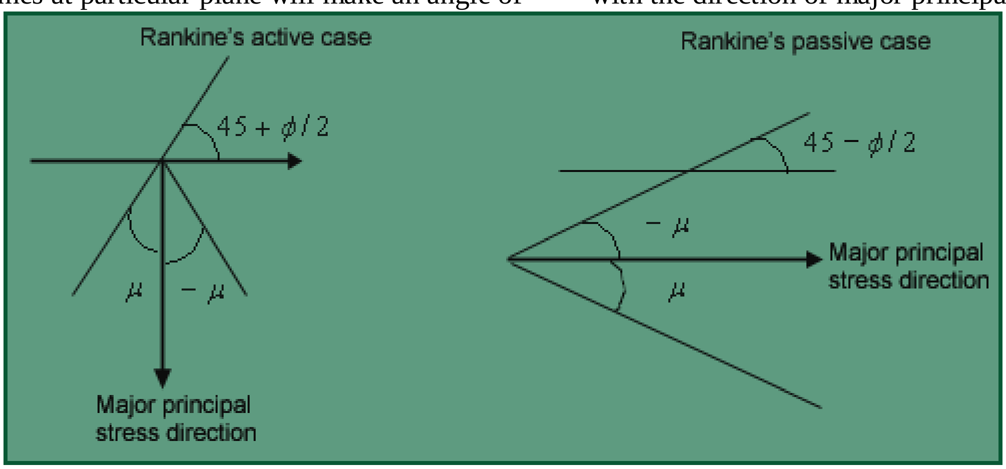
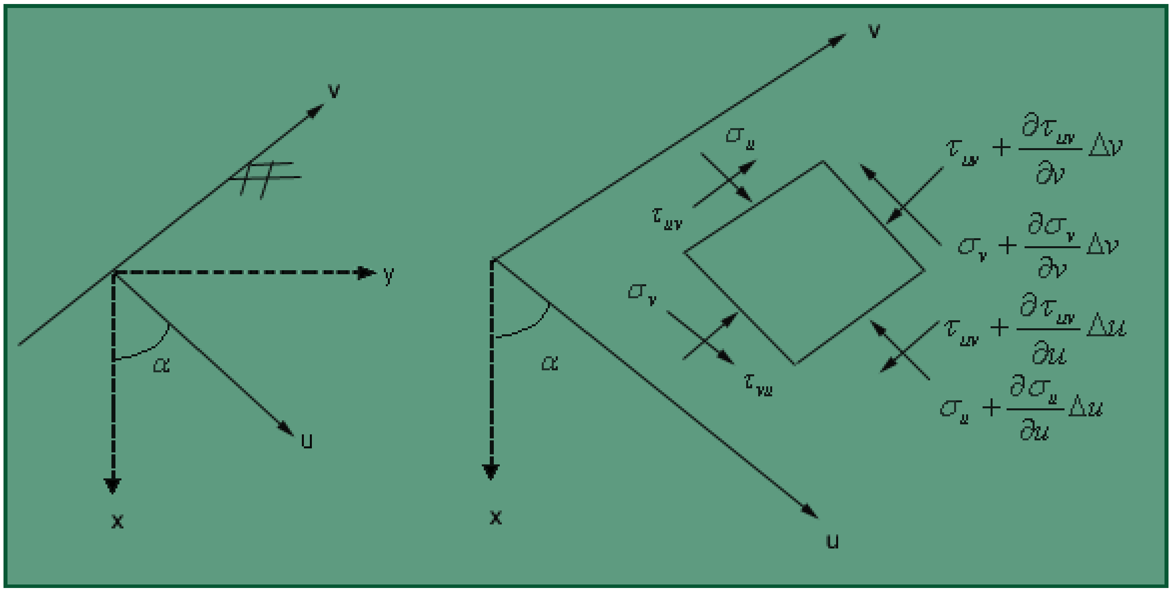


Fig .3.7 Inclination of failure planes

Inclined Ground



Considering the forces in the u and v directions,

$$\frac{\partial \sigma_u}{\partial u} + \frac{\partial \tau_{uv}}{\partial v} = \gamma \cos \alpha$$

$$\frac{\partial \sigma_v}{\partial v} + \frac{\partial \tau_{uv}}{\partial u} = -\gamma \sin \alpha$$

$\sigma_u = \gamma u \cos \alpha$ ---------(9) $\tau_{uv} = -\gamma u \sin \alpha$ ---------(10)

dividing eqn 9 by 10 and simplifying ,

$$\frac{\gamma u \cos \alpha}{-\gamma u \sin \alpha} = \frac{1 + \sin \phi \cos 2\theta}{\sin \phi \sin 2\theta}$$

$$\sin \alpha \cos 2\theta + \sin 2\theta . \cos \alpha = \frac{\sin \alpha}{\sin \phi}$$

$$\sin(2\theta + \alpha) = \frac{-\sin \alpha}{\sin \phi}$$

thus, $$\theta = \frac{1}{2}\left[\sin^{-1}\left(\frac{-\sin \alpha}{\sin \phi}\right) - \alpha\right]$$