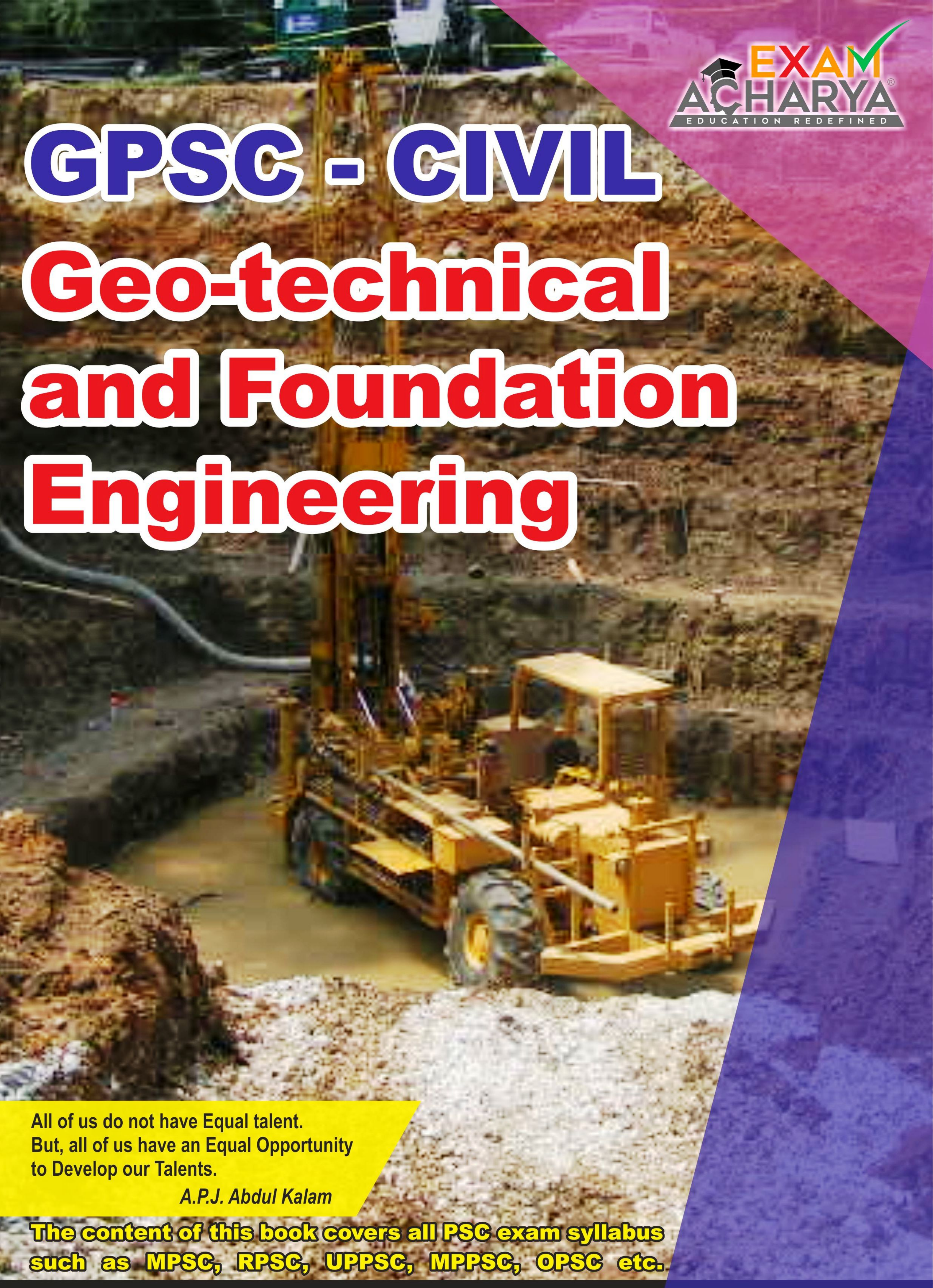


EXAM ACHARYA
EDUCATION REDEFINED

GPSC - CIVIL

Geo-technical and Foundation Engineering

All of us do not have Equal talent.
But, all of us have an Equal Opportunity
to Develop our Talents.

A.P.J. Abdul Kalam

The content of this book covers all PSC exam syllabus such as MPSC, RPSC, UPPSC, MPPSC, OPSC etc.

INDEX

CHAPTER – 1

ORIGIN OF SOIL AND SOIL WATER RELATIONSHIP

DEFINITION OF SOIL

The term 'soil' in soil engineering is defined as an unconsolidated material, composed of solid particles, produced by the disintegration of rocks. The void space between the particles may contain air, water or both. The soil particles may contain organic matter.

