LECTURE NOTES

ON

GEOTECHNICAL ENGINEERING

Diploma in Civil Engineering

By

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Chapter-1

1.1 soil and soil Engineering

The term "**soil**" can have different meanings, depending upon the field in which it is considered.

To a geologist, it is the material in the relative thin zone of the Earth's surface within which roots occur, and which are formed as the products of past surface processes. The rest of the crust is grouped under the term "rock".

To a pedologist, it is the substance existing on the surface, which supports plant life.

To an engineer, it is a material that can be:

- **built on:** foundations of buildings, bridges
- **built in:** basements, culverts, tunnels
- **built with:** embankments, roads, dams
- **supported:** retaining walls

Soil Engineering is a discipline of Civil Engineering involving the study of soil, its behaviour and application as an engineering material.

1.2 Scope of Soil Mechanics

In soil mechanics we study about the various properties of the soil to be used for various engineering contraction works. There are various reasons that as a civil Engineer one must study this new branch of the Engineering science.

Foundations:

All the civil Engineering structures, ultimately rest on the soil. They transfer their whole load to the soil, so we have to construct the foundations to retain these structures. In case of the hard soil/ having sufficient strength we can provide the shallow foundations.
If we know the strength of the soil then we can decide which type of foundation is to be used. If the soil is weak in strength then we have to provide the deep foundations like pile foundation, well foundation etc. It is important to know the method to calculate the method to know the strength of the soil.

Earthen Dams:

There are so many earthen dams constructed to retain the water. The soil to be used for the construction of these earthen dams must be suitable enough to use it in its construction. Various properties of the soil, like it permeability, strength, and density are checked on regular basis to know if the soil compacted to required density or not.
The earthen dams are costly structure and also they have a high risk of getting failed, so they must be constructed with great care, so it is very important to study the properties of the soil

Embankments:

There are embankments constructed to raise the levels of the highways on the plains because there are chances of the floods etc, and also it is required to keep the foundation of the pavement above the water table. The embankments are generally constructed of the soil, which is tested for its various properties. There is need to design a economical embankment which is only possible by studying the various soil properties.

Canals or other retaining and under ground structures:

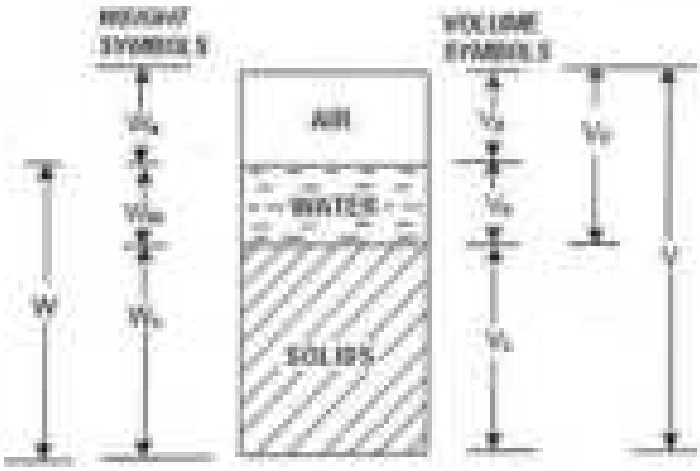
The canals also are formed by the soil which are to be constructed to be impermeable and of enough strength. The retaining structure like the retaining walls, are constructed to retain the earth. The earth properties are important to know about. The properties like the earth pressure, shear strength etc gives us the idea to design the retaining structure.
The soil strata is constantly investigated by the geologist to give the idea of the type of construction to be carried further in case of the tunnelling.

Chapter-2

2.1Soil as a three phase system

Soil is not a coherent solid material like steel and concrete, but is a particulate material. Soils, as they exist in nature, consist of solid particles (mineral grains, rock fragments) with water and air in the voids between the particles. The water and air contents are readily changed by changes in ambient conditions and location.

As the relative proportions of the three phases vary in any soil deposit, it is useful to consider a soil model which will represent these phases distinctly and properly quantify the amount of each phase. A schematic diagram of the three-phase system is shown in terms of weight and volume symbols respectively for soil solids, water, and air. The weight of air can be neglected.

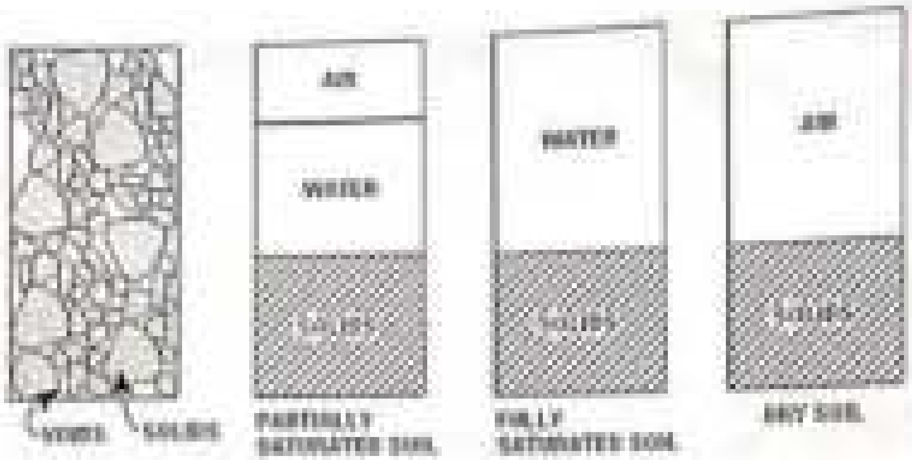


The soil model is given dimensional values for the solid, water and air components.

Total volume, $V = V_a + V_w + V_s$

Soils can be partially saturated (with both air and water present), or be fully saturated (no air content) or be perfectly dry (no water content).

In a saturated soil or a dry soil, the three-phase system thus reduces to two phases only, as shown.



For the purpose of engineering analysis and design, it is necessary to express relations between the weights and the volumes of the three phases.

2.2 Weight volume Relationships

The various relations can be grouped into:

- **Volume relations**
- **Weight relations**
- **Inter-relations**
- As the amounts of both water and air are variable, the volume of solids is taken as the reference quantity. Thus, several relational volumetric quantities may be defined.

Volume relations:

- **1. Void ratio (e)** is the ratio of the volume of voids (V_v) to the volume of soil solids (V_s), and is expressed as a decimal.

$$e = \frac{V_v}{V_s}$$

2. Porosity (n) is the ratio of the volume of voids to the total volume of soil (V), and is expressed as a percentage. $n = \frac{V_v}{V} \times 100$

- Void ratio and porosity are inter-related to each other as follows:

$$e = \frac{n}{1-n} \quad \text{and} \quad n = \frac{e}{(1+e)}$$

- 3. The volume of water (V_w) in a soil can vary between zero (i.e. a dry soil) and the volume of voids. This can be expressed as the **degree of saturation (S)** in percentage.

$S = \frac{V_w}{V_v} x 100$

For a dry soil, S = 0%, and for a fully saturated soil, S = 100%.

- **4. Air content (a_c)** is the ratio of the volume of air (V_a) to the volume of voids.

  $a_c = \frac{V_a}{V_v}$

- **5. Percentage air voids (n_a)** is the ratio of the volume of air to the total volume.

  $n_a = \frac{V_a}{V} x 100 = n x a_c$

Weight relations

Density is a measure of the quantity of mass in a unit volume of material. Unit weight is a measure of the weight of a unit volume of material. Both can be used interchangeably. The units of density are ton/m³, kg/m³ or g/cm³. The following are the **basic weight relations:**

1. The ratio of the mass of water present to the mass of solid particles is called the **water content (w)**, or sometimes the **moisture content.**

$w = \frac{W_w}{W_s}$

Its value is 0% for dry soil and its magnitude can exceed 100%.

2. The mass of solid particles is usually expressed in terms of their **particle unit weight** (γ_s) or **specific gravity (G_s)** of the soil grain solids .

$\gamma_s = \frac{W_s}{V_s} = G_s \gamma_w$

where γ_w = Unit weight of water

For most inorganic soils, the value of G_s lies between 2.60 and 2.80. The presence of organic material reduces the value of G_s.

3. **Dry unit weight** (γ_d) is a measure of the amount of solid particles per unit volume.

$\gamma_d = \frac{W_s}{V}$

4. **Bulk unit weight** $(\gamma_t \text{ or } \gamma)$ is a measure of the amount of solid particles plus water per unit volume.

$$\gamma_t = \gamma = \frac{(W_s + W_w)}{(V_s + V_v)}$$

5. Saturated unit weight (γ_{sat}) is equal to the bulk density when the total voids is filled up with water.

6. Buoyant unit weight (γ') or **submerged unit weight** is the effective mass per unit volume when the soil is submerged below standing water or below the ground water table.

$\gamma' = \gamma_{sat} - \gamma_w$

Inter-relations

It is important to quantify the state of a soil immediately after receiving in the laboratory and prior to commencing other tests. The water content and unit weight are particularly important, since they may change during transportation and storage.

Some physical state properties are calculated following the practical measurement of others. For example, dry unit weight can be determined from bulk unit weight and water content. The following are some **inter-relations:**

1.

$$w = \frac{W_w}{W_s} = \frac{\gamma_w V_w}{G_s \gamma_w V_s} = \frac{V_w}{G_s V_s} = \frac{S V_v}{G_s V_s} = \frac{S e}{G_s}$$

2.

$$\gamma = \frac{(G_s + S e)\gamma_w}{1 + e}$$

3.

$$\gamma = \frac{(1 + w) G_s \gamma_w}{1 + e}$$

4.

$$\gamma_d = \frac{G_s \gamma_w}{1 + e}$$

5.

$$\gamma_d = \frac{\gamma}{1 + w}$$

6.

$$\gamma' = \frac{[(G_s - 1) + (S - 1)e]\gamma_w}{1 + e}$$

7.

$$\gamma' = \frac{(G_s - 1)\gamma_w}{1 + e}$$

Chapter-3 Determination of Index properties

Index properties are the properties of soil that help in identification and classification of soil. These properties are generally determined in the laboratory. In situ density and relative density require undisturbed sample extraction while other quantities can be determined from disturbed soil sampling. Following are the major properties of soils.

1. Water Content
2. Unit weight of Soil/ In-situ density
3. Specific Gravity
4. Consistency Limits
5. Particle Size Distribution
6. Sensitivity and activity of Clays

3.1Water Content(Pycnometer method,Oven drying method)

A.Pycnometer

Pycnometer method is also useful to determine water content. But this is used when the specific gravity of the given soil sample is already known. However, Specific gravity can also be determined using pycnometer.

Apparatus used in pycnometer method are

- Pycnometer
- Weighing balance with an accuracy of 1.0g
- Glass rod
- Vacuum pump

Test procedure of pycnometer method is as follows

- Wash, clean and dry the pycnometer and note down its mass (M_1) along with brass cap and washer using weighing balance with an accuracy of 1.0 g.
- Now place a sample of wet soil about 200 to 400 g in pycnometer and note down its mass (M_2).
- Then add water to the soil in the pycnometer to make it about half full.
- Stir the soil using glass rod to remove air voids of the soil sample. If available connect the vacuum pump to the soil specimen to remove entrapped air.
- Add some more water and after eliminating the entrapped air stop stirring and fix the brass cap. More water is added through hole in brass cap until the water is flush with the hole.
- Now take the mass of pycnometer (M_3).
- Now empty and wash the pycnometer. Then fill it with only water and take its mass (M_4).

Observations and Calculations of Pycnometer Method

The water content (w) of the soil sample using pycnometer method is calculated from the below formula

$$w = \left[\frac{M_2 - M_1}{M_3 - M_4}\left(\frac{G-1}{G}\right) - 1\right] \times 100$$

Where M_1=mass of empty Pycnometer,

M_2= mass of the Pycnometer with wet soil

M_3= mass of the Pycnometer and soil, filled with water,

M_4 = mass of Pycnometer filled with water only.
G= Specific gravity of solids.

Table 2: Observations and Calculations of Pycnometer Method

Sl. No.	Observations and Calculations	Determination No. 1	2	3
	Observation			
1	Mass of empty pycnometer (M_1)			
2	Mass of pycnometer + wet soil (M_2)			
3	Mass of Pycnometer soil filled with water (M_3)			
4	Mass of Pycnometer filled with water only (M_4)			
	Calculations			
5	$M_2 - M_1$			
6	$M_3 - M_4$			
7	$(G - 1) / G$			
8	w (using above formula)			

Result of Pycnometer Method
Water content of the given soil sample = _______%.

B.Oven Drying Method

OBJECTIVE

Determine the natural water content of the given soil sample.

NEED AND SCOPE OF THE EXPERIMENT

In almost all soil tests natural moisture content of the soil is to be determined. The knowledge of the natural moisture content is essential in all studies of soil mechanics. To sight a few, natural moisture content is used in determining the bearing capacity and settlement. The natural moisture content will give an idea of the state of soil in the field.

DEFINITION

The natural water content also called the natural moisture content is the ratio of the weight of water to the weight of the solids in a given mass of soil. This ratio is usually expressed as percentage.

APPARATUS REQUIRED

1. Non-corrodible air-tight container.
2. Electric oven, maintain the temperature between 1050 C to 1100 C.
3. Desiccator.
4. Balance of sufficient sensitivity.

PROCEDURE

1. Clean the container with lid dry it and weigh it (W1).
2. Take a specimen of the sample in the container and weigh with lid (W2).
3. Keep the container in the oven with lid removed. Dry the specimen to constant weight maintaining the temperature between 1050 C to 1100 C for a period varying with the type of soil but usually 16 to 24 hours.

4. Record the final constant weight (W3) of the container with dried soil sample. Peat and other organic soils are to be dried at lower temperature (say 600) possibly for a longer period.

OBSERVATIONS AND RECORDING

Data and observation sheet for water content determination

S.No.	Sample No.	1	2	3
1	Weight of container with lid W_1 gm			
2	Weight of container with lid +wet soil W_2 gm			
3	Weight of container with lid +dry soil W_3 gm			
4	Water/Moisture content W = [(W₂□W₃)/(W₃□W₁)]�100			

RESULT

The natural moisture content of the soil sample is ________

3.2 Specific Gravity

OBJECTIVE

Determine the specific gravity of soil fraction passing 4.75 mm I.S sieve by density bottle.

NEED AND SCOPE

The knowledge of specific gravity is needed in calculation of soil properties like void ratio, degree of saturation etc.

DEFINITION

Specific gravity G is defined as the ratio of thé weight of an equal volume of distilled water at that temperature both weights taken in air.

APPARATUS REQUIRED

1. Density bottle of 50 ml with stopper having capillary hole.
2. Balance to weigh the materials (accuracy 10gm).
3. Wash bottle with distilled water.
4. Alcohol and ether.

PROCEDURE

1. Clean and dry the density bottle

   a. wash the bottle with water and allow it to drain.
   b. Wash it with alcohol and drain it to remove water.
   c. Wash it with ether, to remove alcohol and drain ether.

2. Weigh the empty bottle with stopper (W_1)

3. Take about 10 to 20 gm of oven soil sample which is cooled in a desiccator. Transfer it to the bottle. Find the weight of the bottle and soil (W_2).

4. Put 10ml of distilled water in the bottle to allow the soil to soak completely. Leave it for about 2 hours.

5. Again fill the bottle completely with distilled water put the stopper and keep the bottle

   under constant temperature water baths (T_x°).

6. Take the bottle outside and wipe it clean and dry sure. Now determine the weight of the bottle and the contents (W_3).

7. Now empty the bottle and thoroughly clean it. Fill the bottle with only distilled water and weigh it. Let it be W_4 at temperature (T_x° C).

8. Repeat the same process for 2 to 3 times, to take the average reading of it.

OBSERVATIONS

S. No.	Observation Number	1	2	3
1	Weight of density bottle (W_1 g)			
2	Weight of density bottle + dry soil (W_2 g)			
3	Weight of bottle + dry soil + water at temperature T_x ^{0}C (W_3 g)			
4	Weight of bottle + water (W_4 g) at temperature T_x ^{0}C			
	Specific gravity G at T_x ^{0}C			
	Average specific gravity at T_x ^{0}C			

CALCULATIONS

$$\text{Specific gravity of soil} = \frac{\text{Density of water at } 27^\circ \text{C}}{\text{Weight of water of equal volume}}$$

$$= \frac{(W_2 - W_1)}{(W_4 - W_1) - (W_3 - W_2)}$$

$$= \frac{(W_2 - W_1)}{(W_2 - W_1) - (W_3 - W_4)}$$

The specific gravity of the soil particles lie within the range of 2.65 to 2.85. Soils containing organic matter and porous particles may have specific gravity values below 2.0. Soils having heavy substances may have values above 3.0.

3.3 Particle size distribution

OBJECTIVE

(a). Select sieves as per *I.S* specifications and perform sieving.
(b). Obtain percentage of soil retained on each sieve.
(c). Draw graph between log grain size of soil and % finer.

NEED AND SCOPE OF EXPERIMENT

The grain size analysis is widely used in classification of soils. The data obtained from grain size distribution curves is used in the design of filters for earth dams and to determine suitability of soil for road construction, air field etc. Information obtained from grain size analysis can be used to predict soil water movement although permeability tests are more generally used.

PLANNING AND ORGANISATION

Apparatus

- 1.Balance
- 2.I.S sieves
- 3.Rubber pestle and mortar.
- 4.mechanical Sieve Shaker

The grain size analysis is an attempt to determine the relative proportions of different grain sizes which make up a given soil mass.

KNOWLEDGE OF EQUIPMENT

1.The balance to be used must be sensitive to the extent of 0.1% of total weight of sample taken.

2.I.S 460-1962 are to used. The sieves for soil tests: 4.75 mm to 75 microns.

PROCEDURE

1.For soil samples of soil retained on 75 micron I.S sieve

(a) The proportion of soil sample retained on 75 micron I.S sieve is weighed and recorded weight of soil sample is as per I.S 2720.

(b) I.S sieves are selected and arranged in the order as shown in the table.

(c) The soil sample is separated into various fractions by sieving through above sieves placed in the above mentioned order.

(d) The weight of soil retained on each sieve is recorded.

(e) The moisture content of soil if above 5% it is to be measured and recorded.

2.No particle of soil sample shall be pushed through the sieves.

OBSERVATIONS AND RECORDING

Weight of soil sample:

Moisture content:

I.S sieve number or size in mm	Wt. Retained in each	Percentage on each sieve	Cumulative %age retained on each sieve	% finer	Remarks

	sieve (gm)				
4.75					
4.00					
3.36					
2.40					
1.46					
1.20					
0.60					
0.30					
0.15					
0.075					

GRAPH

Draw graph between log sieve size vs % finer. The graph is known as grading curve. Corresponding to 10%, 30% and 60% finer, obtain diameters from graph are designated as D_{10}, D_{30}, D_{60}.

CALCULATION

1. The percentage of soil retained on each sieve shall be calculated on the basis of total weight of soil sample taken.
2. Cumulative percentage of soil retained on successive sieve is found.

II.HYDROMETER ANALYSIS

OBJECTIVE

Grain size analysis of soils by hydrometer analysis test.

SPECIFIC OBJECTIVE

1. To determine the grain size distribution of soil sample containing appreciable amount of fines.

2. To draw a grain size distribution curve.

NEED AND SCOPE OF THE EXPERIMENT

For determining the grain size distribution of soil sample, usually mechanical analysis (sieve analysis) is carried out in which the finer sieve used is 63 micron or the nearer opening. If a soil contains appreciable quantities of fine fractions in (less than 63 micron) wet analysis is done. One form of the analysis is hydrometer analysis. It is very much helpful to classify the soil as per ISI classification. The properties of the soil are very much influenced by the amount of clay and other fractions.

APPARATUS

1. Hydrometer

2. Glass measuring cylinder-Two of 1000 ml capacity with ground glass or rubber stoppers, about 7 cm diameter and 33 cm high marked at 1000 ml volume.

3. Thermometer- To cover the range 0 to 50° C with an accuracy of 0.5 °C .
4. Water bath.
5. Stirring apparatus.
6. I.S sieves apparatus.
7. Balance-accurate to 0.01 gm.
8. Oven-105 to 110.
9. Stop watch.
10. Desiccators
11. Centimeter scale.

12. Porcelain evaporating dish.
13. Wide mouth conical flask or conical beaker of 1000 ml capacity.
14. Thick funnel-about 10 cm in diameter.
15. Filter flask-to take the funnel.
16. Measuring cylinder-100 ml capacity.
17. Wash bottle-containing distilled water.
18. Filter papers.
19. Glass rod-about 15 to 20 cm long and 4 to 5 mm in diameter.
20. Hydrogen peroxide-20 volume solution.
21. Hydrochloric acid N solution-89 ml of concentrated hydrochloric acid.(specific gravity 1.18) diluted with distilled water one litre of solution.
22. Sodium hexametaphosphate solution-dissolve 33 g of sodium hexametaphosphate and 7 gms of sodium carbonate in distilled water to make one litre of solution.

CALIBRATION OF HYDROMETER

Volume

(a) Volume of water displaced: Approximately 800 ml of water shall be poured in the 1000 ml measuring cylinder. The reading of the water level shall be observed and recorded.

The hydrometer shall be immersed in the water and the level shall again be observed and recorded as the volume of the hydrometer bulb in ml plus volume of that part of the stem that is submerged. For practical purposes the error to the inclusion of this stem volume may be neglected.

(b) From the weight of the hydrometer: The hydrometer shall be weighed to the nearest 0.1 gm.

The weight in gm shall be recorded as the volume of the bulb plus the volume of the stem below the 1000 ml graduation mark. For practical purposes the error due to the inclusion of this stem may be neglected.

Calibration

(a) The sectional area of the 1000 ml measuring cylinder in which the hydrometer is to used shall be determined by measuring the distance between the graduations. The sectional area is equal to the volume include between the two graduations divided by the measured distance between them.

�Place the hydrometer on the paper and sketch it. On the sketch note the lowest and highest readings which are on the hydrometer and also mark the neck of the bulb. Mark the center of the bulb which is half of the distance between neck of the bulb and tip of the bulb.

(b) The distance from the lowest reading to the center of the bulb is (R_h) shall be recorded

(R_h �$=H_L + L/2$).

(c) The distance from the highest hydrometer reading to the center of the bulb shall be measured and recorded.

(d) Draw a graph hydrometer readings vs H_H and R_H. A straight line is obtained. This calibration curve is used to calibrate the hydrometer readings which are taken with in 2 minutes.

(e) From 4 minutes onwards the readings are to be taken by immersing the hydrometer each time. This makes the soil solution to rise, there by rising distance of free fall of the particle. So correction is applied to the hydrometer readings.

(f) Correction applied to the R_h and H_H

$$\frac{R_h - V_h}{A} = \frac{H_h + h/2 - V_h}{2A}$$

V_h= Volume of hydrometer bulb in ml.

A� =Area of measuring cylinder in cm^2.

From these two corrected readings draw graph (straight line)

Grain Size Distribution in Soil-Data and Calculation Chart

Date:

Sample No:

Total weight of dry soil taken, W =

Specific Gravity of soil, G =

Hydrometer No.��_____________ � Wt. Of soil gone into solution , W_s =

Meniscus correction, C_m =������������������������������ Dispersion agent correction =

Reading in water RW���� =

Temperature correction�� =

% finer for wt. Of soil W_s gone into solution���� N=[(100G)/{W_s x (G �1)}] x R

1	2	3	4	5	6	7	8	9	10	11
Date	Time	Elapsed Time	Hydrometer reading upper	Corrected hydrometer	Zr =	Velocity Cms/sec	Equivalent dia. Of	R	N(%finer Than)	REMARKS

1	2	3	4	5	6	7	8	9	10	11
		in Sec	Menisc us R_h ◆ 1000	Readin g (1- lower menisc us C_m)	z'	V=Z◆ r/K or z_r / t	Particle D◆mm		or soil)	

3.4Consistency of soil

The **consistency** of a fine-grained soil refers to its firmness; and it varies with the water content of the soil.

A gradual increase in water content causes the soil to change from ***solid*** to ***semi-solid*** to ***plastic*** to ***liquid*** states. The water contents at which the consistency changes from one state to the other are called **consistency limits** (or **Atterberg limits**).

The three limits are known as the shrinkage limit (W_S), plastic limit (W_P), and liquid limit (W_L) as shown. The values of these limits can be obtained from laboratory tests.

Two of these are utilised in the classification of fine soils:

Liquid limit (W_L) - change of consistency from plastic to liquid state
Plastic limit (W_P) - change of consistency from brittle/crumbly to plastic state

The difference between the liquid limit and the plastic limit is known as the **plasticity index (I_P)**, and it is in this range of water content that the soil has a plastic consistency. The consistency of most soils in the field will be plastic or semi-solid.

LIQUID LIMIT TEST

OBJECTIVE

1.Prepare soil specimen as per specification.
2.Find the relationship between water content and number of blows.
3.Draw flow curve.
4.Find out liquid limit.

NEED AND SCOPE

Liquid limit is significant to know the stress history and general properties of the soil met with construction. From the results of liquid limit the compression index may be estimated. The compression index value will help us in settlement analysis. If the natural moisture content of soil is closer to liquid limit, the soil can be considered as soft if the moisture content is lesser than liquids limit, the soil can be considered as soft if the moisture content is lesser than liquid limit. The soil is brittle and stiffer.

THEORY

The liquid limit is the moisture content at which the groove, formed by a standard tool into the sample of soil taken in the standard cup, closes for 10 mm on being given 25 blows in a standard manner. At this limit the soil possess low shear strength.

APPARATUS REQUIRED

1. Balance 2. Liquid limit device (Casagrende's) 3. Grooving tool 4. Mixing dishes

5. Spatula 6. Electrical Oven

PROCEDURE

1. About 120 gm of air-dried soil from thoroughly mixed portion of material passing 425 micron I.S sieve is to be obtained.

2. Distilled water is mixed to the soil thus obtained in a mixing disc to form uniform paste. The paste shall have a consistency that would require 30 to 35 drops of cup to cause closer of standard groove for sufficient length.

3. A portion of the paste is placed in the cup of LIQUID LIMIT device and spread into portion with few strokes of spatula.

4. Trim it to a depth of 1cm at the point of maximum thickness and return excess of soil to the dish.

5. The soil in the cup shall be divided by the firm strokes of the grooving tool along the diameter through the centre line of the follower so that clean sharp groove of proper dimension is formed.

6. Lift and drop the cup by turning crank at the rate of two revolutions per second until the two halves of soil cake come in contact with each other for a length of about 1 cm by flow only.

7. The number of blows required to cause the groove close for about 1 cm shall be recorded.

8. A representative portion of soil is taken from the cup for water content determination.

9. Repeat the test with different moisture contents at least three more times for blows between 10 and 40.

OBSERVATIONS

Details of the sample:.......

Natural moisture content:........ Room temperature:.............

Determination Number	1	2	3	4
Container number				
Weight of container				
Weight of container + wet soil				
Weight of container + dry soil				
Weight of water				
Weight of dry soil				
Moisture content (%)				
No. of blows				

COMPUTATION / CALCULATION

Draw a graph showing the relationship between water content (on y-axis) and number of blows (on x-axis) on semi-log graph. The curve obtained is called flow curve. The moisture content corresponding to 25 drops (blows) as read from the represents liquid limit. It is usually expressed to the nearest whole number.

INTERPRETATION AND RECORDING

Flow index $I_f = (W_2 - W_1)/(\log N_1/N_2)$ = slope of the flow curve.

Plasticity Index = w_l-w_p =

Toughness Index = I_p/I_f =

PLASTIC LIMIT TEST

NEED AND SCOPE

Soil is used for making bricks , tiles , soil cement blocks in addition to its use as foundation for structures.

APPARATUS REQUIRED

1.Porcelain dish.
2.Glass plate for rolling the specimen.
3.Air tight containers to determine the moisture content.
4.Balance of capacity 200gm and sensitive to 0.01gm
5.Oven thermostatically controlled with interior of non-corroding material to maintain the temperature around 105^0 and 110^0C.

PROCEDURE

1. Take about 20gm of thoroughly mixed portion of the material passing through 425 micron I.S. sieve obtained in accordance with I.S. 2720 (part 1).

2. Mix it thoroughly with distilled water in the evaporating dish till the soil mass becomes plastic enough to be easily molded with fingers.

3. Allow it to season for sufficient time (for 24 hrs) to allow water to permeate throughout the soil mass

4. Take about 10gms of this plastic soil mass and roll it between fingers and glass plate with just sufficient pressure to roll the mass into a threaded of uniform diameter throughout its length. The rate of rolling shall be between 60 and 90 strokes per minute.

5. Continue rolling till you get a threaded of 3 mm diameter.

6. Knead the soil together to a uniform mass and re-roll.

7. Continue the process until the thread crumbles when the diameter is 3 mm.

8. Collect the pieces of the crumbled thread in air tight container for moisture content determination.

9. Repeat the test to atleast 3 times and take the average of the results calculated to the nearest whole number.

OBSERVATION AND REPORTING

Compare the diameter of thread at intervals with the rod. When the diameter reduces to 3 mm, note the surface of the thread for cracks.

