PNS SCHOOL OF ENGINEERING & TECHNOLOGY

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Lecture Note on GEOTECH ENGINEERING

(3RD Semester)

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DEPARTMENT OF CIVIL ENGINEERING

module-1

INTRODUCTION

Preliminary definitions and relationships

A soil mass is a three phase system consisting of soil particles water and air.The void space between the soil grains is filled partly with water and partly with air.However if we take a dry soil mass,the voids are filled with air only.In case of perfectly saturated soil the voids are completely filled with water.The properties depend upon the relative percentage of these constituents their arrangement and a verity of other factors.

Total volume V of soil mass consist of volume of air ,volume of waterVw,volume of solid Vs.The volume of voids is therefore equal to volume of air plus volume of water.The weight of air is considered to be negligible.Hence the weight of total voids is equal to the weight of waterWw.The weight of solids is represented by Wd.The total weight of moist sample is therefore equal to Ww+Wd.

Water content,Density and unit weights

Water content

The water content W also called moisture content, is defined as the ratio of weight of water Ww to the weight of solids in a given mass of soil.

W =Ww/Wd*100

Density of soil

The density of soil is defined as the mass of soil per unit volume.

1.Bulk density

The bulk density or moist density is the total maas M of soil per unit of its total volume.

P = M/V.It is expressed in terms of g/cm3 or kg/m3.

2.Dry density(Pd)

The dry density is the mass of solids per unit of total volume of soil mass.

P =Md/V

3.Density of solids(Ps)

The density of soil solids is the mass of soil solids per unit volume of solids

Ps =Md/Vs

4.Saturated density(P sat)

When the soil mass is saturated its bulk density is called saturated density.Thus saturated density is the ratio of total soil mass of saturated sample to its total volume.

5.Submerged density(p")

The submerged density is the submerged mass of soil solids per unit of total volume of soil mass.

P" =(Md)sub/V

Unit weight of soil mass

The unit weight of soil mass is defined as its weight per unit volume.

1.Bulk unit weight(Y)

The bulk weight or moist unit weight is the total weight of a soil mass per unit of its total volume

Y =W/V

2.Dry unit weight(Yd)

The dry unit weight is the weight of solids per unit of its total volume of soil.

Yd =Wd/V

3.Unit weight of solids(Ys)

The unit weight of of soil solids is the weight of soil solids Wd per unit volume of solids

$Ys = Wd/Vs$

4.Saturated unit weight(Ysat)

When the soil mass is saturated ,its bulk unit weight is called saturated unit weight.

5.Submerged unit weight(Y')

The submerged unit weight Y' is the submerged weight of soil solids per unit of total volume of soil mass.

$Y' = (Wd)sub/V$, $Y' = Y\ sat - Yw$

Specific gravity

Specific gravity G is defined as the ratio of the weight of given volume of soil solids at a given temperature to the weight of an equal volume of distilled water at that temperature both weights are taken in air

$G = Ys/Yw$

Void ratio,porosity and degree of saturation

Void ratio

Void ratio e of a given soil sample is the ratio of volume of voids to the volume of soil solids in the given soil mass

$e = Vv/Vs$

Porosity

The porosity of a given soil sample is the ratio of volume of voids to the total volume of soil mass

$n = Vv/V$

The void ratio e is generally expressed as a fraction,while the porosity n is expressed in percentage.

$n = e/1+e$

$e = n/1-n$

Degree of saturation

Degree of saturation S is defined as the ratio of volume of water present in a given soil mass to the total volume of voids in it.

$S = V_w/V_v$

Percentage air voids

Percentage air voids n_a is defined as the ratio of volume of air voids to the volume of soil mass and is expressed as percentage

$N_a = V_a/V*100$

Air content

The air content a_c is defined as the ratio of volume of air voids to the volume of voids.

$A_c = V_a/V_v$

$V_a = V_v-V_w$

$A_c = 1-V_w/V_v = 1-S$

Relative density

The relative density is defined as the ratio of difference between the void ratio of soil in loosest state e max and its natural void ratio e to the difference between the void ratios in the loosest state and densest state.

$I_D = e_{max}-e/e_{max}-e_{min}$

FUNCTIONAL RELATIONSHIPS

1.Relation between e ,G,w,and S

ew represents the volume of water

e represents the volume of voids and volume of solids equal to unity

$S = Vw/Vv = ew/e$

$ew = eS$

for fully saturated soil sample $ew = e$

$w = Ww/Wd = ew * Yw/Ys*1$

but $G = Ys/Yw$, or $Ys = GYw$

$w = ew*Yw/G*Yw = ew/G$

$e = Wg/S$

for fully saturated soil $S = 1$ and $w = w\ sat.$

$e = w\ sat* G$

2. Relation between e, S and n a

$na = Va/V$

$Va = Vw-Vv = e - ew$

$V = Vs+Vv = 1+e$

$na = e - ew/1+e$ but $ew = Es$

$na = e(1-S)/1+e$

3.Relation between na,ac and n

$ac = Va/Vv, n = Vv/V$

$na = Va/V = n*ac$

4.Relation between Yd,G, and e

$Yd = Wd/V , Yd = Ys*Vs/V$

$Vs = 1$ and $V = 1+e$

$Yd = Ys*1/1+e$,but$Ys = G*Yw$

$Yd = G*Yw/1+e$

5.Relation between Ysat,G,and e(or n)

Ysat =Total weight of saturated soil/Total volume of soil

$= Wsat/V = Wd + Ww/V = Ys*Vs+Yw*Vw/V$

$Vs = 1$,$Vw = e$,$V = 1+e$

$Ysat = Ys *1 + Yw*e/1+e = G*Yw + Yw*e/1+e$

$Vs = 1-n$,$Vw = n$, $V = 1$

$Y sat = Ys (1-n) + Yw*n/1$

$Ysat = G*Yw(1-n) + Yw*n$

6. Relation between Y,G,e and S

$Y = W/V = Ys*Vs + Yw*Vw/V$

For partially saturated sample we have ,$Vs = 1$,$Vw = ew$,$V = 1+e$

$Y = Ys*1+Yw*ew/1+e$

But $Ys = G* Yw$ and $ew = es$

$Y = G*Yw + Yw*es/1+e = (G + es)Yw/1+e$

When the soil is perfectly dry $S=0$

$Yd = G* Yw/1+e$

Similarly when $S=1$, Y becomes

$Y sat = (G + e)Yw/1+e$

7.Relation between Y',G and e

$Y' = Y sat - Y w$

=(G +e)Yw/1+e –Yw

Y' = (G-1)Yw/1+e

8.Relation between Yd,Y and w

W = Ww/Wd

Hence 1+w = Ww+Wd/Wd =w/Wd

Wd = W/1+w

Now Yd = Wd/V =w /(1+w)V

Yd = Y/1+w

9.Relation between Y' , Yd and n

(Wd) sub = 1*Ys -1*Yw =G*Yw – Yw

(Wd)sub =(G-1)Yw and V=1+e

Y' =(Wd)sub/V =(G-1)Yw/1+e

Y' =G*Yw/1+e –Yw/1+e

But G*Yw/1+e =Yd ,and 1/1+e =1-n

Y' =Yd-(1-n)Yw

10.Relation between Ysat,Y,Yd and S

Y =(G+es)Yw/1+e

Y =Yd +S(Ysat-Yd)

Example;A soil sample has a porosity of 40%.The specific gravity of solid is 2.70.calculate void ratio,drydensity,unit weight if the soil is 50% saturated,unit weight if the soil is completely saturated.

Answer;

a)n =40% =0.4,G =2.70

$e = n/1-n = 0.4/1-0.4 = 0.667$

b) $Yd = GYw/1+e = 2.7*9.81/1+0.667 = 15.89$

c) $e=Wg/S, w = Es/G = 0.667*0.5/2.70 = 0.124$

$Yd = 15.89 KN/M3$

$Y = Yd(1+w) = 15.89*1.124 = 17.85 KN/M3$

d)when the soil is fully saturated

$e = Wsat*G$

$Wsat = e/G = 0.667/2.70 = 0.247$

$Ysat = Yd(1+Wsat) = 15.89*1.247 = 19.81 KN/M3$

QUESTIONS

1.What is soil? **2*8**

A soil mass is a three phase system consisting of solid particle ,water and air. The void space between the soil grains is filled partly with air and partly with water.

2.What is water content?

The water content also called moisture content is defined as the ratio between weight of water to weight of solids in a given mass of soil.

3.What is density of soil?

The density of soil is defined as the mass of soil per unit volume.

4.What is saturated density?

When the soil mass is saturated its bulk density is called saturated density.Thus saturated density is the ratio of total soil mass of saturated sample to its total volume.

5.What is unit weight of soil mass?

The unit weight of soil mass is defined as its weight per unit volume.

6.What is specific gravity?

Specific gravity is defined as the ratio between weight of given volume of soil solids at a given temperature to the weight of equal volume of distilled water at that temperature both weights are taken in air.

7.What is the relation between void ratio and porosity?

Void ratio of a soil sample is the ratio of volume of voids to the volume of soil solids in a given mass of soil where as porosity of a soil sample is the ratio of volume of voids to the total volume of soil mass.

$n = e/1+e, e = n/1-n$

8.What is relative density?

The relative density is defined as the ratio of difference between the void ratio of soil in loosest state to its natural void ratio to the difference between the void ratio in the loosest and densest state.

LONG QUESTION 2*10

1.What is functional relationships?Find the relation between e, G,w and S?

2.What is void ratio, porosity and degree of saturation discuss briefly?

DETERMINATION OF INDEX PROPERTIES

The index properties include the determination of 1)water content

2)specific gravity,3)particle size distribution,4)consistency limits,5)in-situ density,6)density index

Water content;

Water content of a soil can be determined by the following methods

1.oven drying method

2.sand bath method

3.alcohol method

4.calcium carbide method

5.pycnometer method

6.radiation method

7.torsion balance method

OVEN DRYING METHOD

This is the most accurate method of determining the water content.A specimen of soil sample is kept in a clean container and put in a thermostatically controlled oven to maintain the temperature between 105"c to110'c.For complete drying sandy soils take about four hours and fat clays take about 14-16 hours.usually the sample is kept for about 24 hours in the oven .water content is calculated from the expression

W =M2-M3/M3-M1*100

M1=Mass of container with liquid

M2 =mass of container with liquid and wet soil

M3=mass of container with liquid and dry soil

2.SAND BATH METHOD

This is the field method of determining rough value of water content where the facility of oven is not available.The container with the soil is placed on the sand bath.The sand bath is heated over a kerosene stove.The soil becomes dry with in ½ to 1 hour.however higher temperature may break the crystalline structure of soil.This method should not be used for organic soils or for soil having higher percentage of gypsum

3.ALCOHOL METHOD

This is a curde field method,The weight soil sample is kept in a evaporating dish and mixed sufficient quantity of methylated sprit.

W = M2-M3/M3-M1*100

M1 =mass of empty dish

M2 =mass of dish +wet soil

M3= mass of dish +dry soil

4.CALCIUM CARBIDE METHOD

In this method,6g of wet soil sample is placed in an air-tight container and is mixed with sufficient quantity of fresh calcium carbide powder.The mixture is shaken vigorously.The acetylene gas exert pressure on a sensitive diaphagram placed at the end of the container.The dial gauge located at the diaphragm reads the water content directly.

5.PYCNOMETER METHOD

This is also a quick method of determining the water content of those soils whose specific gravity G is accurately known.pycnometer is a large size density bottle of about 900ml capacity.a conical brass cap having a 6 mm diameter hole at the top is screwed to the open end of the pycnometer.A rubber washer isplaced between conical cap and rim of the bottle so that there is no leakage of water.

SPECIFIC GRAVITY

The specific gravity of soil solid is determined by

1.a 50ml density bottle

2.a 500ml flask

3.apycnometer

The density bottle method is most accurate and is suitable for all types of soil.The flask or pycnometer is used only for coarse grained soils.The density bottle method is the standard method used in laboratory

QUESTIONS

1.What is index properties?

2.What are the various methods of determining water content?

3.what is specific gravity?

Particle size distribution

The percentage of various size of particle in a given dry sample is found by a particle size analysis or mechanical analysis.The mechanical analysis is performed in two stages

1.sieve analysis

2.sedimentation analysis

Sieve analysis

The sieve size are given in terms of number of openings per inch,The number of openings per square inch is equal to the square of the no of sieve.Inindian standard the sieve are designated by the size of the aperature in mm

A complete sieve analysis can be divided into two parts

1.coarse analysis

2.fine analysis

An oven dried sample of soil is separated into two fractions by sieving it through a 4.75 mm size is termed as gravel fraction and kept for the coarse analysis.while the portion passing through it is subjected to fine analysis.

Sedimentation analysis

In sedimentation analysis the soil fraction finer than 75 micron size is kept in suspension in liquid medium .The analysis is based on stokes law,according to which the velocity at which grains settle out of suspension all other factors being equal,is depend upon the shape weight and size of grains.

V = 1/18D2*Ys-Yw/n

D =diameter of spherical particle

Ys =unit weight of particle(Kn/m3)

Yw=unit weight of water/liquid

n = viscosity

v = terminal velocity

Particle size distribution curve

The results mechanical analysis are plotted to get a particle size distribution curve with the percentade finer N as ordinate and particle diameter as the abscissa.A Particle size distribution curve gives us an idea about the type and gradation of the soil.A curve situated higher up or to the left represents a relatively fine grained soil while the curve situated to the right represents a course grained soil.

A soil is said to be well graded when it has good representation of particle of all sizes.on the other hand a soil is said to be poorly graded if it has excess of certain particle and deficience of other particle.If it has most of the particles of about same size is known as uniformly graded soil.If some intermediate particle are missing such soil is known as gap graded soil or skip graded

For coarse grained soil certain particle sizes such as D10,D30,D60 are important.D10 represents a size in mm such that 10% of the particle are finer than this size .D60 are 60% of the particle are finer than this size.

D10 =Effective size

Uniformity coefficient = Cu =D60/D10

CO-efficient of curvature Cc =(D30)2/D10*D60

Consistency of soils

By consistency is meant the relative easy with which the soil can be deformed.

Atterbergdevided the entire range from liquid to solid state into four stages.

1.Liquid state

2.plastic state

3.semi-solid state

4.solid state

Liquid limit(Wl)

Liquid limit is the water content corresponding to the arbitrary limit between liquid and plastic state of consistency of soil.It is defined as the minimum water content at which the soil is still in the liquid state.

Plastic limit(Wp)

Plastic limit is the water content corresponding to an arbitrary limit between plastic and semi solid state.it is defined as the minimum water content at which the soil just begin to crumble when rolled into a thread approximately 3mm in diameter.

Shrinkage limit(Ws)

Shrinkage limit is defined as the minimum water content at which a reduction in water content will not cause a decrease in volume of soil mass.It is the lowest water content at which a soil can still be completely saturated.

Plasticity index (Ip)

The range of consistency within which a soil exibits plastic properties is called plastic range and indicated by plasticity index.The plasticity index is defined as the numerical difference between the liquid limit and plastic limit of soil.

$Ip = Wl - Wp$

Consistency index(Ic)

The consistency is defined as the ratio of liquid limit minus the natural water content to plasticity index of soil.

$Ic = Wl - W/Ip$

Liquidity index(IL)

The liquidity index is the ratio expressed as the percentage of the natural water content of a soil mass to its plastic limit to its plasticity index.

IL = W1-W2/Ip

Toughness index(It)

The toughness index is defined as the ratio of plasticity index to flow index.

IT =Ip/If

Activity of clays

Activity is defined as the ratio between plasticity index to the percentage weight of solid particles of diameter smaller than two micron present in the soil.

Ac =Ip/Cw

Sensitivity of clays

It is defined as the ratio of its unconfined compression strength in undisturbed state to that in the remodeled state within chang in water content.

S =Qu(undisturbed)/Qu(remoulded)

Sensitivity of most clays generally falls in the range of 1 to8

Questions

1.what is particle size distribution?describe about sieve analysis and sedimentation analysis?

2.what is consistency of soil?

3.what is liquid limit?