DEFINITION OF SOIL MECHANICS

- The term 'Soil Mechanics' was coined by Dr. Karl Terzaghi in 1925, who is also known as the Father of Soil Mechanics.
- According to Terzaghi, 'Soil mechanics is the application of the laws of mechanics and hydraulics to engineering problems dealing with sediments and other unconsolidated accumulations of solid particles produced by the mechanical and chemical disintegration of rock, regardless of whether or not they contain an admixture of organic constituents'.
- Soil mechanics is, therefore, a branch of mechanics which deals with the action of forces on soil and with the flow of water in soil.

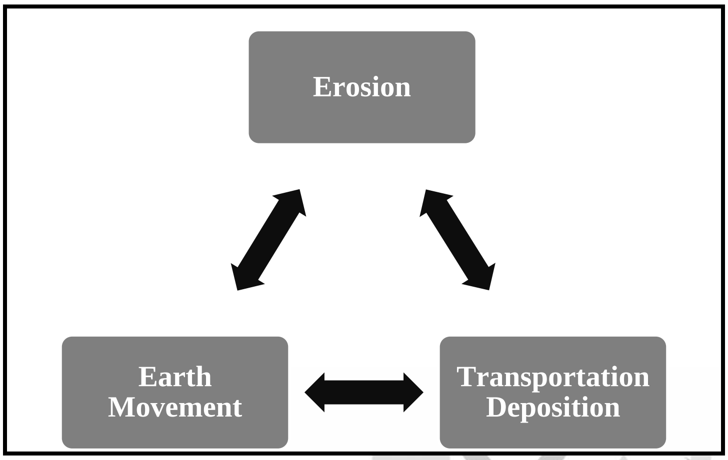
DEFINITION OF SOIL ENGINEERING

- Soil engineering is an applied science dealing with the applications of principles of soil mechanics to practical problems. It has a much wider scope than soil mechanics, as it deals with all engineering problems related with soils. It includes site investigations, design and construction of foundations, earth-retaining structures and earth structures.

DEFINITION OF GEOTECHNICAL ENGINEERING

- Geotechnical engineering is a broader term which includes soil engineering, rock mechanics and geology. Sometimes Geotechnical Engineering is used synonymously with Soil Engineering

ORIGIN OF SOIL



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- In a broad sense, soil may be thought of as an incidental material in vast geological cycle which has been going on continuously for millions of years of geological time.
- The geological cycle consists of 3 phases, Erosion Transportation and deposition & Earth Movement.

Erosion Phase

- The cycle starts with the erosional phase in which there is degradation of exposed rock by weathering processes.
- The weathering processes may be

    (i) Physical weathering

    (ii) Chemical weathering

- Most residual soils are weakly bonded, they have widely varying void ratio. They contain angular rock fragments of varying sizes. Residual soils have better engineering property.

Transportation/Deposition

- In the second phase, the fragmented material is transported by agents such as wind, water or ice to new locations.
- Soil transported from their origin by wind, water, ice or any other agency and has been deposited is called Transported soil. They have generally small grain sizes, large amount of pores.
- Characteristics of soil such as Size of particle, Shape and roundness. Surface texture and, Degree of shortening are influenced by the agency of transportation.
- According to the transporting agency soils are classified as:

  Alluvial deposit → deposited by river water

  Lacustrine deposit → deposited by still water like lakes.

  Marine deposit → deposited by sea water.

  Aeolian deposit → transported by wind

  Glacial deposit → Transported by ice.

- Air transported soil have small size $(20 - 50\mu)$ and they are homogeneous, highly porous and friable.
- Loess is an aeolian soil. They are formed in arid and semi arid regions.
- Alluvial soils are generally rounded and have considerable shorting.

According to Transporting Agency Soils Are Classified as:

<u>Water Transported Soils</u>

Flowing water is one of the most important agents in transportation of soils. Swift running water carries a large quantity of soil either in suspension or by rolling along the bed. Water erodes the hills and deposits the soils in the valleys.

Glacier-Deposited Soils

- Glaciers are large masses of ice formed by the compaction of snow. As the glaciers grow and move they carry with them soils varying in size from fine grained to huge boulders. Soils get mixed with the ice and are transported far away from their original position.
- Drift is a general term used for the deposits made by glaciers directly or indirectly.
- Deposits directly made by melting of glaciers are called till.
- Deposits of glacial till are generally well-graded and can be compacted to a high dry density. These have generally high shearing strength.

Gravity - Deposited Soils

- Soils can be transported through distances under the action of gravity. Rock fragments and soil masses collected at the foot of the cliffs or steep slopes had fallen from higher elevation under the action of the gravitational force. Colluvial soils, such as talus, have been deposited by the gravity. Talus consists of irregular, coarse particles.

Soils Transported by Combined Action

- Sometimes, two or more agents of transportation act jointly and transport the soil. For example, a soil particle may fall under gravity and may be carried by wind to a far off place. It might be picked up again by flowing water and deposited. A glacier may carry it still further.