PRESENTATION OF DATA

Container No.		
Wt. of container + lid, W_1		
Wt. of container + lid + wet sample, W_2		
Wt. of container + lid + dry sample, W_3		
Wt. of dry sample = $W_3 - W_1$		
Wt. of water in the soil = $W_3 . W_2$		

Water content (%) = $(W_2 - W_3) / (W_3 - W_1) * 100$		

Average Plastic Limit=...............

Plasticity Index(Ip) = (LL - PL)=...........   Toughness Index =I_p/I_f

SHRINKAGE LIMIT TEST

OBJECTIVE

To determine the shrinkage limit and calculate the shrinkage ratio for the given soil.

THEORY

As the soil loses moisture, either in its natural environment, or by artificial means in laboratory it changes from liquid state to plastic state, from plastic state to semi-solid state and then to solid state. Volume changes also occur with changes in water content. But there is particular limit at which any moisture change does not cause soil any volume change.

NEED AND SCOPE

Soils which undergo large volume changes with change in water content may be troublesome. Volume changes may not and usually will not be equal.

A shrinkage limit test should be performed on a soil.

1. To obtain a quantitative indication of how much change in moisture can occur before any appreciable volume changes occurs
2. To obtain an indication of change in volume.

The shrinkage limit is useful in areas where soils undergo large volume changes when going through wet and dry cycles (as in case of earth dams)

APPARATUS

1. Evaporating Dish. Porcelain, about 12cm diameter with flat bottom.
2. Spatula
3. Shrinkage Dish. Circular, porcelain or non-corroding metal dish (3 nos) having a flat bottom and 45 mm in diameter and 15 mm in height internally.
4. Straight Edge. Steel, 15 cmm in length.
5. Glass cup. 50 to 55 mm in diameter and 25 mm in height , the top rim of which is ground smooth and level.
6. Glass plates. Two, each 75 � 75 mm one plate shall be of plain glass and the other shall have prongs.
7. Sieves. 2mm and 425- micron IS sieves.
8. Oven-thermostatically controlled.
9. Graduate-Glass, having a capacity of 25 ml and graduated to 0.2 ml and 100 cc one �mark flask.
10. Balance-Sensitive to 0.01 g minimum.
11. Mercury. Clean, sufficient to fill the glass cup to over flowing.
12. Wash bottle containing distilled water.

PROCEDURE

Preparation of soil paste

1. Take about 100 gm of soil sample from a thoroughly mixed portion of the material passing through 425-micron I.S. sieve.

2. Place about 30 gm the above soil sample in the evaporating dish and thoroughly mixed with distilled water and make a creamy paste.

Use water content some where around the liquid limit.

Filling the shrinkage dish

3. Coat the inside of the shrinkage dish with a thin layer of Vaseline to prevent the soil sticking to the dish.

4. Fill the dish in three layers by placing approximately 1/3 rd of the amount of wet soil with the help of spatula. Tap the dish gently on a firm base until the soil flows over the edges and no apparent air bubbles exist. Repeat this process for 2nd and 3rd layers also till the dish is completely filled with the wet soil. Strike off the excess soil and make the top of the dish smooth. Wipe off all the soil adhering to the outside of the dish.

5. Weigh immediately, the dish with wet soil and record the weight.

6. Air- dry the wet soil cake for 6 to 8hrs, until the colour of the pat turns from dark to light. Then oven-dry the to constant weight at 105°C to 110°C say about 12 to 16 hrs.

7. Remove the dried disk of the soil from oven. Cool it in a desiccator. Then obtain the weight of the dish with dry sample.

8. Determine the weight of the empty dish and record.

9. Determine the volume of shrinkage dish which is evidently equal to volume of the wet soil as follows. Place the shrinkage dish in an evaporating dish and fill the dish with mercury till it overflows slightly. Press it with plain glass plate firmly on its top to remove excess mercury. Pour the mercury from the shrinkage dish into a measuring jar and find the volume of the shrinkage dish directly. Record this volume as the volume of the wet soil pat.

Volume of the Dry Soil Pat

10. Determine the volume of dry soil pat by removing the pat from the shrinkage dish and immersing it in the glass cup full of mercury in the following manner.

Place the glass cup in a larger one and fill the glass cup to overflowing with mercury. Remove the excess mercury by covering the cup with glass plate with prongs and pressing it. See that no air bubbles are entrapped. Wipe out the outside of the glass cup to remove the adhering mercury. Then, place it in another larger dish, which is, clean and empty carefully.

Place the dry soil pat on the mercury. It floats submerge it with the pronged glass plate which is again made flash with top of the cup. The mercury spills over into the larger plate. Pour the mercury that is displayed by the soil pat into the measuring jar and find the volume of the soil pat directly.

CALCULATION

First determine the moisture content

Shrinkage limit $(W_S) = (W - (V - V_0) \times \gamma_w / W_0) \times 100$

Where, W = Moisture content of wet soil pat (%)

V = Volume of wet soil pat in cm3

V0 = Volume of dry soil pat in cm3

W0 = Weight of oven dry soil pat in gm

CAUTION

Do not touch the mercury with gold rings.

TABULATION AND RESULTS

S.No	Determination No.	1	2	3
1	Wt. of container in gm, W_1			

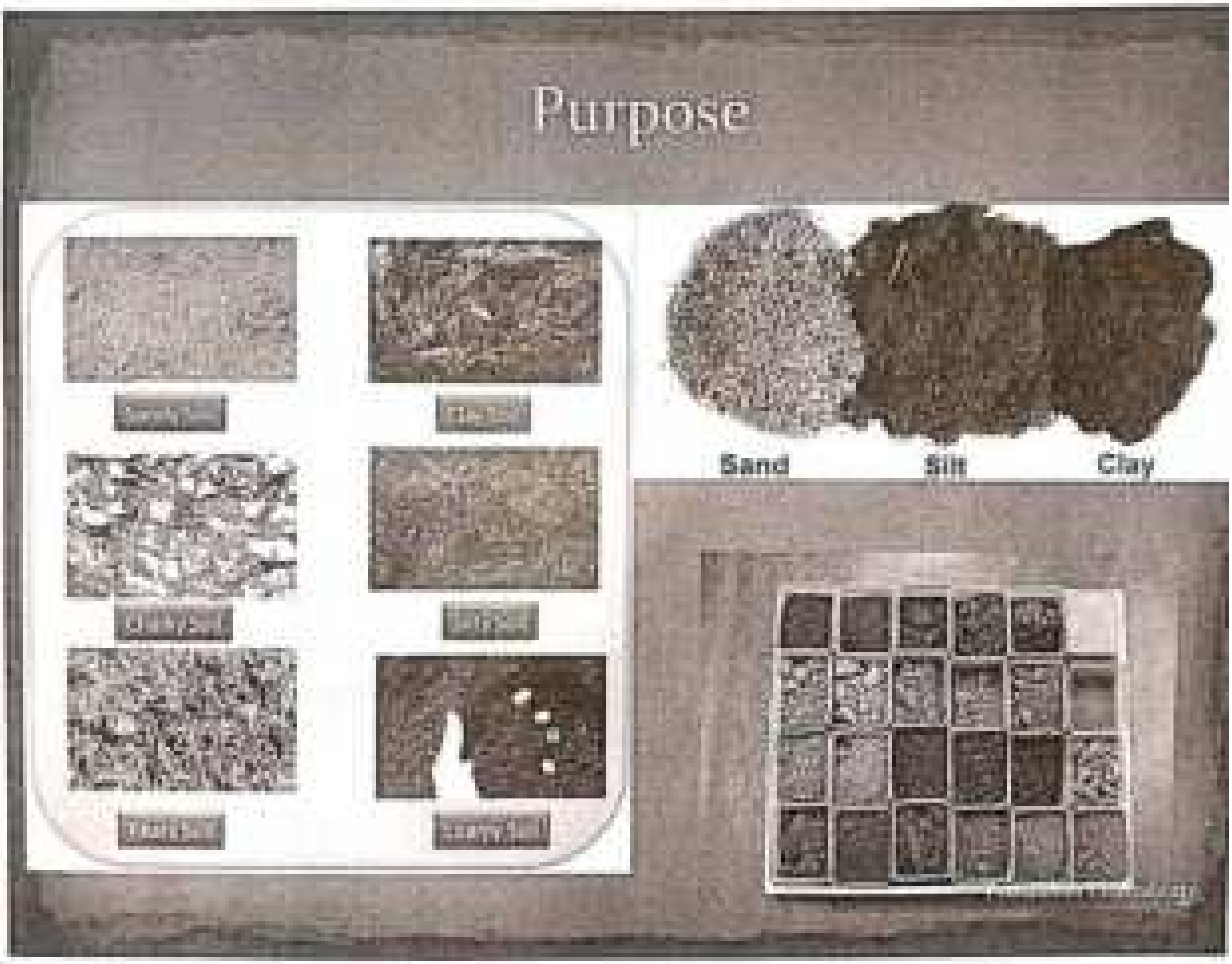
2	Wt. of container + wet soil pat in gm, W_2	
3	Wt. of container + dry soil pat in gm, W_3	
4	Wt. of oven dry soil pat, W_0 in gm	
5	Wt. of water in gm	
6	Moisture content (%), W	
7	Volume of wet soil pat (V), in cm	
8	Volume of dry soil pat (V_0) in cm^3	
9	By mercury displacement method a. Weight of displaced mercury b. Specific gravity of the mercury	
10.	Shrinkage limit (W_S)	
	Shrinkage ratio (R)	

Chapter – ~~3~~4

Classification of Soils

by

Pranamesh Chakraberty


Purpose
Sand
Silt
Clay

Purpose

- Arrange various types of soils into groups according to their engineering [illegible]

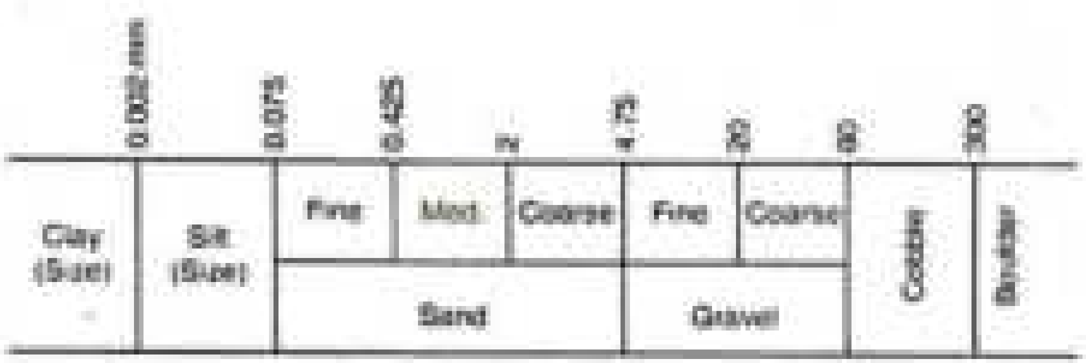
- Soil [illegible] placed in the same group [illegible]

Soil Classification Systems

1. Particle Size Classification
2. Textural Classification
3. Highway Research Board (HRB) Classification
4. Unified Soil Classification System (USCS)
5. Indian Standard Classification System (ISCS)

Introduction

IS Classification of Grain Size



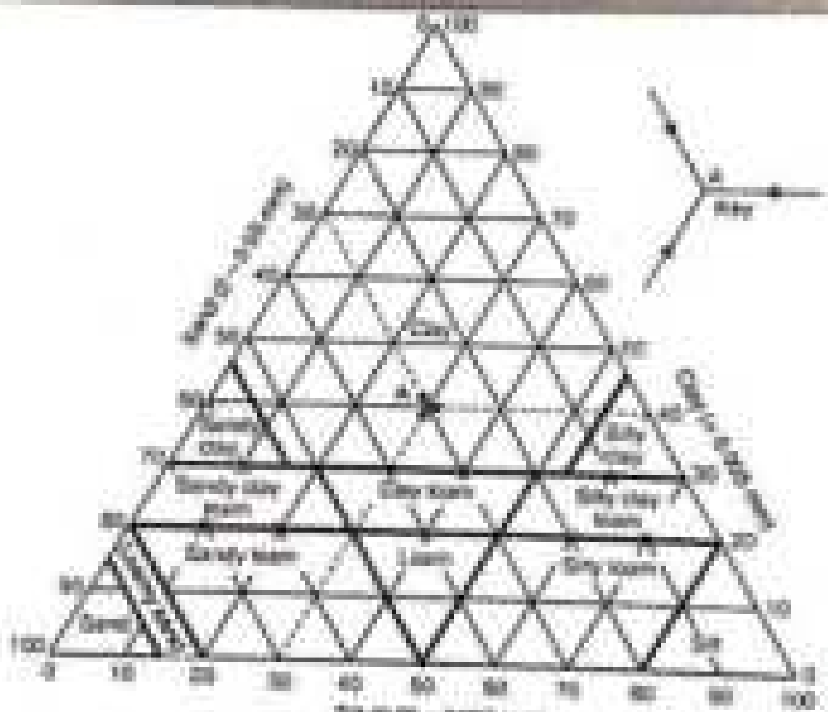
I.S. Classification (IS : 1498-1970)

Textural Classification

Introduction

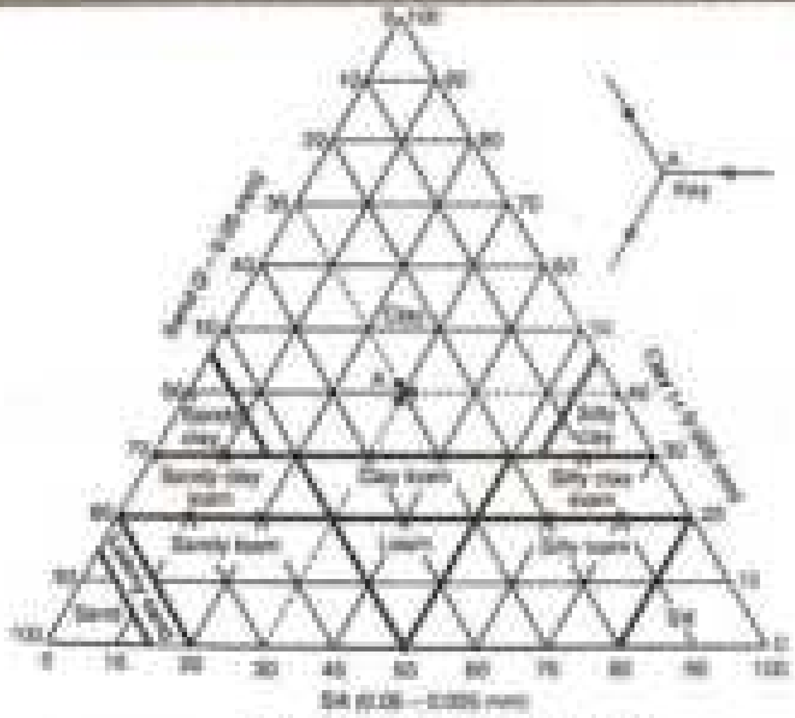
- Soil [illegible]
based on particle size [illegible]
- Most [illegible]
Public Roads [illegible]
- Classification [illegible]
- More suitable [illegible]
- Clay and [illegible]
size than [illegible]

Textural Classification Chart



TEXTURAL CLASSIFICATION CHART
(ADAPTED FROM U.S. PUBLIC ROADS ADMINISTRATION)

Example



TEXTURAL CLASSIFICATION CHART
(ADAPTED FROM U.S. PUBLIC ROADS ADMINISTRATION)

30% sand
30% silt
40% clay

Soil Type
– Clay

Introduction

- Based on both [illegible]
plasticity characteristics
- Mostly [illegible]
- [illegible]
- [illegible]
- [illegible]

Group Index (GI)

[illegible]

Group Index (GI) determination

Group Index of a soil depends on:

1. Amount of material passing 75 micron IS sieve
2. Liquid Limit
3. Plastic Limit.

$$\text{Group index} = 0.2\,a + 0.005\,ac + 0.01\,bd$$

where a = that portion of percentage passing 75 micron sieve greater than 35 and not exceeding 75 expressed as a whole number (0 to 40)

b = that portion of percentage passing 75 micron sieve greater than 15 and not exceeding 55 expressed as a whole number (0 to 40)

c = that portion of the numerical liquid limit greater than 40 and not exceeding 60 expressed as positive whole number (0 to 20)

and d = that portion of the numerical plasticity index greater than 10 and not exceeding 30 expressed as a positive whole number (0 to 20).

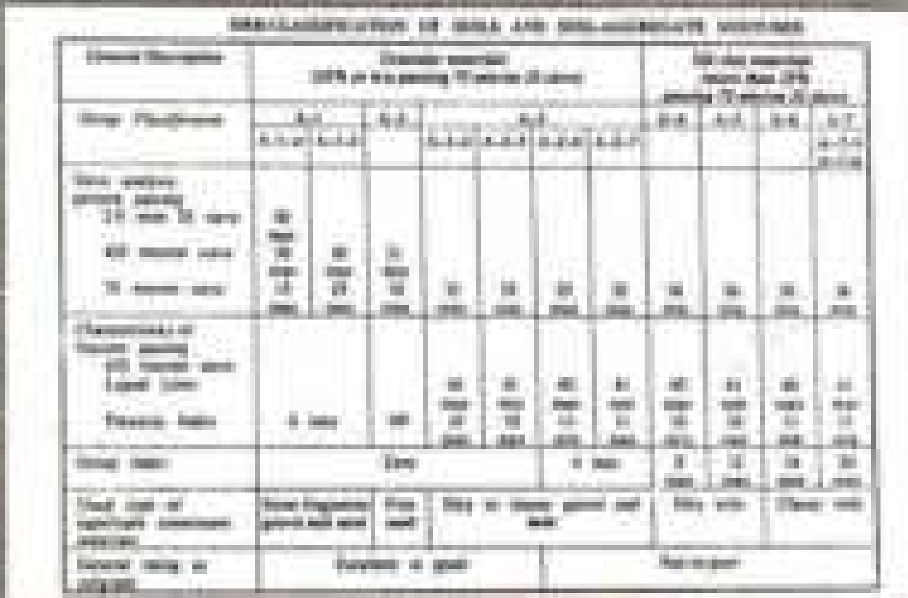
HRB Classification Table

HRB-CLASSIFICATION OF SOILS AND SOIL-AGGREGATE MIXTURES

General Description	Granular materials (35% or less passing 75 micron IS sieve)							Silt clay materials (more than 35% passing 75 micron IS sieve)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve analysis, percent passing											
2.0 mm IS sieve	50 max										
425 micron sieve	30 max	50 max	51 min								
75 micron sieve	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 425 micron sieve											
Liquid Limit				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		NP	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min
Group Index	Zero					4 max		8 max	12 max	16 max	20 max
Usual type of significant constituent materials	Stone fragments gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good					Fair to poor					

For A-7-5, $I_P <= w_L - 30$ For A-7-6, $I_P > w_L - 30$

HRB Classification Procedure



Example

- 56% passes 75 micron sieve.
- Plastic Limit (w_p) =23%
- Liquid Limit (w_L)=36%

Plasticity Index (I_p) = 36-23=13%

Example

Passing 75 micron>35% $I_p < 40\%$ $w_L > 10\%$

$w_L = 36\%$
$I_p = 13\%$

A-6

HRB CLASSIFICATION OF SOILS AND SOIL-AGGREGATE MIXTURES

General Description	Granular materials (35% or less passing 75 micron IS sieve)							Silt clay materials (more than 35% passing 75 micron IS sieve)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve analysis, percent passing											
2.0 mm IS sieve	50 max										
425 micron sieve	30 max	50 max	51 min								
75 micron sieve	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 425 micron sieve											
Liquid Limit				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		NP	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min
Group Index	Zero					4 max		8 max	12 max	16 max	20 max
Usual type of significant constituent materials	Stone fragments gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
Overall rating as subgrade	Excellent to good					Fair to poor					

Example

Group Index Calculation

- 56% passes 75 micron sieve.
- Plastic Limit (w_P) =23%
- Liquid Limit (w_L)=36%
- Plasticity Index (I_P) =13%

Group index = $0.2a + 0.005ac + 0.01bd$

where a = that portion of percentage passing 75 micron sieve greater than 35 and not exceeding 75 expressed as a whole number (0 to 40)

b = that portion of percentage passing 75 micron sieve greater than 15 and not exceeding 55 expressed as a whole number (0 to 40)

c = that portion of the numerical liquid limit greater than 40 and not exceeding 60 expressed as positive whole number (0 to 20)

and d = that portion of the numerical plasticity index greater than 10 and not exceeding 30 expressed as a positive whole number (0 to 20).

a= 56-35=21

b=56-15=41, Take 40

c=0

d=13-10=3

GI=5.4=5

A-6 (5)

Unified Soil Classification System
USCS

Origin of USCS

- First developed by Professor A. Casagrande (1948) for the purpose of airfield construction during World War II
- Afterwards [illegible] modified to enable the system to be applicable to dams, foundations and other construction