4.what is plastic limit?

5.what is shrinkage limit?

6.what is plasticity index?

7.what is sensitivity of clays?

Soil Classification:
(Module-2)

INTRODUCTION:-

It is necessary to adopt a formal system of soil description and classification in order to describe the various materials found in ground investigation. Such a system must be meaningful and concise in an engineering context, so that engineers will be able to understand and interpret. It is important to distinguish between description and classification: Description of soil is a statement that describes the physical nature and state of the soil. It can be a description of a sample, or a soil in situ. It is arrived at by using visual examination, simple tests, observation of site conditions, geological history, etc. Classification of soil is the separation of soil into classes or groups each having similar characteristics and potentially similar behaviour. A classification for engineering purposes should be based mainly on mechanical properties: permeability, stiffness, strength. The class to which a soil belongs can be used in its description. The aim of a classification system is to establish a set of conditions which will allow useful comparisons to be made between different soils. The system must be simple. The relevant criteria for classifying soils are the size distribution of particles and the plasticity of the soil.

For measuring the distribution of particle sizes in a soil sample, it is necessary to conduct different particle-size tests. Wet sieving is carried out for separating fine grains from coarse grains by washing the soil specimen on a 75 micron sieve mesh. Dry sieve analysis is carried out on particles coarser than 75 micron. Samples (with fines removed) are dried and shaken through a set of sieves of descending size. The weight retained in each sieve is measured. The cumulative percentage quantities finer than the sieve sizes (passing each given sieve size) are then determined.

The concentration of particles remaining in the suspension at a particular level can be determined by using a hydrometer. Specific gravity readings of the solution at that same level at different time intervals provide information about the size The resulting data is presented as a distribution curve with grain size along x-axis (log scale) and percentage passing along y-axis (arithmetic scale). Sedimentation analysis is used only for the soil fraction finer than 75 microns. Soil particles are allowed to settle from a suspension. The decreasing density of the suspension is measured at various time intervals. The procedure is based on the principle that in a suspension, the terminal velocity of a spherical particle is governed by the diameter of the particle and the properties of the suspension. In this method, the soil is placed as a suspension in a jar filled with distilled water to which a deflocculating agent is added. The soil particles are then allowed to of particles that have settled down and the mass of soil remaining in solution. The results are then plotted between % finer (passing) and log size.

Indian Standard Soil Classification System:

Fine-grained soils are those for which more than 50% of the material has particle sizes less than 0.075 mm. Clay particles have a flaky shape to which water adheres, thus imparting the property of plasticity. A plasticity chart , based on the values of liquid limit (WL) and plasticity index (IP), is provided in ISSCS to aid classification. The 'A' line in this chart is expressed as IP = 0.73 (WL - 20)

Depending on the point in the chart, fine soils are divided into clays (C), silts (M), or organic soils (O). The organic content is expressed as a percentage of the mass of organic matter in a given mass of soil to the mass of the dry soil solids.Three divisions of plasticity are also defined as follows. Low plasticity WL< 35% Intermediate plasticity 35% < WL< 50% High plasticity WL> 50% The 'A' line and vertical lines at WL equal to 35% and 50% separate the soils into various classes. For example, the combined symbol CH refers to clay of high plasticity

Soil classification using group symbols is as follows:

Group Symbol

Classification Coarse soils

GW	Well-graded GRAVEL

GP	Poorly-graded GRAVEL
GM	Silty GRAVEL
GC	Clayey GRAVEL
SW	Well-graded SAND
SP	Poorly-graded SAND
SM	Silty SAND
SC	Clayey SAND

Fine soils

ML	SILT of low plasticity
MI	SILT of intermediate plasticity
MH	SILT of high plasticity
CL	CLAY of low plasticity
CI	CLAY of intermediate plasticity
CH	CLAY of high plasticity
OL	Organic soil of low plasticity
OI	Organic soil of intermediate plasticity
OH	Organic soil of high plasticity

Activity:

"Clayey soils" necessarily do not consist of 100% clay size particles. The proportion of clay mineral flakes (< 0.002 mm size) in a fine soil increases its tendency to swell and shrink with changes in water content. This is called the activity of the clayey soil, and it represents the degree of plasticity related to the clay content. Activity = (Plasticity index) /(% clay particles by weight) Classification as per activity is: Activity Classification < 0.75 Inactive 0.75 - 1.25 Normal > 1.25 Active

Liquidity Index

In fine soils, especially with clay size content, the existing state is dependent on the current water content (w) with respect to the consistency limits (or Atterberg limits). The liquidity index (LI) provides a quantitative measure of the present state. Classification as per liquidity index is: Liquidity index Classification > 1 Liquid 0.75 - 1.00 Very soft 0.50 - 0.75 Soft 0.25 - 0. 50 Medium stiff 0 - 0.25 Stiff < 0 Semi-solid

Visual Classification

Soils possess a number of physical characteristics which can be used as aids to identification in the field. A handful of soil rubbed through the fingers can yield the following:

SAND (and coarser) particles are visible to the naked eye.

SILT particles become dusty when dry and are easily brushed off hands.

CLAY particles are sticky when wet and hard when dry, and have to be scraped or washed off hands.

QUESTIONS

1.WHAT IS THE PURPOSE OF SOIL CLASSIFICATION?

2.WHAT IS IS SOIL CLASSIFICATION SYSTEM?

3.WHAT IS COARSE GRAINED SOIL?

4.WHAT IS FINE GRAINED SOIL?

5.WHAT IS ACTIVITY OF SOIL MASS?

6.WHAT IS LIQUDITY INDEX OF SOIL?

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4.WHAT IS FINE GRAINED SOIL?

5.WHAT IS ACTIVITY OF SOIL MASS?

6.WHAT IS LIQUDITY INDEX OF SOIL?

PERMEABILITY AND SEEPAGE

PERMEABILITY OF SOIL

Definition of Permeability:

It is defined as the property of a porous material which permits the passage or seepage of water (or other fluids) through its interconnecting voids. A material having continuous voids is called permeable. Gravels are highly permeable while stiff clay is the least permeable, and hence such a clay may be termed impermeable for all practical purpose.

The study of seepage of water through soil is important for the following engineering problems:

1. Determination of rate of settlement of a saturated compressible soil layer.

2. Calculation of seepage through the body of earth dams and stability of slopes for highways.

3. Calculation of uplift pressure under hydraulic structure and their safety against pipin

4. Groundwater flow towards well and drainage of soil.

Darcy's Law (1856) of Permeability:

For laminar flow conditions in a saturated soil, the rate of the discharge per unit time is proportional to the hydraulic gradient.

q = kiA-----------------

v = q/A = ki …

Where q = discharge per unit time A = total cross-sectional area of soil mass, perpendicular to the direction of flow i = hydraulic gradient k = Darcy's coefficient of permeability v = velocity of flow or average discharge velocity

Coefficient of permeability (or) permeability

It is defined as the average velocity of flow that will occur through the total cross-sectional are of soil under unit hydraulic gradient. The coefficient of permeability is denoted as K. It is usually expressed as cm/sec (or) m/day (or) feet/day. When hydraulic gradient is unity, k is equal to V. Thus, the coefficient of permeability, or simply permeability is defined as the average velocity of flow that will occur through the total crosssectional area of soil under unit hydraulic gradient. Dimensions are same as of velocity, cm/sec.

Validity of Darcy's Law:

In accordance with the Darcy's Law, the velocity of flow through soil mass is directly proportion to the hydraulic gradient for laminar flow condition only. It is expected that the flow to be always laminar in case of fine-grained soil deposits because of low permeability and hence low velocity of flow. So v=ki Where i is the hydraulic gradient and k is a constant known as coefficient of permeability. However, in case of sands and gravels flow will be laminar up to a certain value of velocity for each deposit and investigations have been carried out to find a limit for application of Darcy's law. According to researchers, flow through sands will be laminar and Darcy's law is valid so long as Reynolds number expressed in the form is less than or equal to unity as shown below $vDa\gamma w\ \eta g$ ≤ 1Where v = velocity of flow in cm/sec Da = size of particles (average) in cm. It is found that the limiting value of Reynolds number taken as 1 is very approximate as its actual value can have wide variation depending partly on the characteristic size of particles used in the equation

FACTORS AFFECTING PERMEABILITY

1. Size of soil particle

2. Specific Surface Area of Soil Particle

3. Shape of soil particle

4. Void ratio

5. Soil structure

6. Degree of saturation

7. Water properties

8. Temperature

9. Adsorbed water

10. Organic Matter

1. Size of Soil Particle

Permeability varies according to size of soil particle. If the soil is coarse grained, permeability is more and if it is fine grained, permeability is low. The relation between coefficient of permeability (k) and particle size (D) can be shown from equation (1) as follows. $k\alpha D$ 2

2. Specific Surface Area of Particles

Specific surface area of soil particles also affects the permeability. Higher the specific surface area lower will be the permeability. $K\alpha$ 1/ $Specific\ Surface$

Area

3. Shape of Soil Particle

Rounded Particles will have more permeability than angular shaped. It is due to specific surface area of angular particles is more compared to rounded particles. **4. Void Ratio**

In general, Permeability increases with void ratio. But it is not applicable to all types of soils. For example, Clay has high void ratio than any other types of soil but permeability for clays is very low. This is due to, the flow path through voids in case of clays is extremely small such that water cannot permit through this path easily. The relation between coefficient of permeability and void ratio can be expressed as For Clay $k\alpha\ Ce$ 3/ 1+e Where, C = Shape of the flow path, e = Void ratio. For coarse grained soil, "C" can be neglected. Hence $k \propto e$ 3 /1+e

5. Soil Structure

Structure of any two similar soil masses at same void ratio need not be same. It varies according to the level of compaction applied. If a soil contains flocculated structure, the particles are in random orientation and permeability is more in this case. If the soil contains dispersed structure, the particles are in face to face orientation hence, permeability is very low. The permeability of stratified soil deposits also varies according to the flow direction. If the flow is parallel, permeability is more. If it is perpendicular, permeability is less

. 6. Degree of Saturation

Partially saturated soil contains air voids which are formed due to entrapped air or gas released from the percolating fluid or water. This air will block the flow path thereby reduces the permeability. Fully saturated soil is more permeable than partially saturated soil

7. Water Properties

Various properties of water or fluid such as unit weight and viscosity also effects the permeability. However, unit weight of water will not affect much since it does not change much with temperature. But when temperature is increased viscosity decreases rapidly. From equation (1), permeability increase when viscosity decreases. $k\alpha\ \gamma w$/u

8. Temperature

Temperature also affects the permeability in soils. The permeability is inversely proportional to the viscosity of the fluid. It is known that viscosity varies inversely to the temperature. Hence, Permeability is directly related to temperature. Greater the temperature, higher will be the permeability. That is the reason; seepage is more in summer seasons than in winter. $k\alpha\ 1/\ \mu\ \alpha\ temperature$

9. Adsorbed Water

Adsorbed water is the water layer formed around the soil particle especially in the case of finegrained soils. This reduces the size of the void space by about 10%. Hence, permeability reduces.

10. Organic matter

Presence of organic matter decreases the permeability. This is due to blockage of voids by the organic matter.

CONSTANT HEAD PERMEABILITY TEST

The constant head permeability test is a laboratory experiment conducted to determine the permeability of soil. The soils that are suitable for this test are sand and gravels. Soils with silt content cannot be tested with this method. The test can be employed to test granular soils either reconstituted or disturbed.

What is Coefficient of Permeability?

The coefficient of permeability, k is defined as the rate of flow of water under laminar flow conditions through a porous medium area of unit cross section under unit hydraulic gradient. The coefficient of permeability (k) is obtained from the relation k = $qL/\ Ah$ = $QL/\ Aht$

Where q= Discharge, Q = Total volume of water, t=time period, h= Head causing flow, L= Length of specimen, A= Cross-sectional area

FALLING HEAD PERMEABILITY TEST

Falling head permeability test is one of several techniques by which the permeability of soil is determined. It is used to evaluate the permeability of fairly less previous soil particularly for grained soil sample. Permeability is the measure of the ability of soil to allow water to flow its pores or voids. A falling head permeameter The soil sample is kept in a vertical cylinder of cross-sectional area A. A transparent stand pipe of cross sectional area, a, is attached to the test cylinder. The test cylinder is kept in a container filled with water, the level of which is kept constant by overflows. Before the commencement of the test the soil sample is saturated by allowing the water to flow continuously through the sample from the stand pipe. After saturation is complete, the stand pipe is filled with water up to a height of h0 and a stop watch is started. Let the initial time be t0. The time t1 when the water level drops from h0 to h1 is noted. The hydraulic conductivity k can be determined on the basis of the drop in head (h0 – h1) and the elapsed time (t1 – t0) required for the drop as explained below.

Let h be the head of water at any time t. Let the head drop by an amount dh in time dt. The quantity of water flowing through the sample in time dt from Darcy's law is dQ = $kiAdt$ = $k\ h\ L\ Adt$where, i = h/L the hydraulic gradient. The quantity of discharge dQ can be expressed as dQ = -adh Since the head decreases as time increases, dh is a negative can be equated $-adh\ h$ = kAd $/L$The discharge Q in time (t1 – t0) can be obtained by. Thereforecan be rearranged and integrated as follows $-a \int dh\ h\ h1\ h0$ = $KA\ L \int dt\ t1\ t0$ Or $aln\ h0\ h1$ = $KA\ L$ ($t1 - t0$The general expression for k is K = aL ($t1-t0$) $ln\ h0\ h1$ OR $\boldsymbol{K}$ = $\mathbf{2.3}\boldsymbol{aL}$ ($\boldsymbol{t1-t0}$) $\boldsymbol{log10\ h0/\ h1}$ ------------------

LONG QUESTIONS

1.WHAT IS PERMEABILITY?DEFINE DRACY"S LAW?

2.WHAT ARE THE FACTORS AFFECTING PERMEABILITY?DISCUSS IT?

3.DESCRIBE ABOUT FALLING HEAD PERMEABILITY TEST?

4.DESCRIBE ABOUT CONSTANT HEAD PERMEABILITY TEST?

COMPACTION

Introduction:

Compaction is the process of increasing the bulk density of a soil or aggregate by driving out air. For any soil, at a given compactive effort, the density obtained depends on the moisture content. An "Optimum Moisture Content" exists at which it will achieve a maximum density. Compaction is the method of mechanically increasing the density of soil. The densification of soil is achieved by reducing air void space. During compaction, air content reduces, but not water content It is not possible to compact saturated soil. It should be noted that higher the density of soil mass, stronger, stiffer, more durable will be the soil mass.

Hence, Compaction

1) Increases density

2) Increases strength characteristics

3) Increases load-bearing capacity

4) Decreases undesirable settlement

5) Increases stability of slopes and embankments

6) Decreases permeability

7) Reduces water seepage

8) Reduces Swelling & Shrinkage

9) Reduces frost damage

10) Reduces erosion damage

11) Develops high negative pore pressures (suctions) increasing effective stress

Factors affecting Compaction

1. Water Content
2. Amount of Compaction
3. Method of Compaction
4. Type of Soil
5. Addition of Admixtures

Effect of Water Content

1. With increase in water content, compacted density increases up to a stage, beyond which compacted density decreases.

2. The maximum density achieved is called MDD and the corresponding water content is called OMC.

3. At lower water contents than OMC, soil particles are held by the force that prevents the development of diffused double layer leading to low inter-particle repulsion

. 4. Increase in water results in expansion of double layer and reduction in net attractive force between particles. Water replaces air in void space

5. Particles slide over each other easily increasing lubrication, helping in dense packing.

6. After OMC is reached, air voids remain constant. Further increase in water, increases the void space, thereby decreasing dry density

Effect of Amount of Compaction

1. As discussed earlier, effect of increasing compactive effort is to increase MDD And reduce OMC (Evident from Standard & Modified Proctor’s Tests).

2. However, there is no linear relationship between compactive effort and MDD

Effect of Method of Compaction

The dry density achieved by the soil depends on the following characteristics of compacting method.