SOME SPECIAL / TYPICAL SOILS

a. Loess: A loose deposit of windblown silt that has been weakly cemented with calcium carbonate and montmorillonite.

b. Bentonite: A chemically weathered volcanic ash.

c. Peat: A highly organic soil; fibrous and highly compressible.

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Construction, Planning and Management

"All Birds find shelter during a rain.
But Eagle avoids rain by flying above
the Clouds."

A.P.J. Abdul Kalam

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Desert Soils

- These are windblown deposits of sand.
- Dune sand is non plastic uniformly graded fine sand.
- Problems associated with these soils are of soil stabilization for roads and runways, reducing settlement under static and dynamic loads and reducing its perviousness to make it suitable for storage and transport of water.

SOIL WATER RELATIONSHIP

- A soil mass consists of solid particles which form a porous structure. The voids in the soil mass may be filled with air, with water or partly with air and partly with water. The three constituents are blended together to form a complex material. However, for convenience, all the solid particles are segregated and placed in the lower layers of the three-phase diagram. Likewise, water and air particles are placed separately, as shown. The 3-phase diagram is also known as Block diagram.
- It may be noted that the three constituents cannot be actually segregated, as shown. A 3 -phase diagram is an artifice used for easy understanding and convenience in calculation.
- Soil can be either two-phase or three-phase composition.
- Fully saturated soil and fully dry soil are two phase system.
- Partially saturated soil are three phase system.

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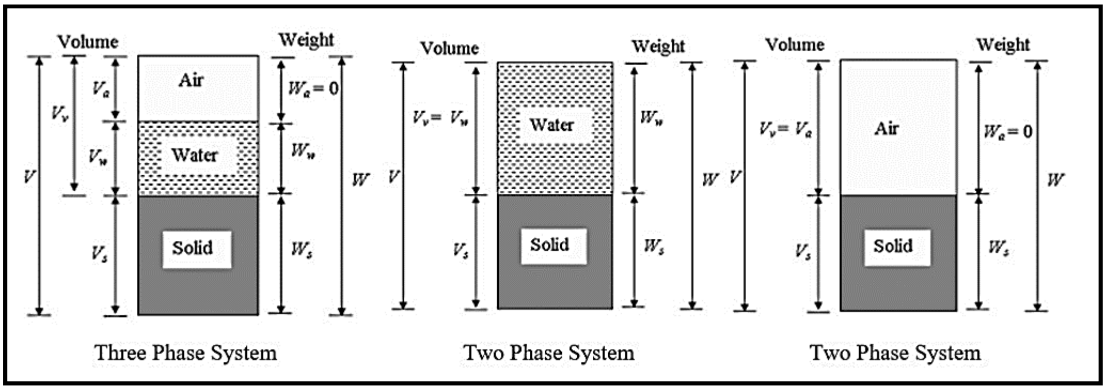

Three Phase System Two Phase System Two Phase System

2. Void Ratio(e)

$$e = \frac{v_v}{V_s}; e > 0$$

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- Void ratio is defined as ratio of volume of voids to the volume of solids.
- It is denoted by letter symbol (e) and generally expressed as a decimal fraction eg. 0.20, 0.45 etc.
- There is no upper limit of void ratio in soil suspension and in macro-porous soils like loess, V_v could be much greater than V_s
- Void ratio of fine grained soil are generally higher than those of coarse grained soil.
- Size of void in coarse grained soil are generally larger than that in fine grained soil

3. Porosity (n)

$$\mathbf{n} = \frac{V_{\mathbf{v}}}{V} \times \mathbf{100}\ ;\ \mathbf{100} > n > 0$$

- Porosity is defined as ratio of volume of voids to the total volume expressed as a percentage. It is also known as percentage voids.
- Porosity is denoted by letter symbol (n) and is commonly expressed as a percentage.

$$V_{\mathbf{v}} = V_{\mathbf{a}} + \mathbf{V_w}$$

$$\mathbf{V} = \mathbf{V_a} + \dot{\mathbf{V}}_{\mathbf{w}} + \mathbf{V_s}$$

- The porosity of soil can not exceed 100% hence it has a upper limit of 100%.
- Both porosity and void ratio are measure of denseness (or looseness) of soil.

- Air content of a soil mass is defined as a ratio of Volume of air voids to the total volume of voids. It is denoted by letter symbol a_c and commonly expressed as a percentage.

7. Bulk Unit Weight (γ_t)

$$\gamma_t = \frac{W}{V} = \frac{W_s + W_w}{V_s + V_w + V_a}$$

- Bulk unit weight of soil mass is defined as the weight per unit volume of soil mass.
- It is denoted by letteror γ or γ_t and is generally expressed as $\frac{kN}{m^3}, \frac{N}{m^3}, \frac{kgf}{cm^3}$

> **Note**
> Density (γ) is mass per unit volume.