Four Major Divisions

Course Grained

Fine Grained

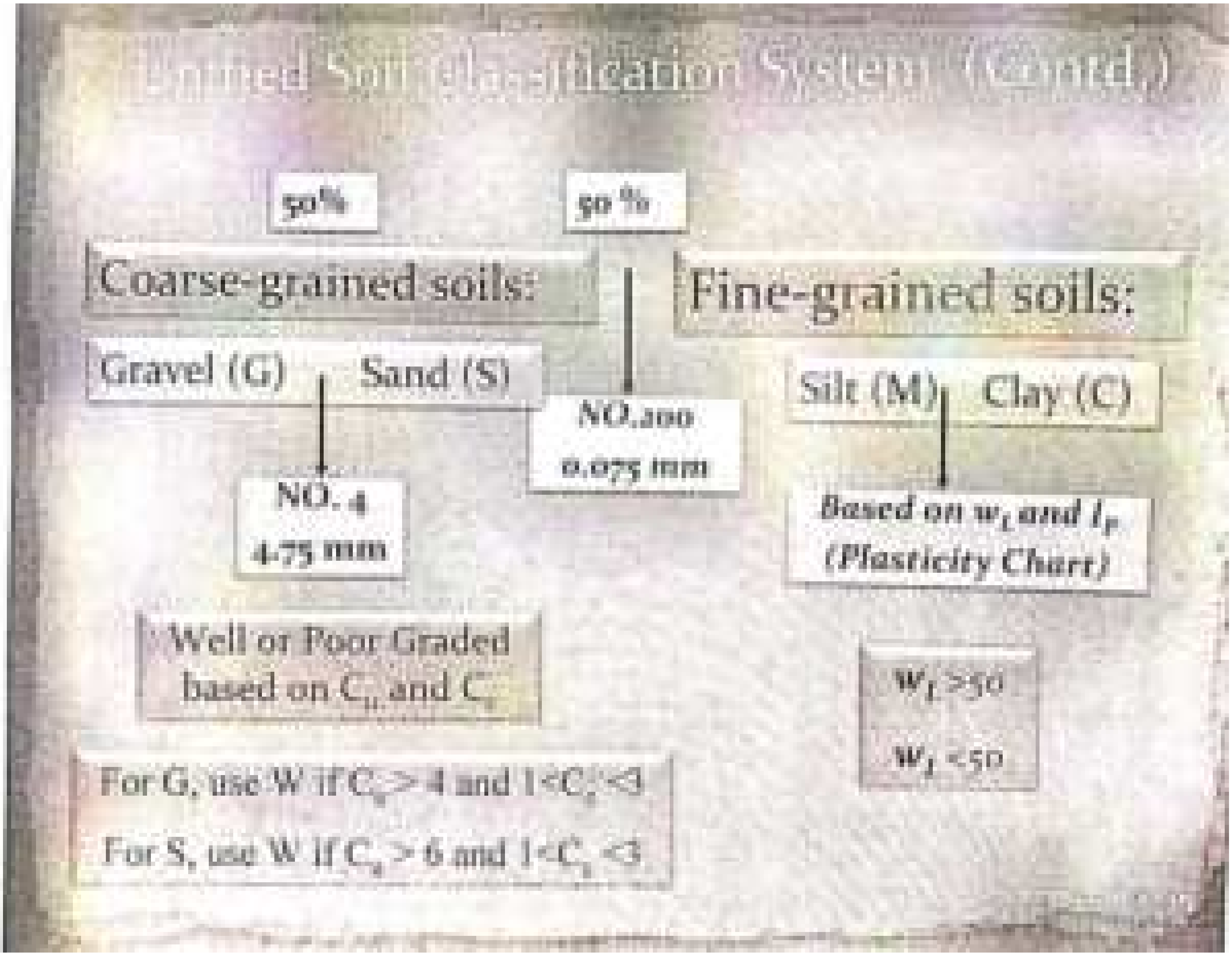
Organic Soil

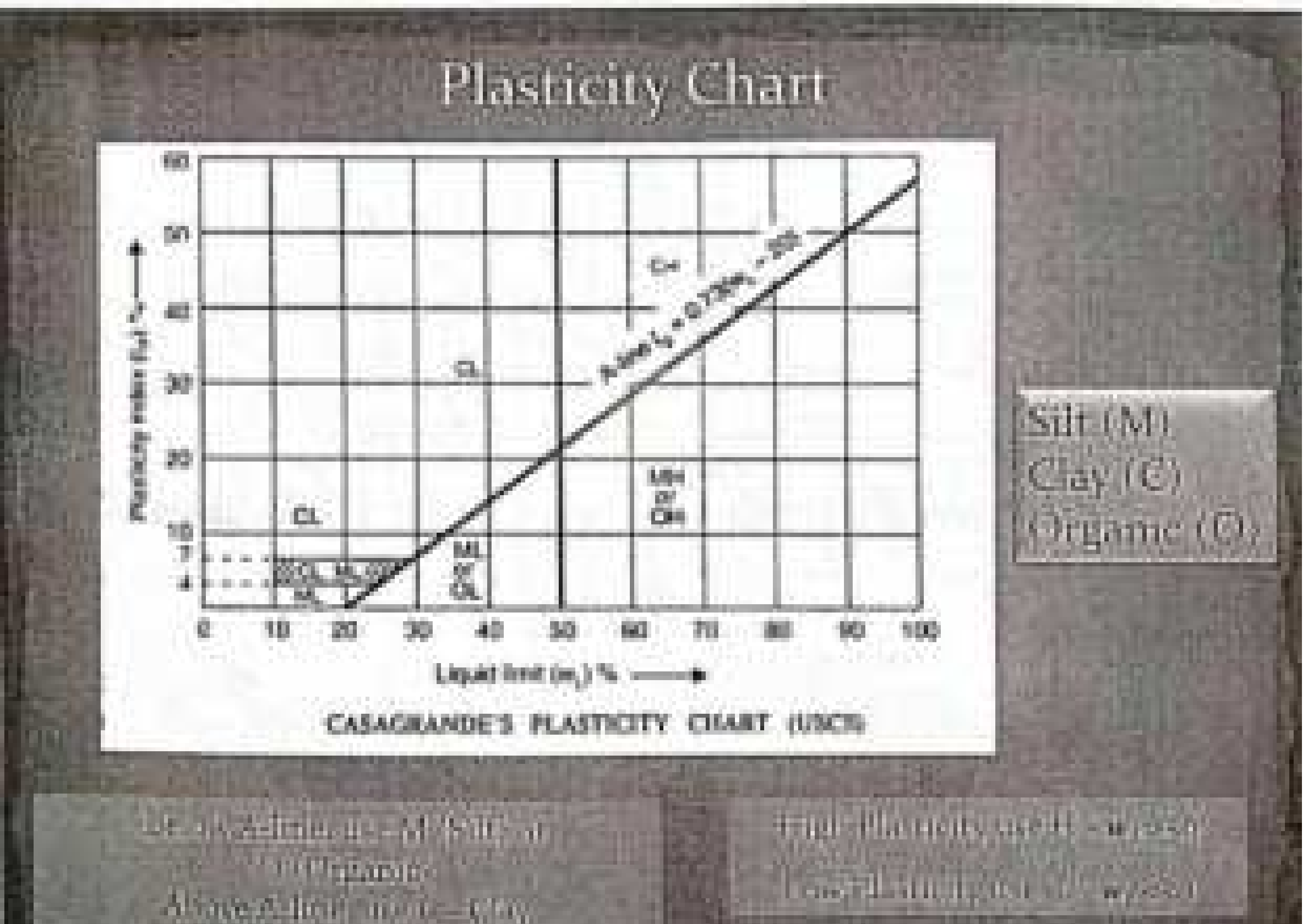
Peat

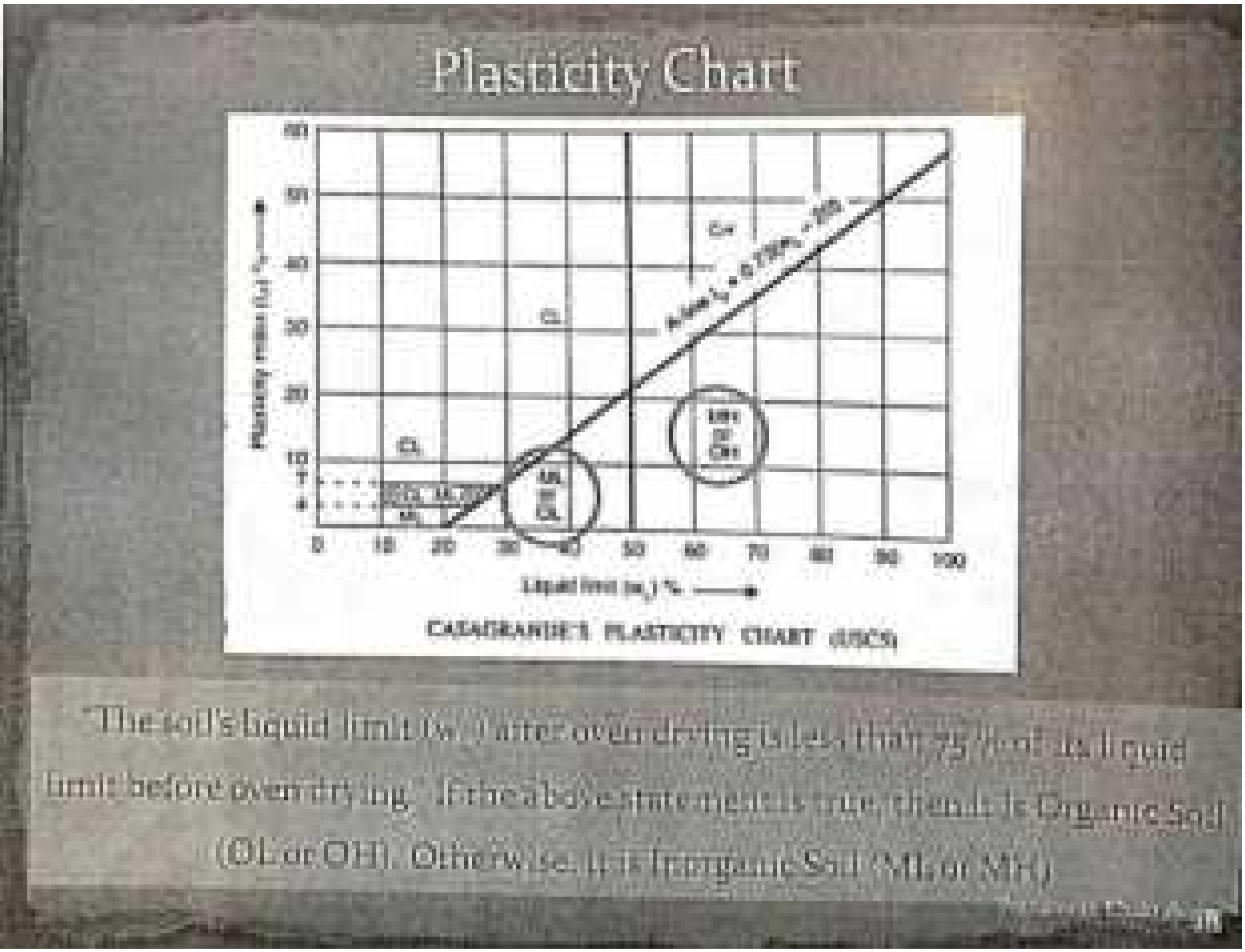
Classification Groups

Classification Group Symbols

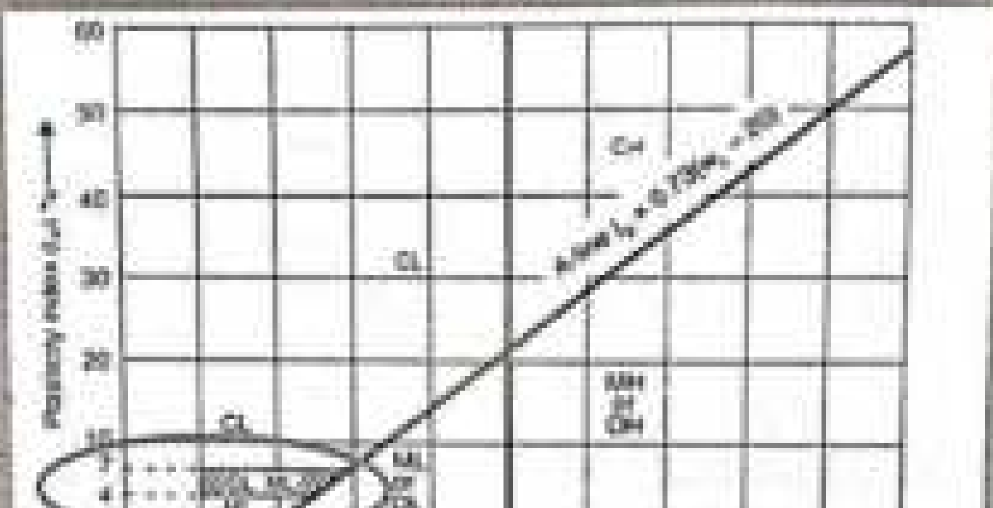
Main Soil Type	Prefix	Subgroup	Suffix	Classification Group symbols
Gravel	G	Well-graded	W	GW
		Poorly-graded	P	GP
		Silty	M	GM
		Clayey	C	GC
Sand	S	Well-graded	W	SW
		Poorly-graded	P	SP
		Silty	M	SM
		Clayey	C	SC
Silt	M	LL < 50%	L	ML
		LL > 50%	H	MH
Clay	C	LL < 50%	L	CL
		LL > 50%	H	CH
Organic	O	LL < 50%	L	OL
		LL > 50%	H	OH
Peat	Pt			Pt


Unified Soil Classification System (Contd.)
50%
50 %
Coarse-grained soils:
Fine-grained soils:
Gravel (G) Sand (S)
Silt (M) Clay (C)
NO.200
0.075 mm
NO. 4
4.75 mm
Based on w_L and I_P
(Plasticity Chart)
Well or Poor Graded
based on C_u and C_c
$w_L > 50$
$w_L < 50$
For G, use W if $C_u > 4$ and $1 < C_c < 3$
For S, use W if $C_u > 6$ and $1 < C_c < 3$


Plasticity Chart
Silt (M)
Clay (C)
Organic (O)
Liquid limit (w_L) %
CASAGRANDE'S PLASTICITY CHART (USCS)
CL
ML
MH or OH
ML or OL
0
10
20
30
40
50
60
70
80
90
100


Plasticity Chart
CL
OL
CL
CL-ML
ML
ML or OL
MH or OH
A-line
Liquid limit (w_L) %
CASAGRANDE'S PLASTICITY CHART (USCS)
"The soil's liquid limit (w_L) after oven drying is less than 75 % of its liquid limit before oven drying." If the above statement is true, then it is Organic Soil (OL or OH). Otherwise it is Inorganic Soil (ML or MH)

Plasticity Chart



CASAGRANDE'S PLASTICITY CHART (USCS)

When I_p and w_L are in the hatched portion of the plasticity chart, the soil is given dual symbol (CL-ML).

Soil possessing properties of more than one group are termed as boundary soil and designated by dual group symbol.

USCS at a glance

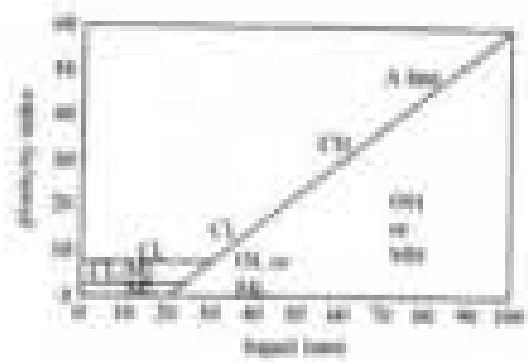
COARSE More than 50% retained sieve #200	Gravel more than 50% coarse fraction retained on sieve #4	Less than 5% fines	$C_u > 4, 1 \le C_c \le 3$	— GW
			Not satisfying GW	— GP
		More than 12% fines	Below 'A' line	— GM
			Above 'A' line	— GC
	Sand less than 50% coarse fraction retained on sieve #4	Less than 5% fines	$C_u > 6, 1 \le C_c \le 3$	— SW
			Not satisfying SW	— SP
		More than 12% fines	Below 'A' line	— SM
			Above 'A' line	— SC

FINE

Less than 50% retained sieve #200

$LL < 50$

$LL > 50$



ML

CL

OL

MH

CH

OH

ORGANIC SOILS — Pt

(Sivakugan et al., 2001)

Group Symbols and Group Names

Group Symbol	Typical Name
GW	Well-graded gravels.
GP	Poorly-graded gravels.
GM	Silty gravels.
GC	Clayey gravels
SW	Well-graded sands.
SP	Poorly-graded sands.
SM	Silty sands.
SC	Clayey sands.

Group Symbols and Group Names

Group Symbol	Typical Name
CL	Inorganic clays of low plasticity.
ML	Inorganic silts with slight plasticity.
OL	Organic soil of low plasticity.
CH	Inorganic clays of high plasticity.
MH	Inorganic silts with high plasticity.
OH	Organic soil of high plasticity.
Pt	Peat.

Example

Passing No.200 sieve 30 %
Passing No.4 sieve 70 %

LL= 33
PI= 12

Passing No.200 sieve 30%

Passing No.4 sieve 70 %

LL= 33

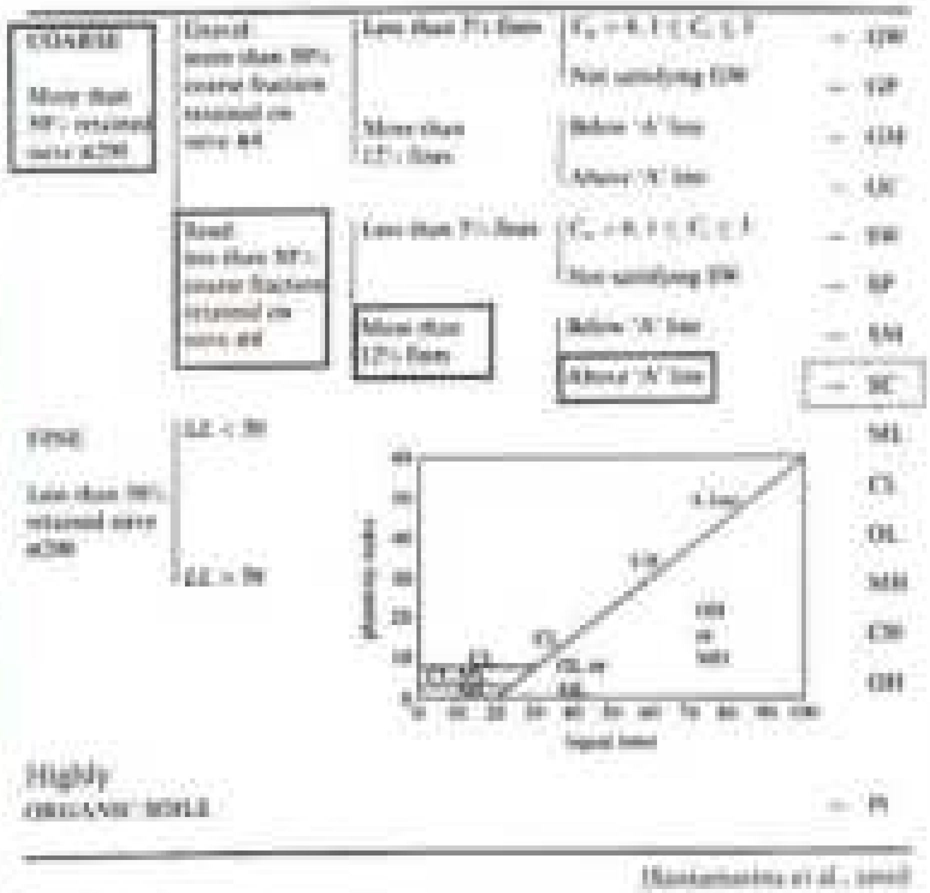
PI= 12

PI= 0.73(LL-20), A-line

PI=0.73(33-20)=9.49

Soil type= SC

- Clayey sand



Indian Standard Classification System (ISCS)

IS:1498-1970

Introduction

Based on [illegible] system

Modification [illegible]

- The grade [illegible] groups [illegible] intermediate [illegible] against two groups(low and [illegible])

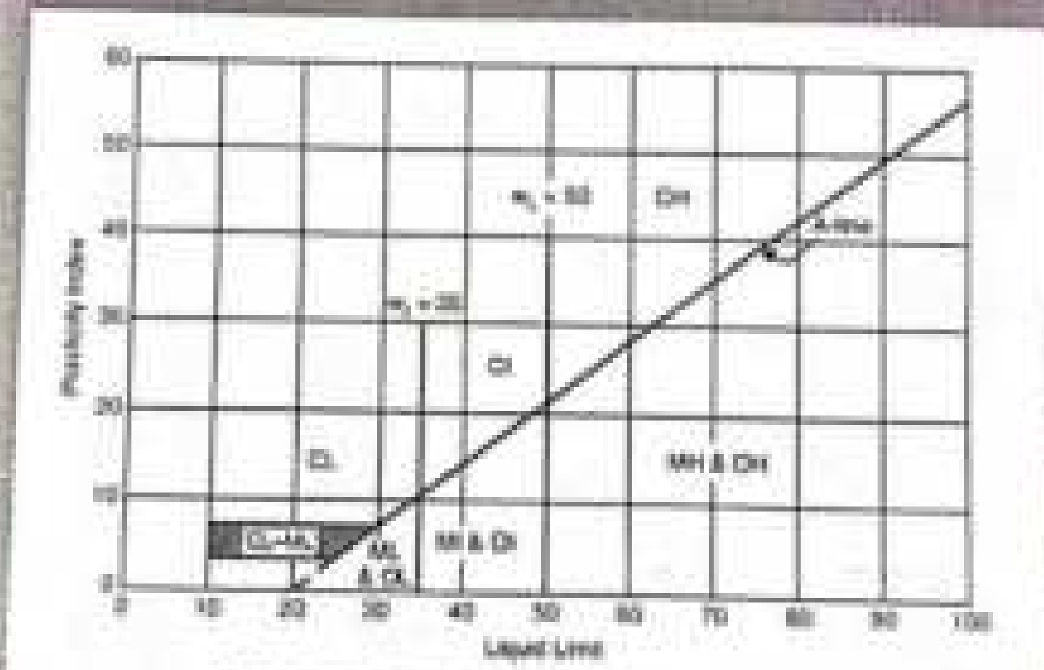
- W_1 [illegible] W_2 [illegible] W_3 [illegible]

- The [illegible] into 8 groups [illegible] against [illegible] in use

Classification Group Symbols

Main Soil Type	Prefix	Subgroup	Suffix	Classification Group symbols
Gravel	G	Well-graded	W	GW
		Poorly-graded	P	GP
		Silty	M	GM
		Clayey	C	GC
Sand	S	Well-graded	W	SW
		Poorly-graded	P	SP
		Silty	M	SM
		Clayey	C	SC
Silt	M	LL < 35%	L	ML
		35<LL<50	I	MI
		LL > 50%	H	MH
Clay	C	LL < 35%	L	CL
		35<LL<50	I	CI
		LL > 50%	H	CH
Organic	O	LL < 35%	L	OL
		35<LL<50	I	OI
		LL > 50%	H	OH
Peat	Pt			Pt

Plasticity Chart



PLASTICITY CHART (IS SOIL CLASSIFICATION SYSTEM)

Below A-line, use M (Silt) or O (Organic)

Above A-line, use C - Clay

High Plasticity use H - $w_L > 50$

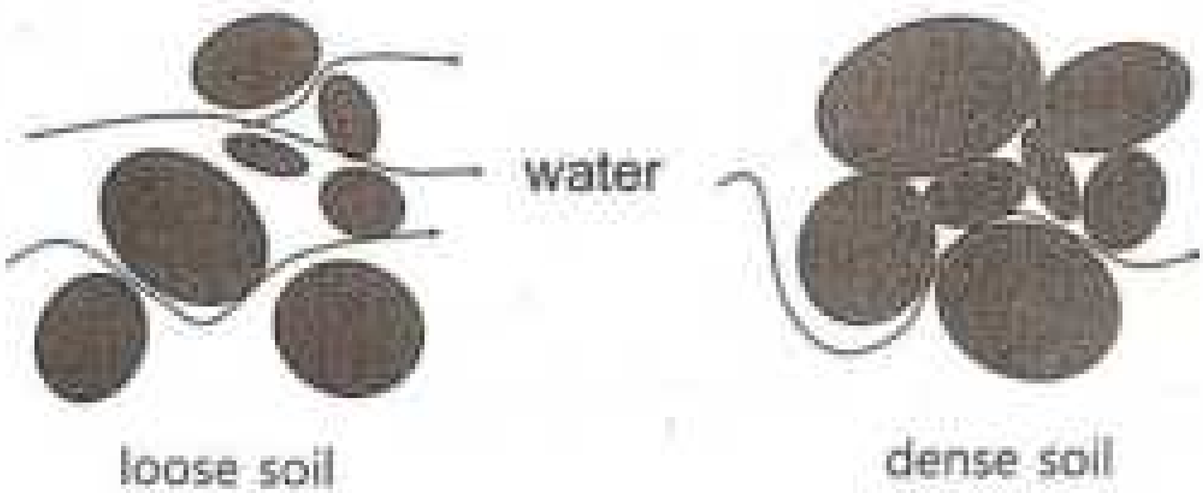
Intermediate Plasticity use I – $35 < w_L > 50$

Low Plasticity use L - $w_L < 35$

INTRODUCTION

Definition

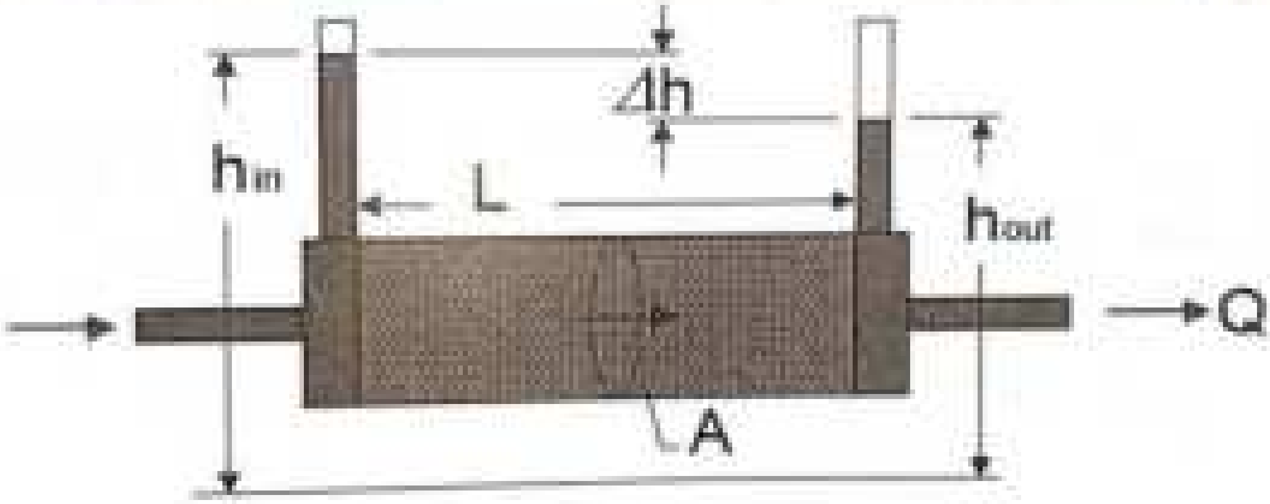
It is the property of soil which allows the flow of water through it.



Importance of Permeability

- The design of earth dams is very much based upon the permeability of the soils used.
- The stability of slopes and retaining structures can be greatly affected by the permeability of the soils involved.
- Filters made of soils are designed based upon their permeability
- Estimating the quantity of underground seepage

Darcy's law



Where,

A is the cross section of soil sample
L is the length of the soil sample
h_{in} is the head at the inlet
h_{out} is the head at the outlet
Q is the discharge
q is the rate of discharge per unit time (t) = Q/t

Darcy's law

It states that "In a saturated soil, under laminar flow condition, the rate of flow of water through given sample of soil is directly proportional to hydraulic gradient"

$$V=q/A=ki$$

$$q=kiA$$

Where,

V is the superficial velocity (m/sec)
k is the co-efficent of permeability (m/sec)
i is the hydraulic gradient= $(h_{in}-h_{out})/L$

Superficial velocity

It is defined as discharge per unit cross section area of soil

$$V=q/A$$

Where,

V is the superficial velocity (m/sec)
q is the discharge per unit time
A is the area of the soil sample

Seepage velocity

It is defined as discharge per unit cross section area of voids to the direction of the flow soil

$$V_s = q/A_s$$

Where,

V_s is the seepage velocity (m/sec)
q is the discharge per unit time
A_s is the area of voids

Relationship between superficial velocity and seepage velocity is

$$V_s = V/n$$

n is the porosity

Factors affecting permeability

- Particle size
- Properties of pore water   • Void ratio
- Degree of saturation
- Presence of entrapped air & other foreign matter
- Structural arrangement
- Stratification of soil

Factors affecting permeability

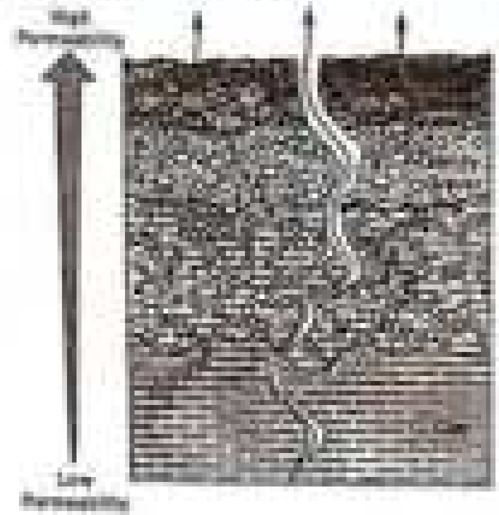
Particle size

The Permeability varies approximately as the square of diameter of the soil

$$k=100D^2_{10}$$

Where,

D_{10} is the effective diameter of the soil



Factors affecting permeability

Property of pore water

The Permeability of the soil varies directly with density & inversely proportional to the viscosity of the water

$$k \propto \gamma_w/\mu$$

$$k=1/\mu$$

$$k\mu=\text{constant}$$

Factors affecting permeability

Void ratio

Increase in the void ratio increases the area available for flow hence permeability increases.

$$k \propto e^3/1+e$$

Where,

e is the void ratio for the soil permeability k

Factors affecting permeability

Degree of saturation

Higher the degree of saturation, higher will be the permeability.

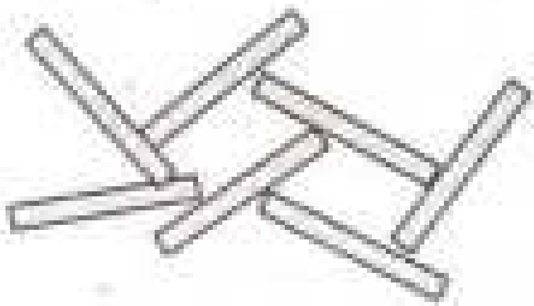
Presence of entrapped air & Other foreign matter

The entrapped air and foreign matter will block the voids in soil results in decreasing in permeability

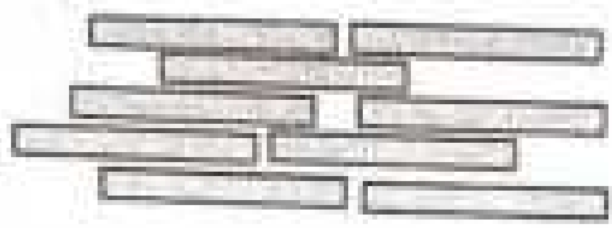
Factors affecting permeability

Structural arrangement

For same void ratio the permeability of the soil will be more in flocculated structure as compare to Dispersed structure.



Flocculated structure



Dispersed structure

Factors affecting permeability

Stratification of soil

Stratified soil deposits have grater permeability parallel to the plane when compare to perpendicular to the plane.

k1>k2

Laboratory Testing to find coefficient of permeability

Two standard laboratory tests are us to determine the coefficient of permeability of soil

- The constant-head test
- The falling-head test.

Laboratory Testing to find coefficient of permeability

The constant-head test

- The constant head test is used primarily for coarse-grained soils.
- This test is based on the assumption of laminar flow (Darcy's Law apply)

$$k = \frac{V \cdot L}{h \cdot A \cdot t}$$

Where:
Q = volume of water collection
A = cross section area of soil specimen
t = duration of water collection

Laboratory Testing to find coefficient of permeability

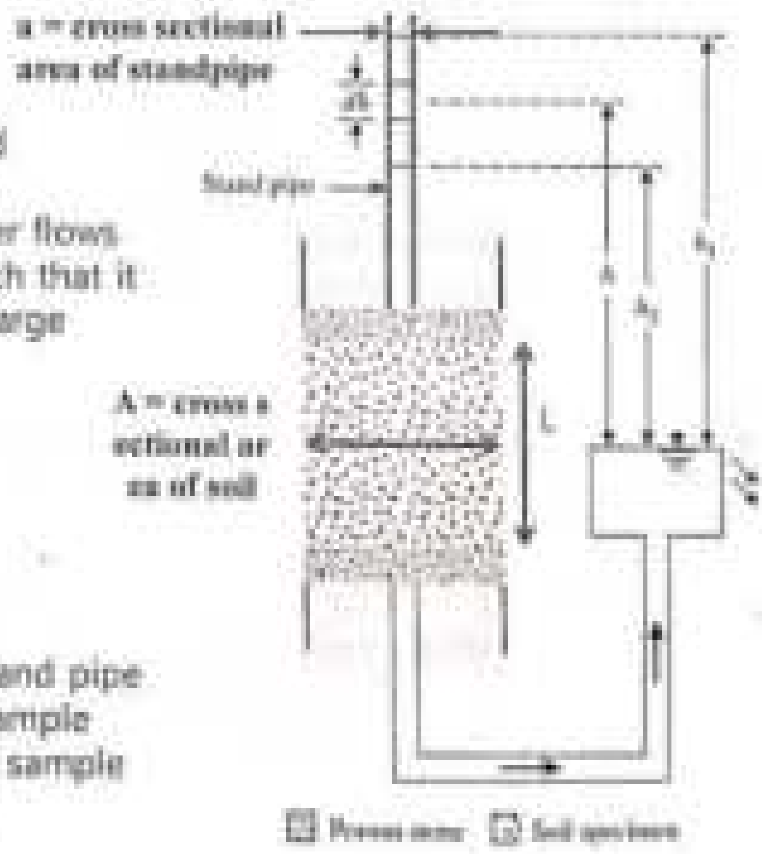
Falling Head Test

- Variable head method is adopted for highly impervious soils
- This test is conducted when water flows through the soil is very small such that it is very difficult to measure discharge

$$k = \frac{2.30aL}{At}\log_{10}\frac{h_1}{h_2}$$

Where,

h_1 is the initial head
h_2 is the final head
a is the c/s area of the stand pipe
A is the c/s area of soil sample
L is the length of the soil sample
t is the time



Chapter 6 COMPACTION AND CONSOLIDATION

What is Compaction of Soil?

Compaction of soil is the pressing of soil particles close to each other by mechanical methods. Air during compaction of soil is expelled from the void space in the soil mass and therefore the mass density is increased.

Compaction of soil is done to improve the engineering properties of the soil. Compaction of soil is required for the construction of earth dams, canal embankments, highways, runways and many other structures.

Methods of Testing Compaction of Soil

Standard Proctor's Test for Compaction of Soil

To assess the amount of compaction of soil and water content required in the field, compaction tests are done on the same soil in the laboratory. The test provides a relationship between the water content and the dry density.

The water content at which the maximum dry density is attained is obtained from the relationship provided by the tests. Proctor used a standard mould of 4 inches internal diameter and an effective height of 4.6 inches with a capacity of 1/30 cubic foot.

The mould had a detachable base plate and a removable collar of 2 inches height at its top. The soil is compacted in the mould in 3 layers, each layer was given 25 blows of 5.5 pounds rammer falling through a height of 12 inches.

IS: 2720 part VII recommends essentially the same specification as in Standard Proctor test, some minor modifications. The mould recommended is of 100mm diameter, 127.3 mm height and 1000 ml capacity.