1. Weight of compacting equipment

2. Type of compaction

3. Area of contact of

4. Time of exposure

5. Each of these approaches will yield different compactive effort. Further, suitability of a particular method depends on type of soil.

Effect of Type of Soil

1. Maximum density achieved depends on type of

2. Coarse grained soil achieves higher density at lower water content and fine grained soil achieves lesser density, but at higher water content.

Effect of Addition of Admixtures

1. Stabilizing agents are the admixtures added to soil.

2. The effect of adding these admixtures is to stabilize the soil.

3. In many cases they accelerate the process of densification.

Types of field Compaction Equipment:

1. Smooth Wheeled Steel Drum Rollers

2. Pneumatic Tyred Rollers

3. Sheepsfoot Rollers

4. Impact Rollers

5. Vibrating Rollers

6. Hand Operated vibrating plate & rammer compactors

Smooth wheeled steel drum rollers

1. Capacity 20 kN to 200 Kn
2. Self propelled or towed
3. Suitable for well graded sand, gravel, silt of low plasticity
4. Unsuitable for uniform sand, silty sand and soft clay

Pneumatic Tyred Rollers

1. Usually two axles carrying rubber tyred wheels for full width of track.
2. Dead load (water) is added to give a weight of 100 to 400 kN.
3. Suitable for most coarse & fine soils
4. Unsuitable for very soft clay and highly variable soil.

Sheepsfoot Roller

1. Self propelled or towed
2. Drum fitted with projecting club shaped feet to provide kneading action.
3. Weight of 50 to 80 kN
4. Suitable for fine grained soil, sand & gravel with considerable fines

Impact Roller

1. Compaction by static pressure combined with impact of pentagonal roller.
2. Higher impact energy breaks soil lump and provides kneading action.

Vibrating Drum

1. Roller drum fitted with vibratory motion
2. . 2. Levels and smoothens ruts

Plate & Rammer Compactor:

It is used for backfilling trenches, smaller constructions and less accessible locations

Field Compaction Control

It is extremely important to understand the factors affecting compaction in the field and to estimate the correlation between laboratory and field compaction. Field compaction control depends on

(i) Placement wate rcontent,
Type of equipment for compaction
(ii) Lift thickness
(iii) (iv) Number of passes based on soil type & degree of c ompaction desired
Placement water content is the water content at which the ground is compacted in the field. It is desirable to compact at or close to optimum moisture content achieved in laboratory so as to increase the efficiency of compaction. However, in certain jobs the compaction is done at lower than or higher than OMC (by about 1– 2 %) depending on the desired function as detailed .

Characteristics of the compactor:

(1) Mass, size

(2) Operating frequency and frequency range

Characteristics of the soil:

(1) Initial density

(2) Grain size and shape

(3) Water content

Construction procedures:

(1) Number of passes of the roller

(2) Lift thickness

(3) Frequency of operation vibrator

(4) Towing speed

QUESTIONS

1.WHAT IS COMPACTION?

2.WHAT ARE THE FACTORS AFFECTING COMPACTION?

3.WHAT IS THE EFFECT OF WATER CONTENT ON COMPACTION?

4.WHAT IS THE EFFECT OF TYPE OF SOIL ON COMPACTION?

5.WHAT IS THE EFFECT ON ADDITION OF ADMIXTURE ON COMPACTION?

6.WHAT IS SMOOTH WHEEL TYPE ROLLER?

7.WHAT IS IMPACT ROLLER?

8.WHAT IS VIBRATING DRUM?

Consolidations

When a soil mass is subjected to a compressive force, its volume decreases. The property of the soil due to which a decrease in volume occurs under compressive force is known as the compressibility of soil.

The water is squeezed out of the clay over a long time (due to low permeability of the clay).

The compression of soil can occur due to:

1. Compression of solid particles and water in the voids
2. Compression and expulsion of air in the voids
3. Expulsion of water in the voids

The compression of saturated soil under a steady static pressure is known asconsolidation. It is entirely due to expulsion of water from the voids

INITIAL, PRIMARY AND SECONDARY CONSOLIDATION

Initial Consolidation:

When a load is applied to a partially saturated soil, a decrease in volume occurs due to expulsion and compression of air in the voids. A small decrease in volume occurs due to compression of solid particles. The reduction in volume of the soil just after the application of the load is known as initial consolidation or initial compression. For saturated soils, the initial consolidation is mainly due to compression of solid particles.

Primary Consolidation

After initial consolidation, further reduction in volume occurs due to expulsion of water from the voids. When a saturated soil is subjected to a pressure, initially all the applied pressure is taken up by water as an excess pore water pressure. A hydraulic gradient will develop and the water starts flowing out and a decrease in volume occurs. This reduction in volume is called as the primary consolidation of soil

Secondary Consolidation

The reduction in volume continues at a very slow rate even after the excess hydrostatic pressure developed by the applied pressure is fully dissipated and the primary consolidation is complete. The additional reduction in the volume is called as the secondary consolidation.

Granular soils have no cohesion.
$c = 0$ & $c' = 0$

For unconsolidated undrained test, in terms of total stresses, $\phi_u = 0$

For normally consolidated clays, $c' = 0$ & $c = 0$.

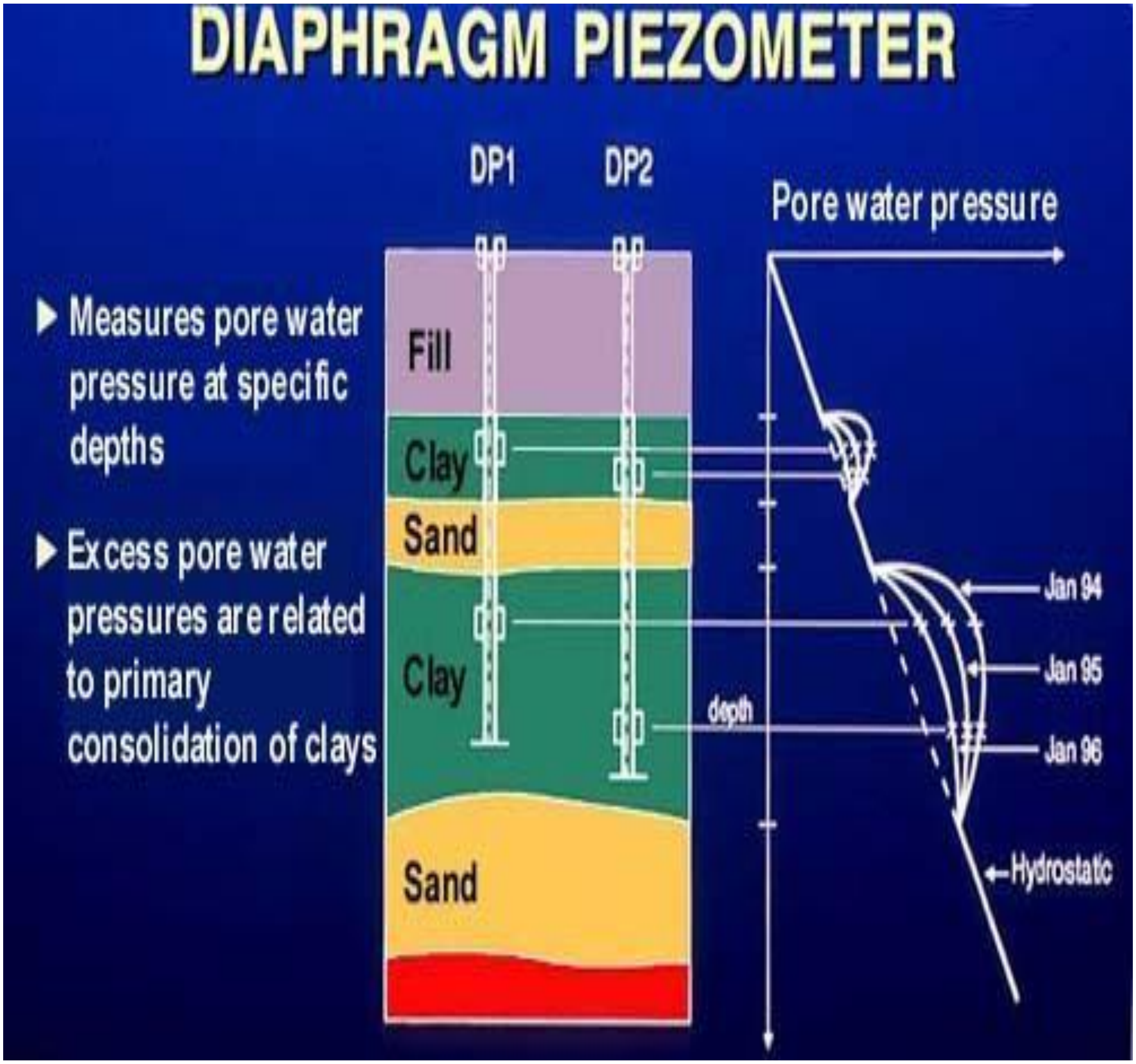


Fig: Shows Diaphragm piezometer

Questions

1. **What is consolidatioons ?**
2. **What is the distinction between compaction and consolidation ?**

SHEARING STRENGTH OF SOILS

Shear strength may be defined as the resistance to shearing stresses and a consequenttendency for shear deformation.

Soil derives its shearing strength from the following

1. resistance due to interlocking of particles
2. frictional resistance between the individual soil grains
3. adhesion between soil particles or cohesion

Necessity of studying Shear Strength of soils:

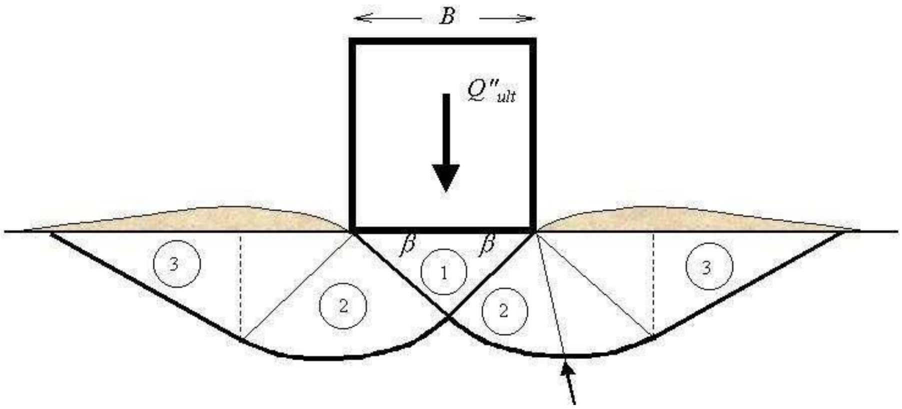
- Soil failure usually occurs in the form of "shearing" along internal surface withinthe soil. **Shear Strength**
- Thus, structural strength is primarily a function of shear strength.
- The strength of a material is the greatest stress it can sustain.
- The safety of any geotechnical structure is dependent on the strength of the soil.
- If the soil fails, the structure founded on it can collapse

Thus shear strength is "The capacity of a material to resist the internal and externalforces which slide past each other"

Significance of Shear Strength :

- Engineers must understand the nature of shearing resistance in order to analyze soilstability problems such as;

- Bearing capacity
- Slope stability



- Lateral earth pressure on earth-retaining structure

Fig: Shear Failure under Foundation Load

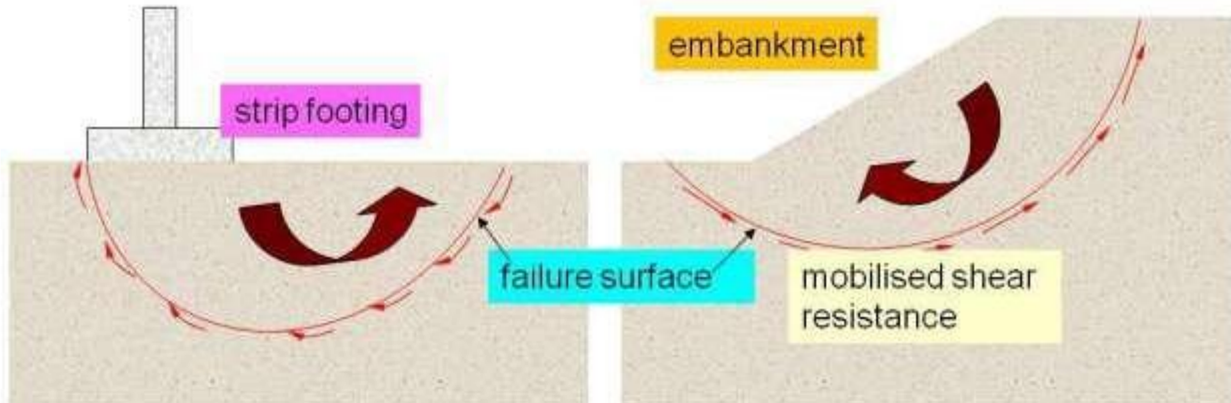


Fig. Shear Failure

At failure, shear stress along the failure surface reaches the shear Thus shear strength of soil is "The capacity of a soil to resist the internal and external forces which slide past each other"

Shear Strength in Soils :

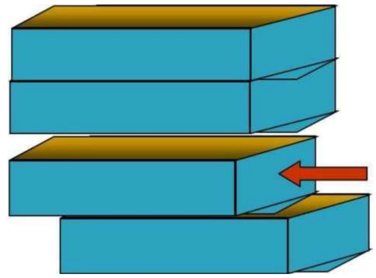
- The shear strength of a soil is its resistance to shearing stresses.
- It is a measure of the soil resistance to deformation by continuous displacement of itsindividual soil particles.
- Shear strength in soils depends primarily on interactions between particles.
- Shear failure occurs when the stresses between the particles are such that they slide orroll past each other

Components of shear strength of soils
Soil derives its shear strength from two sources: –

- Cohesion between particles (stress independent component)
- Cementation between sand grains.
- Electrostatic attraction between clay particles – Frictional resistance

and interlockingbetween particles (stress dependent component)

Cohesion: Cohesion (C), is a measure of the forces that cement particles of soils



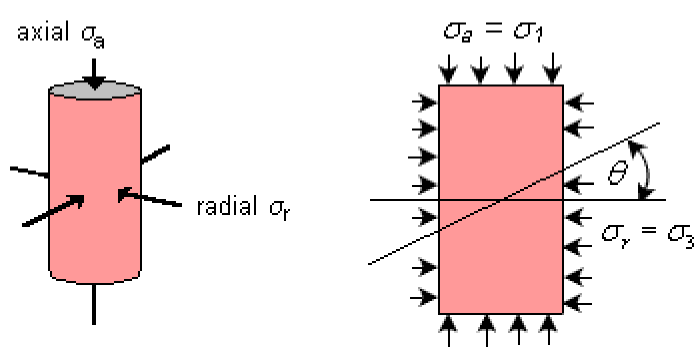
Internal Friction: Internal Friction angle (*f*), is the measure of the shear strength of soils due to friction.

Factors Influencing Shear Strength: The shearing strength, is affected by:

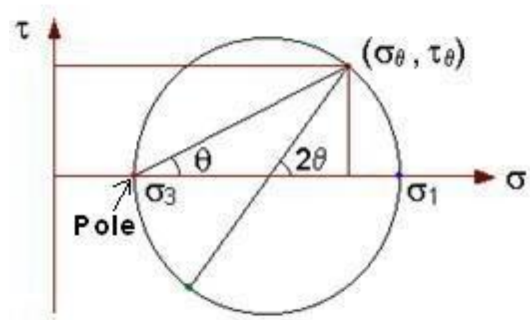
- Soil composition: mineralogy, grain size and grain size distribution, shape of particles, pore fluid type and content, ions on grain and in pore fluid.
- Initial state: State can be describe by terms such as: loose, dense, overconsolidated, normally consolidated, stiff, soft, etc.
- Structure: Refers to the arrangement of particles within the soil mass; the manner in which the particles are packed or distributed. Features such as layers, voids, pockets, cementation, etc, are part of the structure.

Mohr Circle of Stresses

In soil testing, cylindrical samples are commonly used in which radial and axial stresses act on principal planes. The vertical plane is usually the minor principal plane whereas the horizontal plane is the major principal plane. The radial stress (σ_r) is the minor principal stress (σ_3), and the axial stress (σ_a) is the major principal stress (σ_1).