8. Unit weight of Solids (γ_S)

$$\gamma_S = \frac{W_s}{V_s}$$

- Unit weight of solids is the weight of soil solids per unit volume of solids alone. It is also called as “absolute unit weight” of a soil.
- It is denoted by letter γ_s

9. Unit- Weight of Water (γ_w)

$$\gamma_w = \frac{W_w}{V_w}$$

- Unit weight of water is the weight per unit volume of water.
- It is denoted by letter symbol γ_w.

> Note
> - The value of γ_w changes with temperature but usually we take $\gamma_w = 9.81 kN/m^3$ which is at 4°C.

Note
- Soil in Submerged condition will be in saturated state but soil in saturated condition need not to be submerged. For example, soil mass below water table is in submerged as well as saturated condition whereas soil mass in capillary zone is in saturated condition only.

13. Specific Gravity of Solids (G)

$$G = \frac{\gamma_s}{\gamma_w}$$

- The specific gravity of solids is defined as the ratio of the unit weight of solids (absolute unit weight of soil) to the unit weight of water. It is denoted by letter G and is a Unit less quantity.
- This is also known as "Absolute specific gravity" or "Grain specific gravity"

14. Mass Specific gravity of soil(G_m)

$$G_m = \frac{\gamma_t}{\gamma_w}$$

- Mass specific gravity is defined as the ratio of bulk unit weight of soil to unit weight of water.
- It is denoted by letter symbol G_m and is a unit less quantity.

15. Relative Density (D_r)

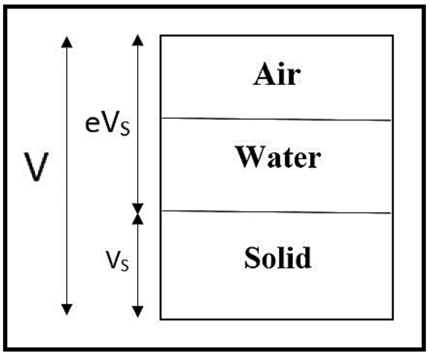
$$(D_r) = \frac{e_{\max} - e}{e_{\max} - e_{\min}}$$

- The relative density is a parameter used in sandy and gravelly soils.
- The value of $e_{\min}$ & $e_{\max}$ represents the soil in very dense and loose conditions, respectively and are determined by a standard laboratory test.
- Loose soil have low values of D_r, While dense soils have high values.
- The theoretically lowest possible value of D_r is 0% and highest theoretical possible value is 100%. Thus D_r, is often more useful than void ratio (e) because

2. $V = \frac{V}{1+e}$

$\Rightarrow V_S(1+e) = V$

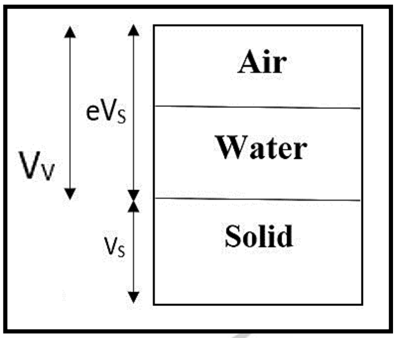
$$\Rightarrow \boldsymbol{V_S = \frac{V}{1+e}}$$



3. $n = \frac{e}{1+e}$

$$\Rightarrow n = \frac{V_v}{V} = \frac{eV_s}{V_S + V_v} = \frac{eV_S}{V_S + eV_S} = \frac{e}{1+e}$$

$$\Rightarrow \boldsymbol{n = \frac{e}{1+e}}$$



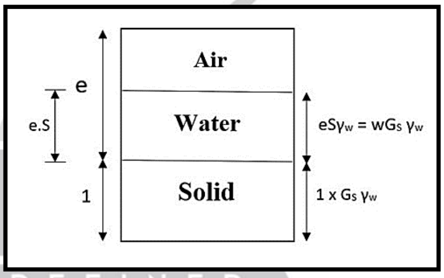
4. $eS = wG_s$

$$e = \frac{V_v}{V_s} = \frac{V_v}{V_w} \times \frac{V_w}{V_s} = \frac{1}{S} \times \frac{\frac{Ww}{\gamma_s}}{\frac{W_S}{\gamma_S}}$$

$$e = \frac{1}{S} \times \frac{W_w}{W_s} \times \frac{\gamma_s}{\gamma_w} = \frac{1}{S} \times wG_S$$