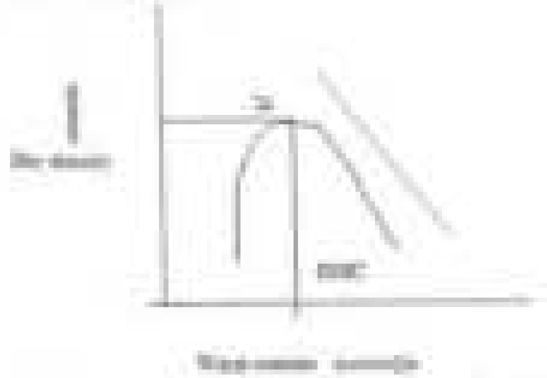
The rammer recommended is of 2.6 kg mass with a free drop of 310mm and a face diameter of 50mm. The soil is compacted in three layers. The mould is fixed to the detachable base plate. The collar is of 60mm height.

Procedure of Proctor's Test for Compaction of Soil

About 3kg of air dried soil is taken for the test. It is mixed with 8% water content and filled in the mould in three layers and giving 25 blows to each layer. The volume of the mould and mass of the compacted soil is taken. The bulk density is calculated from the observations. A representative sample is placed in the oven for determination of water content. The dry density is found out from the bulk density and water content. The same procedure is repeated by increasing the water content.

Presentation of Results of Proctors Test

Compaction curve



Compaction curve

A compaction curve is plotted between the water content as abscissa and the corresponding dry density as ordinate. It is observed that the dry density initially increases with an increase in water content till the maximum density is attained.

With further increase in water content the dry density decreases. The water content corresponding to maximum dry density is known as the optimum water content (O.W.C) or the optimum moisture content (O.M.C).

At a water content more than the optimum, the additional water reduces the dry density as it occupies the space that might have been occupied by the solid particles.

For a given water content, theoretical maximum density is obtained corresponding to the condition when there are no air voids (degree of saturation is 100%). The theoretical maximum density is also known as saturated dry density. The line indicating theoretical maximum density can be plotted along with the compaction curve. It is known as the zero air void line.

Modified Proctor Test for Compaction of Soil

The modified Proctor test was developed to represent heavier compaction than that in the standard Proctor test. The test is used to simulate field conditions where heavy rollers are used. The test was standardised by American association of State Highway Officials and is, therefore also known as modified AASHO test.

In this, the mould used is same as that in the Std Proctor test. However, the rammer used is much heavier and has a greater drop than that in the Std Proctor test. Its mass is 4.89 kg and the free drop is 450mm. The soil is compacted in five equal layers, each layer is given 25 blows. The compactive effort in modified Proctor test is 4.56 times greater than in the Std Proctor test. The rest of the procedure is same.

Factors Affecting Compaction of Soil

Water Content

At low water content, the soil is stiff and offers more resistance to compaction. As the water content is increased, the soil particles get lubricated. The soil mass becomes more workable and the particles have closer packing. The dry density of the soil increases with an increase in the water content till the O.M.C is reached.

Amount of compaction
The increase in compactive effort will increase the dry density at lower water content to a certain extent.

Type of soil
The dry density achieved depends upon the type of soil. The O.M.C and dry density for different soils are different.

Method of compaction
The dry density achieved depends on the method of compaction.

Effect of Compaction on Properties of Soil
1. Effect of Compaction on Soil Structure
Soils compacted at a water content less than the optimum generally have a flocculated structure. Soils compacted at water content more than the optimum usually have a dispersed structure.

2. Effect of Compaction of Soil on Permeability
The permeability of a soil depends upon the size of voids. The permeability of a soil decreases with an increase in water content on the dry side of optimum water content.

3. Swelling

4. Pore water pressure

5. Shrinkage

6. Compressibility

7. Stress-strain relationship

8. Shear strength

Methods of Compaction of Soil used in Field
Several methods are used in the field for compaction of soils. The choice of method will depend upon the soil type, the maximum dry density required and economic consideration. The commonly used methods are:

1. Tampers

2. Rollers

3. Vibratory compactors

The compaction depends upon the following factors:

- Contact pressure
- Number of passes
- Layer thickness
- Speed of roller

Types of rollers

- Smooth Wheel rollers
- Pneumatic tyred rollers
- Sheepsfoot rollers

Field Compaction Equipment

There is a wide range of compaction equipment. The compaction achieved will depend on the thickness of lift (or layer), the type of roller, the no. of passes of the roller, and the intensity of pressure on the soil. The selection of equipment depends on the soil type as indicated.

Equipment	Most suitable soils	Least suitable soils
Smooth steel drum rollers (static or vibratory)	Well-graded sand-gravel, crushed rock, asphalt	Uniform sands, silty sands, soft clays
Pneumatic tyred rollers	Most coarse and fine soils	Very soft clays
Sheepsfoot rollers	Fine grained soils, sands and gravels with > 20% fines	Uniform gravels, very coarse soils
Grid rollers	Weathered rock, well-graded coarse soils	Uniform materials, silty clays, clays
Vibrating plates	Coarse soils with 4 to 8% fines	
Tampers and rammers	All soil types	

CONSOLIDATION

According to Karl von Terzaghi "consolidation is any process which involves a decrease in water content of saturated soil without replacement of water by air." In general it is the process in which reduction in volume takes place by expulsion of water under long term static loads.

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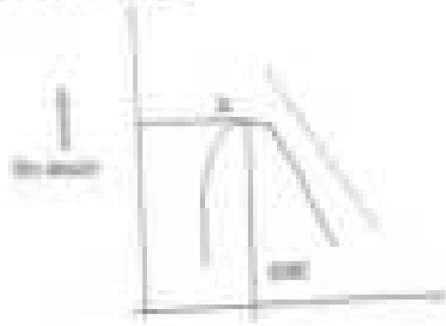

Water content [illegible]

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Chapter-5(Contd…)

Seepage pressure, the phenomenon of quick sand

Seepage in an upward direction reduces the effective stress within the soil. When the water pressure at a point in the soil is equal to the total vertical stress at that point, the effective stress is zero and the soil has no frictional resistance to deformation. For a surface layer, the vertical effective stress becomes zero within the layer when the upward hydraulic gradient is equal to the critical gradient.[14] At zero effective stress soil has very little strength and layers of relatively impermeable soil may heave up due to the underlying water pressures. The loss in strength due to upward seepage is a common contributor to levee failures. The condition of zero effective stress associated with upward seepage is also called liquefaction, quicksand, or a boiling condition. Quicksand was so named because the soil particles move around and appear to be 'alive' (the biblical meaning of 'quick' – as opposed to 'dead'). (Note that it is not possible to be 'sucked down' into quicksand. On the contrary, you would float with about half your body out of the water.)

Flow Nets

Graphical form of solutions to **Laplace equation** for two-dimensional seepage can be presented as flow nets. Two orthogonal sets of curves form a flow net:

- **Equipotential lines** connecting points of equal total head **h**
- **Flow lines** indicating the direction of seepage down a hydraulic gradient

Two flow lines can never meet and similarly, two equipotential lines can never meet. The space between two adjacent flow lines is known as a **flow channel**, and the figure formed on the flownet between any two adjacent flow lines and two adjacent equipotential lines is referred to as a **field**. Seepage through an embankment dam is shown.

Flow net

The network formed by the flow lines and is the pattern which shows the path of flow as well as the dissipation of potential in a system of seepage trough a layer of soil.

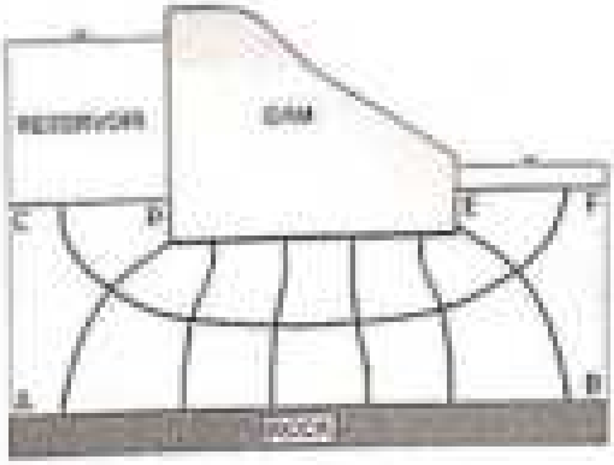
Properties of flow net

1.The flow lines and equipotential lines meet at right angles to each other
2.Two flow line never cross each other.
3.Two equipotential lines never cross each other.
4.Flow and equipotential lines are smooth curves.

5.the quantity of water flowing through each flow channel is same ,
6.same potential drop occurs between the successive equipotential lines.
7.Two flow lines or equipotential line never start from the same point.
8.Smaller the field , greater will be hydraulic gradient.

APPLICATION OF FLOW NET

The graphical properties of a flow net can be used in obtaining solutions for many seepage problems such as:



1. ***Estimation of seepage losses from reservoirs:*** It is possible to use the flow net in the transformed space to calculate the flow underneath the dam.

2. ***Determination of uplift pressures below dams:*** From the flow net, the pressure head at any point at the base of the dam can be determined. The uplift pressure distribution along the base can be drawn and then summed up.

3. ***Checking the possibility of piping beneath dams:*** At the toe of a dam when the upward exit hydraulic gradient approaches unity, boiling condition can occur leading to erosion in soil and consequent piping. Many dams on soil foundations have failed because of a sudden formation of a piped shaped discharge channel. As the stored water rushes out, the channel widens and catastrophic failure results. This is also often referred to as piping failure.

Chapter-8

Lateral Earth Pressure

In 1929 Terzaghi (The Father of Soil Mechanics) conducted experiments on the retaining wall and showed the relation of pressure on the wall if wall changes its position i.e to move towards to the backfill, outwards of it or remain at its place. **There are three types of earth pressures on the basis of the movement of the wall.**

1. Earth Pressure at rest
2. Active Earth Pressure
3. Passive Earth Pressure

These are explained below.

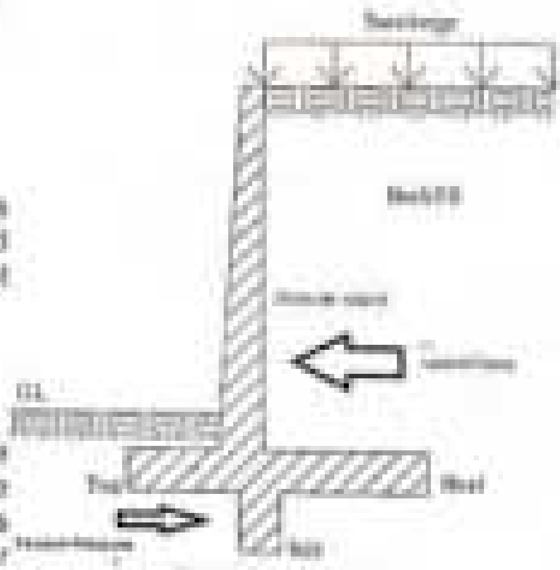


Figure 1: Retaining Wall

Pressure at rest:

When the wall is at rest and the material is in its natural state then the pressure applied by material is known as Earth Pressure at Rest. It is represented by P_o.

Active earth pressure:

When the wall moves away from the backfill, there is a decrease in the pressure on the wall and this decrease continues until a minimum value has reached after which their is no reduction in the pressure and the value will become constant. This kind of pressure is known as active earth pressure.

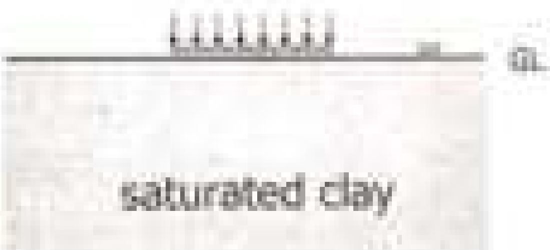
Passive earth pressure:

When the wall moves towards the back fill, there is an increase in the pressure on the wall and this increase continues until a maximum value has reached after which their is no increase in the pressure and the value will become constant. This kind of pressure is known as passive earth pressure.

Chapter – 6

What is Consolidation?

When a saturated clay is loaded externally,



The reduction in volume will takes place by expulsion of water from voids over a long time

Compaction VS Consolidation

COMPACTION

1. Application of dynamic load.
2. Expulsion of air
3. Short Process

CONSOLIDATION

1. Application of static load.
2. Expulsion of water
3. Long Process

Types of Consolidation

The total compression of a saturated clay strata under excess effective pressure may be considered as the sum of

- 1. Immediate compression,
- 2. Primary consolidation, and
- 3. Secondary compression.

or,

1. Initial consolidation
2. Primary consolidation
3. Secondary consolidation

1. Immediate compression

The portion of the settlement of a structure which occurs more or less simultaneously with the applied loads is referred to as the initial or immediate settlement. This settlement is due to the immediate compression of the soil layer under undrained condition and is calculated by assuming the soil mass to behave as an elastic soil.

Total settlement:

or, $S_T = S_i + S_c + S_s$

1) Initial Consolidation: The small reduction in the vol^m of soil just after application of the load is known as initial consolidation.

2) Primary consolidation: After initial consolidation, further reduction in volume occurs due to expulsion of water from voids

3) Secondary Consolidation/Creep: The effect of Continu consolidation after complete dissipation of excess pore water pressure (1° consolidat^n) is called Secondary Consoli

- It is due to expulsion of highly viscous water & plasti readjustment of particles

Immediate	Primary	Secondary
1. Due to distortion or elastic deformation with no change in water content.	1. Decrease in voids volume due to squeeze of pore water out of the soil	1. Due to gradual changes in the particulate str. of the soil.
2. Occurs rapidly during application of load	2. Occurs in saturated fine grained soils (Low co-efficient of permeability), Time Dependent	2. Occurs very slowly, long after the primary consolid. is completed. Time dependent
3. Quite small quantity in dense sands, gravels & stiff clays.	3. Only significant in clay & silts	3. Most significant in saturated soft clays & organic soils & peats.

2. Primary consolidation

If the rate of compression of the soil layer is controlled solely by the resistance of the flow of water under the induced hydraulic gradients, the process is referred to as primary consolidation. The portion of the settlement that is due to the primary consolidation is called primary consolidation settlement or compression.

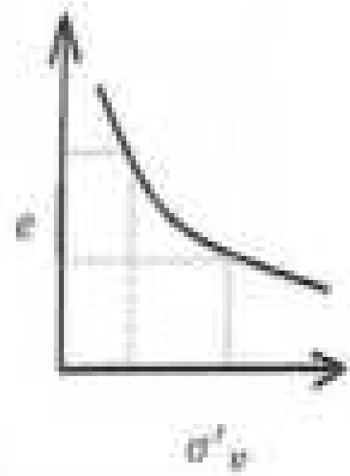
Change in volume by expulsion of water

3. Secondary Consolidation

Compression due to the compression and rearrangement of the clay particles and clay layer. It is linear with logarithm of the time.

Basic definitions

1. coefficient of compressibility:



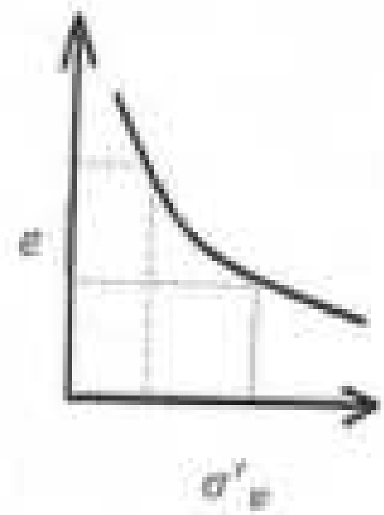
Coefficient of compressibility is defined as change in void ratio due to per unit change in effective stress. It is denoted by (a_v)

$$a_v = -\frac{\Delta e}{\Delta \sigma'_v}$$

Basic definitions

2. Coefficient of Volume Change:

The coefficient of volume change is defined as the volumetric strain per unit increase in effective stress. It is denoted by (m_v)



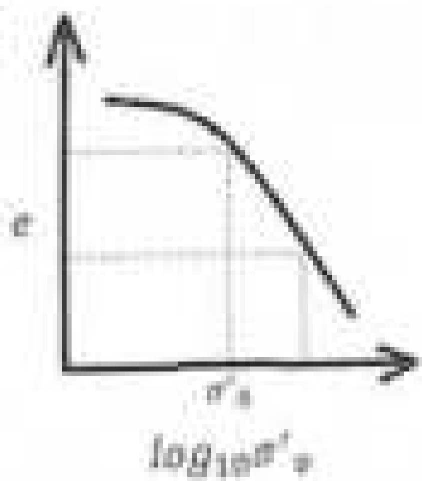
$$m_v = -\frac{\frac{\Delta V}{V}}{\Delta \sigma_v'}$$

Basic definitions

3. Compression Index:

The compression index is defined as the slope of the linear portion of the void ratio (e) verses $log_{10}\sigma'_v$. It is denoted by (c_c)

$$c_c = -\frac{\Delta e}{log_{10}(\frac{\sigma'_0 + \Delta\sigma'_v}{\sigma'_0})}$$



Terzaghi and Peck

c_c=0.009 (LL-10) for undisturbed

c_c=0.007 (LL-10) for remolded

Basic definitions

4. Recompression Index:

The recompression index is defined as the slope during reloading (c_r)

$$c_r = -\frac{\Delta e}{log_{10}(\frac{\sigma'_2}{\sigma'_1})}$$

Terzaghi and Peck

c_C=0.009 (LL-10) for undisturbed

c_C=0.007 (LL-10) for remolded

Normally consolidated clay and over consolidated clay

- **Normally consolidated clay:** A soil is normally consolidated when it has never been subjected to stress higher than the present stress.
- **Over consolidated clay:** A soil which has experienced higher stress in the past than the present stress.
- Cause of over consolidation
  i. Removal of the overburden; excavation, erosion, landslide etc.
  i. Removal of the structure
  ii. Variation in pore water pressure

Is not type of soil but is pressure history.

$OCR = \frac{past\ stress}{present\ stress} > 1$

Over consolidated clay

Spring Analogy Method :

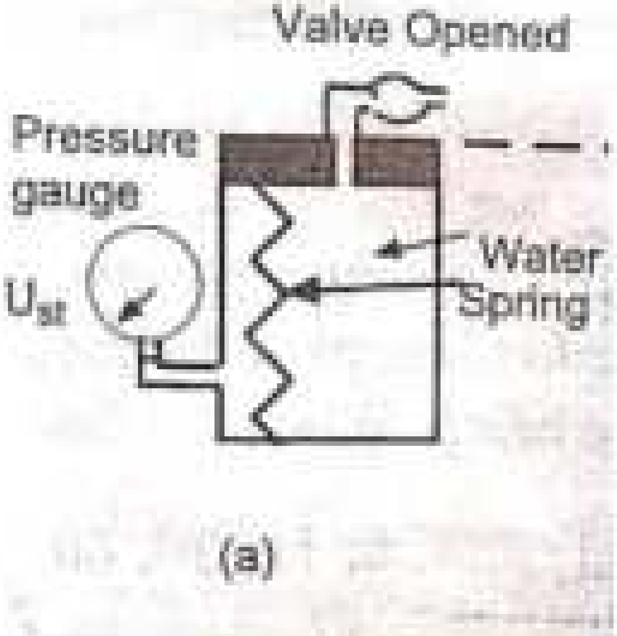
- Terzaghi's model consists of Cylindrical Vessel with a piston attached to spring.
- Space between springs filled with H_2O. Piston is perforated to allow passage of water.
- Piezometers inserted to measure pr. head due to excess porepr.

Terzaghi Correlated : Spring mass compression process ⇝ Consolidation of Saturated Clay Subjected to uniform Load σ

Spring → Soil Skeleton
Water → Water in voids
Spring & Surrounding H_2O → Saturated Soil

Terzaghi spring analogy

* Compression is 1D & flow will be in vertical directn



(a)

- Valve is opened; The system is equilibrium.

This is similar to the soil before loading

- When the pressure σ is applied this will be borne by water surrounding the spring.

  $\sigma = u$ at time $t = 0$

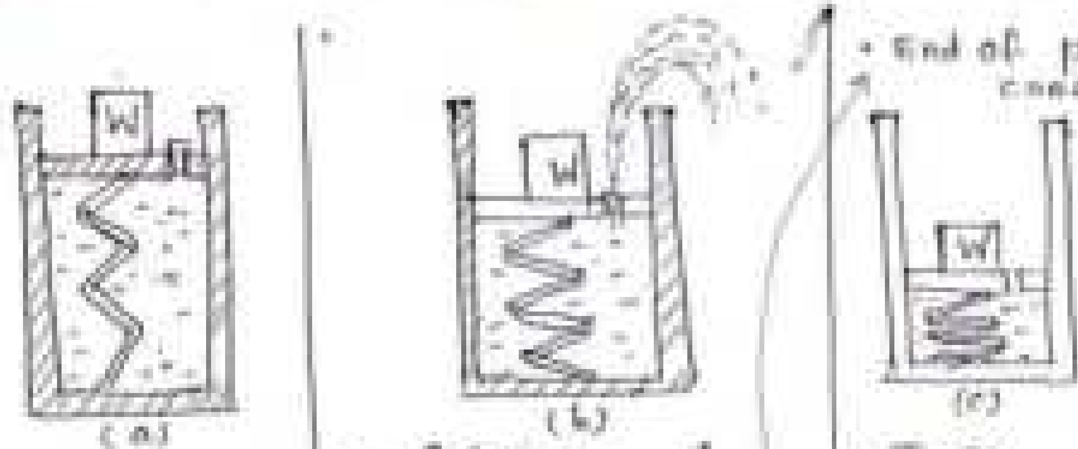
  u → excess hydrostatic pr. due to this water level in all the piezometer reach the same ht. 'h' given by $h = \frac{u}{\gamma_w}$

  $\sigma = u$ & $\sigma' = 0$ at time $t = 0$

  & there will be no volume change.

- Due to spring get compressed & they begin to carry a portion of the applied load.

  After sometime 't' there will be flow of water thru perforation in upward direⁿ there will be reduction in volume.

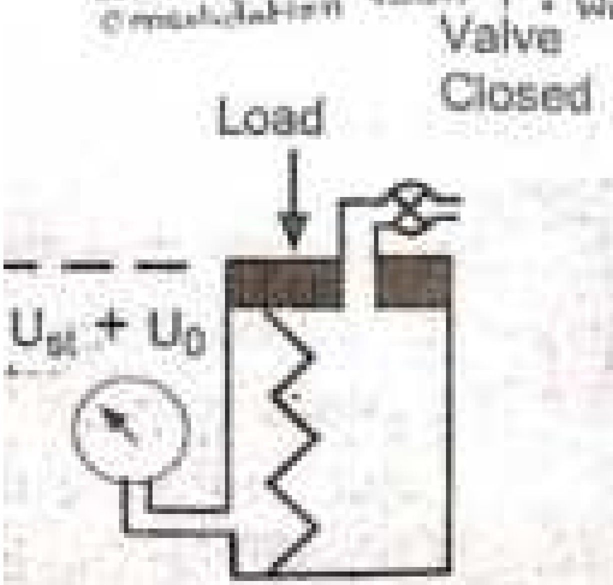


- End of primary consolidation

- $T = 0$
- $\sigma = u$ & $\sigma' = 0$
- Soil = Spring
- Water = water void
- After application of load – primary consolidation start

- $0 < T < \infty$
- $\sigma = u + \sigma'$
- As load resisted by H_2O
- Flow is in upward dir
- Reduction in volume
- Spring doesn't carry any load
- Water carry external Pr

- $T = \infty$
- All the load resisted by spring
- Water doesn't carry any load
- End of consolidation
- Soil solids carry external pressure
- Excess pore H_2O pr. = 0



(b)

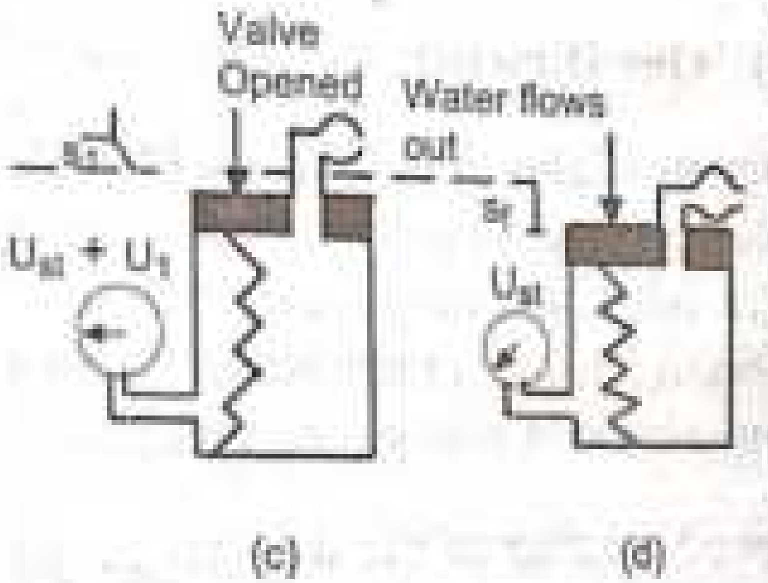
- The valve is closed and the piston is loaded. The pressure increase in the gauge is equal to the increased load.

This is similar to the condition just after loading.

→ This signifies a reduction in excess hydrostatic pressure or pore H_2O pr. & increase in effective stress

→ At time $t = \infty$ when no more pore H_2O flow out the excess hydrostatic pr. will be zero & the entire load is carried by spring.

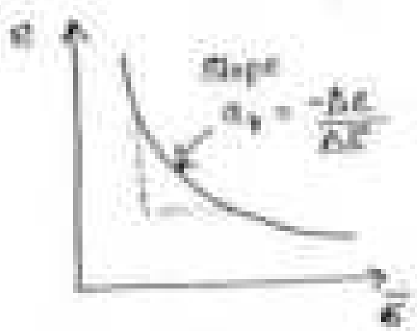
$\sigma = \sigma'$ & $u = 0$ at time $t = \infty$



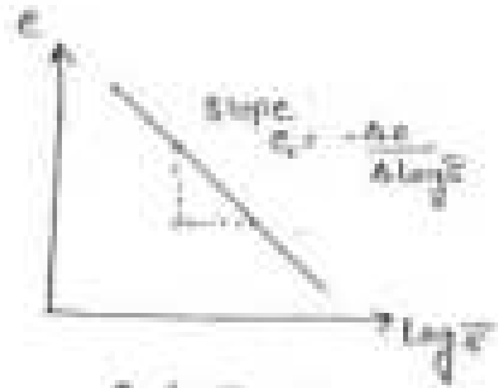
When valve is opened the piston start to move down and the pore water pressure is gauge reduced. The applied load is now share by both spring and water.

At final stage; pressure gauge shows Ust pressure all load is taken by spring.

- The amount of settlement in the spring is depends on the stiffness of spring (compressibility characteristic of soil)
- The rate of settlement depends on the opening (permeability of soil)



$e - \bar{\sigma}$ plot



$e - \log \bar{\sigma}$ plot

Consolidation test

To determine the compressibility characteristics of soil one dimensional consolidation (Oedometer) test is carried out.

Objective of test:

1. To determine the amount of deformation
2. To determine the rate of deformation

Consolidation test is used to determine the rate & magnitude of settlement in soils.