To visualise the normal and shear stresses acting on any plane within the soil sample, a graphical representation of stresses called the Mohr circle is obtained by plotting the principal stresses. The sign convention in the construction is to consider compressive stresses as positive and angles measured counter-clockwise also positive.



Draw a line inclined at angle θ with the horizontal through the pole of the Mohr circle so as to intersect the circle. The coordinates of the point of intersection are the normal and shear stresses acting on the plane, which is inclined at angle θ within the soil sample.

$$\frac{(\sigma_1+\sigma_3)}{2} \quad \frac{(\sigma_1-\sigma_3)}{2}$$

Normal stress

$$\tau_\theta = \frac{(\sigma_1-\sigma_3)}{2}\sin 2\theta$$

Shear stress

The plane inclined at an angle 45° of to the horizontal has acting on it the maximumshear stress equal to $\frac{\sigma_1-\sigma_3}{2}$ $\frac{\sigma_1+\sigma_3}{2}$,

and the normal stress on this plane is equal to .

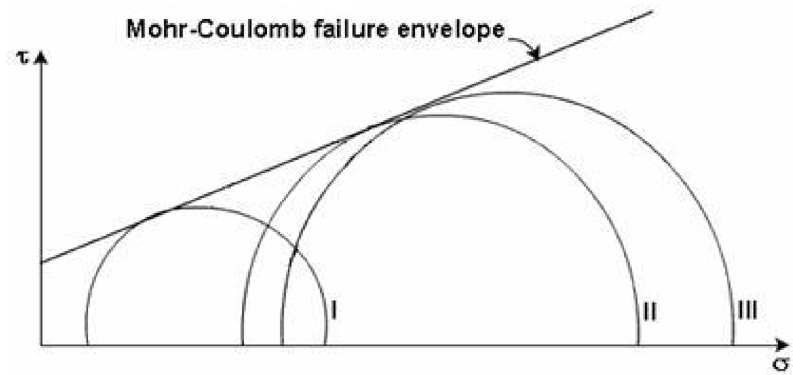
The plane with the maximum ratio of shear stress to normal stress is inclined at an angle $45^0+\frac{\alpha}{2}$

of to the horizontal, where a is the slope of the line tangent to the Mohr circle and passing through the origin.

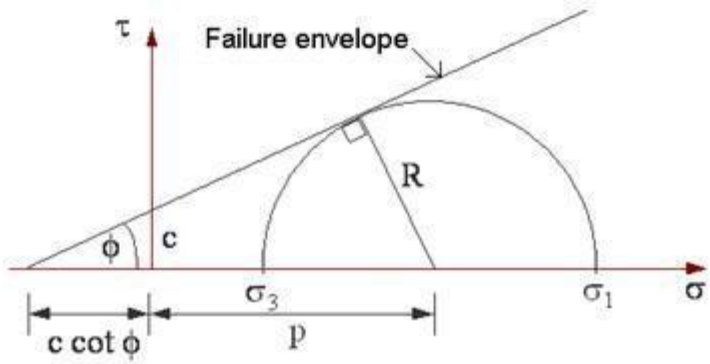
When the soil sample has failed, the shear stress on the failure plane defines the shear strength of the soil. Thus, it is necessary to identify the failure plane. Is it the plane on which the maximum shear stress acts, or is it the plane where the ratio of shear stresses to normal stress is the maximum?

For the present, it can be assumed that a failure plane exists and it is possible to apply principal stresses and measure them in the laboratory by conducting a triaxial test. Then, the Mohr circle of stress at failure for the sample can be drawn using the known values ofthe principal stresses.

If data from several tests, carried out on different samples upto failure is available, a series of Mohr circles can be plotted. It is convenient to show only the upper half of the Mohr circle. A line tangential to the Mohr circles can be drawn, and is called the Mohr- Coulomb failure envelope.



If the stress condition for any other soil sample is represented by a Mohr circle that lies below the failure envelope, every plane within the sample experiences a shear stress which is smaller than the shear strength of the sample. Thus, the point of tangency of the envelope to the Mohr circle at failure gives a clue to the determination of the inclination of the failure plane. The orientation of the failure plane can be finally determined by the pole method.



The Mohr-Coulomb failure criterion can be written as the equation for the line that represents the failure envelope. The general equation is

$$\tau_f = c + \sigma_f . \tan\phi$$

Where τ_f = shear stress on the failure plane

c = apparent cohesion

σ_f = normal stress on the failure plane

f = angle of internal friction

The failure criterion can be expressed in terms of the relationship between the principalstresses. From the geometry of the Mohr circle,

$$\sin\phi = \frac{R}{c.\cot\phi + p} = \frac{\frac{\sigma_1 - \sigma_3}{2}}{c.\cot\phi + \frac{\sigma_1 + \sigma_3}{2}}$$

Rearranging,

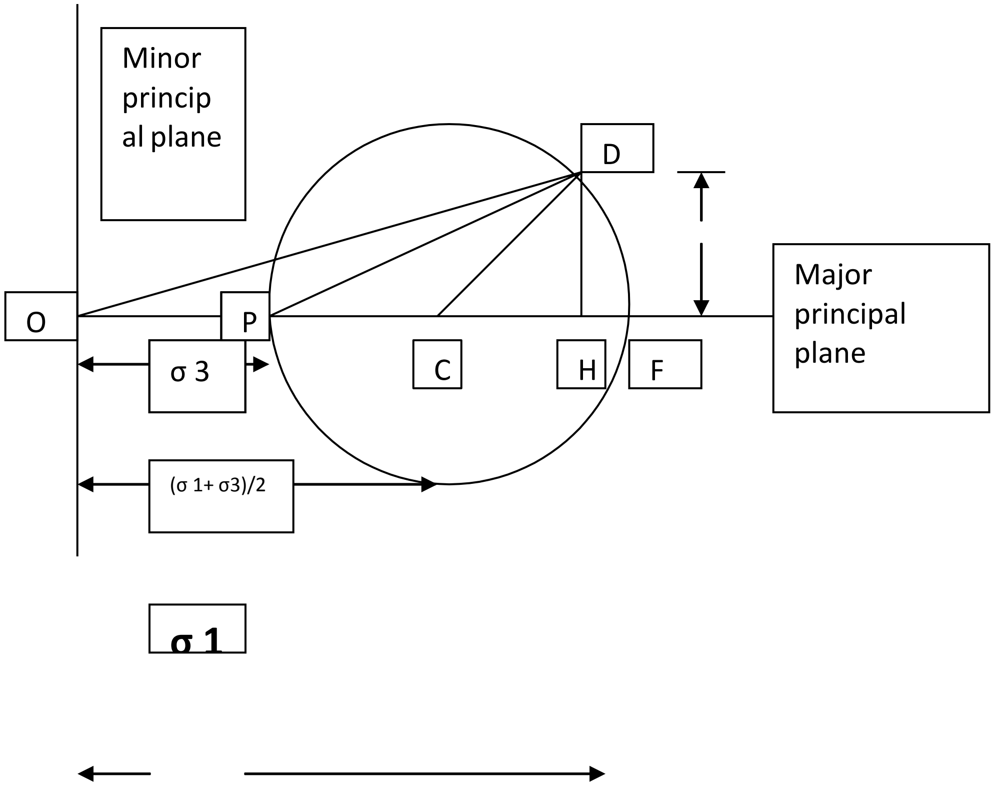
$$\sigma_1 = \sigma_3\left(\frac{1+\sin\phi}{1-\sin\phi}\right) + 2c\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}$$

where

$$\frac{1+\sin\phi}{1-\sin\phi} = \tan^2\left[\frac{\pi}{4} + \frac{\phi}{2}\right]$$

MOHR'S CIRCLE

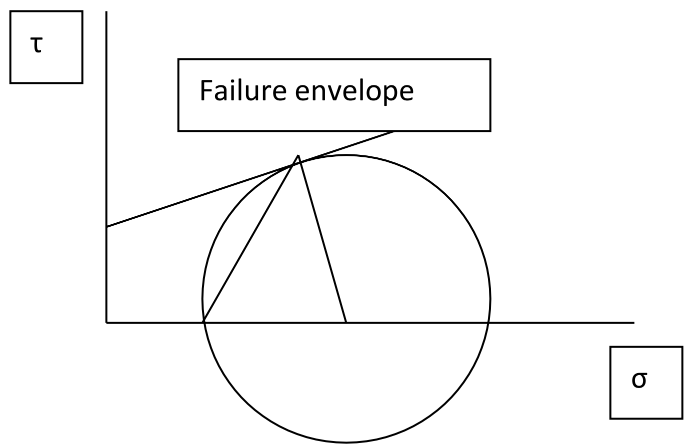
Otto Mohr, a German scientist deviced a graphical method for the determination of stresses on a plane inclined to the major principal planes. The graphical construction is known as Mohr's circle. In this method, the origin O is selected and the normal stresses are plotted along the horizontal axis and the shear stresses on the vertical axis.



To construct a Mohr circle, first mark major and minor principal stresses on X axis. Mark the centre point of that as C. A circle is drawn with c as centre and CF as radius. Each point on the circle gives the stresses **σ** and **τ** on a particular plane. The point E is known as the pole of the circle.

1. Mohr's circle can be drawn for stress system with principal planes inclined to co-ordinate axes
2. Stress system with vertical and horizontal planes are not the principal planes

MOHR-COULOMB THEORY



The soil is a particulate material. The shear failure in soils is by slippage of particles due to shear stresses. According to Mohr, the failure is caused by a critical combination of normal and shear stresses. The soil fails when the shear stress on the failure plane at failure is a unique function of the normal stress acting on that plane. Since the shear stress of the failure plane is defined as the shear strength (s) the equation for that can be written as

$S = f(\sigma)$

The Mohr theory is concerned with the shear stress at failure plane at failure. A plot can be made between the shear stresses and the normal stress at failure. The curve defined by this is known as the failure envelope. The shear strength of a soil at a point on a particular plane was expressed by Coulomb as a linear function of the normal stress on that plane as,

$$S = C + \sigma \tan \varphi$$

In this C is equal to the intercept on Y axis and phi is the angle which the envelope make with X axis.

DIFFERENT TYPES OF SHEAR TESTS AND DRAINAGE CONDITIONS

The following tests are used to measure the shear strength of the soil

1. Direct shear test
2. Triaxial compression test
3. Unconfined compression test
4. Vane shear test

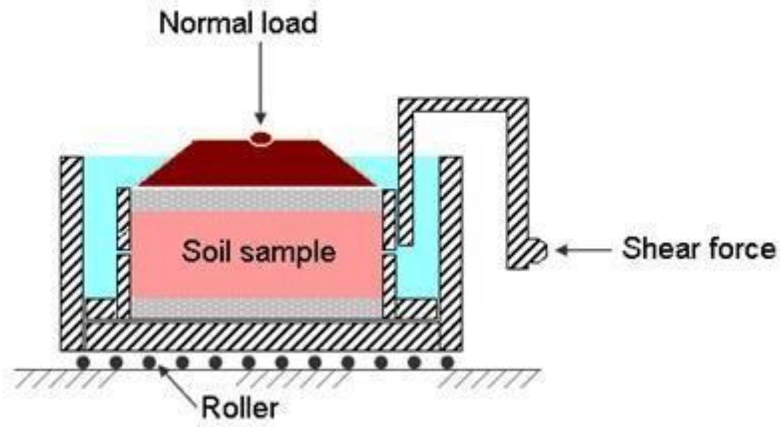
Depending upon the drainage conditions, there are three types of tests

- Unconsolidated-Undrained condition
- Consolidated - Undrained condition
- Consolidated-Drained condition

DIRECT SHEAR TEST

Apparatus:

The test is conducted in a soil specimen in a shear box which is split in to two halves along the horizontal plane at its middle. The size of the shear box is 60 x 60 x 50 mm. the box is divided horizontally such that the dividing plane passes through the centre. Thetwo halves are held together by locking pins the box is also provided with gripper plates plain or perforated according to the testing conditions



Test Procedure:

A soil specimen of size 60 x 60 x 25 mm is taken. It is placed in the direct shear box and compacted. The upper grid plate, porous stone and pressure pad is placed on the specimen. Normal load and shear load is be applied till failure

Presentation of results:

- Stress – strain curve
- Failure envelope
- Mohr's circle

Merits:

1. the sample preparation is easy
2. as the thickness of the sample is very less, the drainage is quick
3. it is ideally suited for conducting drained tests on cohesionless soils
4. the apparatus is relatively cheap

Demerits:

1. the stress conditions are known only at failure
2. the stress distribution on the failure plane is not uniform
3. the area of shear gradually decreases as the test progresses
4. the orientation of the failure plane is fixed
5. control of drainage conditions is very difficult
6. measurement of pore water pressure is not possible

Tests on sands and gravels can be performed quickly, and are usually performed dry as it is found that water does not significantly affect the drained strength. For clays, the rate of shearing must be chosen to prevent excess pore pressures building up.

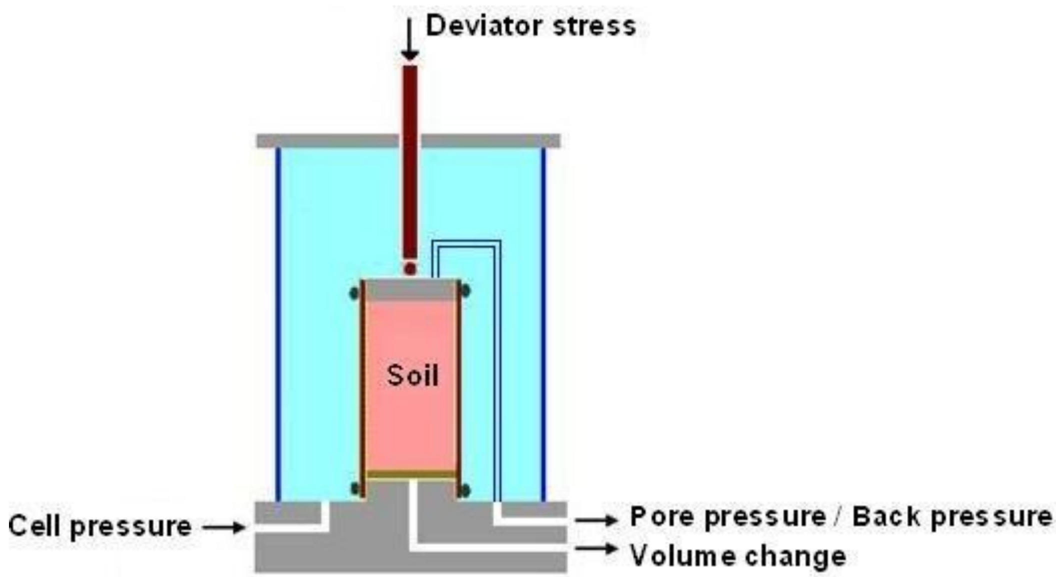
As a vertical normal load is applied to the sample, shear stress is gradually

applied horizontally, by causing the two halves of the box to move relative to each other. The shear load is measured together with the corresponding shear displacement. The change of thickness of the sample is also measured.

A number of samples of the soil are tested each under different vertical loads and the value of shear stress at failure is plotted against the normal stress for each test. Provided there is no excess pore water pressure in the soil, the total and effective stresses will be identical. From the stresses at failure, the failure envelope can be obtained.

TRIAXIAL COMPRESSION TEST

The triaxial test is carried out in a cell on a cylindrical soil sample having a length to diameter ratio of 2. The usual sizes are 76 mm x 38 mm and 100 mm x 50 mm. Three principal stresses are applied to the soil sample, out of which two are applied water pressure inside the confining cell and are equal. The third principal stress is applied by a loading ram through the top of the cell and is different to the other two principal stresses. A typical triaxial cell is shown.



The soil sample is placed inside a rubber sheath which is sealed to a top cap and bottom pedestal by rubber O-rings. For tests with pore pressure measurement, porous discs are placed at the bottom, and sometimes at the top of the specimen. Filter paper drains may be provided around the outside of the specimen in order to speed up the consolidation process. Pore pressure generated inside the specimen during testing can be measured by means of pressure transducers.