$\Rightarrow eS = wG_s$

$\Rightarrow$**eS = wG**

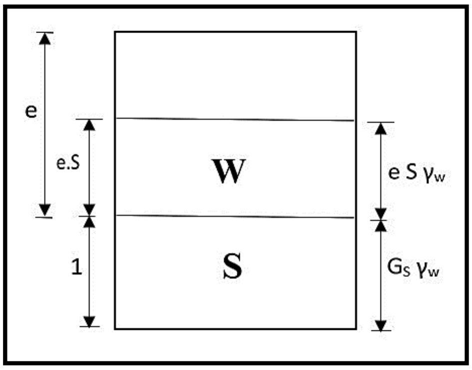


5. $\gamma_t = \frac{G_S + Se}{1+e}\gamma_w$

$$\gamma_t = \frac{W}{V} = \frac{W_S + W_w}{V_S + V_V} = \frac{W_S\left(1 + \frac{W_w}{W_S}\right)}{V_S\left(1 + \frac{V_V}{V_S}\right)}$$

$$= \frac{W_S(1+w)}{V_S(1+e)}$$

$$\gamma_t = \frac{G_S\gamma_w(1+w)}{1+e} = \frac{G_S + Se}{1+e} \cdot \gamma_w$$



8. $\gamma' = \frac{G_S-1}{1+e}\gamma_w$

We know that γ' = submerged unit wt = $\gamma_{sat} - \gamma_w$

$$= \frac{G_S + e}{1+e}\gamma_w - \gamma_w = \frac{G_s - 1}{1+e}\gamma_w$$

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

9. $\gamma_d = \frac{\gamma_t}{1+w}$

$$\gamma_t = \frac{W}{V} = \frac{W_s+W_w}{V} = \frac{W_s(1+w)}{V} = \gamma_d\,(1+w)$$

$$\Rightarrow \boldsymbol{\gamma_d} = \frac{\boldsymbol{\gamma_t}}{\mathbf{1}+\boldsymbol{w}}$$

10. $\gamma_d = \frac{\gamma_w G_S}{1+\frac{WG_S}{S}}$

We know that $\gamma_d = \frac{G_S\gamma_w}{1+e} = \frac{\boldsymbol{G_s\gamma_w}}{\mathbf{1}+\frac{\boldsymbol{wG_s}}{\boldsymbol{S}}}$

11. $1 - n_a = 1 - \frac{V_a}{V} = \frac{V_w+V_s}{V} = \frac{w_w}{\gamma_w V} + \frac{w_s}{G_s\gamma_w V}$

$$= \frac{\gamma_d}{G_S\gamma_w} + \frac{wW_S}{\gamma_w V} = \frac{\gamma_d}{\gamma_w}\left(w + \frac{1}{G_S}\right)$$

$$\Rightarrow \boldsymbol{\gamma_d} = \frac{(\mathbf{1}-\boldsymbol{n_a})\boldsymbol{G_s\gamma_w}}{\mathbf{1}+\boldsymbol{wG_s}}$$

WATER CONTENT DETERMINATION

- Water content of soil is an important soil parameter which significantly influences the behaviour of soil, particularly of cohesive soils.
- It is important to quantify the state of soil immediately after it is received in the testing laboratory and just prior to commencing any other tests (example, shear strength test, compression test etc.)

$$w = \frac{W_2 - W_3}{W_3 - W_1} \times 100\%$$

Where,

$W_1 =$ Weight of container
$W_2 =$ Weight of container + moist sample
$W_3 =$ Weight of container + dried sample
Weight of water $= W_2 - W_3$
Weight of solids $= W_3 - W_1$