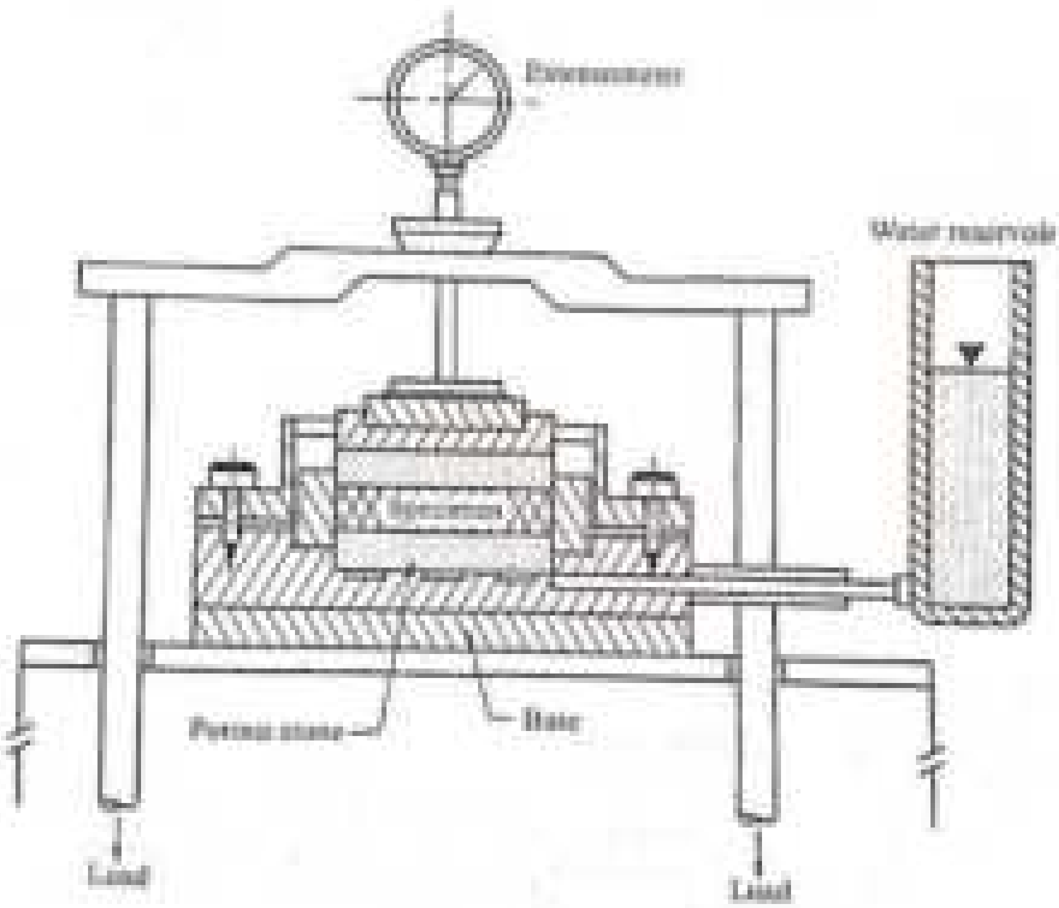
- The settlement values obtained by this test are due to primary consolidation only which is 90% of total consolidation.
- This test is very much helpful for foundation design

Result : (i) coefficient of compressibility (a_v) =

(ii) Co-efficient of vol^m change (m_v) =

(iii) Compression Index (C_c) =

(iv) Co-efficient of consolidation (C_v) =


Water reservoir
Specimen
Porous stone
Base
Load
Load

- Procedure:

1. Sample is placed in the cutting ring in between two porous stone.
2. The loading beam is then brought into contact and dial gauge is set at zero.
3. When first load of $10kN/m^2$ is applied reading of dial gauge is taken at 1/4, ½, 1, 2, 4, 8, 16, 30, 60, 120, 240, 1440 mins.
4. Now load is doubled and dial gauge reading is taken as in step 3. Load is doubled upto $640kN/m^2$
5. Unloading is done by removing $3/4^{th}$ load and reading is observed as earlier.

Calculation

- Determination of void ratio

A. Height of solids method.

$$V_s = \frac{W}{G_s \gamma_w} \qquad e = \frac{Ah - Ah_s}{Ah_s} = \frac{h - h_s}{h_s}$$

B. Change in void ratio method.

$$\frac{\Delta h}{h} = \frac{\Delta V}{V} = \frac{\Delta e}{1+e}$$

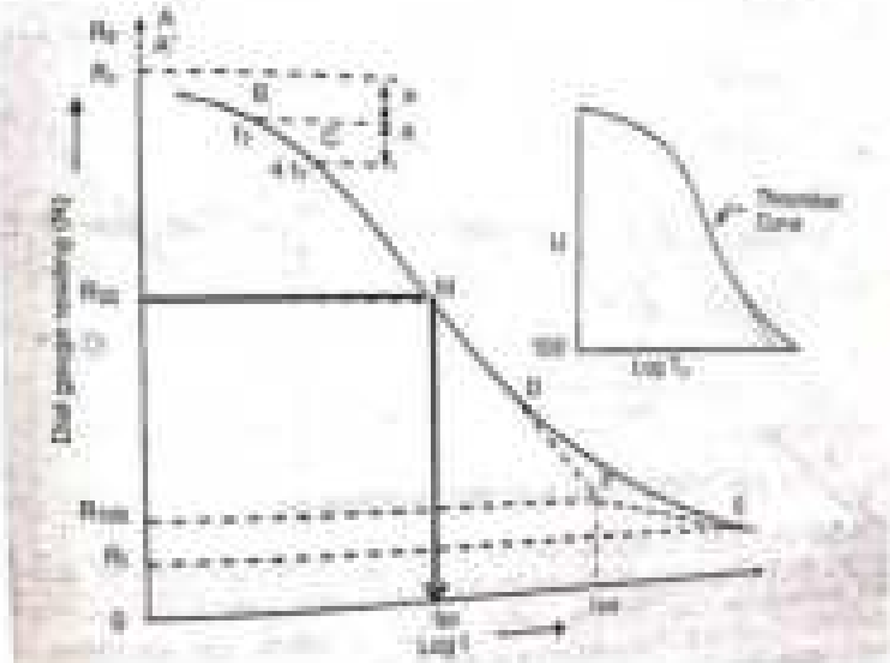
h= final height and, e= final void ratio

$C_v = 0.197 \frac{d^2}{t_{50}}$ Log fitting method

$C_v = 0.848 \frac{d^2}{t_{90}}$ square fitting method

Determination of coefficient of consolidation (C_v)

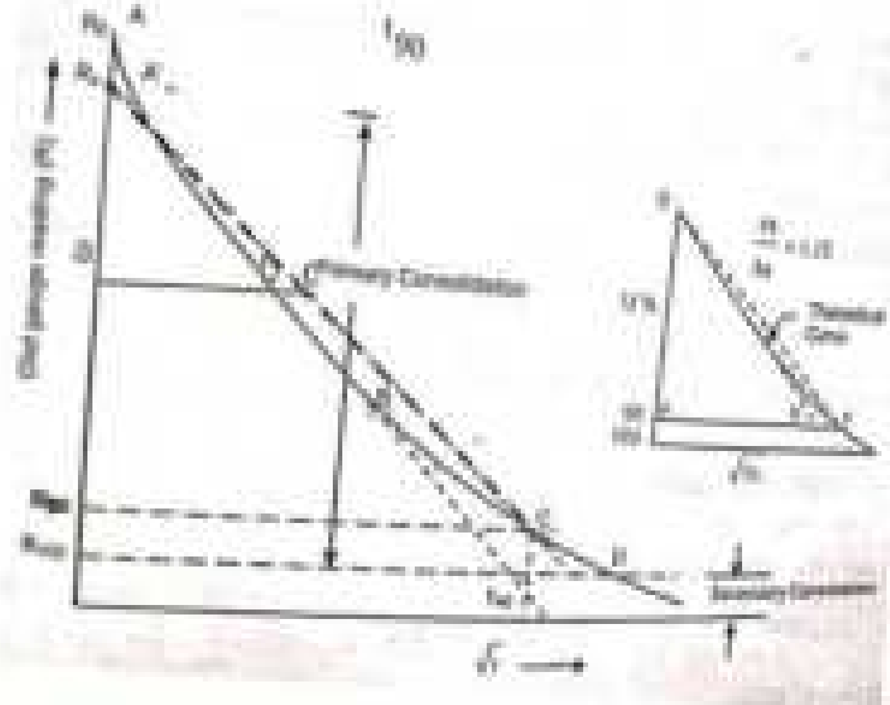
1. Logarithm of time : Casagrande method



R50=(Rc+R100)/2
Tv=(Cv t)/d²

Tv=0.197
For 50% consolidation

2. Square root time: Taylor method



$Tv=(Cv\ t)/d^2$

$Tv=0.848$
For 90% consolid-
ation

* Time factor $\boxed{T_v = \frac{C_v t}{d^2}}$

(Problem based formula)

$C_v \to cm^2/sec$

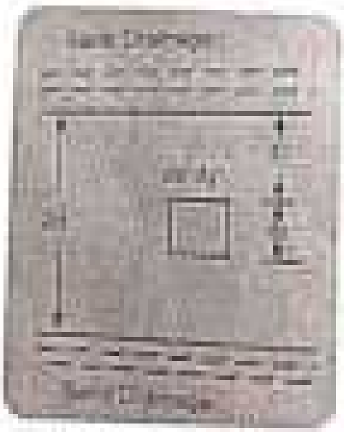
23

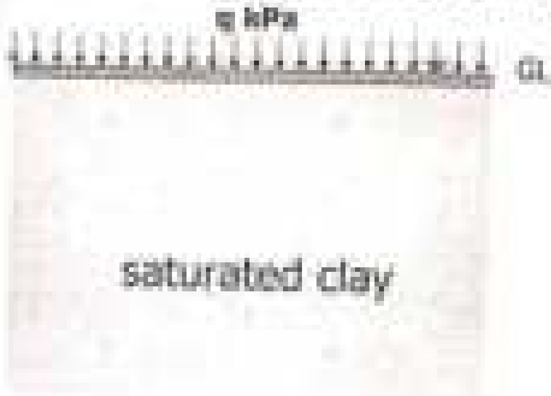
Terzaghi theory of one dimensional consolidation

Assumptions

1. The soil is homogenous
2. The soil is fully saturated
3. The solid particles and water are incompressible
4. The flow is one dimensional
5. Darcy's law is valid
6. k and m_v remains constant
7. There is unique relationship betn void ratio and effective stress and remain constant

One Dimensional Consolidation





According to Darcy's law

$$V_z = -k_z \times i$$

$$V_z = -k_z \times \frac{dh}{dz}$$

$$V_z = -k_z \times \left(\frac{\partial u}{\partial z}\right)\frac{1}{\gamma_w}$$

Inflow $= dx \times dy \times V_z$

Out flow =

$$dx \times dy \times \left(V_z + \frac{\partial V_z}{\partial z} \times dz\right)$$

Difference

$$dq = \frac{\partial V_z}{\partial z} \times dz \times dx \times dy$$

$$dq = \frac{dV}{dt} = \frac{\partial V_z}{\partial z} \times dz \times dx \times dy$$

or

$$dq = \frac{\partial\left(-k_z \times \left(\frac{\partial u}{\partial z}\right)\frac{1}{\gamma_w}\right)}{\partial z} \times dz \times dx \times dy \text{-------}1$$

Now we know, $\frac{dV}{V} = m_v \times \Delta\sigma'_v$

$$\text{Or, } dV = dx \times dy \times dz \times m_v \times \Delta\sigma'_z$$

Or, $dq = \frac{dV}{dt} = dx \times dy \times dz \times m_v \times \frac{\Delta\sigma'_v}{dt}$ --------2

$$\frac{\Delta\sigma'_v}{dt} = -\frac{du}{dt}$$

$$\frac{du}{dt} = \frac{-k_z}{m_v \times \gamma_w} \times \frac{\partial^2 u}{\partial z^2} = c_v \times \frac{\partial^2 u}{\partial z^2} \text{..........}3$$

Solution of one-dimensional equation is complicated. The approximate solution used to calculated degree of consolidation

i. When U < 0.6 : $T_v = U^2 \frac{\pi}{4}$

ii. When U > 0.6 :

$$T_v = -0.933 \log(1 - U) - 0.085$$

Settlement computations

Two different ways to estimate the consolidation settlement:

(a) using m_v

$$\text{settlement} = m_v \, \Delta\sigma \, H$$

(b) using e-log σ_v' plot

next slide

$$settlement = \frac{\Delta e}{1 + e_o} H$$

Settlement computations

~ computing Δe using e-log σ_v' plot

If the clay is normally consolidated,

the entire loading path is along the VCL.

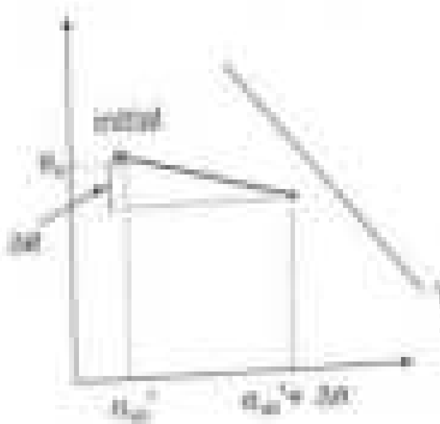
$$S = \frac{H}{1+e_0} C_c \log \frac{\sigma_{vo}' + \Delta\sigma'}{\sigma_{vo}'}$$

Settlement computations

» computing Δe using e-log σ_v' plot

If the clay is <u>overconsolidated</u>, and remains so by the end of consolidation,

$$S = \frac{H}{1+e_0} C_r \log \frac{\sigma_{vo}' + \Delta\sigma'}{\sigma_{vo}'}$$



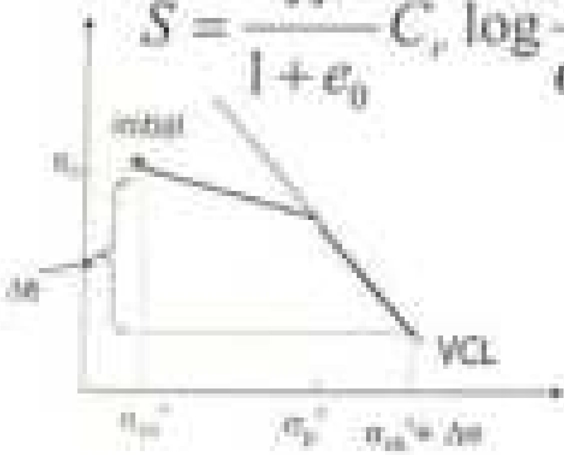
note the use of C_r

Settlement computations

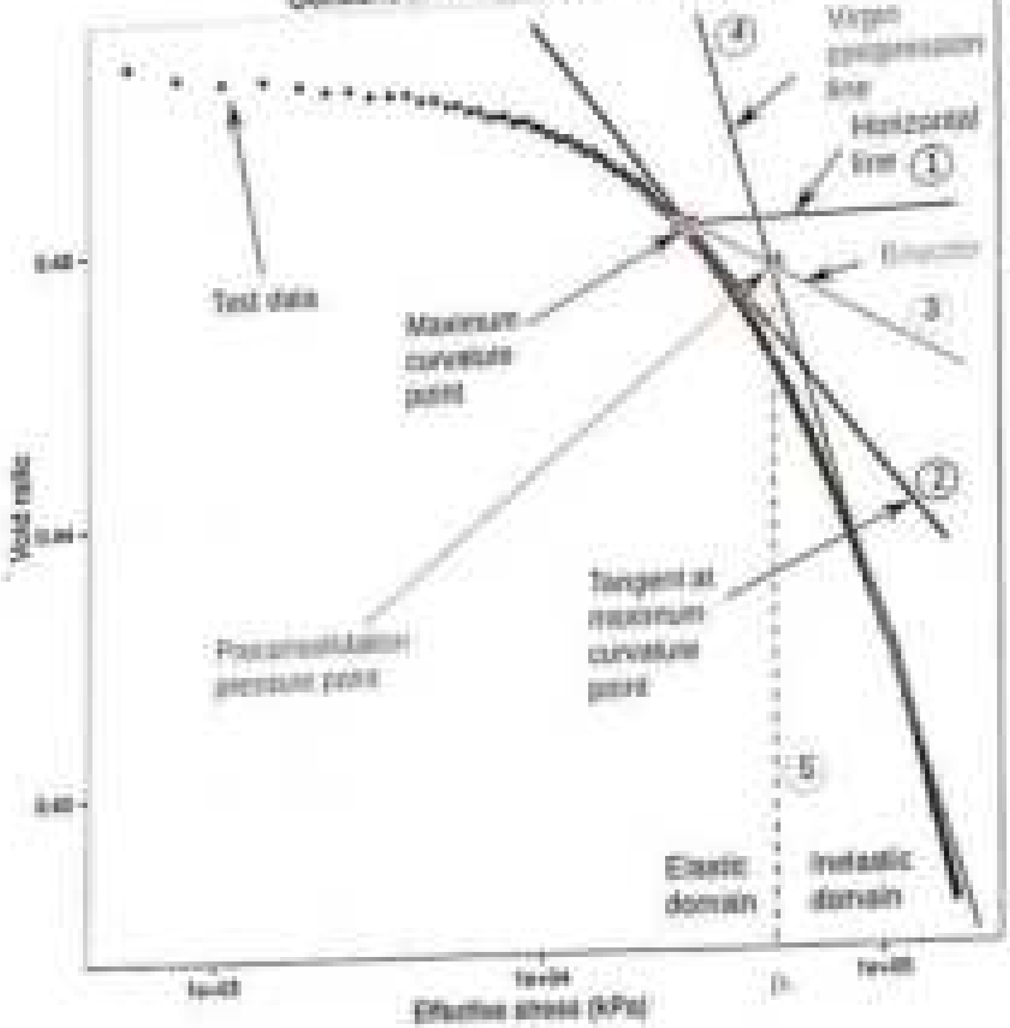
~ computing Δe using e-log σ_v' plot

If an overconsolidated clay becomes normally consolidated by the end of consolidation,

$$S = \frac{H}{1+e_0} C_r \log\frac{\sigma_p'}{\sigma_{vo}'} + \frac{H}{1+e_0} C_c \log\frac{\sigma_{vo}' + \Delta\sigma'}{\sigma_p'}$$



Consolidation curve for saturated clay



Ch- 9

FOUNDATION ENGINEERING

Foundation is the lowest part of the building or the civil structure that is in direct contact with the soil which transfers loads from the structure to the soil safely.

Generally, the foundation can be classified into two, namely **shallow foundation** and **deep foundation.**

Functions of foundations

1. Provide overall lateral stability for the structure
2. Foundation serve the function of providing a level surface for the construction of substructure
3. Load Distribution is carried out evenly
4. The load intensity is reduced to be within the safe bearing capacity of the soil
5. The soil movement effect is resisted and prevented
6. Scouring and the undermining issues are solved by the construction of foundation

In the following table the main differences between shallow and deep foundation are given:

	Sources	Shallow Foundation	Deep Foundation
1	Definition	Foundation which is placed near the surface of the earth or transfers the loads at a shallow depth is called shallow foundation.	Foundation which is placed at a greater depth or transfers the loads to deep strata is called deep foundation.
2	The depth of foundation	The depth of shallow foundation is generally about 3 meters or the depth of foundation is less than the footing ~~wi~~ width $B_f > D_f$	Greater than shallow foundation.

	Sources	Shallow Foundation	Deep Foundation
3	Cost	Shallow foundation is cheaper.	Deep foundations are generally more expensive than shallow foundation.
4	Feasibility	Shallow foundations are easier to construct.	The construction process of a deep foundation is more complex.
5	Mechanism of load transfer	Shallow foundations transfer loads mostly by end bearing.	Deep foundations rely both on end bearing and skin friction, with few exceptions like end bearing pile.
6	Advantages	Construction materials are available, less labor is needed, construction procedure is simple at an affordable cost etc.	Foundation can be provided at a greater depth, Provides lateral support and resists uplift, effective when foundation at a shallow depth is not possible, can carry huge load etc.
7	Disadvantages	Possibility of a settlement, usually applicable for lightweight structure, weak against lateral loads etc.	More expensive, needs skilled labors, complex construction procedure, can be time-consuming and some types of deep foundations are not very flexible etc.
8	Types	Isolated foundation, strip foundation, mat foundation, combined foundation etc.	Pier foundation, pile foundation, caissons etc.

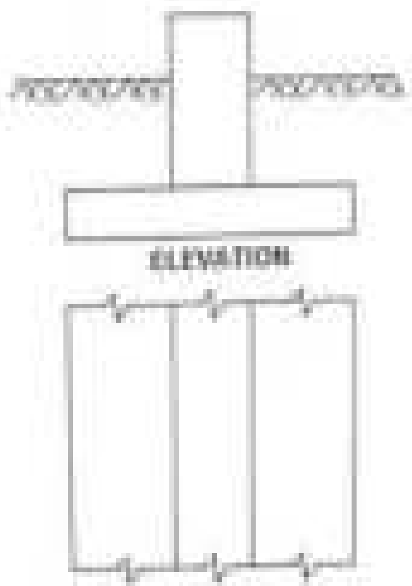
Shallow foundations are constructed where soil layer at shallow depth (upto 1.5m) is able to support the structural loads. The depth of shallow foundations are generally less than its width.

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

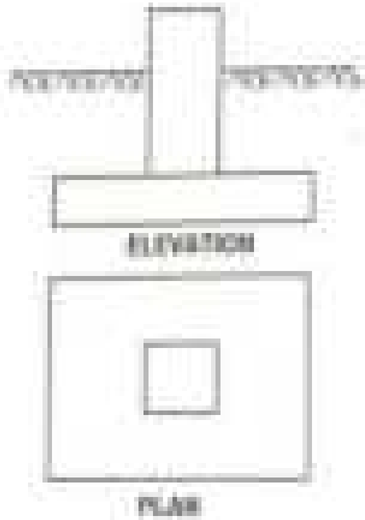
Strip Footing

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.



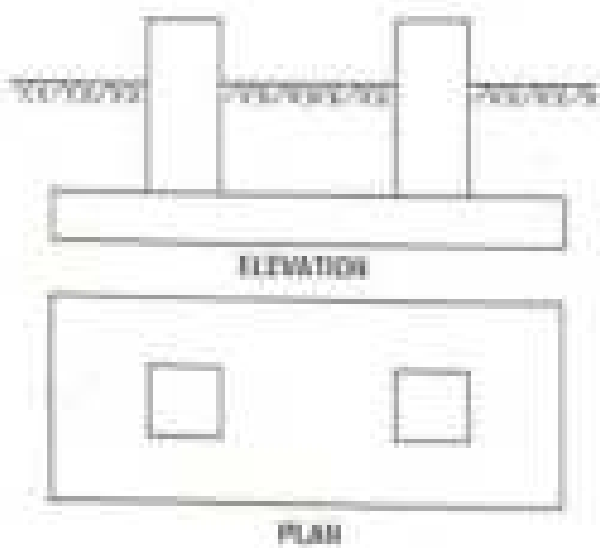
Spread Footing

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.



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 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.

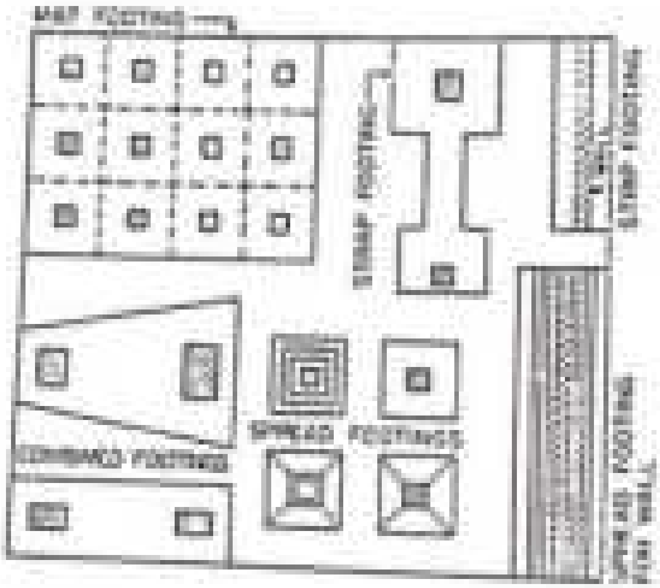
ELEVATION

PLAN

Mat or Raft Foundation

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-homogenous soils or where there is a large variation in the loads on individual columns.



Deep foundation is required to carry loads from a structure through weak compressible soils or fills on to stronger and less compressible soils or rocks at depth, or for functional reasons. Deep foundations are founded too deeply below the finished ground surface for their base bearing capacity to be affected by surface conditions, this is usually at depths >3 m below finished ground level.

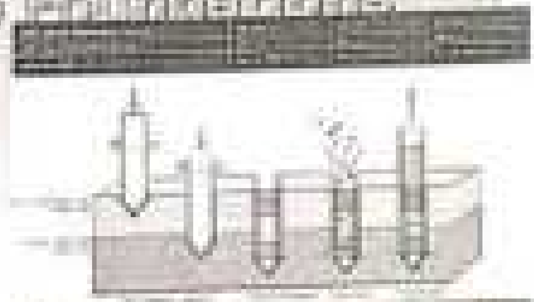
Deep foundation can be used to transfer the loading to a deeper, more competent strata at depth if unsuitable soils are present near the surface.

Deep foundation are further classified into the following types ;

1. Pile foundation
2. Well foundation
3. Caisson foundation

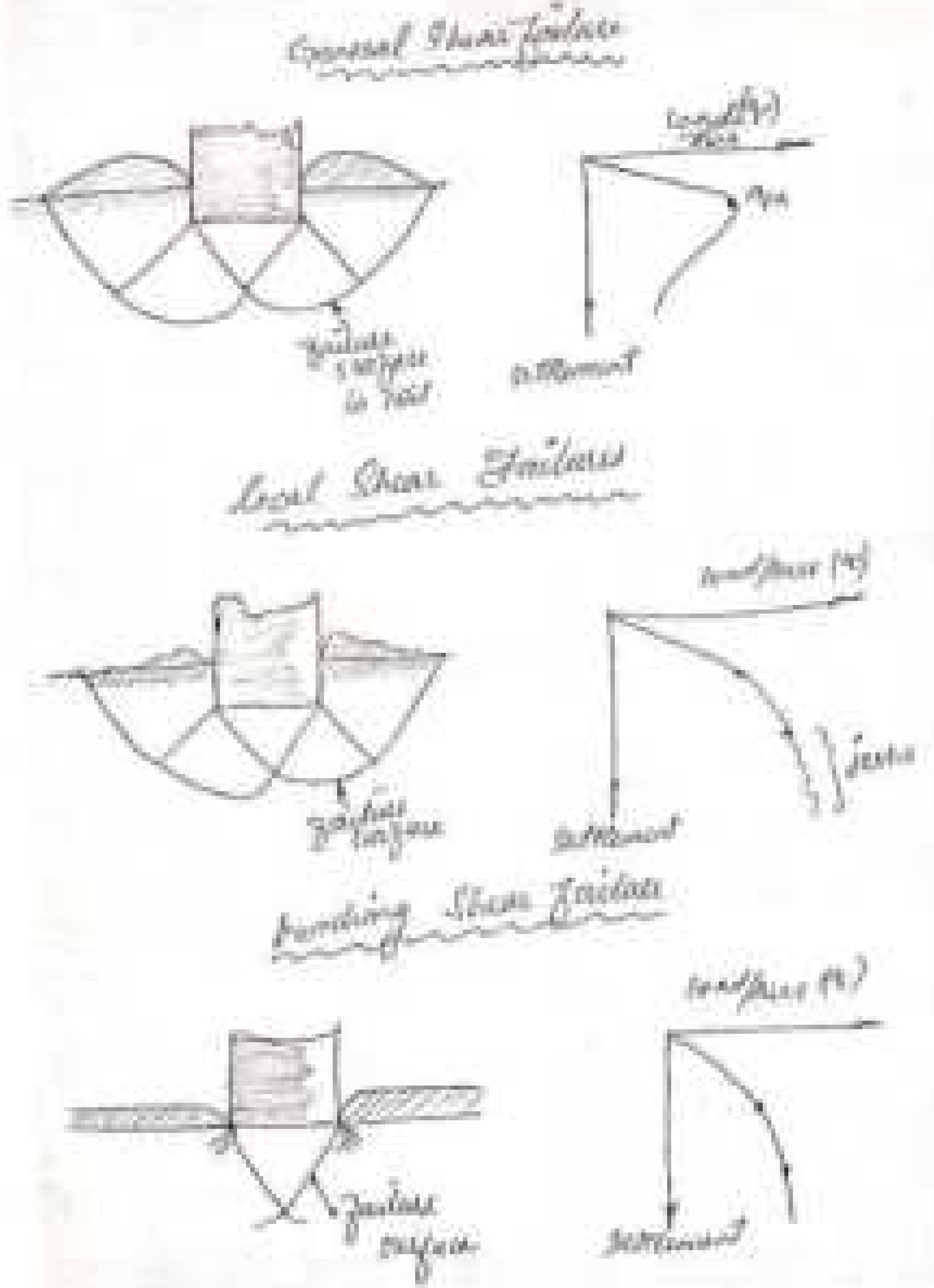
Examples of Deep Foundations

- Pile foundations
- Pier foundations
- Wells or Caissons foundations.



Pier Foundation	Caisson	Pile Foundation
Pier foundation is a type of deep foundation, which consists of a cylindrical column of large diameter to support and transfer large superimposed loads to firm strata below.	Caissons are watertight structures made up of wood, steel or reinforced concrete built above the ground level and then sunken into the ground.	Pile foundation is a type of deep foundation, in which the loads are taken to a low level by means of vertical timber, concrete or steel.
The types of pier foundations are masonry or concrete piers and drilled caissons.	The types of caissons are box, open, pneumatic, monolithic, floating, excavated etc.	The types of pile foundation are end-bearing piles, friction piles, compaction piles, anchor piles, tension or uplift piles, sheet and batter piles etc.

Pier Foundation	Caisson	Pile Foundation
Pier is inserted down to the bedrock.	Caisson is putting a box into underwater and pouring it with concrete.	Pile is a column of material driven by a piledriver.
Pier has a footing.	Caisson doesn't have a footing.	Pile doesn't have a footing.
Pier is typically dug out and cast in place using forms.	Caissons are driven into surface condition.	Piles are driven into surface condition.