- It is used for the determination of shear characteristics of all types of soils under different drainage conditions. In this a cylindrical specimen is stressed under conditions of axial symmetry. In the first stage of the test, the specimen is subjected to an all round confining pressure on the sides, top and bottom.
- This stage is known as the consolidation stage. In the second stage of the test called shearing stage, an additional axial stress called deviator stress is applied on the top of the specimen through a ram. Thus the total stress in the axial direction at the time of shearing is equal to the confining stress plus the deviator stress.
- The vertical sides of the specimen are principal planes. The confining pressure is the minor principal stress. The sum of the confining stress and deviator stress is the major principal stress.

- Triaxial apparatus consists of a circular base with a central pedestal. The specimen is placed on the pedestal. The pedestal has one or two holes which are used in the drainage function or pore pressure measurement.
- A triaxial cell is placed to the base plate. It is a Perspex cylinder. There are three tie rods which support the cell. A central ram is there for applying axial stress. An air release valve and an oil release valve are attached to the cell. The apparatus also have special features like,

- Mercury control system.
- Pore water pressure measurement device.
- Volume changes measure.

Triaxial test on cohesive soil

CU, UU and CD tests can be conducted on soil specimen. The specimen is placed in the pedestal inside a rubber membrane. The confining pressure and axial pressure is applied till failure.

Triaxial test on cohesion less soil

The procedure is same as that in the cohesive soil only the sample preparation isdifferent. A metal former, a membrane and a funnel are used for the sample preparation.

Merits

1. There is complete control over the drainage conditions

2. Pore pressure changes and volumetric changes can be measured directly
3. The stress distribution in the failure plane is uniform
4. The specimen is free to fail on the weakest plane
5. The state of stress at all intermediate stages up to failure is known
6. The test is suitable for accurate research work

Demerits

1. The apparatus is elaborate, costly and bulky
2. The drained test takes a longer period as compared with that in a direct shear test
3. The strain condition in the specimen are not uniform
4. It is not possible to find out the cross sectional area of the specimen accuratelyunder large strains
5. The test simulates only axi symmetric problems
6. The consolidation of the specimen in the test is isotropic whereas in the field,consolidation is generally anisotropic.

Computation of various parameters

1. Post consolidation

   dimensions $V_0 = Lo \times (\pi/4. Do^2)$

   $Do = [Vo/(\pi/4 \times Lo)]^{1/2}$
2. Cross sectional area during

   shearing stage $A = Ao/(1-\xi_1)$
3. Stresses

   Deviator

   stress=P/A

   Principal stresses

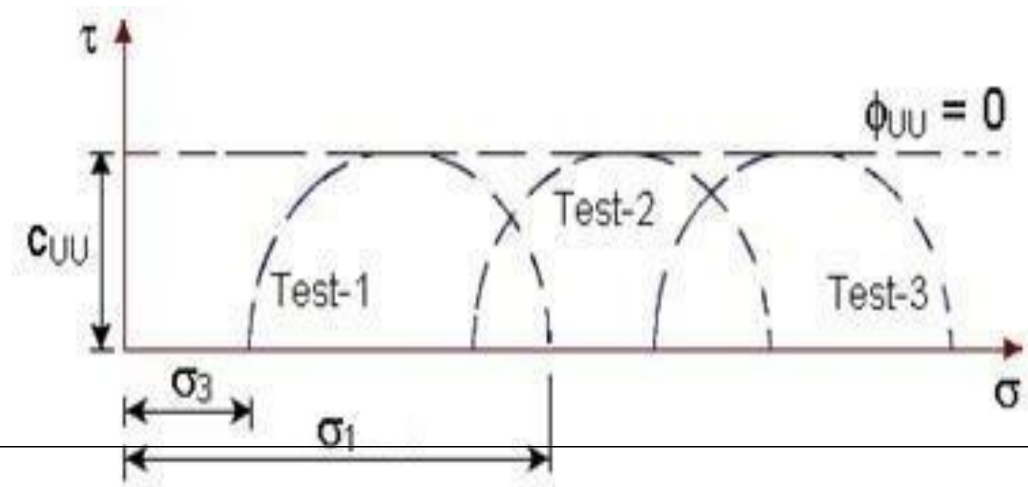
   $\sigma 1 = \sigma 3 + (\sigma 1 - \sigma 3)$
4. Compressive strength

   The deviator stress at failure is known as the compressive strength of soil

Presentation of results of triaxial test

- Stress-strain curves
- Mohr envelopes in terms of total stress and effective stress

UU Test

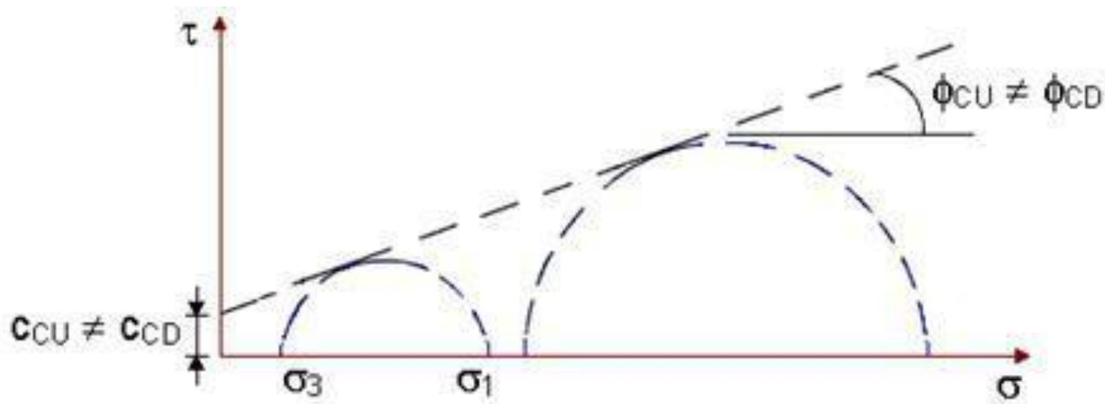


All Mohr circles for UU test plotted in terms of total stresses have the same diameter.The failure envelope is a horizontal straight line and hence

It can be represented by the equation:

$$\tau_f = c_{UU} = \frac{\sigma_1 - \sigma_3}{2}$$

CU & CD Tests:



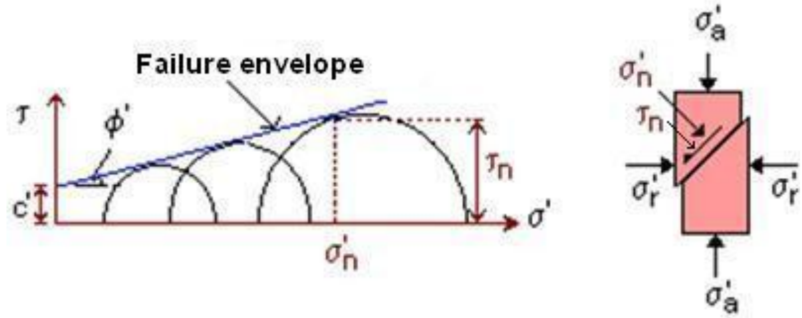
For tests involving drainage in the first stage, when Mohr circles are plotted in terms of total stresses, the diameter increases with the confining pressure. The resulting failure envelope is an inclined line with an intercept on the vertical axis.

It is also observed that $C_{CU} \neq C_{CD}$ and $\emptyset_{CU} \neq \emptyset_{CD}$

It can be stated that for identical soil samples tested under different triaxial conditions of UU, CU and CD tests, the failure envelope is not unique.

Effective Stress Parameters

If the same triaxial test results of UU, CU and CD tests are plotted in terms of effective stresses taking into consideration the measured pore water pressures, it is observed that all the Mohr circles at failure are tangent to the same failure envelope, indicating that shear strength is a unique function of the effective stress on the failure plane.



This failure envelope is the shear strength envelope which may then be written as

$$\tau_f = c' + \sigma'.\tan\phi'$$

Where c' = cohesion intercept in terms of effective stress
f' = angle of shearing resistance in terms of effective stress

If σ'_n is the effective stress acting on the rupture plane at failure, τ_n is the shear stress onthe same plane and is therefore the shear strength.

The relationship between the effective stresses on the failure plane is

$$\sigma'_1 = \sigma'_3\left(\frac{1+\sin\phi'}{1-\sin\phi'}\right) + 2c'\sqrt{\frac{1+\sin\phi'}{1-\sin\phi'}}$$

UNCONFINED COMPRESSION TEST

The unconfined compression test is a special form of triaxial test in which the confining pressure is zero. The test can be conducted only on clayey soils which can stand without confinement. There are two types of UCC machines machine with a spring and machine with a proving ring

A compressive force is applied to the specimen till failure. The compressive load can be measured using a proving ring.

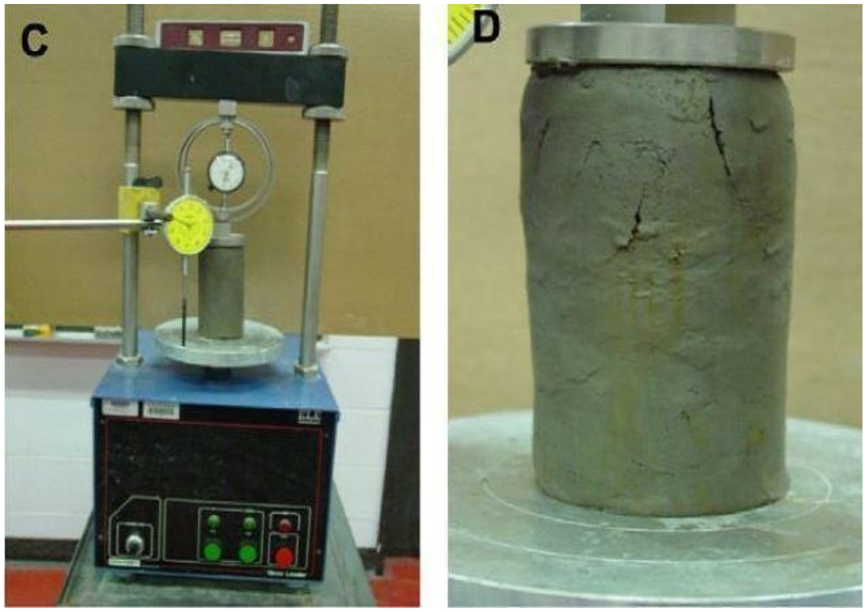


Fig. Unconfined Compressive
Strength Test

Presentation of results

In this test the minor principal stress is zero. The major principal stress is equal to thedeviator stress. The Mohr circle can be drawn for stress conditions at failure.

Merits

1. The test is convenient, simple and quick
2. It is ideally suited for measuring the unconsolidated undrained shear strength ofintact saturated clays
3. The sensitivity of the soil can be easily

determinedDemerits

1. The test cannot be conducted on fissured clays
2. The test may be misleading for soils of which the angle of shearing resistance isnot zero.

VANE SHEAR TEST

The undrained strength of soft clays can be determined in a laboratory by vane shear test. The test can also be conducted in the field on the soil at the bottom of bore hole. The apparatus consists of a vertical steel rod having

four thin stainless steel blades or vanes fixed at its bottom end. Height of the vane should be equal to twice the diameter. For conducting test in a laboratory, a specimen of dia 38mm and height 75mm is prepared and fixed to the base of the apparatus. The vane is slowly lowered in to the specimen till the top of the vane is at a depth of 10 to 20 mm below the top of the specimen. The readings of the strain indicator and torque indicator are taken

Shear strength $S = T/[\pi(D^2H_1/2 + D^3/12)]$ Where T = Torque applied

D = Diameter of vane

H_1 = Height of vane

Fig. Laboratory Vane Shear Test

Merits

1. The test is simple and quick
2. It is ideally suited for determination of the in-situ undrained shear strength of nonfissured, fully saturated clay
3. The test can be conveniently used to determine the

sensitivity of the soilDemerits

1. The test cannot be conducted on the fissured clay or the clay containing silt orsand laminations
2. The test does not give accurate results when the failure envelope is not horizontal.

Question

1. **What is the concept of shear strength ?**
2. **What is mohr coulomb failure theory ?**

3. What is cohesion and angle of internal frictions ?
4. What are methods of measurements of shear strength ?

EARTH RETAINING STRUCTURES

Earth retaining structures or systems are used to hold back earth and maintain a difference in the elevation of the ground surface as shown in Figure 10-1. The retaining wall is designed to withstand the forces exerted by the retained ground or "backfill" and other externally applied loads, and to transmit these forces safely to a foundation and/or to a portion of the restraining elements, if any, located beyond the failure surface.

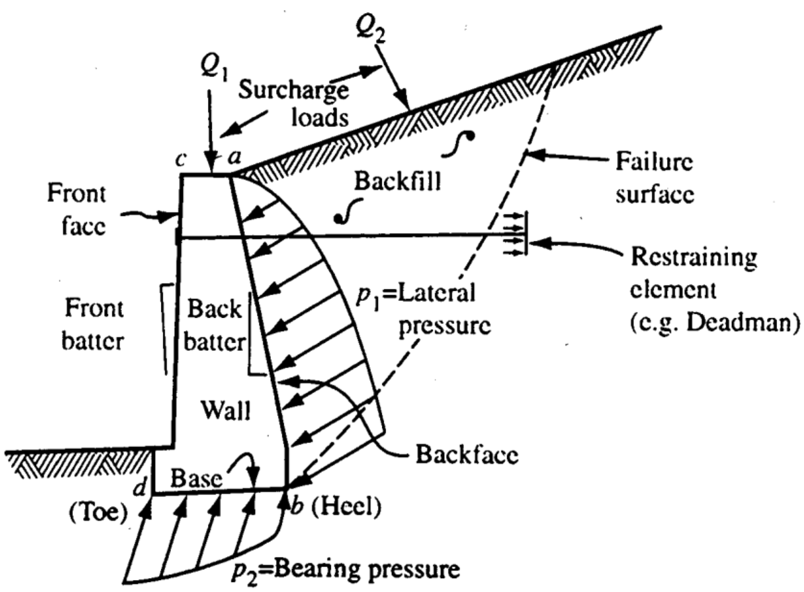


Figure 10-1. Schematic of a retaining wall and common terminology.

In general, the cost of constructing a retaining wall is usually high compared with the cost of forming a new slope. Therefore, the need for a retaining wall should be assessed carefully during preliminary design and an effort should be made to keep the retained height as low as possible.

In highway construction, retaining walls are used along cuts or fills where space is inadequate for construction of cut slopes or embankment slopes. Bridge abutments and foundation walls, which must support earth fills, are also designed as retaining walls.

Typical applications for earth retaining structures in highway construction include:

- new or widened highways in developed areas;
- new or widened highways at mountain or steep slopes;
- grade separation;
- bridge abutments, wing walls and approach embankments;

- culvert walls;
- tunnel portals and approaches;
- flood walls, bulkheads and waterfront structures;
- cofferdams for construction of bridge foundations;
- stabilization of new or existing slopes and protection against rockfalls; and
- groundwater cut-off barriers for excavations or depressed roadways.