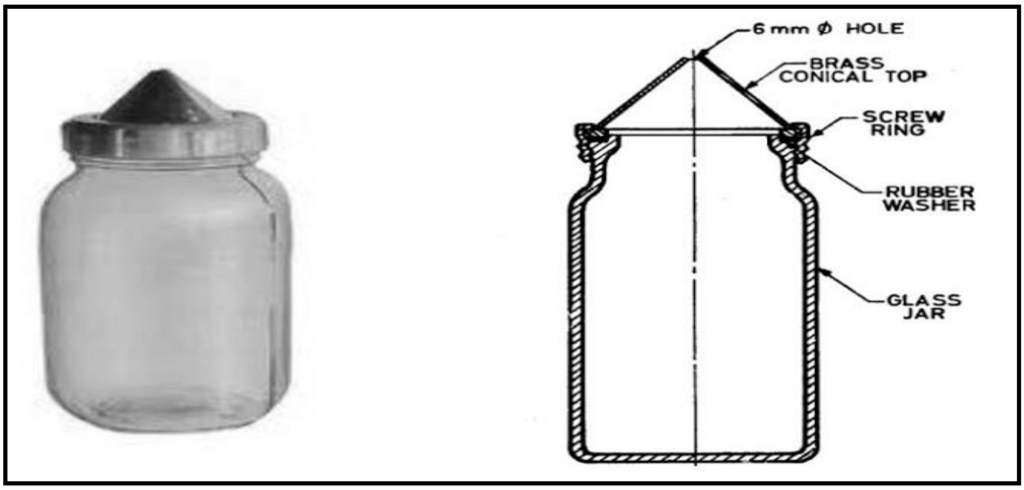
Pycnometer Method

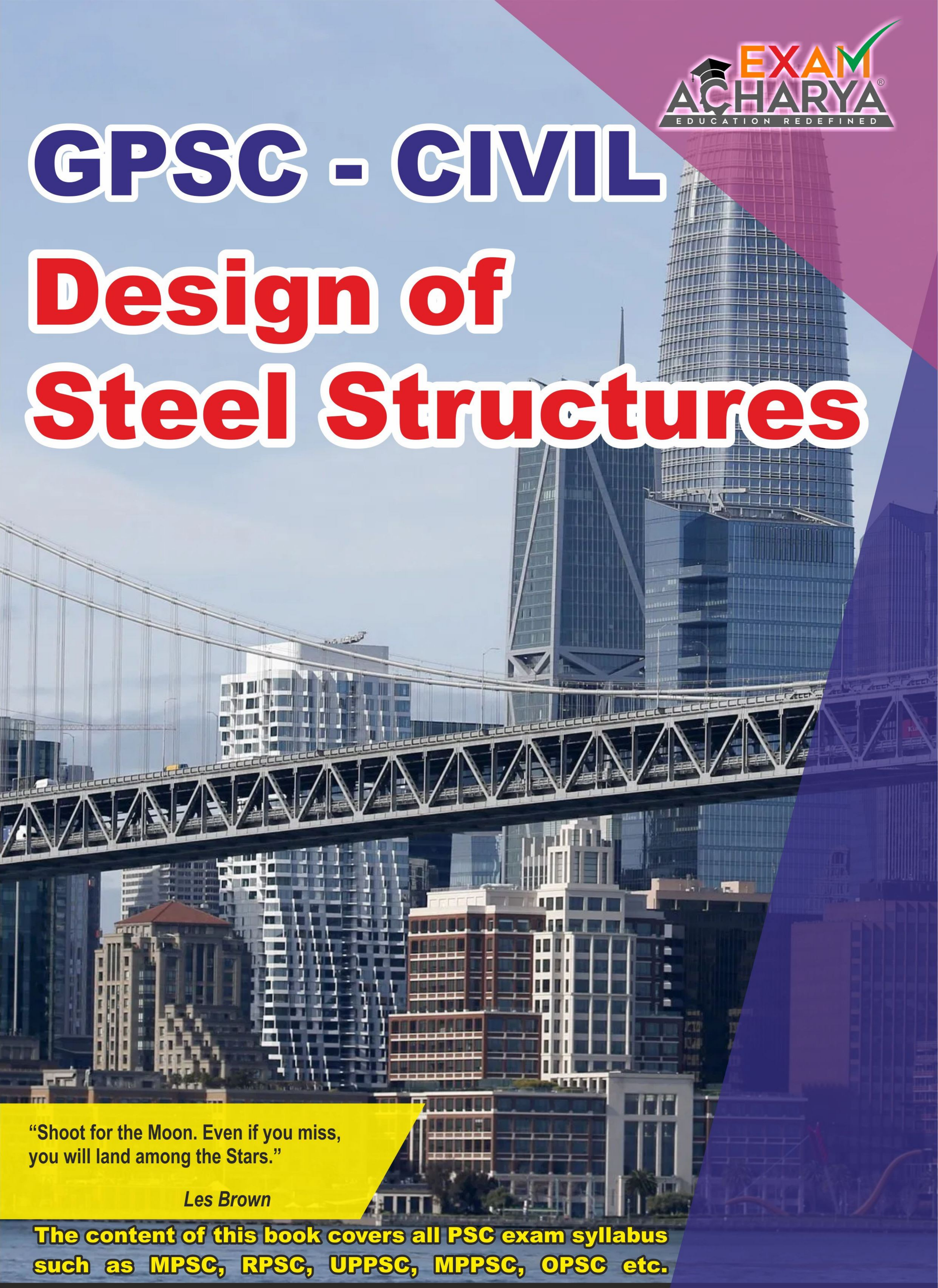
- Quick method
- Capacity of pycnometer= 900ml.
- A conical cap provided with a 6 mm diameter hole at the top can be screwed on to the glass bottle.
- Used when specific gravity of soil solids is known
- Let $W_1 = Wt.$ of empty dried pycnometer bottle

  $W_2 = Wt.$ of pycnometer + Soil

  $W_3 = Wt.$ of pycnometer + Soil + Water

  $W_4 = Wt.$ of pycnometer + Water.





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Design of Steel Structures

"Shoot for the Moon. Even if you miss, you will land among the Stars."