Shear failure of Soil

There are three modes of shear failure, i.e. General, Local and Punching shear failures depending upon the compressibility of soil and depth of footing with respect to its breadth (i.e D/B Ratio). When the ultimate bearing capacity of the soil is reached, it may fail in one of the following three failure mode depending upon the type of soil and depth to width ratio of the footing (i.e. D/B)

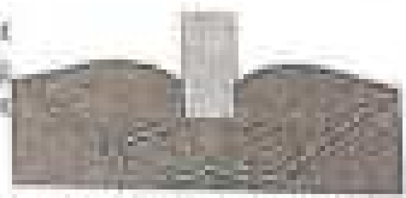
- General Shear Failure
- Local Shear Failure
- Punching Shear Failure

1. General Shear Failure

- In this mode a slight downward movement of the footing develops fully plastic zones and a sudden failure takes place with a considerable bulging of the ground surface adjacent to the footing
- Characterized by well defined failure pattern, consisting of a wedge and slip surface and bulging (heaving) of soil surface adjacent to the footing
- Sudden collapse occurs, accompanied by tilting of the footing
- This type of failure occurs in case of dense sand or stiff cohesive soil supporting the footing
- Failure load is well defined
- The load-settlement diagram is similar to stress-strain for dense sand or over-consolidated clay as shown
- The ultimate load is well defined on this curve as shown typically in figure given below

2. Local shear failure

Failure pattern consists of wedge and slip surface but is well defined only under the footing. Slight bulging of soil surface occurs. Tilting of footing is not expected



- In this mode a large deformation takes place under the footing before the development of failure zones, i.e. large vertical settlement takes place before slight bulging of the ground surface
- Tilting of footing is not expected
- Ultimate load is not well defined
- It takes place in moderately compressible soils or loose sand i.e Occurs in soil of high compressibility
- Yielding takes place close to the lower edges of the footing
- Several yield developments may occur accompanied by settlement in a series of jerks
- The bearing pressure at which the first yield takes place is referred to as the first-failure pressure or first failure load

3. Punching shear failure

- Failure pattern is not well defined
- No bulging of ground surface and no tilting of footing occurs

- The yield surfaces are vertical planes immediately adjacent to the sides of the foundation
- The ground surface may be dragged down thus, no bulging of the surface takes place
- Failure take place immediately below footing and surrounding soil remains relatively unaffected
- Large settlements-ultimate load is not well defined
- Punching Shear Failure takes place in weak compressible soils with considerable vertical settlement i.e Occurs in soil of very high compressibility
- It also occurs in the soil of low compressibility, if the foundation is located at considerable depth.
- After the first yield the load-settlement curve will steepen slightly, but remain fairly flat
- Punching Shear Failure may also take place in soil of low compressibility, if the foundation is located at a considerable depth.

Modes of Shear Failure (Summary)

	General	Local	Punching
Relative Settlement	Less	Large	Large
Bulging	Significant	Less	No
Tilting of Footing	Expected	Not expected	Not expected
Ultimate Load	Well defined	Not well defined	Not well defined
Failure Pattern	Wedge + Slip Surface + Bulging	Wedge + Slip Surface + Bulging(no or less)	Not well defined
Occurs in (Soil Type)	Dense	Less compressible	Highly Compressible

GEO-TECHNICAL ENGG (NUMERICALS)

Q1) A fully saturated soil sample has a mass of 130gm & has a volume of $64 cm^3$. After oven drying the mass of soil sample is 105gm. Assuming that the volume doesn't change during drying condition, determine the following

(i) Sp. gravity of soil
(ii) Void ratio
(iii) Porosity
(iv) dry density

$M = 130 gm$ $\qquad$ $V = 64 cm^3$

$M_d = 105 gm$ $\qquad$ $S = 100\%$

$$M_w = M - M_d = 130 - 105 = 25 gm$$

water content $\omega = \frac{M_w}{M_d} = \frac{25}{105} = 23.8\%$

$$\rho = \frac{M}{V} = \frac{130}{64} = 2.03 gm/cc$$

$$\rho_d = \frac{\rho}{1+\omega} \quad \text{OR} \quad \rho_d = \frac{M_d}{V}$$

$$= \frac{2.03}{1 + 23.8\%} \qquad = \frac{105}{64}$$

$$= 1.64 g/cc \qquad = 1.64 gm/cc \text{ (Ans)}$$

$wG = Se$

$\Rightarrow 23.8\% \, G = 1 \times e$

$\Rightarrow e = 0.238G$

$\rho = \dfrac{(G+Se)\rho_w}{1+e}$ or, $\rho_d = \dfrac{G\rho_w}{1+e}$

$\Rightarrow 2.03 = \dfrac{G+e}{1+e}$ $1.64 = \dfrac{G\times1}{1+e}$

$\Rightarrow 2.03 = \dfrac{G+0.238G}{1+0.238G}$ $\Rightarrow 1.64 + 1.64e = G$

$\Rightarrow 1.64 + 1.64 \times 0.238 = G$

$\Rightarrow G = 2.687$

$\Rightarrow 2.03 + 2.03 \times 0.238G = G + 0.238G$

$\Rightarrow 2.03 + 0.483G = G + 0.28G$

$\Rightarrow 0.797G = 2.03$

$\Rightarrow G = 2.54$

$\Rightarrow 2.03 + 2.03e = G + e$

$\Rightarrow 2.03 + 1.03e = G$

$\Rightarrow 2.03 + 1.03 \times 0.238G = G$

$\Rightarrow 2.03 + 0.245G = G$

$\Rightarrow G = 2.687$ (Ans)

$e = 0.238G = 0.64$ (Ans)

$\eta = \dfrac{e}{1+e} = \dfrac{0.64}{1+0.64} = 0.39$ (Ans)

Q5) A soil sample of height 8cm and area of cross section of 120 cm² was subjected to falling head permeability test. In a time interval of 5 minutes, the head dropped by 20cm. If the c/s area of the stand pipe is 2cm², calculate the coefficient of permeability of sample.

Falling head Permeability Test :

$L = 8cm$
$A = 120 cm^2$ $a = 2cm^2$
$t = 5 mins$
$h_1/h_2 = 20$
$K = ?$

$$K = \frac{2.303\,aL}{At} \log\left(\frac{h_1}{h_2}\right)$$

$$= \frac{2.303 \times 2 \times 8}{120 \times 5} \log(20)$$

$$= 0.079 \ cm/min$$

Q3) A strip footing is reqd to carry a net load of 900KN at a depth of 1m. Taking a FOS of 3. Determine width of footing. Take $\phi = 30°$, $\gamma = 19 KN/m^3$, $C = 20 KN/m^2$, $N_c = 37.2$, $N_q = 22.5$, $N_\gamma = 19.7$ (Assume general shear failure)

Strip footing?

$P = 900 kN$, F.O.S. = 3

$D_f = 1m$, $B_f = ?$

Sol: $\gamma = 19 KN/m^3$, $\phi = 30°$ & $C = 20 KN/m^2$

Using IS code formula :-

$$\boxed{q_{nu} = CN_c s_c d_c i_c + q(N_q - 1) s_q d_q i_q + 0.5\gamma B_f N_\gamma s_\gamma d_\gamma i_\gamma}$$

$\boxed{q = \gamma D_f}$

* For vertical footing $i_c = i_q = i_\gamma = 1$
* For strip footing $s_c = s_q = s_\gamma = 1$

* $N_c = 37.2$, $N_q = 22.5$, $N_\gamma = 19.7$

$d_\gamma = d_q = 1 + 0.1\left[\frac{D_f}{B_f}\tan\left(45^\circ + \frac{\phi}{2}\right)\right]$

$= 1 + 0.1\left[\frac{1}{B_f}\tan\left(45^\circ + \frac{30^\circ}{2}\right)\right]$

$= 1 + \frac{0.173}{B_f}$ for $\phi > 10^\circ$

$d_\gamma = d_q = 1$ for $\phi < 10^\circ$

$d_c = 1 + 0.2\frac{D_f}{B_f}\tan\left(45^\circ + \frac{\phi}{2}\right)$ for $\phi > 10^\circ$ & $\phi < 10^\circ$

$= 1 + \frac{0.346}{B_f}$

Substituting all values we get

$q_{nu} = 20 \times 37.2 \times 1 \times \left(1 + \frac{0.346}{B_f}\right) \times 1$

$+ 19 \times 1 \times (22.5 - 1) \times 1 \times \left(1 + \frac{0.173}{B_f}\right) \times 1$

$+ 0.5 \times 19 \times B_f \times 19.7 \times 1 \times \left(1 + \frac{0.173}{B_f}\right) \times 1$

$= 744 + \frac{257.42}{B_f} + 408.5 + \frac{70.67}{B_f}$

$+ 187.15 B_f + 32.37$

$= 1184.87 + 187.15 B_f + \frac{328.09}{B_f}$

$$q_s = \frac{q_{nu}}{F} + \gamma D_f$$

$$\frac{Load}{Area} = \frac{q_{nu}}{FOS} + 19 \times 1$$

$$\Rightarrow \frac{900\,KN}{D_f \times B_f} = \frac{1}{FOS}\left[1184.87 + 187.15B_f + \frac{328.0}{B_f}\right] + 19$$

$$\Rightarrow \frac{900}{1 \times B_f} = \frac{1}{3}\left[1184.87 + 187.15B_f + \frac{328.0}{B_f}\right] + 19$$

$$\Rightarrow \frac{900}{B_f} = 395 + 62.4B_f + \frac{109.4}{B_f} + 19$$

$$\Rightarrow 900 = 395B_f + 62.4B_f^2 + 109.4 + 19B_f$$

$$\Rightarrow 790.6 = 414B_f + 62.4B_f^2$$

$$\Rightarrow 62.4B_f^2 + 414B_f - 790.6 = 0$$

$$\Rightarrow B_f = 1.5m \text{ (Ans)}$$

Q. A soil has liquid limit of 25% & a flow index of 12.5%. If the plastic limit is 17%, determine the plasticity index & toughness index. If the water content of soil in its natural condition in the field is 20%, find the liquidity index & consistency index.

$W_L = 25\%$ $W_P = 17\%$ $w = 20\%$

$I_f = 12.5\%$ $I_P = ?$ $I_t = ?$ $I_L = ?$ $I_c = ?$

$I_P = W_L - W_P$

$= 25\% - 17\%$

$= 8\%$ (Ans)

Toughness $I_t = \frac{I_P}{I_f}$

$= \frac{8\%}{12.5\%}$

$= 0.64$

$= 64\%$ (Ans)

$$I_L = \frac{W - W_P}{I_P} \times 100\%$$

$$= \frac{20\% - 17\%}{8\%} \times 100\%$$

$$= 37.5\% \quad (\text{Ans})$$

$$I_C = \frac{W_L - W}{I_P} \times 100\%$$

$$= \frac{25\% - 20\%}{8\%} \times 100\%$$

$$= 62.5\% \quad (\text{Ans})$$

Q) A plate bearing test on a pure clayey soil, failure occurred at a load of 125KN. The size of the plate was 45cm x 45cm & the test was conducted at a depth of 1.5m below ground level. Find out the ultimate bearing capacity for a 1.2m wide continuous wall footing with its base at a depth of 1.5m below ground level. The unit wt. of clay may be taken as $19\,KN/m^3$ & $N_c = 5.7$, $N_q = 1$, $N_\gamma = 0$

Size of plate 4.5cm x 4.5cm → square.
For the plate load test on square plate.

Square: $\boxed{q_f = 1.3cN_c + qN_q + 0.4B\gamma N_\gamma}$

$= 1.3cN_c + \gamma D_f N_q + 0.4B_f \gamma N_\gamma$

$q_f = \dfrac{\text{Load}}{\text{Area}}$

$= \dfrac{125\,KN}{0.45 \times 0.45\,m^2}$

$= 617.28\,KN/m^2$

$617.28 = 1.3c \times 5.7 + 19 \times 1.8 \times 1 + 0$

$\Rightarrow c = 78.68 \ KN/m^2$

For Strip footing

$$\boxed{q_f = cN_c + \gamma D_f N_q + 0.5 B \gamma N_\gamma}$$

$\Rightarrow q_f = (78.68 \times 5.7) + (19 \times 1.8 \times 1) +$

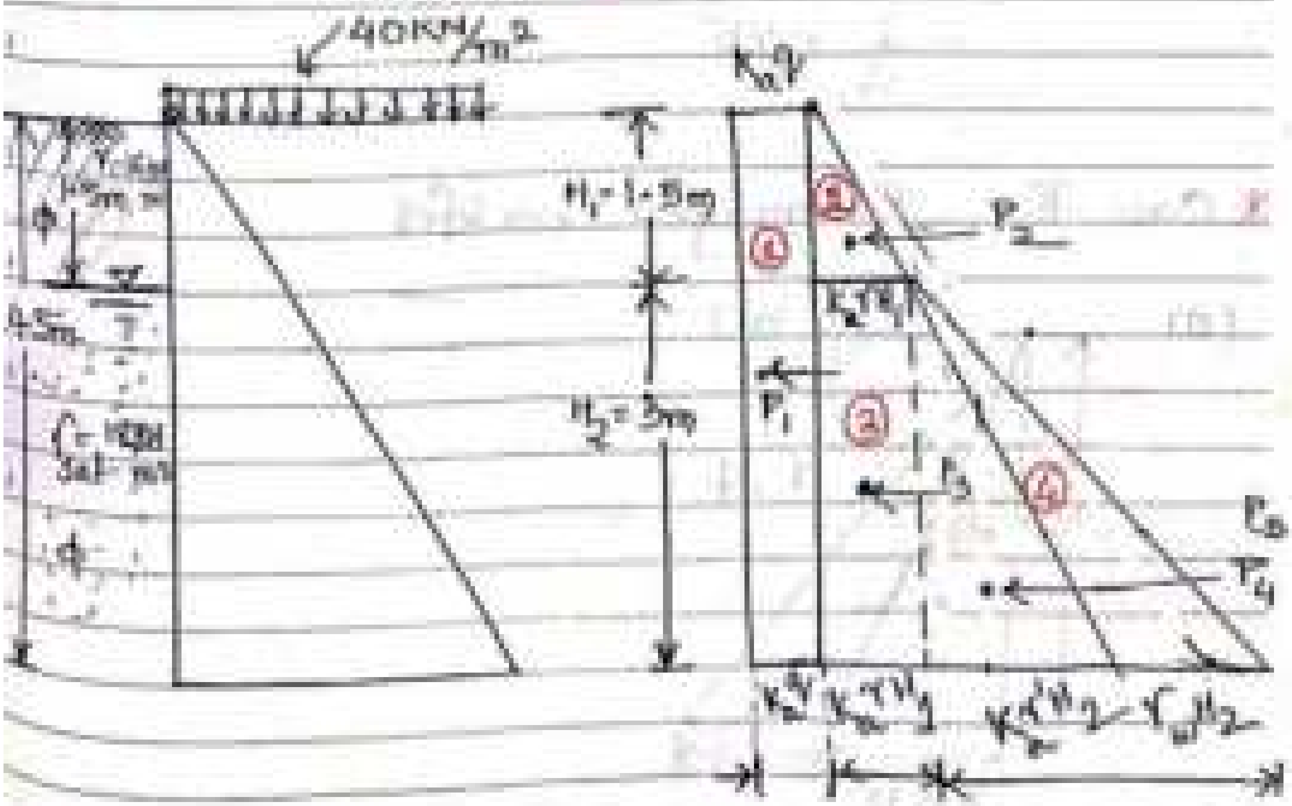
$\Rightarrow q_f = 482.67 \ KN/m^2$

$q_s = \frac{q_{nu}}{F} + \gamma D_f$

$= \frac{448.67}{3} + 19 \times 1.8$

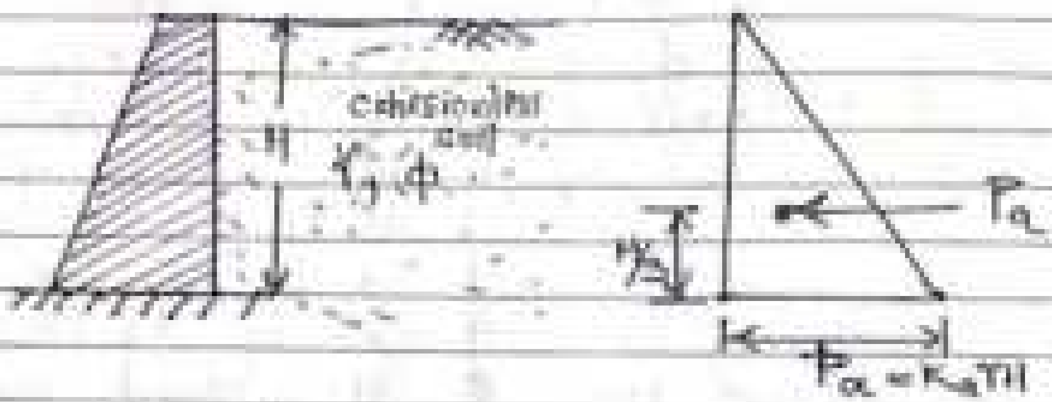
$= 183.67 \ KN/m^2$

Q) A retaining wall 4.5m high has a smooth vertical back with cohesionless soil backfill. If there is an uniformly distributed surcharge load of 40 kN/m² over the backfill determine the magnitude & point of application of active earth pressure per meter length of the wall. The backfill is saturated below 1.5m from the top. Take unit weight of dry backfill = 16 kN/m³, unit wt of saturated backfill = 18 kN/m³, $\phi = 30°$ (For both dry & saturated case)



Rankine's Theory

★ Case - I Backfill with no surcharge



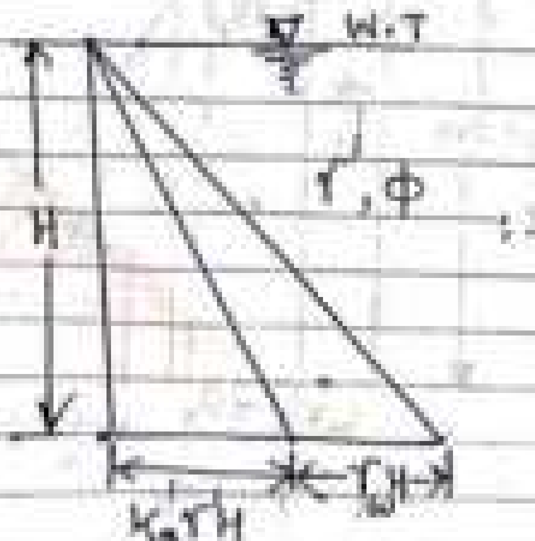
$P_a = p_a \times area$ $K_a = \frac{1 - \sin\phi}{1 + \sin\phi}$

$= K_a \gamma H \times \frac{1}{2} \cdot H$

$= \frac{1}{2} K_a \gamma H^2$

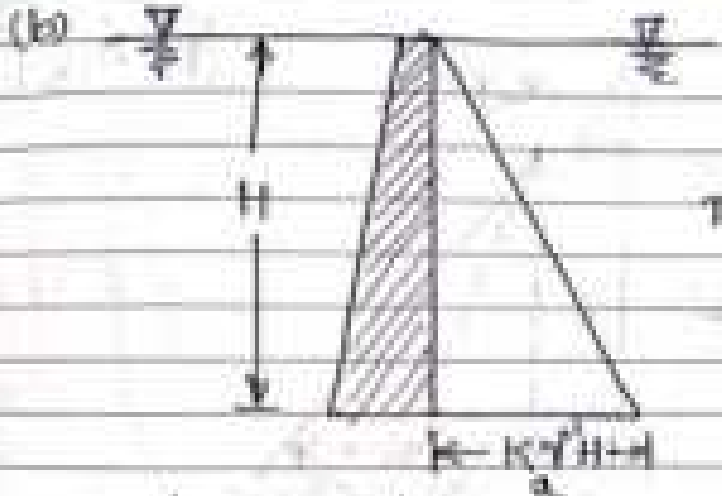
★ Case - II Submerged backfill

(a)



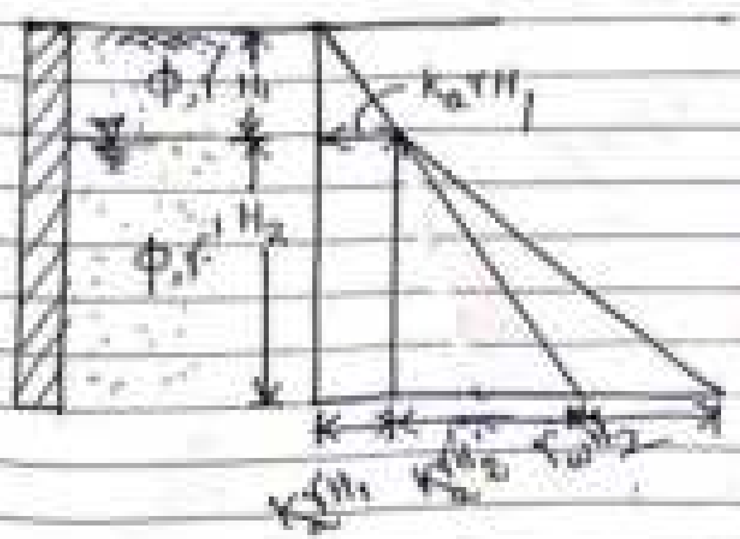
$$p_a = K_a \gamma' H + \gamma_w H$$

(b)



$$p_a = K_a \gamma' H$$

(c) Partially Saturated/Submerged

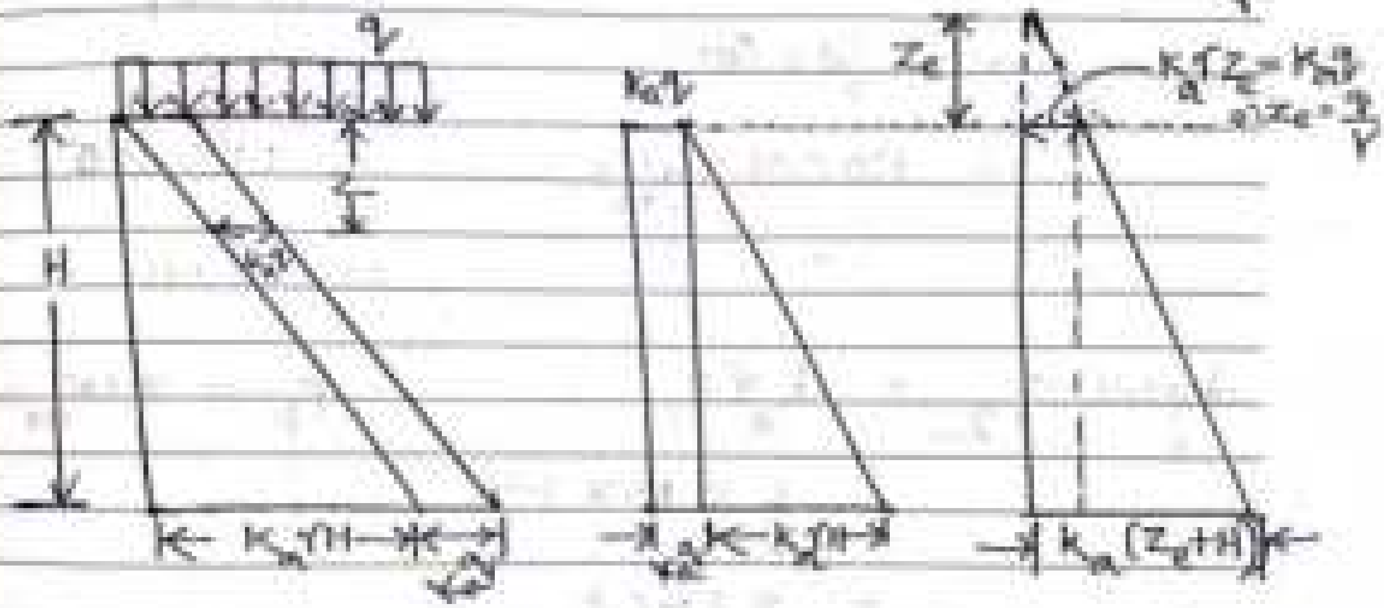


$$p_a = K_a \gamma H_1 + K_a \gamma' H_2 + \gamma_w H_2$$

$(\phi_1 > \phi_2)$

$$p_a = K_{a_1} \gamma H_1 + K_{a_2} \gamma' H_2 + \gamma_w H_2$$

Case - (iii) Backfill with uniform surcharge



$$p_a = K_a \gamma H + K_a q \quad \& \quad z_e = \frac{q}{\gamma}$$

Pressure

Surcharge $p_1 = K_a q$ $\qquad K_a = \frac{1-\sin\phi}{1+\sin\phi}$

$= \frac{1}{3} \times 40$ $\qquad = \frac{1-\sin 30^\circ}{1+\sin 30^\circ}$

$= 13.33 \, kN/m^2$

$= \frac{1}{3}$

dry soil $p_2 = K_a \gamma H_1$ $\qquad p_3 = 8 \, kN/m^2$

$= \frac{1}{3} \times 16 \times 1.5$

$= 8 \, kN/m^2$

Submerged Soil $p_4 = K_a \gamma' H_2$

$= \frac{1}{3} \times 18 \times 3$

$= 18 \, kN/m^2$

water $p_5 = \gamma_w H_2$

$= 9.81 \times 3$

$= 29.43 \, kN/m^2$

$P_1 = p_1 \cdot$ area of the diagram with at CG $H/2$

$= 13.33 \times 4.5$ KN/m at $\frac{4.5}{2}$

$= 60$ KN/m at 2.25 m from base.

$P_2 = p_2 \cdot$ area acting @ $\frac{1}{3} \cdot H_1 + H_2$

$= 8 \times \frac{1}{2} \times 1.5$ $= \frac{1}{3} \times 1.5 + 3$

$= 6$ KN/m $= 3.5$ m from base

$P_3 = p_3 \cdot$ area acting @ $\frac{H_2}{2} = \frac{3}{2}$

$= 8 \times 3$

$= 24$ KN/m acting @ 1.5m from base

$P_4 = p_4 \cdot$ area @ $\frac{1}{3} H_2$ from base

$= 18 \times \frac{1}{2} \times H_2$ $= \frac{3}{3} = 1$m

$= 18 \times \frac{1}{2} \times 3$

$= 27$ KN/m @ 1m from base

$P_5 = p_5 \times$ Area @ $\frac{H_3}{3}$ from base $= 3/3 = 1$

$= 29.43 \times \frac{1}{2} \times 3$

$= 44.14$ kN/m @ 1m from base

Total $\Sigma P_i = P_1 + P_2 + P_3 + P_4 + P_5$

$= 60 + 6 + 24 + 27 + 44.14$

$= 161.14$ kN/m (Ans)

≈ 160 kN

$CG = \frac{\Sigma P x_i}{\Sigma P_i}$

$= \frac{(60 \times 2.25) + (6 \times 3.5) + (24 \times 1.5) + (27 \times 1) + (44.14)}{161.14}$

$= \frac{263.14}{161.14}$

$= 1.63$ m (Ans)

Q) A soil sample of 8cm height & 100 cm^2 c/s area was subjected to the falling head permeability test. In a time interval of 10 min, the head dropped from 75cm to 25cm. If the c/s area of the stand pipe is 2 cm^2. Compute co-efficient of permeability.