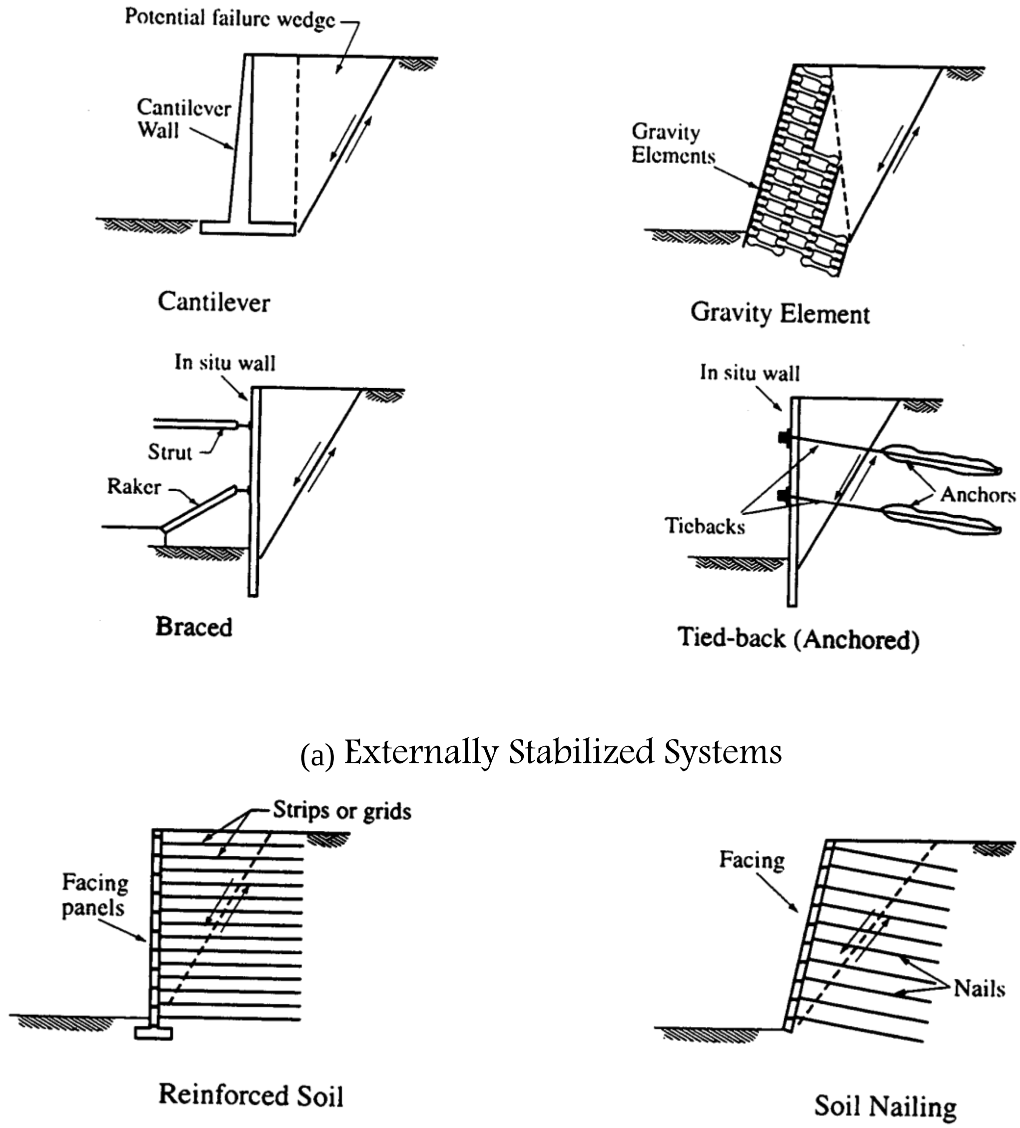
Figure 10-2 provides schematic illustrations of several retaining wall systems traditionally used in highway applications. A great number of wall systems have been developed in the past two decades by specialty contractors who have been promoting either a special product or a specialized method of construction, or both. Due to the rapid development of these diversified systems and their many benefits, the design engineer is now faced with the difficult task of having to select the best possible system; design the structure; and ensure its proper construction.

An important breakthrough in the design of earth retaining structures (ERS) that occurred in this era was the recognition that the earth pressure acting on a wall is a function of the type of wall and the amount and distribution of wall movement. Classical earth pressure theories, which were developed by Coulomb (1776) and Rankine (1857), were formalized for use by Caquot and Kerisel (1948) and others. Sophisticated analyses of soil-structure interaction and wall/soil movements began in the 1960s with the development of finite difference and finite element analytical procedures. The simultaneous advancement of geotechnical instrumentation equipment and monitoring procedures made the "observational method" of design (Peck, 1969) popular and cost effective.

Since 1970 there has been a dramatic growth in the number of methods and products for retaining soil. O'Rourke and Jones (1990) describe two trends in particular that have emerged since 1970. First, there has been an increasing use of reinforcing elements, either by incremental burial to create reinforced soils (MSE walls), or by systematic in situ installation to reinforce natural soils or even existing fills (soil nailing); see Figure 10-2b. Mechanically stabilized earth and soil nailing have changed the ways we construct fill or cut walls, respectively, by providing economically attractive alternatives to traditional designs and construction methods. Second, there has been an increasing use of polymeric products to reinforce the soil and control drainage. Rapid developments in polymer manufacturing have supplied a wide array of geosynthetic materials. The use of these products in construction has encouraged a multitude of different earth retention schemes.

The rapid development of these new trends and the increased awareness of the impact of construction on the environment, have led to the emergence of the concept of "earth walls."

> In this concept, the soil supports itself or is incorporated into the structure and assumes a major structural or load carrying function. With this concept, structural member requirements of the system are reduced, or eliminated altogether. Examples of recently developed earth walls include the soil-reinforcement systems discussed above, as well as systems involving chemical treatment of the in-situ soil such as jet grouting or deep soil mixing.



(b) Internally Stabilized Systems

Figure 10-2. Variety of retaining walls (after O'Rourke and Jones, 1990)

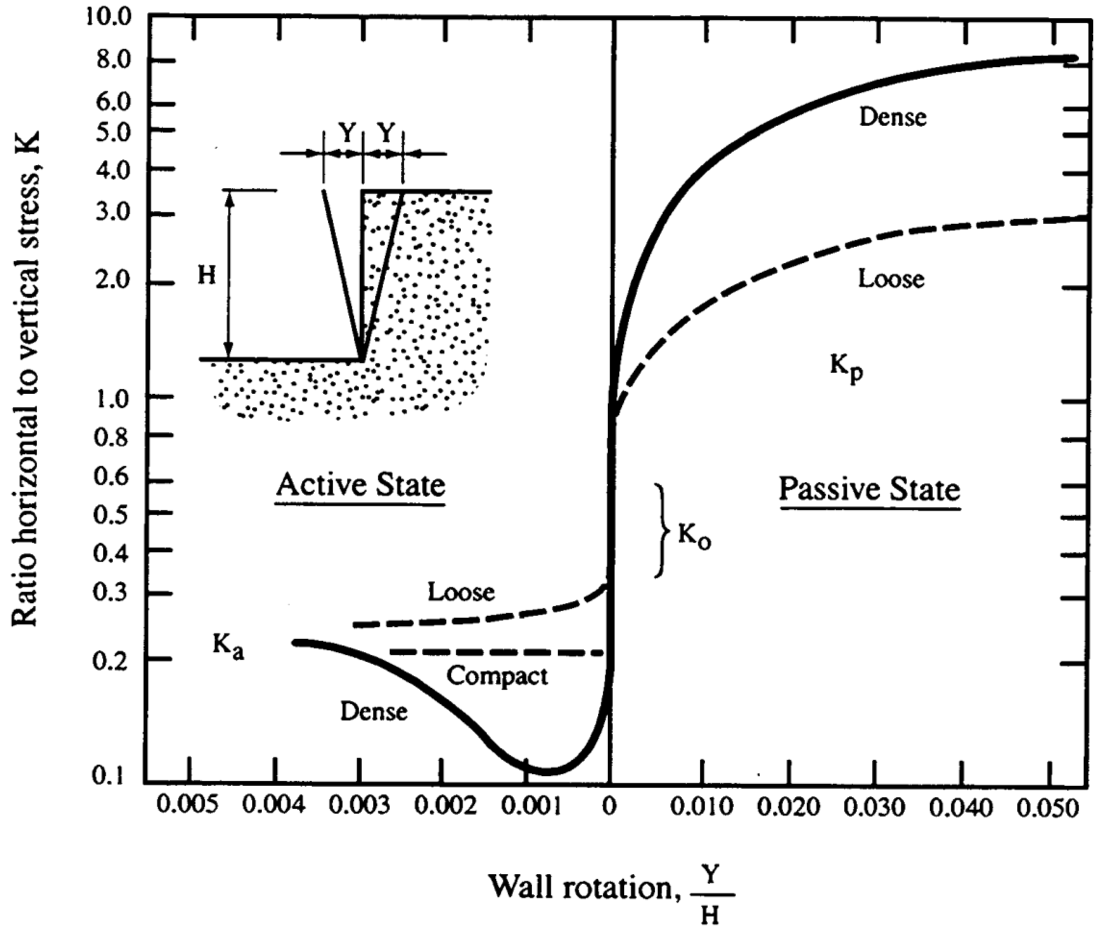
10.1 LATERAL EARTH PRESSURES

Some of the basic concepts of lateral earth and water pressures were discussed in Chapter 2. It is recommended that the reader should review Section 2.9 before proceeding further in this Chapter. Here the principles of lateral earth pressure are explained on the basis of deformation. A total lateral pressure diagram consistent with the assumed deformations is developed for use in assessing the forces acting on the wall from the backfill or retained ground. This section focuses primarily on theoretical earth pressure diagrams, which are most commonly used in the design of rigid gravity structures, nongravity cantilevered walls, MSE walls, and anchored walls with stiff structural facings such as diaphragm walls.

A wall system is designed to resist lateral earth pressures and water pressures that develop behind the wall. Earth pressures develop primarily as a result of loads induced by the weight of the backfill and/or retained in-situ soil, earthquake ground motions, and various surcharge loads. For purposes of earth retaining system design, three different types of lateral earth pressure are usually considered: (1) at-rest earth pressure; (2) active earth pressure; and (3) passive earth pressure. These conditions are shown in Figure 10-4 relative to lateral deformation of the walls. The conditions are defined as follows:

- At-rest earth pressure is defined as the lateral earth pressure that exists in level groundfor a condition of no lateral deformation.

- Active earth pressure is developed as the wall moves away from the backfill or the retained soil. This movement results in a decrease in lateral pressure relative to the at-rest condition. A relatively small amount of lateral movement is necessary to reachthe active condition.



Magnitude of Wall Rotation to Reach Failure

Soil type and condition	Rotation, Y/H	
	Active	Passive
Dense cohesionless	0.001	0.02
Loose cohesionless	0.004	0.06
Stiff cohesive	0.010	0.02
Soft cohesive	0.020	0.04

Figure 10-4. Effect of wall movement on wall pressures (after Canadian GeotechnicalSociety, 1992).

- Passive earth pressure is developed as the wall moves towards the backfill or the retained soil. This movement results in an increase in lateral pressure relative to the at-rest condition. The movements required to reach the passive condition are approximately ten times greater than those required to develop active earth pressure.

Each of these earth pressure conditions can be expressed in general form by:

$$p_h = Kp_o \qquad 10\text{-}1$$

where p_h is the lateral earth pressure at a given depth behind the wall, p_o, is the vertical stress at the same depth, and K is the earth pressure coefficient that has a value related to the at-rest condition (K_o), active conditions of movement, (K_a), or passive conditions of movement,(K_p).

As shown in Figure 10-4, the magnitudes of these earth pressure coefficients follow the relationship of $K_p > K_o > K_a$. The relationship between the magnitude of retaining wall movement, in this case rotation, Y/H, into or away from the retained material about its toe, and the horizontal pressure exerted by the soil is presented in Figure 10-4, with angular movement along the x axis and the mobilized coefficient of lateral earth pressure on the y axis. Figure 10-4 can also be used to estimate the state of stress for walls with uniform horizontal translation equal to Y. As illustrated in this figure, significantly larger lateral displacements are required to mobilize the passive resistance than those required to develop active pressures. The maximum values of K_a and K_p correspond to fully mobilized pressures that represent active and passive failure conditions, respectively.

When the estimated wall movement is less than the value required to fully mobilize active or passive pressure, the earth pressure coefficient can be adjusted proportionally based on the graphical relationship presented in Figure 10-4.

10.1.1 At-Rest Lateral Earth Pressure

The at-rest earth pressure represents the lateral effective stress that exists in a natural soil in its undisturbed state. For cut walls constructed in near normally consolidated soils, the at- rest earth pressure coefficient, K_o, can be approximated by the equation (Jaky, 1944):

$$K_O = 1 - \sin \phi' \tag{10-2}$$

where ϕ' is the effective (drained) friction angle of the soil. The magnitude of the at-restearth pressure coefficient is primarily a function of soil shear strength and degree of overconsolidation, which, as indicated in Chapter 7, may result from natural geologicprocesses for retained natural ground or from compaction effects for backfill soils.

In overconsolidated soils, K_o can be estimated as (Schmidt, 1966):

$$K_O = (1 - \sin \phi')(OCR)^{\Omega} \qquad \text{10-3}$$

where Ω is a dimensionless coefficient, which, for most soils, can be taken as $\sin \phi'$ (Mayne and Kulhawy, 1982) and OCR is the overconsolidation ratio.

Usually, Equations 10-2 and 10-3 for the at-rest earth pressure coefficient are sufficiently accurate for normally to lightly overconsolidated soils provided the overconsolidation ratio has been evaluated from laboratory consolidation testing. For moderately to heavily overconsolidated clays, or where a more accurate assessment is required, laboratory triaxial tests on undisturbed samples and in-situ testing such as pressuremeter testing may be used.

For normally consolidated clay, K_o is typically in the range of 0.55 to 0.65; for sands, the typical range is 0.4 to 0.5. For lightly over consolidated clays (OCR ≤ 4), K_o may reach a value up to 1; for heavily over consolidated clays (OCR > 4), K_o values may be greater than 2 (Broker and Ireland, 1965). For heavily overconsolidated soils, values for K_o can be very large. A relatively stiff wall would be required to resist the large forces resulting from the lateral earth pressures in this case. For walls constructed in such soils, consideration should be given to performing pressuremeter tests, which provide a direct measure of lateral pressures in the ground.

In the context of wall designs consisting of steel soldier beams or sheet-pile wall elements, design earth pressures based on at-rest conditions are not typically used since at-rest earth pressures imply that the wall system undergoes no lateral deformation. This condition may be appropriate for heavily preloaded, stiff wall systems, but designing to a requirement of zero wall movement for flexible wall systems is not practical.

10.1.2 Active and Passive Lateral Earth Pressures

As discussed in Chapter 2, in stability analyses active and passive earth pressures are developed as a result of soil displacement within a failure zones developed behind the wall (active) or in front of the wall (passive) assuming that the wall displaces outward. For the purpose of illustration Figure 10-5 shows the two conditions with respect to wall movement relative to the backfill only. In one case the wall moves away from the backfill (active case) in the other case the wall moves into the backfill (passive case) As shown in the figure, the failure zone for both cases is typically bounded by the back face of the wall and a failure surface through the retained soil mass along which the soil has attained limiting equilibrium. In addition to the effect of lateral movements on the values of K_a and K_p shown in Figure 10-4, the magnitude of the active and passive earth pressure coefficients are functions of the soil shear strength, the backfill geometry, i.e., horizontal backfill surface or sloping ground surface above the wall, the orientation of the surface where the wall contacts the backfill or retained soil, i.e., vertical or inclined, and the friction and cohesive forces that develop on this surface as the wall moves relative to the retained ground.

Active and passive earth pressure coefficients based on a plane wedge theory, which considers the effect of wall friction, sloping backfill and sloping wall face, was first proposed by Coulomb (1776) and are shown in Figure 10-5. The pressures calculated by using these coefficients are commonly known as the Coulomb earth pressures. Since Coulomb's method is based on limit equilibrium of a wedge of soil, only the magnitude and direction of the earth pressure is found. Pressure distributions and the location of the resultant are assumed to be triangular.

For simple cases involving vertical walls retaining homogeneous soil with a level ground surface, without friction between the soil and the wall face, and without the presence of groundwater, the formulas for computing the earth pressure coefficients can be simplified considerably by substituting, $\square = \theta = \beta = 0$ in Coulomb's equations, as shown in Figure 10-5. For such simplified cases, K_a and K_p can be expressed by Equations 10-4 and 10-5, respectively:

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \tan^2(45 - \phi/2)$$

Questions

1. What is the theory of earth pressure ?
2. What is active earth pressure ?
3. What is passive earth pressure ?
4. What is the earth pressure at rest ?
5. What is rankines formula ?