Les Brown

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- The gauge reads water content with respect towet soil. i.e., $w_r = \frac{W_w}{(W_s)_{wet}}$
- Actual water content

$$w = \frac{w_r}{1 - w_r} \times 100\%$$

Sand Bath Method

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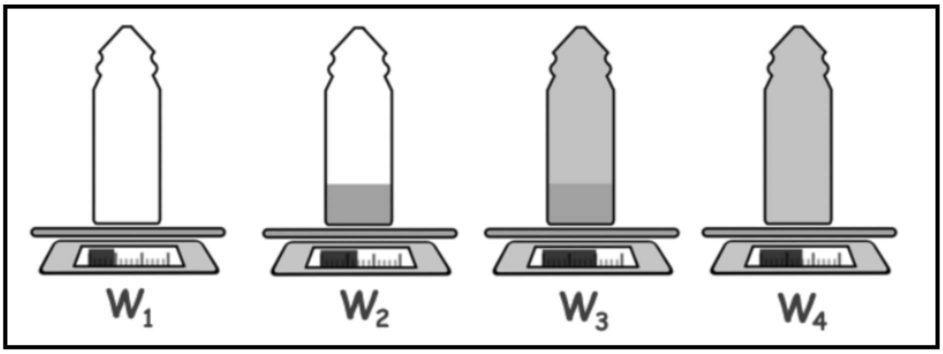
- Quick, field method
- Used when electric oven is not available.
- Soil sample is put in a container & dried by placing it in a sand bath, which is heated on kerosene store.
- Water content is determined by using same formula as in oven drying method.

Torsion Balance Moisture Meter Method

- Quick method for use in laboratory.
- Infrared radiations are used for drying sample.
- Principle: The torsion wire is prestressed accurately to an extent equal to 100% of the scale reading. Then the sample is evenly distributed on the balance pan to counteract the prestressed torsion and the scale is brought back to zero. As the sample dries, the loss in weight is continuously balanced by the rotation of a drum calibrated directly to read moisture % on wet basis.

Radiation Method

- Radio-active isotopes are used for the determination of water content of soils. A device containing radio-active isotopes material, such as cobalt 60, is placed in a capsule. It is then lowered in a steel casing A, placed in a bore hole as shown in Fig. The steel casing has a small opening on its one side through which rays can come out. A detector is placed inside another steel casing B, which also has an opening facing that in casing A.



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

$$\Rightarrow G = \frac{W_s}{W_s - W_3 + W_4}$$

Note

1 Specific gravity values are generally reported at 27°C (in India)

2 If T°C is the test temperature then Sp. Gr. at 27°C is given by,

$$G_{27^\circ C} = G_{T^\circ C} \times \frac{\text{Unit}Wt.\text{ of water at }T^\circ\text{C}}{\text{Unit Wt. of water at }27^\circ\text{C}}$$

3 If kerosene (better wetting agent) is used instead of water then,

$$G = \frac{W_s}{W_s - W_3 + W_4} \times K \; [K = \text{Sp. gr. of Kerosene}]$$

4 G can also be determined indirectly by using shrinkage limit.

MEASUREMENT OF UNIT WEIGHT

- The Bulk Unit weight of the sample is total weight per Unit Volume of soil mass.
- The weight of soil sample can be determined to a high degree of precision in comparison to the volume of sample.
- The methods basically differ in procedure for the measurement of the Volume.
- Dry unit weight can be determined from the bulk unit weight by using the formula.

$$\boldsymbol{\gamma_d = \frac{\gamma_t}{1 + w}}$$

Water Displacement Method

- The volume of the specimen is determined in this method by water displacement. As the soil mass disintegrates when it comes in contact with water, the sample is coated with paraffin wax to make it impervious.
- A test specimen is trimmed to more or less a regular shape and weighed. It is then coated with a thin layer of paraffin wax by dipping it in molten wax.
- The specimen is allowed to cool and weighed. The difference between the two observations is equal to the mass of the paraffin.
- The waxed specimen is then immersed in a water-displacement container. The volume of the specimen is equal to the volume of water which comes out of the outflow tube.
- The actual volume of the soil specimen is less than the volume of the waxed specimen.
- The volume of the wax is determined from the mass of the wax peeled off from the specimen after the test and the mass density of wax.

$$\text{Now, } \boldsymbol{V = V_t - \frac{(M_t - M)}{\rho_p}}$$

Where,

V = volume of specimen

V_t = volume of waxed specimen

M_t = mass of waxed specimen

M = mass of specimen

ρ_p = mass density of paraffin (approximately $0.998\ \mathrm{gm/m^3}$)

- A representative sample of the soil is taken from the middle of specimen for the water content determination.
- Once the mass, volume and the water content of the specimen have been determined, the bulk density and the dry density are found.

- Radiation methods for determination of bulk density of soil are quick and convenient and are gaining popularity.

INDEX PROPERTY OF SOIL

- Those properties which help to access the engineering behaviour of a soil and which assist in determining its classification accurately are termed as index properties. Index properties include indices which help in determining the engineering behaviour such as

  (a) Strength

  (b) Load-bearing capacity

  (c) Swelling and shrinkage

  (d) Settlement etc.