$L = 8cm$ $\quad t = 10 min$

$A = 100 cm^2$ $\quad h_1 = 75 cm$

$a = 2 cm^2$ $\quad h_2 = 25 cm$

$$k = \frac{aL}{At} \ln\left(\frac{h_1}{h_2}\right)$$

$$= \frac{2 \times 8}{100 \times 10 min} \ln\left(\frac{75}{25}\right)$$

$$= 0.01757 \ cm/min$$

$$= 2.9 \times 10^{-4} \ cm/sec$$

Q) An undistributed sample of clay 2.5cm thick consolidation 50% in 35min when tested in the laboratory with drainage being allowed at top & bottom. The clay layer from which the sample was taken is 425mm thick in the field. How much time will it take to consolidate 50% with double drainage, If the clay stratum has only single drainage, then calculate the time to consolidate 50%

Clay in Lab

$H = 2.5\text{cm}$

$U = 50\%$

$t = 35\text{min}$

Double drainage

$d = \frac{H}{2} = \frac{2.5}{2} = 1.25\text{cm}$

Clay in field

$H = 42.5\text{cm}$

$U = 50\%$

$t = ?$ For doub drai / for sing drainage

$$\boxed{C_v = T_v \frac{d^2}{t}}$$

$$T_v = \frac{\pi}{4}(U\%)^2 = 0.196$$

$$= 0.196 \times \frac{1.25^2}{35} \text{ cm}^2/\text{min}$$

$$= 1.45 \times 10^{-4} \text{ cm}^2/\text{sec}$$

Field

Double drainage

$d = \frac{H}{2}$

$= \frac{42.5}{2}$

$= 21.25 \text{ cm}$

$U = 50\%$

$T_v = \frac{\pi}{4}(U\%)^2$

$= 0.196$

$C_v = 1.45 \times 10^{-4} \text{ cm}^2/\text{sec}$

$C_v = \frac{T_v d^2}{t}$

$\Rightarrow 1.45 \times 10^{-4} = 0.196 \times \frac{21.25^2}{t}$

$\Rightarrow t = 7$ days

Single drainage

$d = H$

$= 42.5 \text{ cm}$

$U = 50\%$

$T_v = \frac{\pi}{4}(U\%)^2$

$= 0.196$

$C_v = 1.45 \times 10^{-4} \text{ cm}^2/\text{sec}$

$C_v = \frac{T_v d^2}{t}$

$\Rightarrow 1.45 \times 10^{-4} = \frac{0.196 \times 42.5^2}{t}$

$\Rightarrow t = 28$ days

Or, Same Soil → C_v same
Consolidatⁿ 50% → T_v same

$$C_v = \frac{T_v d^2}{t}$$

$$\Rightarrow \boxed{t \propto d^2}$$

$$\Rightarrow \frac{t_{1 \to \text{lab}}}{t_{2 \to \text{field (double)}}} = \frac{d_1^2}{d_2^2}$$

$$\Rightarrow \frac{35\,\text{min}}{t_2} = \frac{1.25^2}{21.25^2}$$

$$\Rightarrow t_2 = 10115\,\text{min}$$

$$= 7\text{ days (Ans)}$$

$$\boxed{t \propto d^2}$$

$$\frac{t_{1 \to \text{lab}}}{t_{2 \to \text{field (single)}}} = \frac{d_1^2}{d_2^2}$$

$$\Rightarrow \frac{35\,\text{min}}{t_2} = \frac{1.25^2}{42.5^2}$$

$$\Rightarrow t_2 = 40460\,\text{min} = 28\text{ days (Ans)}$$

Q) The following properties were determined for two types of soils A & B

Property		A	B
Liquid limit	w_L	60%	35%
Plastic limit	w_P	20%	20%
Moisture content	w	40%	27%
Degree of saturation	S	100%	100%
Sp. gravity of grains	G	2.7	2.65

Which of the two soils:
(i) contains more clay fractions
(ii) has a greater dry density
(iii) has greater void ratio &
(iv) has a greater saturated unit weight

Also classify these soils as per plasticity chart of I.S classification system

(iii) Void ratio $\boxed{e = \frac{wG}{S}}$

$$(e)_{Soil\,A} = \frac{40\% \times 2.7}{100\%} = 1.08$$

$$(e)_{Soil\,B} = \frac{27\% \times 2.65}{100\%} = 0.72$$

Soil A has greater void ratio. (Ans.)

(iii) Dry density $\boxed{\rho_d = \frac{G \rho_w}{1+e}}$

$$(\rho_d)_{Soil\,A} = \frac{2.7 \times 1}{1 + 1.08} = 1.3\ g/cc$$

$$(\rho_d)_{Soil\,B} = \frac{2.65 \times 1}{1 + 0.72} = 1.54\ g/cc$$

Soil B has greater dry density. (Ans)

(iv) Saturated unit weight $\gamma_{sat} = \frac{(G+e)\gamma_w}{1+e}$

$$(\gamma_{sat})_{Soil\,A} = \frac{(2.7 + 1.08)}{1 + 1.08} 10$$

$$= 18.17\ kN/m^3$$

$$(\gamma_{sat})_{Soil\,B} = \left(\frac{2.65 + 0.72}{1 + 0.72}\right) 10$$

$$= 19.6\ kN/m^3$$

Soil B has greater saturated unit w

(i) Plastic Index $\boxed{I_P = W_L - W_P}$

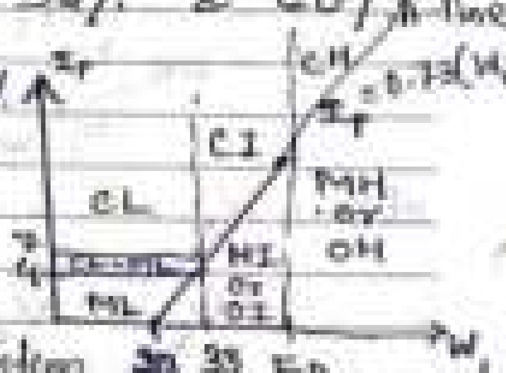
$$(I_P)_{Soil\,A} = 60\% - 28\% = 32\%$$

$$(I_P)_{Soil\,B} = 35\% - 20\% = 15\%$$

$(I_P)_{Soil\,A}$ & $(W_L)_{Soil\,A} \approx 32\%$ & 60%

$(I_P)_{Soil\,B}$ & $(W_L)_{Soil\,B} \approx 15\%$ & 35%

Soil A $\rightarrow$ CH
Soil B $\rightarrow$ CL } (Ans)



$\therefore$ Soil B has more clay fraction (Ans)

Soil A : A-Line = 29.2 = 0.73(60 - 20)
Soil B : A-Line = 0.73(35-20) = 11

Q) Tests on a fill reveal that one cubic meter of soil in the fill weighs 16.24kN & after being dried 14kN. If the sp. gravity of soil solids is 2.65, determine the water content, void ratio, porosity & degree of saturation of the soil mass in moist state. Also draw the phase diagram of the soil indicating volume & mass/weight.

$W = 16.24\,kN$

$W_s = 14\,kN$

$W_w = 16.24\,kN - 14\,kN = 2.24\,kN$

water: $V_w = \frac{W_w}{\gamma_w} = \frac{2.24\,kN}{10\,kN/m^3} = 0.224\,m^3$

Soil solid: $V_s = \frac{W_s}{\gamma_s} = \frac{14\,kN}{2.65 \times 10} = 0.53\,m^3$

or $\frac{W}{G\gamma_w}$

air: $V_a = 1 - (V_w + V_s) = 0.246\,m^3$

$V_v = V_a + V_w = 0.246 + 0.224 = 0.47$

Moisture content $w = \frac{W_w}{W_s}$

$= \frac{2.24\,kN}{14\,kN}$

$= 0.16 = 16\%$ (Ans)

Void ratio $e = \frac{V_v}{V_s}$

$= \frac{0.47\,m^3}{0.53\,m^3}$

$= 0.89$ (Ans.)

Degree of saturation $S = \frac{wG}{e}$

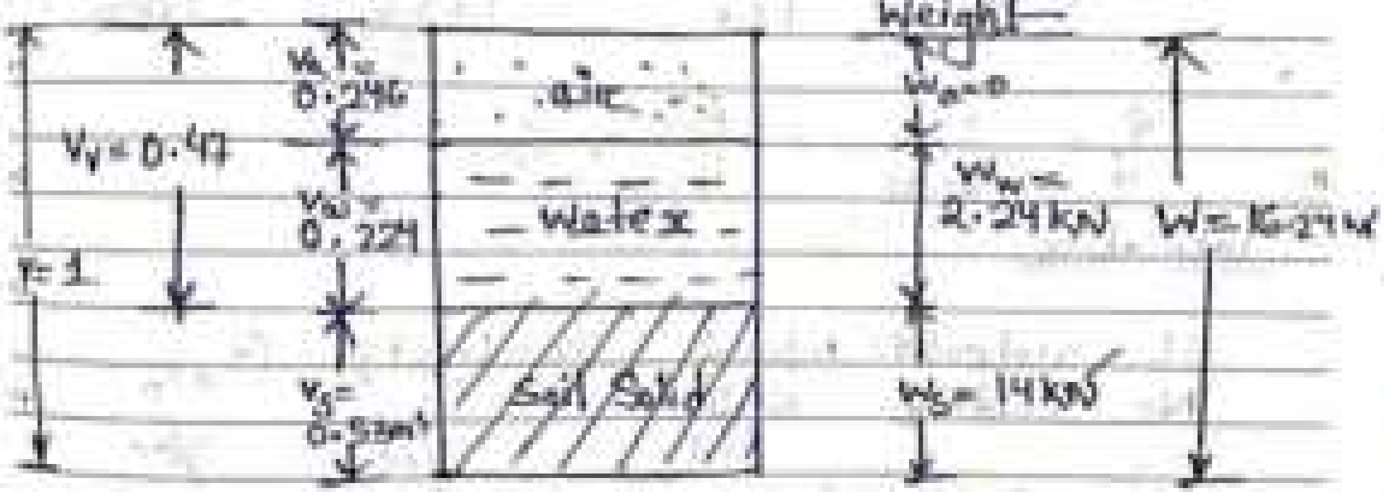
$= \frac{16\% \times 2.65}{0.89}$

$= 47.6\%$ (Ans)

porosity $\eta = \frac{e}{1+e}$

$= \frac{0.89}{1+0.89}$

$= 0.47$ (Ans)



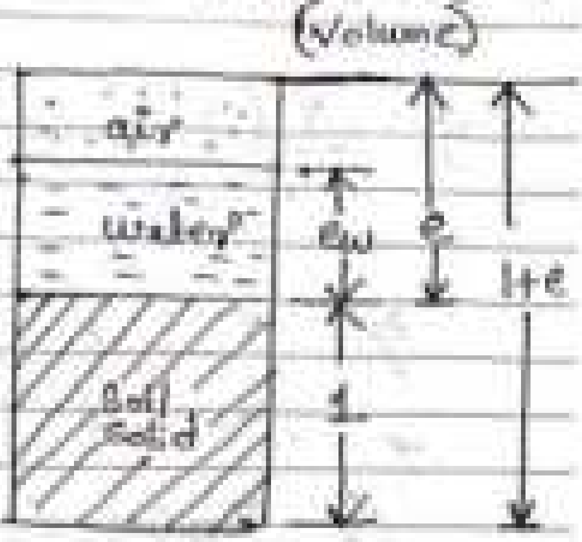
Q) The following observations were made in a standard proctor test on a soil.

Mass of wet soil (kg)	1.7	1.89	2.03	1.99	1.95	1.92
Water content (%)	7.7	11.5	14.6	17.5	19.7	21.2

Volume of the mould = 945cc, specific gravity of soil G = 2.65. Determine maximum dry density & optimum moisture content.

Q) Prove that :

$$S = \frac{W}{\frac{\gamma_w}{\gamma}(1+w) - \frac{1}{G}}$$



From phase diagram :

$$\gamma = \frac{W}{V}$$

$$\Rightarrow \gamma = \frac{\gamma_s \cdot 1 + \gamma_w \cdot e_w}{1+e}$$

$$\Rightarrow \gamma = \frac{G\gamma_w + \gamma_w(Se)}{1+e}$$

$$\Rightarrow \gamma(1+e) = G\gamma_w + \gamma_w Se$$

$$\Rightarrow \frac{\gamma(1+e) - G\gamma_w}{\gamma_w \cdot e} = S$$

$$\Rightarrow \frac{\gamma(1+e)}{\gamma_w \cdot e} - \frac{G\gamma_w}{\gamma_w \cdot e} = S$$

$$\Rightarrow \frac{\gamma(1+e)}{\gamma_w \cdot e} = \frac{G}{e} + S$$

$$\Rightarrow \frac{r(1+e)}{r_w \cdot e} = \frac{G}{e} + S$$

$$\Rightarrow \frac{r}{r_w}\frac{(1+e)}{\cancel{e}} = \frac{G+Se}{\cancel{e}}$$

$$\Rightarrow \frac{r(1+e)}{r_w} = G + wG$$

$$\Rightarrow \frac{r(1+e)}{r_w} = G(1+w)$$

$$\Rightarrow \frac{r}{r_w}(1+e) = G(1+w)$$

$$\Rightarrow \frac{r_w}{r(1+e)} = \frac{1}{G(1+w)}$$

$$\Rightarrow \frac{r_w}{r}\frac{(1+w)}{1+e} = \frac{1}{G}$$

$$\Rightarrow \frac{r_w}{r}(1+w) = \frac{1+e}{G}$$

$$\Rightarrow \frac{\gamma_w}{\gamma}(1+w) = \frac{1}{G} + \frac{e}{G}$$

$$= \frac{1}{G} + \frac{wG/S}{G}$$

$$\Rightarrow \frac{\gamma_w}{\gamma}(1+w) = \frac{1}{G} + \frac{w}{S}$$

$$\Rightarrow \frac{\gamma_w}{\gamma}(1+w) - \frac{1}{G} = \frac{w}{S}$$

$$\Rightarrow S = \frac{w}{\frac{\gamma_w}{\gamma}(1+w) - \frac{1}{G}}$$

(Proved)

Q) A cylinder of soil fails under an axial stress of $70\,kN/m^2$. The failure plane makes an angle of $52°$ with the horizontal. Calculate the value of cohesion & angle of internal friction of the soil.

Vertical stress $\sigma_1 = 70\,kN/m^2$

Angle of failure plane $\alpha = 52°$

$c = ?$ $\quad \phi = ?$

$$\boxed{\sigma_1 = 2 \cdot c \cdot \tan}$$

$\Rightarrow 70 = 2 \cdot c \cdot \tan 52^\circ$

$\Rightarrow c = 27.34 \text{ kN/m}^2$ (Ans)

$$\boxed{\alpha = 45^\circ + \frac{\phi}{2}}$$

$\Rightarrow 52^\circ = 45^\circ + \frac{\phi}{2}$

$\Rightarrow \phi = 14^\circ$ (Ans)

Q) Calculate the group Index of the soil hav
% passing 75μ IS sieve - 50%
$w_L = 30\%$ $w_p = 12\%$ $I_p = 30 - 12 = 18$

HRB Soil Classification

Group Index | $G.I. = 0.2a + 0.005ac + 0.01bd$

$a = \%$ passing $[35\% - 75\%]$ $(0-40)$
$b = \%$ passing $[15\% - 55\%]$ $(0-40)$
$c = w_L [40 - 60] = (0 - 20)$
$d = I_p [10 - 30] = (0 - 20)$

$a = 50\% - w_L = 50 - 30 = 20$
$b = 50\% - w_p = 50 - 12 = 38$
$c = 0$ $\because w_L \ngtr 40\%$
$d = 18 - 10 = 8$

$$\begin{cases} a = 50-35 = 15 \\ b = 50-15 = 35 \\ c = 0 \\ d = 18-10 = 8 \end{cases}$$

$G.I = 0.2 \times 15 + 0.005 \times 15 \times 0 + 0.01 \times 35 \times 8$

$= 3 + 0 + 2.8$

$= 5.8$ (Ans) A-4

Q) Determine the flow index from the following test data

No. of blows	Water Content
$n_1 = 38$	$w_1 = 16\%$
$n_2 = 20$	$w_2 = 20\%$

Flow Index $I_f = \dfrac{w_1 - w_2}{\log_{10}\left(\frac{n_1}{n_2}\right)}$

$= \dfrac{20-16}{\log_{10}\left(\frac{38}{20}\right)}$

$= 14.39$

Q) A compacted soil sample with a bulk unit weight of 19.62 kN/m³ has a water content of 20%. Find γ_d & S. Assume G = 2.65

$\gamma = 19.62$ kN/m³ $\quad w = 20\%$

$G = 2.65$

$$\gamma_d = \frac{\gamma}{1+w} = \frac{19.62}{1+0.20} = 16.35 \text{ kN/m}^3$$

$$\gamma_d = \frac{G\gamma_w}{1+e}$$

$$\Rightarrow 16.35 = \frac{2.65 \times 9.81}{1+e}$$

$$\Rightarrow e = 0.59$$

$$S = \frac{wG}{e}$$

$$= \frac{0.2 \times 2.65}{0.59}$$

$$= 89\%$$

Air content $a_c = 1 - S$

$= 11\%$

(3) A falling head permeameter accomodates a soil sample of 12cm high & 60cm² in c/s area. The permeability of the sample is expected to be 2×10^{-4} cm/sec. If it is desired that the head in the stand pipe to fall from 30cm by 12cm in 30minutes, determine the size of stand pipe.

$L = 12cm$
$A = 60cm^2$
$k = 2\times10^{-4}$ cm/sec
$h_1 = 30$
$h_2 = 12$
$t = 30minute = 30\times60 = 1800sec$
$a = ?$

$$k = \frac{aL}{At}\ln\left(\frac{h_1}{h_2}\right)$$

$$\Rightarrow 2\times10^{-4} = \frac{a\times12}{60\times1800}\ln\left(\frac{30}{12}\right)$$

$$\Rightarrow a = 1.96\ cm^2$$

(4) A clay layer 4m thick is sandwiched between layers of sand. Calculate the time the clays layer will take to reach 50% consolidation. The coefficient of consolidation is 3×10^{-3} cm²/sec.

$C_v = 3 \times 10^{-4}\ cm^2/sec$

$H = 4m \rightarrow$ double drainage $d = \frac{H}{2} = \frac{4}{2} = 2$

$U = 50\%$ $\qquad = 200\,cm$

$t = ?$

$T_v = \frac{\pi}{4}(U\%)^2$

$= 0.196$

$C_v = T_v \frac{d^2}{t}$

$\Rightarrow 3 \times 10^{-4} = 0.196 \times \frac{200^2}{t}$

$\Rightarrow t = 26133333.33\ sec$

$= 302.5$ days

Q) Determine the depth at which a circular footing of 1.5m diameter be founded to provide a FOS of 3.0 if it has to carry an ultimate load of 3500 kN. Assume the following data

$\phi = 30°,\ c = 10\,kN/m^2,\ \gamma = 18\,kN/m^3$

$N_c = 37.2 \quad N_q = 22.5, \quad N_\gamma = 19.7$

The terzaghi's bearing capacity eq^n for circular footing is

$$q_{nf} = 1.3cN_c + \gamma D(N_q - 1) + 0.3\gamma BN_\gamma$$

$$= 1.3 \times 10 \times 37.2 + 18 \times D(22.5-1) + 0.3 \times 18 \times 1.8 \times 19.7$$

$$= 483.6 + 387D + 191.484$$

$$= 675 + 387D.$$

$$q_s = \frac{q_{nu}}{F.O.S} + \gamma D.$$

$$\Rightarrow \frac{3600}{\frac{\pi}{4} \times 1.8^2} = \frac{675 + 387D}{2.5} + 18D$$

$$\Rightarrow = 241 + 158D + 18D$$

$$\Rightarrow 387 = 156D$$

$$\Rightarrow D = 2.5m \text{ (Ans)}$$

9.2 Bearing Capacity of soil, Terzaghi's Formula & I.S. code formula for strip, circular & square footing

- Bearing capacity of soil is the supporting power of a soil or rock.

Terzaghi's Formula — I.S. Code Formula.

c-φ soil

Strip footing $q_f = cN_c + \bar{\sigma} N_q + 0.5 \gamma B N_\gamma$

Circular $q_f = 1.3cN_c + \bar{\sigma} N_q + 0.3 \gamma B N_\gamma$

Square $q_f = 1.3cN_c + \bar{\sigma} N_q + 0.4 \gamma B N_\gamma$

$\bar{\sigma} = \gamma D_f$

cohesive soil ($\phi = 0$; $c > 0$)

circular : $q_f = 1.3cN_c + \bar{\sigma} = 7.4c + \bar{\sigma}$

Strip ~~Square~~ : $q_f = 5.7c + \bar{\sigma}$

Square $q_f = 1.3cN_c + \bar{\sigma}$

Non-cohesive Soils ($\phi > 0$, $c = 0$)

strip : $q_f = \bar{\sigma} N_q + 0.5 \gamma B N_\gamma$

circular : $q_f = \bar{\sigma} N_q + 0.3 \gamma B N_\gamma$

Square : $q_f = \bar{\sigma} N_q + 0.4 \gamma B N_\gamma$

N_c, N_q, N_γ from Terzaghi's Bearing capacity factors Table

$q_{nf} = q_f - \gamma D$; $q_s = \frac{q_{nf}}{F.O.S} + \gamma D$

I.S. code formula

(i) For general shear failure

$$q_{nf} = c N_c s_c d_c i_c + \bar{\sigma}(N_q - 1) s_q d_q i_q + \frac{1}{2} B \gamma N_\gamma s_\gamma d_\gamma i_\gamma$$

N_c, N_q, N_γ = Bearing capacity factor

s_c, s_q, s_γ = shape factor

Strip $s_c = s_q = s_\gamma = 1$

Square 1.3, 1.2, 0.8

Circle 1.3, 1.2, 0.6

i_c, i_q, i_γ = inclination factor

For vertical load $i_c = i_q = i_\gamma = 1$

d_c, d_q, d_γ = depth factor

$$d_c = 1 + 0.2 \frac{D}{B} \tan\left(45^\circ + \phi/2\right)$$

$$d_q = d_\gamma = 1 \quad \text{for } \phi < 10^\circ$$

or, $$d_q = d_\gamma = 1 + 0.1 \frac{D}{B} \tan\left(45^\circ + \frac{\phi}{2}\right) \quad \phi > 10^\circ$$

$$q_s = \frac{q_{nf}}{F.O.S} + \gamma D$$

(b) For Local shear failure :-

$$q_{nf} = \frac{2}{3} c N_c S_c d_c i_c + \bar{\sigma}(N_q - 1) S_q d_q i_q + \frac{1}{2} B \gamma N_\gamma S_\gamma d_\gamma i_\gamma$$

9.3 Machine foundation : (Workshop)

- Machine foundations are special types of foundations required for machines, machine tools and heavy equipments.
- These foundations are designed considering the shocks & vibrations (dynamic force) resulting from operation of machines.

Soil dynamics

- Soil dynamics deals with the engineering behaviour of soils subjected to time varying loads & loads applied very rapidly.
- In soil dynamics applied loads vary with time.
- The governing equations are thus of wave propagation / vibration theory principles.

Free Vibration

- When there no external excitations force on body or body itself vibrate without any external force, then the vibration in the body is called natural vibration/free vibration.
- This type of vibration occurs when a system is set off with an initial input and then allow to vibrate freely.

Forced vibration

- Vibrations that occur under the excitation of external forces
- Forced vibration occur at the frequency of the exciting force.
- Periodic input is required

Natural frequency

Frequency → The number of cycles of motion in a unit of time cycles/sec

Natural frequency → The frequency with which an elastic system vibrates under the action of forces inherent in the system.

Types of Machines

- There are several types of machines such as reciprocating machines, rotary machines
- Transmits time varying (dynamic) loading to the underlying soil

Ex: (i) Reciprocating ~~machine~~ pumps & rotary machines transmit sinusoidal loading

(ii) Punch presses, shredders & forge hammers transmit impact type of loading to the underlying soil.

cut holes in material

Metal shredding

Types of Machine foundation

centrifugal
Impact
Reciprocating

- Dynamic forces from these machines produce vibrations in the foundations & make it uncomfortable for the people working around them.

- If the vibrations are excessive, they can cause damage to the connecting piping or even the machine itself.

- Consulting Geotechnical Engineers should make sure that the vibrations of the designed foundation are within the prescribed limits as stipulated by the manufacturers of the machine.

Q) A square footing 3m × 3m is built up in homogeneous bed of sand of unit wt 22kN/m³ & having an angle of shearing resistance of 30°. The depth of the base of the footing is 1.8m below the ground surface. Calculate the safe load that can be carried by the footing with FOS of 3 against complete shear failure. Use Terzaghi's Analysis. Take $N_c = 65$, $N_q = 50$ & $N_\gamma = 55$

Square footing : $B_f = 3m$, $D_f = 1.8$

$\gamma = 22 kN/m^3$, sand

$\phi = 30°$

$F.O.S = 3$

Terzaghi's Analysis :-

$$q_f = 1.3 \cdot c N_c + \bar{\sigma} N_q + 0.4 \gamma B N_\gamma$$

$$= 0 + \gamma . D_f N_q + 0.4 \gamma B_f N_\gamma$$

$$= 22 \times 1.8 \times 50 + 0.4 \times 22 \times 3 \times$$

$$= 3432 \ kN/m^2$$

$q_{nf} = q_f - \gamma D_f = 3392.4 \ kN/m^2$

$$q_s = \frac{q_f}{FOS} + \gamma D_f$$

$$= \frac{3432}{3} + 22 \times 1.8$$

Safe load = 1183.6×3^2
= 10.6 kN

Q) A soil sample of 6 cm height & 100 cm² c/s area was subjected to the falling head permeability test. In a time interval of 10 min, the head dropped from 70 cm to 20 cm. If c/s area of stand pipe is 2 cm². Compute the coefficient of permeability of soil sample.

$L = 6cm$
$A = 100 cm^2$
$t = 10 min$
$h_1 = 70m, \; h_2 = 20cm$
$a = 2cm^2$

$$K = \frac{aL}{At} \ln\left(\frac{h_1}{h_2}\right)$$

$$= \frac{2 \times 6}{100 \times 10 \times 60} \ln\left(\frac{70}{20}\right)$$

$$= 2.5 \times 10^{-4} \; cm/sec$$

Q) Calculate the intensities of active & passive earth pr. at a depth of 10m in dry cohesionless sand with an angle of internal friction of 28° & unit wt of $18\ kN/m^3$.

$H = 10m$

$\phi = 28°$

$\gamma = 18\ kN/m^3$

$$K_a = \frac{1 - \sin 28°}{1 + \sin 28°} = 0.36$$

$$K_p = \frac{1 + \sin 28°}{1 - \sin 28°} = 2.77$$

Intensity of pressure

$$p_a = K_a \gamma H$$

$$= 0.36 \times 18 \times 10$$

$$= 64.8\ kN/m^2$$

$$p_p = K_p \gamma H$$

$$= 2.77 \times 18 \times 10$$

$$= 498.6\ kN/m^2.$$

Q) A moist soil has a mass of 633gm & a volume of 300cc at a water content of 11%. Assuming G = 2.68 determine e, S, n. Also water content at which soil gets fully saturated w/o any increase in volume

$M = 633gm \qquad V = 300\ cc$
$w = 11\% \qquad G = 2.68$

$\rho_d = \dfrac{\rho}{1+w} \qquad \rho = \dfrac{M}{V} = \dfrac{633}{300} = 2.11 g/cc$

$= \dfrac{2.11}{1+0.11}$

$= \dfrac{2.11}{1.11}$

$= 1.9\ g/cc$

$\rho_d = \dfrac{G\rho_w}{1+e}$

$\Rightarrow 1.9 = \dfrac{2.68 \times 1}{1+e}$

$\Rightarrow e = \dfrac{2.68}{1.9} - 1$

$\Rightarrow e = 0.41$

$$\eta = \frac{e}{1+e}$$

$$= \frac{0.41}{1+0.41}$$

$$= 0.29$$

$$S = \frac{wG}{e}$$

$$= \frac{0.11 \times 2.68}{0.41}$$

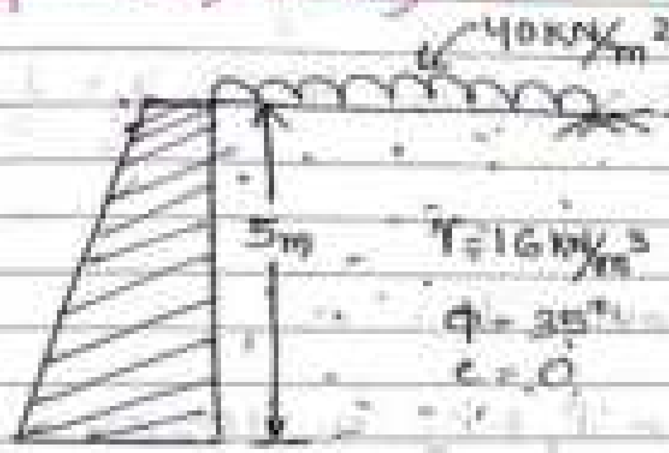
$$= 71.9\%$$

$$S_{100\%} = \frac{w\,G}{e}$$

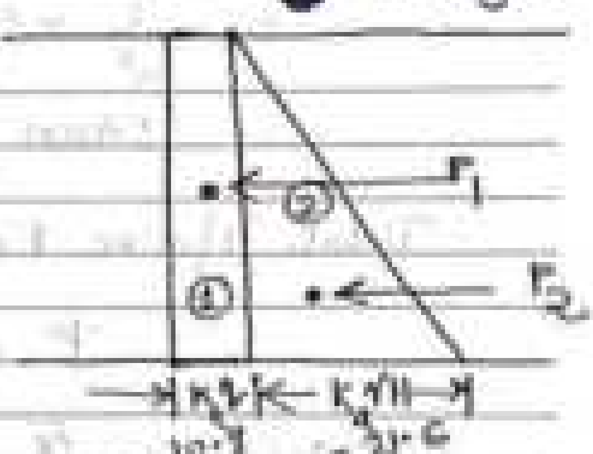
$$\Rightarrow 1 = \frac{w \times 2.68}{0.41}$$

$$\Rightarrow w = 15.3\%$$

Q) A retaining wall 5m high has a smooth vertical back. There is uniformly distributed surcharge load of $40 kN/m^2$ over the backfill. Determine the magnitude & point of application of active pressure per mtr. length of wall (Take unit wt. of backfill $= 16 kN/m^3$, $\phi = 35°$, $c = 0$)



Pressure diagram



$p_1 = K_a q$

$= 0.27 \times 40$

$= 10.8\ kN/m^2$

$K_a = \dfrac{1 - \sin\phi}{1 + \sin\phi}$

$= \dfrac{1 - \sin 35°}{1 + \sin 35°}$

$= 0.27$

$p_2 = K_a \gamma H$

$= 0.27 \times 16 \times 5$

$= 21.6\ kN/m^2$

P_1 = Area ① of pr diagram

$= 10.8 \times 5$ @ CG $x_1 = \frac{5}{2} = 2.5$

$= 54\,kN$

P_2 = Area ② of pr. diagram

$= \frac{1}{2} \times 5 \times 21.6$ @ CG $x_2 = \frac{1}{3} \times 5 = 1$

$= 54\,kN$

Total Active Earth Pr.