Foundation engineering

Functions Of Foundation

- Reduction of Load Intensity. ...
- Even distribution of load. ...
- Provision of level surface. ...
- Lateral stability Foundation anchors the super-structure to the ground, thus imparting lateral stability to the super-structure. ...
- Safety against undermining. ...
- Protection against soil movements.

SHALLOW FOUNDATIONS

A shallow foundation is a type of building foundation that transfers structural load to the earth very near to the surface, rather than to a **subsurface layer or a range of depths**, as does a deep foundation.**subsurface layer or a range of depths**, as does a deep foundation.

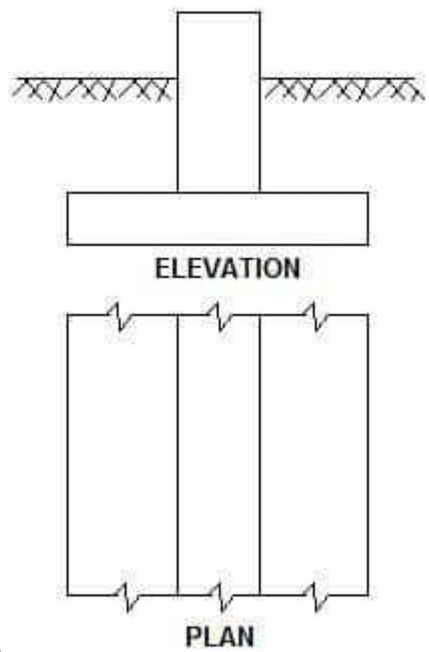
Different Types of Shallow Foundations

The different types of shallow foundation are:

1. Strip footing
2. Spread or isolated footing
3. Combined footing Strap or cantilever footing
4. Mat or raft Foundation

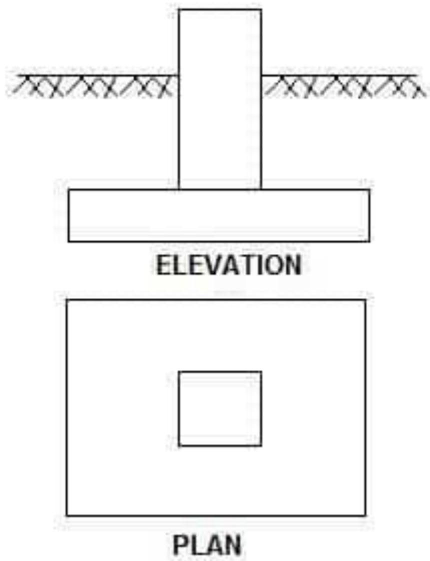
1. Strip Footing

A strip footing is provided for a load-bearing wall. A strip footing is also provided for a row of columns which are so closely spaced that their spread footings overlap or nearly touch each other. In such a case, it is more economical to provide a strip footing than to provide a number of spread footings in one line. A strip footing is also known as continuous footing.



2. Spread or Isolated Footing or Individual Footing

A spread footing also called as isolated footing, pad footing and individual footing is provided to support an individual column. A spread footing is circular, square or rectangular slab of uniform thickness. Sometimes, it is stepped or haunched to



spread the load over a large area.

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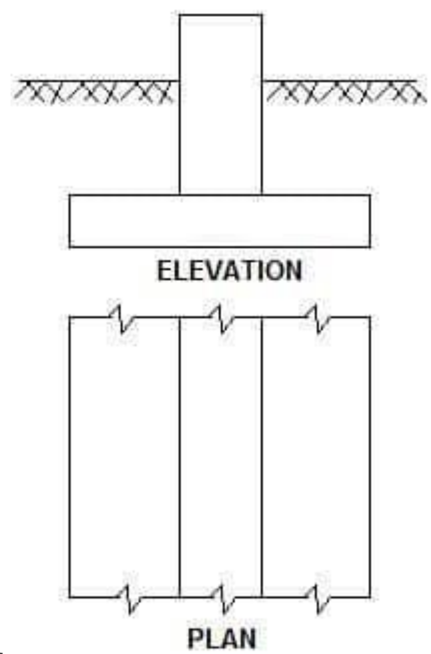

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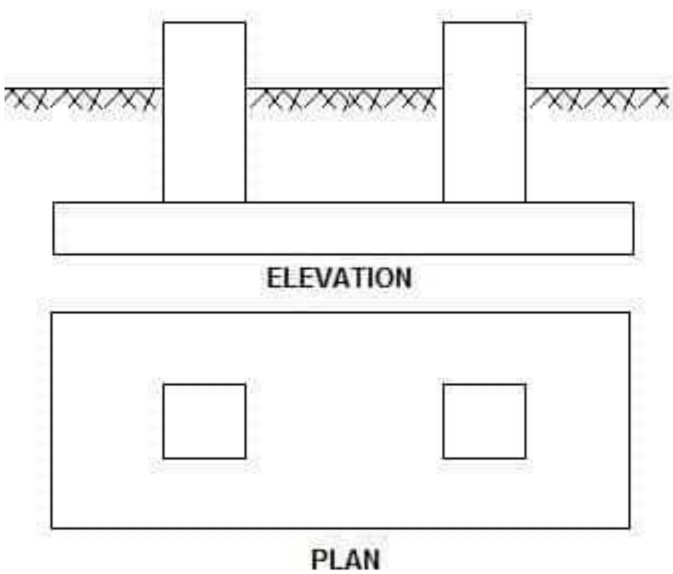
A spread footing also called as isolated footing, pad footing and individual footing is provided to support an individual column. A spread footing is circular, square or rectangular slab of uniform thickness. Sometimes, it is stepped or haunched to spread the load over a large area.

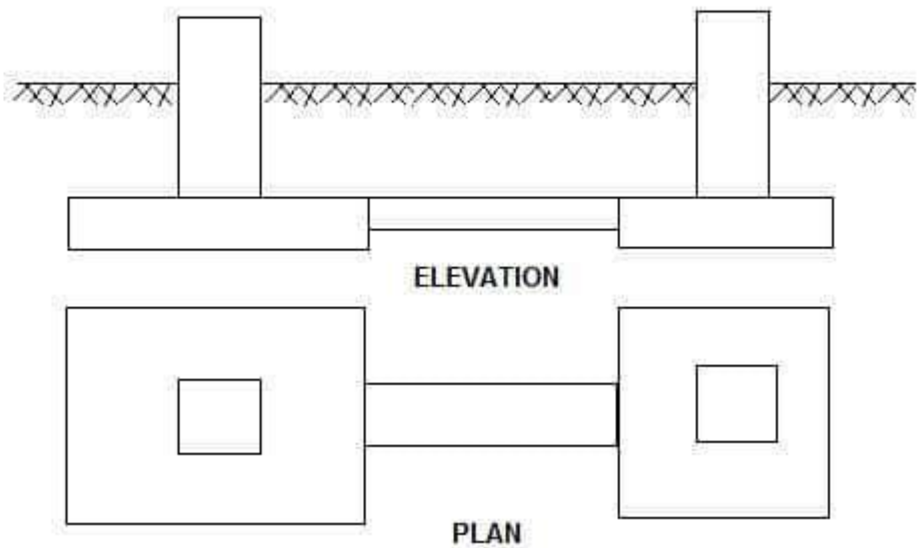
Combined Footing

A combined footing supports two columns. It is used when the two columns are so close to each other that their individual footings would overlap. A combined footing is also provided when the property line is so close to one column that a spread footing would be eccentrically loaded when kept entirely within the property line. By combining it with that of an interior column, the load is evenly distributed. A combined footing may be rectangular or trapezoidal in plan.



. Strap or Cantilever Footing

A strap (or cantilever) footing consists of two isolated footings connected with a structural strap or a lever. The strap connects the two footings such that they behave as one unit. The strap is designed as a rigid beam. The individual footings are so designed that their combined line of action passes through the resultant of the total load. a strap footing is more economical than a combined footing when the allowable soil pressure is relatively high and the distance between the columns is large.



Mat or Raft Foundations

A mat or raft foundation is a large slab supporting a number of columns and walls under the entire structure or a large part of the structure. A mat is required when the allowable soil pressure is low or where the columns and walls are so close that individual footings would overlap or nearly touch each other. Mat foundations are useful in reducing the differential settlements on non-homogeneous soils or where there is a large variation in the loads on individual columns.

Deep foundations

A deep foundation is **a type of foundation that transfers building loads to the earth farther down from the surface than a shallow foundation does to a subsurface layer or a range of depths**.

Types-

Types of Deep Foundation

The types of deep foundations in general use are as follows:

1. Basements
2. Buoyancy rafts (hollow box foundations)
3. Caissons
4. Cylinders
5. Shaft foundations
6. Pile foundations

1. Basement foundation

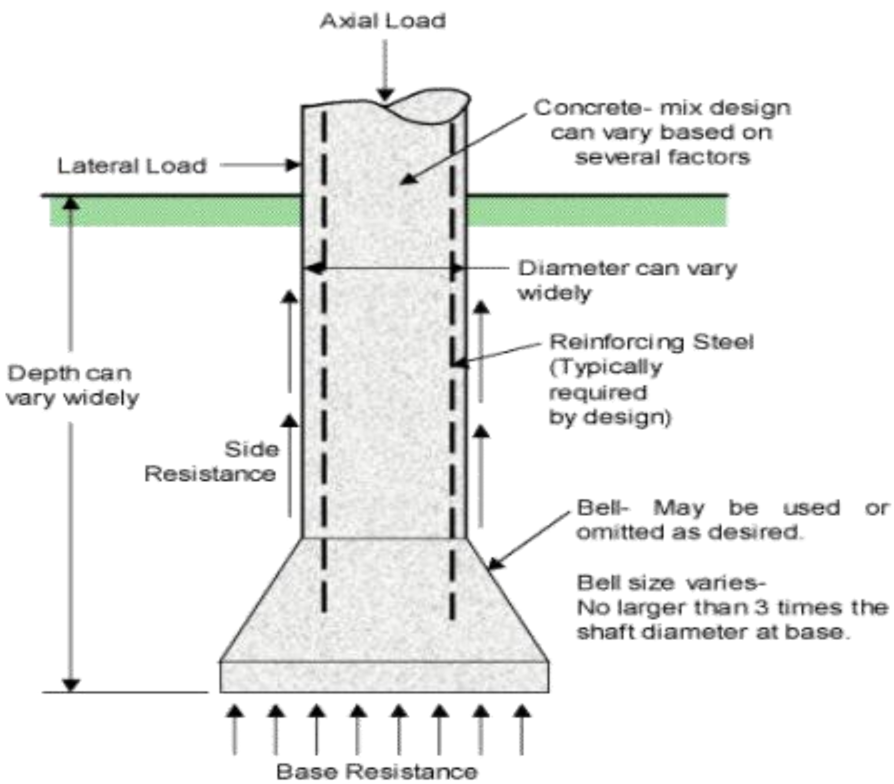
These are hollow substructures designed to provide working or storage space below ground level. The structural design is governed by their functional requirements rather than from considerations of the most efficient method of resisting external earth and hydrostatic pressures. They are constructed in place in open excavations.

2. Buoyancy Rafts (Hollow Box Foundations)

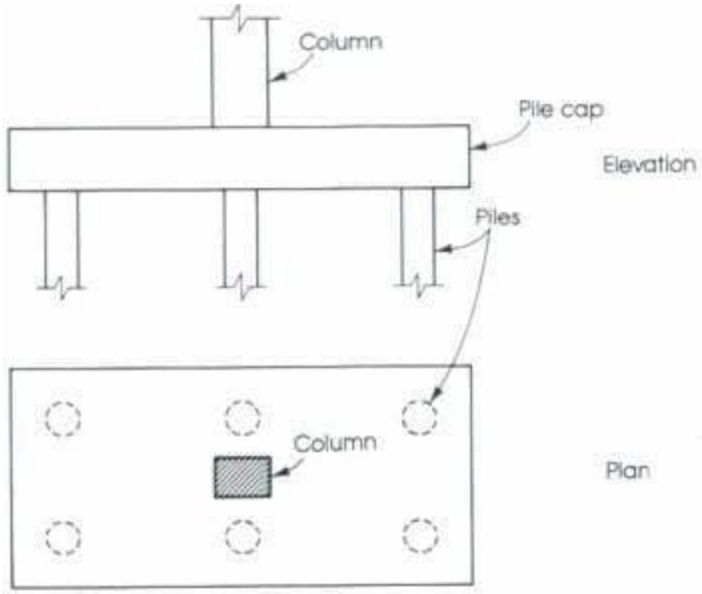
Buoyancy rafts are hollow substructures designed to provide a buoyant or semi-buoyant substructure beneath which the net loading on the soil is reduced to the desired low intensity. Buoyancy rafts can be designed to be sunk as caissons, they can also be constructed in place in open excavations. Read More about **Buoyancy rafts (hollow box foundations)**

3. Caissons Foundations

Caissons are hollow substructures designed to be constructed on or near the surface and then sunk as a single unit to their required level.



Pile foundations



Pile foundations are relatively long and slender members constructed by driving preformed units to the desired founding level, or by driving or drilling-in tubes to the required depth – the tubes being filled with concrete before or during withdrawal or by drilling unlined or wholly or partly lined boreholes which are then filled with concrete.

General Shear Failure – This type of failure occurs in stiff clay or dense sand. In this type of failure, failure takes place at a very small strain. The load settlement curve shows a well-defined peak as shown in Fig.1. At failure entire soil mass within the failure wedge participates and well defined rupture surfaces develop. The failure is accompanied by a considerable bulging of sheared mass of soil. There is only marginal difference between the load causing local shear failure and general Shear failure.

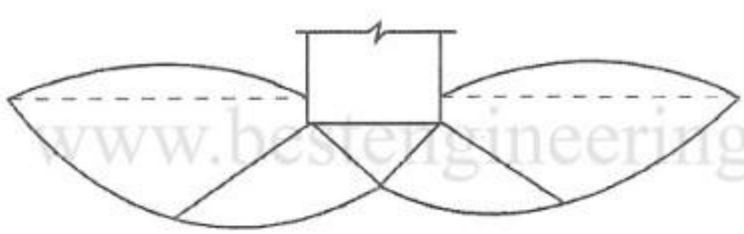
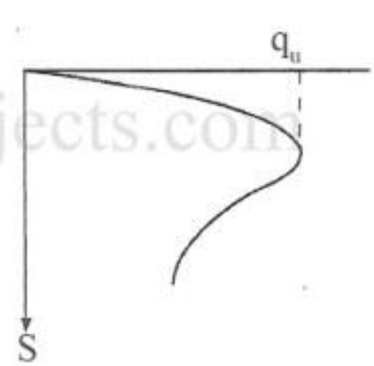
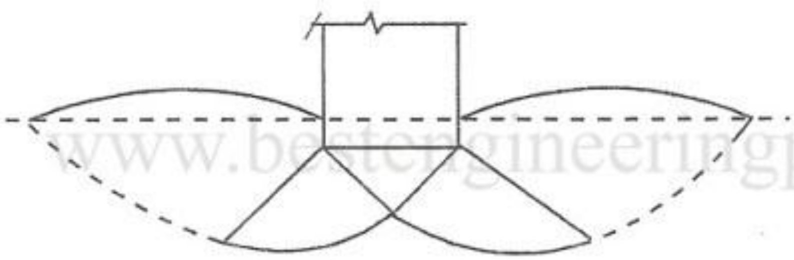


Fig 1 General shear failure



Local Shear Failure – This type of failure occurs in medium dense sand with relative density between 35 – 70 %. In this type of failure, failure takes place at a very large strain. The load settlement curve does not show a well-defined peak as shown in Fig.2. At failure only a small portion of soil underneath the footing participates and well-defined rupture surfaces develop only at points directly below the footing. Bulging of soil at surface begins when strain exceeds about 8 %. The curve shows increase in resistance after failure.



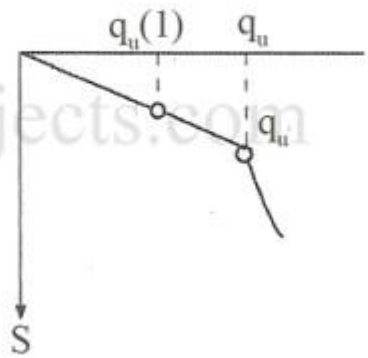
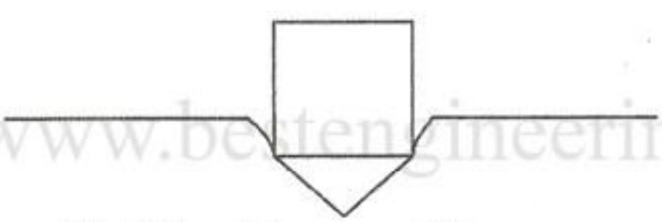


Fig 2 Local shear failure

Punching Shear Failure – This type of failure occurs in loose sand or soft clay with relative density less than 35 %. In this type of failure, footing penetrates into the soil without any bulging in the soil at the surface. Increase in vertical load increases the vertical movement and compression in the foundation soil. The failure is accompanied by vertical shear around the perimeter of the footing. At failure, soil outside the loaded area does not participate and there will be no movement of soil on the sides of the footing. This type of failure is shown in Fig.3.



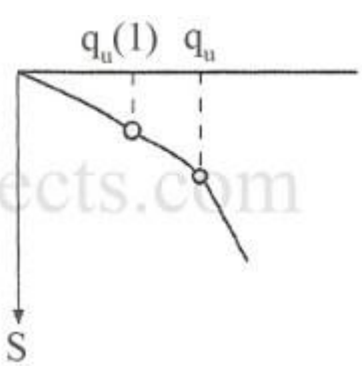


Fig 3 Punching shear failure

Some Definition

Ultimate Bearing Capacity (Q_{ult}) – It is the maximum soil pressure at the base of the foundation which causes shear failure of the supporting soil.

Net Ultimate Bearing Capacity ($(Q_{ult})_{net}$) – It is the maximum soil pressure in excess of overburden at the base of the foundation which causes shear failure of the supporting soil.

Safe Bearing Capacity (q_s) – It is the safe soil pressure at the base of the foundation which the soil will resist safely without any risk of shear failure irrespective of any settlement that may occur.

Net Safe Bearing Capacity (q_{ns}) – It is the safe soil pressure at the base of the foundation in excess of overburden which the soil will resist safely without any risk of shear failure irrespective of any settlement that may occur.

Safe Bearing Pressure (q_u) – The intensity of loading that will cause a permissible settlement specified for a given structure.

Allowable Bearing Capacity (q_a) – It is the maximum safe soil pressure at the base of the foundation which neither causes shear failure nor produces any settlement in excess of a specified value.

The bearing capacity of soil is defined as the capacity of the soil to bear the loads coming from the foundation. The pressure which the soil can easily withstand against load is called allowable bearing pressure.

Types of Bearing Capacity of Soil

Following are some types of bearing capacity of soil:

1. Ultimate bearing capacity (q_u)

The gross pressure at the base of the foundation at which soil fails is called ultimate bearing capacity.

2. Net ultimate bearing capacity (q_{nu})

By neglecting the overburden pressure from ultimate bearing capacity we will get net ultimate bearing capacity.

$$q_{nu} = q_u - \Upsilon D_f$$

3. Net safe bearing capacity (q_{ns})

By considering only shear failure, net ultimate bearing capacity is divided by certain factor of safety will give the net safe bearing capacity.

$$q_{ns} = q_{nu}/ F$$

Where F = factor of safety = 3 (usual value)

4. Gross safe bearing capacity (q_s)

When ultimate bearing capacity is divided by factor of safety it will give gross safe bearing capacity.

$$q_s = q_u/F$$

Net safe settlement pressure (q_{np})

The pressure with which the soil can carry without exceeding the allowable settlement is called net safe settlement pressure.

6. Net allowable bearing pressure (q_{na})

This is the pressure we can used for the design of foundations. This is equal to net safe bearing pressure if $q_{np} > q_{ns.}$ In the reverse case it is equal to net safe settlement pressure.

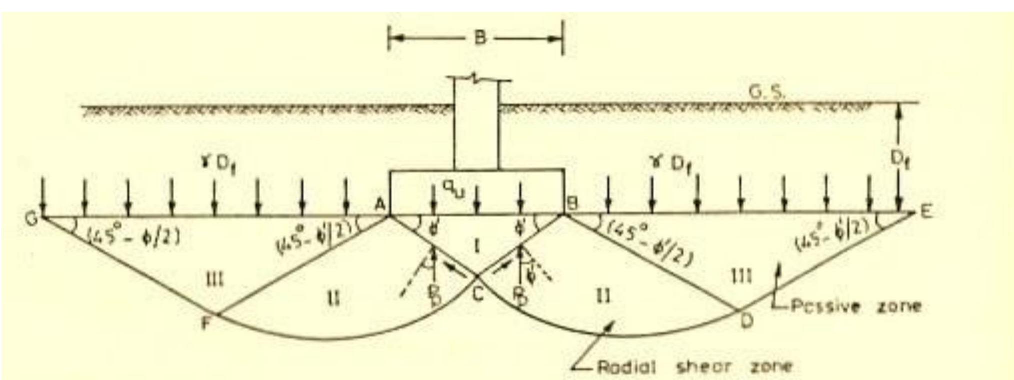
Calculation of Bearing Capacity

For the calculation of bearing capacity of soil, there are so many theories. But all the theories are superseded by Terzaghi's bearing capacity theory.

1. Terzaghi's bearing capacity theory

Terzaghi's bearing capacity theory is useful to determine the bearing capacity of soils under a strip footing. This theory is only applicable to shallow foundations. He considered some assumptions which are as follows.

1. The base of the strip footing is rough.
2. The depth of footing is less than or equal to its breadth i.e., shallow footing.
3. He neglected the shear strength of soil above the base of footing and replaced it with uniform surcharge. (γD_f)
4. The load acting on the footing is uniformly distributed and is acting in vertical direction.
5. He assumed that the length of the footing is infinite.
6. He considered Mohr-coulomb equation as a governing factor for the shear strength of soil.



As shown in above figure, AB is base of the footing. He divided the shear zones into 3 categories. Zone -1 (ABC) which is under the base is acts as if it were a part of the footing itself. Zone -2 (CAF and CBD) acts as radial shear zones which is bear by the sloping edges AC and BC. Zone -3 (AFG and BDE) is named as Rankine's passive zones which are taking surcharge (y D_f) coming from its top layer of soil. From the equation of equilibrium, Downward forces = upward forces Load from footing x weight of wedge = passive pressure + cohesion x CB sinΦ

$$q_u \text{ X } B + \left(\frac{1}{4} \Upsilon B^2 \sin \phi\right) = 2P_p + 2c' \text{ x } \left(\frac{B}{2 \cos\phi} . \text{Sin}\phi\right)$$

Where P_p = resultant passive pressure = $(P_p)_y$ + $(P_p)_c$ + $(P_p)_q$ $(P_p)_{y \text{ is}}$ derived by considering weight of wedge BCDE and by making cohesion and surcharge zero. $(P_p)_{c \text{ is}}$ derived by considering cohesion and by neglecting weight and surcharge. $(P_p)_q$ is derived by considering surcharge and by neglecting weight and cohesion.

Therefore,

$$q_u \text{ X } B = 2((P_p)_y + (P_p)_c + (P_p)_q) + \left(\frac{Bc'}{\cos\phi} . \text{Sin}\phi\right) - \left(\frac{1}{4} \Upsilon B^2 \sin \phi\right)$$

$$2(P_p)_y - \frac{1}{4} \Upsilon B^2 \sin \phi = B \text{ x } 0.5 \Upsilon B N_y$$

$$2(P_p)_y + Bc' \tan \phi = B c' N_c$$

By substituting,

$$2(P_p)_q = B \Upsilon D_f N_q$$

So, finally we get **q_u = c'N_c + y D_f N_q + 0.5 y B N_y** The above equation is called as Terzaghi's bearing capacity equation. Where q_u is the ultimate bearing capacity and N_c, N_q, N_y are the Terzaghi's bearing capacity factors. These dimensionless factors are dependents of angle of shearing resistance (). **Equations to find the bearing capacity factors**

$$N_c = \cot\phi\left[\frac{a^2}{2\cos^2(45+\phi/2)} - 1\right]$$

$$N_q = \left[\frac{a^2}{2\cos^2(45+\phi/2)}\right] \text{ and}$$

$$N_\gamma = 0.5\tan\phi\left[\frac{K_p}{\cos^2\phi} - 1\right]$$

are: Where $a = e^{\left(\frac{3\pi}{4} - \frac{\phi}{2}\right)\tan\phi}$ Kp = coefficient of passive earth pressure.

The ***plate load*** *test* is a field test, which is performed to determine the ultimate bearing capacity of the soil and the probable settlement under a given load. This test is very popular for the selection and design of the shallow foundation.

For performing this test, the plate is placed at the desired depth, then the load is applied gradually and the settlement for each increment of the load is recorded. At one point a settlement occurs at a rapid rate, the total load up to that point is calculated and divided by the area of the plate to determine the **ultimate bearing capacity** of soil at that depth. The ultimate bearing capacity is then divided by a safety factor (typically 2.5~3) to determine the **safe bearing capacity**.

Plate Load Test Apparatus / Equipment

The following plate load test apparatus is necessary for performing the test.

1. A steel plate is at least 300 mm square and 6 mm thick.
2. Hydraulic jack & pump
3. A hydraulic jack with a capacity of at least 1.5 times the anticipated test load.
4. A set of steel shims, at least 6 mm thick.
5. Reaction beam or reaction truss
6. A dial gauge, with a range of 0-250 mm and an accuracy of 0.02 mm.
7. Pressure gauge

8. A loading frame with a capacity of at least 1.5 times the anticipated test load. The frame should be designed so that it can be firmly attached to the ground, and so that the load can be applied to the center of the plate.
9. Necessary equipment for the loading platform.
10. A steel rule or tape measure is at least 3 m long.
11. Tripod, Plumb bob, spirit level, etc.
12. A hammer.
13. A set of wrenches.
14. A clean, dry cloth.

Plate Load Test Procedure

The necessary steps to perform a plate load test is written below-

1. Excavate test pit up to the desired depth. The pit size should be at least 5 times the size of the test plate (B_p).
2. At the center of the pit, a small hole or depression is created. The size of the hole is the same as the size of the steel plate. The bottom level of the hole should correspond to the level of the actual foundation. The depth of the hole is created such that the ratio of the depth to width of the hole is equal to the ratio of the actual depth to the actual width of the foundation.
3. A mild steel plate is used as a load-bearing plate whose thickness should be at least 25 mm thickness and size may vary from 300 mm to 750 mm. The plate can be square or circular. Generally, a square plate is used for square footing and a circular plate is used for circular footing.
4. A column is placed at the center of the plate. The load is transferred to the plate through the centrally placed column.
5. The load can be transferred to the column either by gravity loading method or by truss method.

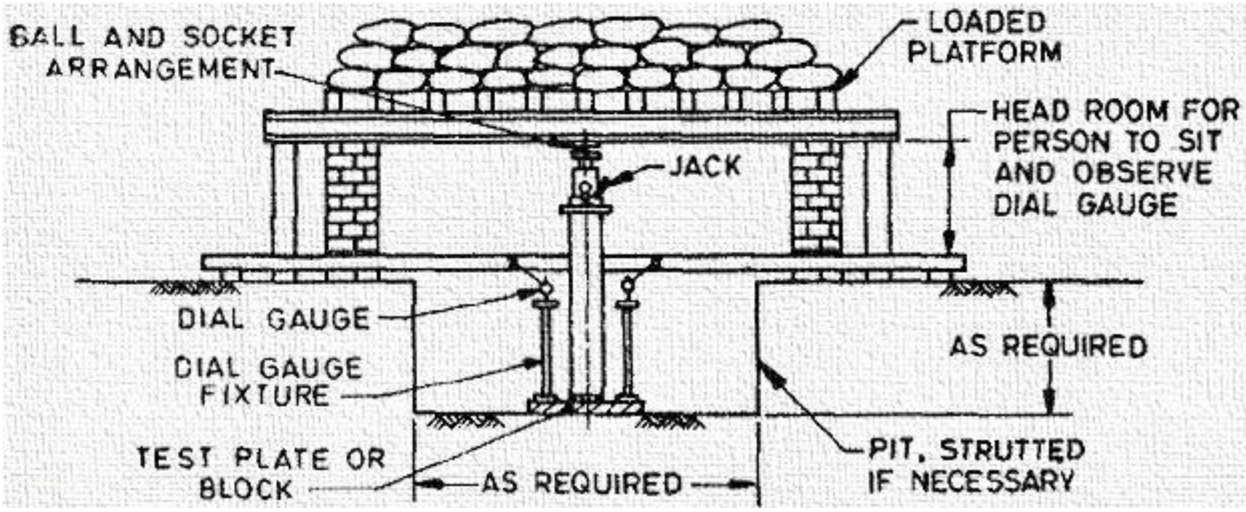


Figure: Test Setup for Plate Load Test

6. For gravity loading method a platform is constructed over the column and load is applied to the platform by means of sandbags or any other dead loads. The hydraulic jack is placed in between column and loading platform for the application of gradual loading. This type of loading is called reaction loading.
7. At least two dial gauges should be placed at diagonal corners of the plate to record the settlement. The gauges are placed on a platform so that it does not settle with the plate.
8. Apply seating load of .7 T/m^2 and release before the actual loading starts.
9. The initial readings are noted.
10. The load is then applied through the hydraulic jack and increased gradually. The increment is generally one-fifth of the expected safe bearing capacity or one-tenth of the ultimate bearing capacity or any other smaller value. The applied load is noted from the pressure gauge.
11. The settlement is observed for each increment and from dial gauge. After increasing the load-settlement should be observed after 1, 4, 10, 20, 40, and 60 minutes and then at hourly intervals until the rate of settlement is less than .02 mm per hour. The readings are noted in tabular form.
12. After completing the collection of data for a particular loading, the next load increment is applied and readings are noted under new load. This

increment and data collection is repeated until the maximum load is applied. The maximum load is generally 1.5 times the expected ultimate load or 3 times of the expected allowable bearing pressure.

Standard Penetration Test (SPT)

The standard penetration test is an in-situ test that is coming under the category of penetrometer tests. The standard penetration tests are carried out in borehole. The test will measure the resistance of the soil strata to the penetration undergone. A penetration emphirical correlation is derived between the soil properties and the penetration resistance. The test is extremely useful for determining the relative density and the angle of shearing resistance of cohesionless soils. It can also be used to determine the unconfined compressive strength of cohesive soils.

Tools for Standard Penetration Test

The requirements to conduct SPT are:

1. Standard Split Spoon Sampler
2. Drop Hammer weighing 63.5kg
3. Guiding rod
4. Drilling Rig.
5. Driving head (anvil).

Procedure for Standard Penetration Test

The test is conducted in a bore hole by means of a standard split spoon sampler.Once the drilling is done to the desired depth, the drilling tool is removed and the sampler is placed inside the bore hole. By means of a drop hammer of 63.5kg mass falling through a height of 750mm at the rate of 30 blows per minute, the sampler is driven into the soil. This is as per IS -2131:1963. The number of blows of hammer required to drive a depth of 150mm is counted. Further it is driven by 150 mm and the blows are counted. Similarly, the sampler is once again further driven by 150mm and the number of blows recorded. The number of blows recorded for the first 150mm not taken into consideration.. The number of blows recorded for last two 150mm intervals are added to give the **standard penetration number (N)**. In other words,

N = No: of blows required for 150mm penetration beyond seating drive of 150mm.

Questions

1. **What are the functions of foundations ?**
2. **What is shallow foundations &Explain its types ?**
3. **What is deep foundations & Explain its types ?**
4. **What are different types of shear failure ?**
5. **What is the bearing capacity of soil ?**
6. **Define different type of bearing capacity ?**
7. **What is plate load test ?**
8. **What is standard penetrations test ?**

THANK YOU