$P = P_1 + P_2 = 108\,kN$

C.G $\bar{x} = \dfrac{P_1 x_1 + P_2 x_2}{P_1 + P_2}$

$= \dfrac{54 \times 2.5 + 54 \times 1.67}{54 + 54}$

$= 2.1\,m$

Q) An undisturbed sample of clay, 2.5cm thick consolidated 50% in 30min when tested in the laboratory with drainage allowed at top & bottom. The clay layer from which the sample was taken is 350cm thick in the field. How much time will it take to consolidate 50% with double drainage? If the clay stratum has only single drainage, calculate the time to consolidate 50%.

Lab

$H = 2.5cm$

$U = 50\%$. $t = 30min$

Double drainage $d = \frac{H}{2} = \frac{2.5}{2} = 1.25cm$

Field

$H = 350cm$

$t = ?$

$U = 50\%$.

t — Double drainage = ? $d_1 = \frac{H}{2} = \frac{350}{2} = 175cm$

t — Single drainage = ? $d_2 = H = 350cm$

$C_v = \frac{T_v d^2}{t}$ (T_v same)

$\boxed{C_v \propto \frac{d^2}{t}}$ (C_v same)

$\Rightarrow t \propto d^2$

$t \propto d^2$

$\Rightarrow \dfrac{t}{t_1} = \dfrac{d^2}{d_1^2}$ and $\dfrac{t}{t_2} = \dfrac{d^2}{d_2^2}$

$\Rightarrow \dfrac{30\text{min}}{t_1} = \dfrac{1.25^2}{175^2}$ | $\Rightarrow \dfrac{30}{t_2} = \dfrac{1.25^2}{350^2}$

$\Rightarrow t_1 = \dfrac{30 \times 175^2}{1.25^2}\text{min}$ | $\Rightarrow t_2 = \dfrac{30 \times 350^2}{1.25^2}$

$= 588000\text{ min}$ | $= 2352000\text{ min}$

$= 1.13$ Year | $= 4.5$ year

Q. A partially saturated sample of soil has a density of 1950 kg/m³ & a water content of 21%. If the specific gravity of solids is 2.7, calculate the degree of saturation & void ratio. If the sample subsequently gets saturated determine its saturated density.

$\rho = 1950 \ kg/m^3$

$w = 21\%$

$G = 2.7$

$S = ? \quad e = ?$

$\rho_d = \frac{\rho}{1+w}$

$\Rightarrow \rho_d = \frac{1950}{1+0.21}$

$= 1611.57 \ kg/m^3$

$\rho_d = \frac{G\rho_w}{1+e}$

$\Rightarrow 1611.57 = \frac{2.7 \times 1000}{1+e}$

$\Rightarrow 1611.57 = \frac{2700}{1+e}$

$\Rightarrow e = \frac{2700}{1611.57} - 1$

$\Rightarrow e = 0.68$

$$S = \frac{wG}{e}$$

$$= \frac{0.21 \times 2.7}{0.68}$$

$$= 83\%$$

$$\rho_{sat} = \frac{(G+e)\rho_w}{1+e}$$

$$= \frac{(2.7+0.68)\,1000}{1+0.68}$$

$$= 2012 \text{ kN/m}^3$$

Q) The consistency limits of a soil sam are LL - 52% , PL - 32% , SL - 17%. If the specimen of this soil shrinks from a volume of 10cm^3 at liquid li to 6.01cm^3 at shrinkage limit, calculate the specific gravity of soil solids.

Volume

$W_L = 52\% \rightarrow 10\ cm^3$

$W_p = 32\%$

$W_s = 17\% \rightarrow 6.01\ cm^3$

$G = ?$

$$S.R. = \frac{\left(\frac{V_1 - V_d}{V_d}\right)}{W_1 - W_s} \times 100$$

$$= \frac{\left(\frac{10 - 6.01}{6.01}\right)}{52 - 17} \times 100$$

$$= 1.89$$

$$\boxed{S.R. = \frac{\gamma_d}{\gamma_w} = \frac{\rho_d}{\rho_w} = 1.89}$$

$$Sp.\ Gravity\ G = \frac{1}{\frac{\gamma_w}{\gamma_d} - \frac{W_s}{100}} \text{ or } \frac{1}{\frac{\rho_w}{\rho_d} - \frac{W_s}{100}}$$

$$= \frac{1}{\frac{1}{1.89} - \frac{17}{100}} = 2.78 \text{ (Ans)}$$

Student's Doubt

1) Relationship b/w γ, G, e, S

2) " " γ_d, G, e, S

3) SR, G, V.R Problem

4) Critical hydraulic gradient

5) Passive earth pr theory

6) Square footing R correction . Problem

7) air content

8) Zero air void line

9) Sp. gravity

10) G.I. Problem

11) Seepage pressure Problem

Q) The Atterberg limit of a clay soil are liquid limit 52%, plastic limit 30% & shrinkage limit 18%. If the specimen of the soil shrinkage from a volume of 39.5cm³ at liquid limit to a volume of 24.2cm³ at the shrinkage limit. Calculate the specific gravity?

L.L = 52%. $\quad w_L = 52\% \quad V_1 = 39.5cm^3$
P.L = 30%.
S.L = 18%. $\quad w_S = 18\% \quad V_2 = 24.2cm^3$

(a) Shrinkage Ratio

$$S.R = \frac{\frac{V_1 - V_2}{V_2}}{W_L - W_S} \times 100$$

$$= \frac{\frac{39.5 - 24.2}{24.2}}{52 - 18} \times 100$$

$$= \frac{0.63}{34} \times 100$$

$$= 1.85$$

(b) Volumetric Shrinkage

$$V.R = \frac{V_1 - V_2}{V_2} \times 100$$

$$= 63\%.$$

(3) Sp. Gravity $G = \dfrac{1}{\dfrac{\gamma_w}{\gamma_d} - \dfrac{w_s}{100}} = \dfrac{1}{\dfrac{\rho_w}{\rho_d} - \dfrac{w_s}{100}}$

$$\boxed{S.R = \frac{\gamma_d}{\gamma_w} = \frac{\rho_d}{\rho_w}}$$

$\Rightarrow 1.85 = \dfrac{\rho_d}{1 g/cc}$

$\Rightarrow \rho_d = 1.85 g/cc$

$$\Rightarrow G = \frac{1}{\dfrac{1}{1.85} - \dfrac{12}{100}}$$

$$= 2.77$$

Q) A square footing located at a depth of 1.3m below the ground has to carry a safe load of 800kN. Find the size of the footing if the desired factor of safety is 3. The soil has e = 0.55, S = 50%, G = 2.67. c = 8 kN/m², φ = 30°

Square footing : $B_f \times B_f \rightarrow$ width

$D_f \longrightarrow$ depth = 1.3m

Terzaghi's Analysis,

$$\boxed{q_{nf} = \frac{1}{3} c \cdot N_c + \bar{\sigma}(N_q - 1) R_{w_1} + 0.4 \gamma B N_\gamma R_{w_2}}$$

$c = 8\ kN/m^2$, $\bar{\sigma} = \gamma D_f$

$= 1.3 \times \gamma$

$= 1.3 \dfrac{(G + Se)\gamma_w}{1+e}$

$= 1.3 \times \dfrac{(2.67 + 0.5 \times 0.55)}{1 + 0.55} \times 9.81$

$= 1.3 \times 18.64\ kN/m^3$

$= 24.23\ kN/m^2$

$\gamma = 18.64\ kN/m^3$

$R_{w_1} = 1$, $R_{w_2} = 1$

For [redacted]°, $\phi = 30°$, Terzaghi's Bearing capa
factors from table.

$$\begin{cases} N_c = 37.2 \\ N_q = 22.5 \\ N_\gamma = 19.7 \end{cases}$$

$$q_{nf} = \frac{1}{3} \times 8 \times 37.2 + 24.23\,(22.5 - 1) \times$$
$$+ 0.4 \times 18.64 \times B_f \times 19.7 \times 1$$
$$= 99.2 + 520.95 + 146.88\,B_f$$

$$q_s\gamma = \frac{q_{nf}}{FOS} + \gamma D_f$$

$$= \frac{99.2 + 520.95 + 146.88B_f}{3} + 24$$

$$= 33 + 173.65 + 49B_f + 24.23$$

$$= 230.88 + 49B_f \quad \text{——} \; (1)$$

$$q = \frac{\text{Load}}{\text{Area}}$$

$$= \frac{800\,kN}{B_f \times B_f} = \frac{800}{B_f^2} \quad \text{—— (2)}$$

Equating (1) & (2)

$$\Rightarrow 230.88 + 49 B_f = \frac{800}{B_f^2}$$

$$\Rightarrow 230.88 B_f^2 + 49 B_f^3 = 800$$

$$\Rightarrow 230.88 B_f^2 + 49 B_f^3 - 800 = 0$$

$$\Rightarrow 49 B_f^3 + 230.88 B_f^2 - 800 = 0$$

$$B_f = 1.6\ m$$

Q) Laboratory test on soil sample obtained from a highway project site reveal that 56% of the soil passing 75μ sieve & the liquid & plastic limits of soil are 36% & 23% respectively. Determine the group index of soil.

% passed through 75μ sieve = 56%

$w_L = 36\%$, $w_p = 23\%$

$G.I = ?$ $I_p = 36 - 23 = 13$

$G.I = 0.2a + 0.005\,a.c + 0.01\,bd$

$a = 56\% - 35\%$ (fixed) $= 21\%$

$b = 56\% - 15\% = 41$

$c = 0$ ∵ $L.L = 36\% < 40\%$

$d = 13 - 10 = 3$

$= 0.2 \times 21 + 0.005 \times 21 \times 0 + 0.01 \times 41 \times 3$

$= 4.2 + 0 + 1.23$

$= 5.43$

Q) Calculate the co-efficient of permeability of a soil sample 6cm in height & 50cm² in c/s area, if a quantity of water equal to 450ml passed down in 10mins, under an effective constant head of 40cm on oven drying, the test specimen has mass of 498 kg. taking the $G_s = 2.65$. Calculate the seepage velocity of water during the test.

$K = ?$ $V_s = ?$

Soil Sample

$L = 6cm$

$A = 50cm^2$

$Q = 430ml$

$t = 10min$

$h = 40cm$

$V_s = \frac{V}{n}$

Oven dried Mass = 498kg . M_d

$G_s = 2.65$

$M_d = 498 kg = 498 \times 10^3 g$

$V = 6cm \times 50cm^2 = 300cm^3$

$$\boxed{\rho_d = \frac{M_d}{V}}$$

$$= \frac{498 \times 10^3 g}{300 cc}$$

$$= 1.6 \times 10^3 g/cc$$

$$\rho_d = \frac{G\rho_w}{1+e}$$

$$\Rightarrow 1.66 \times 10^3 = \frac{2.65 \times 1}{1+e}$$

$$\Rightarrow e = \frac{2.65}{1.66} - 1$$

$$\Rightarrow e = 0.6$$

porosity $n = \frac{e}{1+e}$

$$= \frac{0.6}{1+0.6}$$

$$= 0.38$$

Constant head Test

$$\boxed{k = \frac{QL}{Aht}}$$

$$= \frac{430\text{ml} \times 6\text{ cm}}{50_{\text{cm}} \times 40_{\text{cm}} \times 10_{\text{min}}}$$

$$= \frac{0.43\text{L} \times 6\text{cm}}{50 \times 40 \times 600\text{ cm}^3\text{-sec}}$$

$1 m^3 = 1000 L$
$\Rightarrow 10^6 cm^3 = 1000 L$
$\Rightarrow 1000 cm^3 = 1 L$

$$= \frac{0.43 \times 10^3 \; cm^3 \times 6\,cm}{12 \times 10^5 \; cm^2\text{-}sec}$$

$$= 2.15 \times 10^{-3} \; cm/sec$$

$$V = ki = k \cdot \frac{h}{L}$$

$$= 2.15 \times 10^{-3} \times \frac{40\,cm}{6\,cm}$$

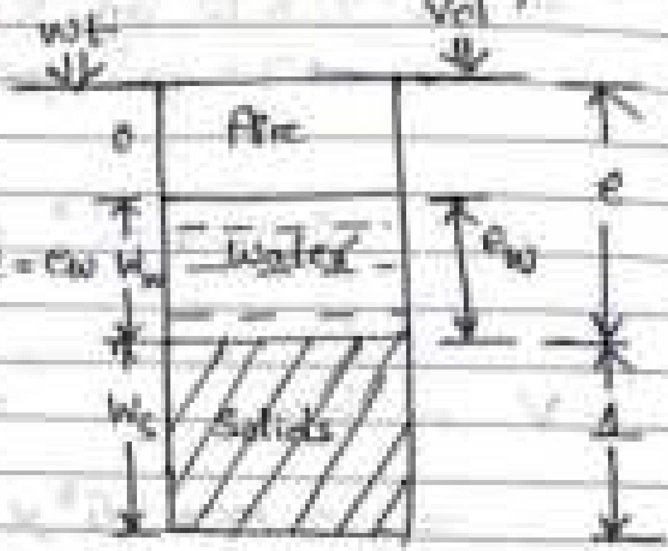
$$= 14.33 \times 10^{-3} \; cm/sec$$

$$V_s = \frac{V}{n}$$

$$= \frac{14.33 \times 10^{-3} \; cm/sec}{0.38}$$

$$= 37.72 \times 10^{-3} \; cm/sec$$

Q) Relationship b/w W, G, e, S or γ, G, e, S



Degree of saturation

$$S = \frac{V_w}{V_v} = \frac{e_w}{e}$$

$$\Rightarrow S \cdot e = e_w$$

Sp gravity

$$G = \frac{\gamma_s}{\gamma_w}$$

I: water content

$$w = \frac{W_w}{W_s}$$

$$= \frac{V_w \cdot \gamma_w}{V_s \cdot \gamma_s}$$

$$= \frac{e_w}{1} \times \frac{\gamma_w}{\gamma_s}$$

$$= e_w \times \frac{1}{G}$$

$$= \frac{e_w}{G}$$

$$\boxed{w = \frac{S \cdot e}{G}}$$ Proved

II. $\gamma = \frac{W}{V}$

$= \frac{W_w + W_s}{V}$

$= \frac{\gamma_w . V_w + \gamma_s . V_s}{1+e}$

$= \frac{\gamma_w . e_w + \gamma_s . 1}{1+e}$

$= \frac{\gamma_w \left(e_w + \frac{\gamma_s}{\gamma_w} \right)}{1+e}$

$= \frac{\gamma_w (Se + G)}{1+e}$

$\Rightarrow \gamma = \frac{(G + Se)\gamma_w}{1+e}$ (Proved)

Q) Critical hydraulic gradient (i_c)
This is the hydraulic gradient (h/L) at which the soil becomes unstable the effective stress becomes zero.

- Consider a soil in which the flow of wat[er] is upward, this will creat an upward seepage pr.

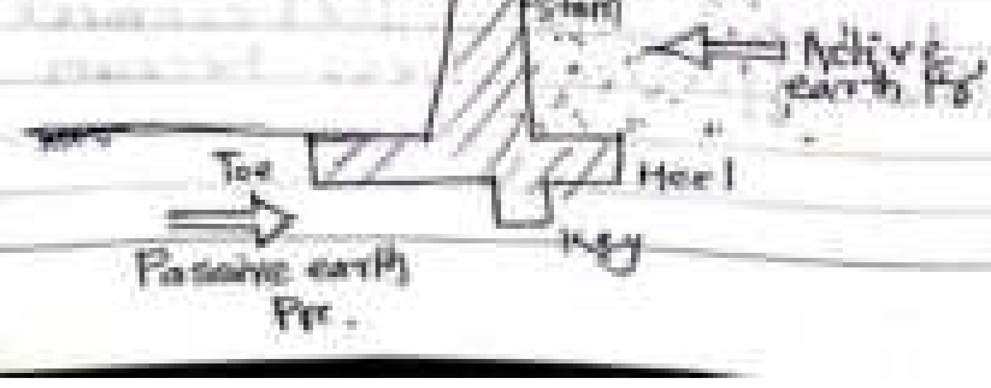
- If the upward flow of water is large enough the seepage pr. will negate th[e] effective stress & the soil will become unstable. In this situation the soil is said to be in 'quick' condition or 'sand boiling'

$$\boxed{i_c = \frac{G-1}{1+e}}$$

Q) Passive Earth Pressure :-

Earth Pressure :-

Active Earth Pr.	Passive Earth Pr.
i) wall moves away from backfill	i) wall moves towards backfi[ll]



Passive Earth Pr. $\boxed{P_p = \frac{1}{2} K_p \gamma H^2}$

$K_p = \frac{1+\sin\phi}{1-\sin\phi}$ coefficient of passive earth pr

γ = unit wt. of soil
H = height of retaining wall
ϕ = Angle of internal friction

Q) Define air content, specific gravity

air content $a_c = \frac{\text{Volm of air } V_a}{\text{Volm of voids } V_v}$

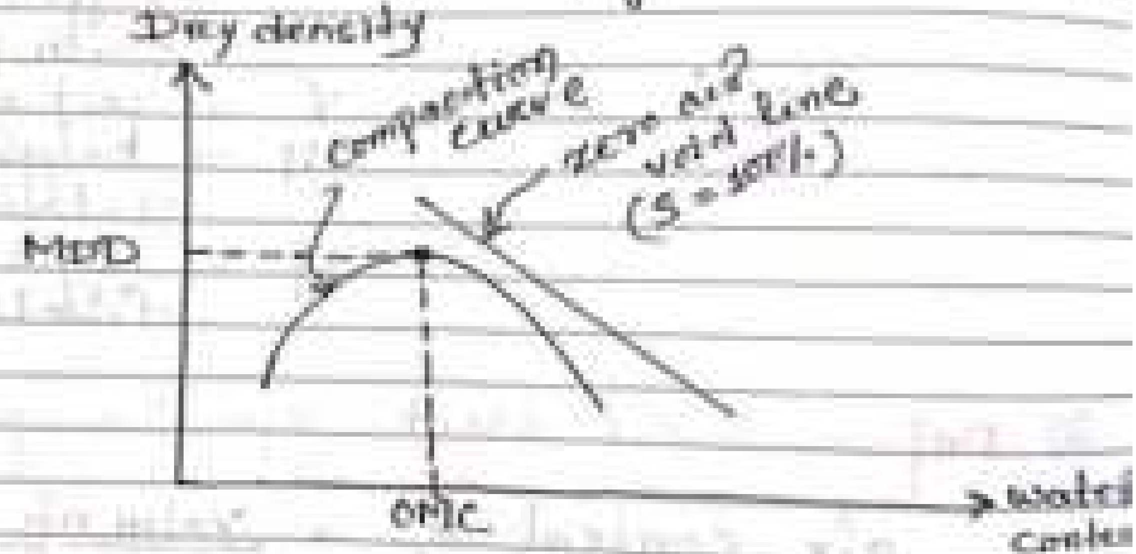
% air void $n_a = \frac{\text{volm of air}}{\text{total volm}} = \frac{V_a}{V}$

Sp. gravity G is the ratio of specific gravity of water. It can obtained by measuring the wt. of solid to the wt of water occupying equivalent volume of water

$G = \frac{\rho_s}{\rho_w}$

Q) Zero void line

The zero air void curve is a plot of the dry unit wt. against water content at 100% degree of saturation.



compact^n curve : $\gamma_d = \frac{G\gamma_w}{1+e} = \frac{G\gamma_w}{1+\frac{wG}{S}}$

zero void line : $\gamma_d = \frac{G\gamma_w}{1+\frac{wG}{1}} = \frac{G\gamma_w}{1+wG}$

Ch-9 SD-II

Design of Masonry structure -

- Masonry structures are those structures which are built from individual units laid & bound together by mortar
- Masonry is commonly used for walls.
- Brick & concrete block are the most commonly used material.
- Masonry has high compressive strength under vertical loads but has low tensile strength against twisting or stretching unless reinforced.

9.1 Design consideration for masonry wall

1. Load bearing wall
2. Non-Load bearing wall.

① A Load bearing wall is part of the structure of the building, used to support floors, ceiling, roof & other walls.

② Non-load bearing wall / Partition wall is used to divide rooms but doesn't hold anything up apart from its own weight.

(ii) You can remove a non-load bearing wall with no repercussions.
But a load bearing wall can be removed but you have to redistribute the load path.

- Partition wall is an interior non load bearing wall to divide the larger space into smaller spaces.

- These walls are made up of glass, fiber boards or brick masonry.

- Panel wall is generally made of wood & is an exterior non-load bearing wall in framed construction.

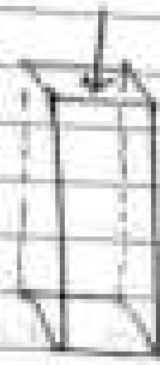
- It is used for aesthetics of the building both inside & outside.

- It remains totally supported at each storey but subjected to lateral load.

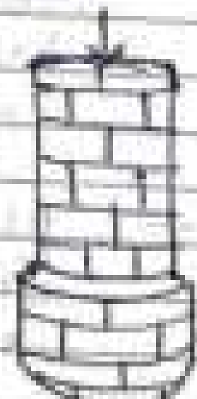
Curtain Wall is an outer covering of a building in which the outer walls are non-structural, but merely keep the weather out & the occupants in.
or
Skin of building

Ex: Glass curtain wall
- Transparent
- Light weight
- Wide spread usage.

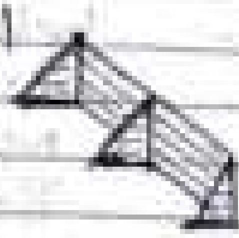
Masonry Column:
(brick + mortar)



Masonry Piers



Buttress wall



Masonry wall footing

Brick Masonry footing

Permissible Stresses Allowable stresses

Grade of Mortar

N O S M

Explain :—

1 : 1 : 6 N — above ground
cement lime sand
exterior, Interior
Load bearing
soft stone (masonry) home-owner

O — Above ground
Interior
Non-load bearing
repair work

S — below grade/ground
Masonry foundations
Manholes
Retaining walls
Sewers
Brick Patios & Pavements

M – heavy load
below ground
foundations
retaining wall
drive ways

Ch - 6
Design of Steel beams :—

- A beam is a structural member which takes the vertical loads by means of bending.

OR,

Beam is the member which is subjected to bending & designed to take bending moment.

- Classification by material
  - Steel beam
  - RCC beam
  - Wooden beam

- Classification by support conditions
  - Simply supported
  - Cantilever
  - Fixed end beam
  - Overhanging bm
  - Continuous beam

- Classification by cross section
  - I - beam
  - Rectangular beam
  - T beam

—

NE 6. e-Shodh Sindhu
→ No digital library

[MIC]

NE 1. Smart India Hackathon
→ No PG students

NE 2. Institute | Innovation council

3. Atal ranking

4. Drug Discovering hackathon.

" " NITTT ✓ Scheme AICTE; FD
5 yr Less

30 DECEMBER
SUNDAY

2018 NOVEMBER

M	T	W	T	F	S	S
			1	2	3	4
5	6	7	8	9	10	11
12	13	14	15	16	17	18
19	20	21	22	23	24	25
26	27	28	29	30		

NE 11. Sports Faculty Program
→ No such faculties designated as Sports officers are available

NE 12 Emeritus Professor.
No superannuated Professor

NE 13. ISTE Orientation/Refresher Prog.
→ No regular faculties with research Publication
→ ~~No self financing p~~

(B). Institution

NE 1. MODROB
→ 10 Yr. Existence of institution not there
→ 5yr Existence for rural areas

NE 2. CAFEB
→ No huge amount of SC/ST Students availa

NE 3. Idea lab
No brilliant innovation with Prof/ Students

NE 4. GOC
→ 8yrs existence of institution
→ No A.Pro/ Pro + 10yr Experien

NE 5. SPICES
→ club formation & with 1lakh (50 students)
→ 10 years faculty experience reqd

2018

NE 1. Short term Training Programme (UT - J&K, Ladak & NER) STTP
Shortfall - Location Criteria

NE 2. Research Promotion Scheme (RPS)

General Rural/NER Management ADF

→ No Research facilities in institute
→ Faculty with Research Experience & publications not available

NE 6. Recognition of New QIP Centre
→ No scheme document available on website

NE (4) Technical book writing & translation to Indian language. 1lakh funding (E) eligible

NE (8). Professional development scheme (Travel Grant)
Only applicable for PHD holder
NBA Accredited

NE 9. Distinguished Visitor Professor.
→ No professor in age of 40-75 yr. available
→ 10 yr. Exp. in Industry

NE 10. Technical University.
→ No partner university available
→ 10 yrs Institution criteria not satisfied
→ No adequate experience in research field.

NAME, ADDRESS & E-MAIL | PHONES | MOBL/FAX

Buckling on long column

- Circular
- Rectangular Column

longitudinal dimension

least lateral dimension

$\boxed{L > 20D}$ → cause buckling

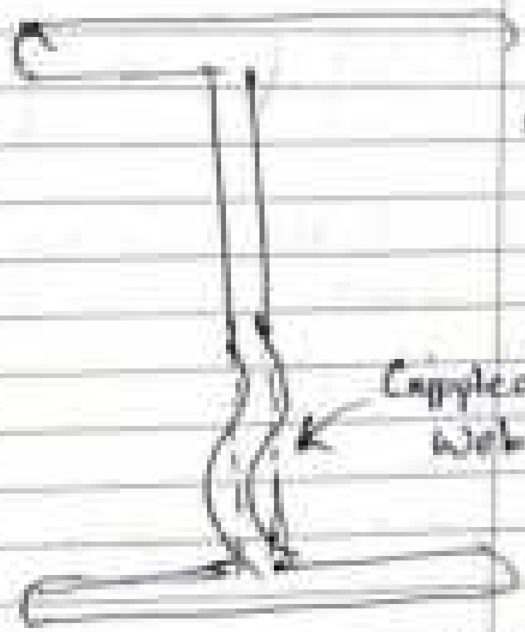
$\boxed{P > P_{cr}}$

P_{cr} - Euler formula

Radius of gyration

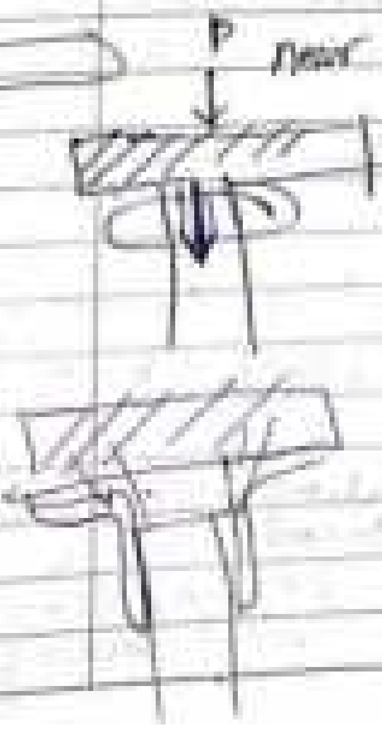
→ the distance from an axis at which the mass of a body may be assumed to be concentrated

$K = \sqrt{I/m}$ - I = mass moment of inertia, m = mass



Crippled web → Localized buckling

→ Stress concentration more than place crippled i.e. near support



σ ↑ → A ↓

stress concentration get increase so web crippling

NAME ADDRESS & E-MAIL | PHONES | MOB./FAX

Internet online UPS — "Server-2 method"

Data Speed — Kindly kindly ... Speed ... broadband

Note Sheet

Server

60 Antivirus → Get ... After

CAM CAD, STAAD Pro, Revit → Install

Lateral load

Load transverse load ... bending

Load along axis ⇒ Buckling

AC

Window blinds

Dept. Printer

Shoe stand

Drilling

2marks

Q1. Web buckling Vs web crippling —

Q2. Define radius of gyration

Q3. Butt joint adv. over lap joint

Q4. Pitch & staggered pitch

Q5. Partial safety factor in LSM design

Q6. Tubular steel section are suitable for ——

Q7. Define net section area of tension members

Q8. Types of Buckling in compression member

5marks

Q1. Assumption in design of bearing bolts

Q2. Explain block shear failure with sketch.

8marks

Q1. Crushing in tubular steel compression members & Design consideration for masonry wall footing