These properties may be relating to

1 Individual soil grain

2 Aggregate soil mass

- The properties of individual particles can be determined from a remoulded, disturbed sample. These depend upon the individual grains their mineralogical composition, size and shape of grains and are independent of soil formation.
- The soil aggregate properties depend upon the mode of soil formation, soil history and soil structure. These properties should be determined from undisturbed samples or preferably from in situ tests

Type of Soil	Index Property
Coarse soil	Particle size, Relative density, Grain Shape
Fine soil	Atterberg's Limit & Consistency

80 mm – 4.75 mm	Gravel	Coarse Grained soil
4.75 mm – 2 mm 2 mm – 0.475 mm 0.475 mm – 75 μ	Coarse sand Medium sand } Sand Fine sand	
75 $\mu - 2\ \mu$	Silt	Fine Grained soil
Less then 2 μ	Clay	

Sieve Aanalysis

Coarse sieve analysis

- Adopted for gravel
- Sieve size are 80, 20 10, 4.75 mm (4No's)

Always dry analysis done

Fine sieve analysis

- Adopted for sand
- Sieve size are 2 mm 1 mm, 600 μ, 425 μ, 212 μ, 150 μ, 25μ, (7 No's)

Dry or Wet analysis done

Smallest sieve size = 75 μ

- Cobbles and Boulders have size greater than 80 & 300 mm respectively (They are not grouped as soil)
- Properties of coarse grained soil (cohesionless) to a greater extent depend on grain size distribution.
- Properties of fine grained soil depends little on grain size distribution. They rather depend on structure, shape of grains, geological origin etc.
- Interparticle forces are more important in case of fine grained soil.
- Grain size analysis helps in determining the gradation and uniformity of soil. This knowledge helps in construction of earth dam, embankments, filters etc.

$1 \leq C_c \leq 3 \rightarrow$ well graded soil

For well graded sand, $C_u > 6$, $1 \leq C_c \leq 3$

For well graded gravel, $C_u > 4$, $1 \leq C_c \leq 3$

Larger the value of C_u, larger is the range of particles in soil.

Concept of "Percentage Finer"

- % retained on a particular sieve

$$= \frac{\text{Weight of soil retained on that sieve}}{\text{Total weight of soil taken}} \times 100$$

- Cumulative % retained = sum of % retained on all sieves of larger sizes and the % retained on that particular sieve.
- "Percentage finer" than the sieve under reference = 100% - Cumulative % retained.

SEDIMENTATION ANALYSIS

- Most convenient for determining the grain size distribution of the soil fraction finer than 75 μm.
- The analysis is based on stokes's law.
- If a single sphere is allowed to fall freely through a liquid of infinite extent, its vertical velocity is first increased rapidly under the action of gravity, but a constant velocity called the terminal velocity is reached within a short time. According to stokes law, the terminal velocity is given by,

$$\mathbf{V = \frac{g}{18} \cdot \frac{\rho_s - \rho_w}{\mu} \cdot D^2}$$

ρ_s = Density of grains (g/cm^3)

ρ_w = Density of water (g/cm^3)

μ = Viscosity of water

Analysis of Fine Grained Soils

- First step involved in preparation of soil sample, which is mixed with water and suspension is made.

Treatment given to soil sample:

1. **Pretreatment:** given before making of soil suspension to remove organic matters and calcium compounds.

   for organic matter – oxidizing agent is used (e.g., H_2O_2)

   for calcium compounds – Acids are used (e.g., HCI)

2. **Post treatment:** given after preparation of soil suspension to break the flocs formed due to presence of surface electric charges.

   The dispersing (defloculating) agents used are sodium hexameta phosphate or calgon, sodium oxalate, etc.

   The analysis is carried out by the Hydrometer method or the Pipette method. The principle of the test is same in both methods. The difference lies only in the method of making observations

PIPETTE METHOD

- In this method, the weight of solids per cc of suspension is determined directly by collecting 10 cc of soil suspension from a specified sampling depth.
- If m_d = dry mass (obtained after drying the sample) then, mass present in unit vol. of pipette

$$= \frac{m_d}{Vol. of\ pipette\ (V_p)} = \frac{m_d}{10\ ml.(V_p)}$$

- If M_d= total mass of soil dissolved in total volume of water (V)

  then mass/unit volume = $\frac{M_d}{V}$

- Therefore, % finer is given by = %

$$N = \frac{m_d/V_p}{M_d/V}$$

- % finer is given as:

$$N = \frac{G}{G-1} \cdot \gamma_w \cdot \frac{V}{W} \cdot \frac{R_H}{10} \%$$

Where,

G = Sp. gr. of soil solids

R_H = Final corrected value of hydrometer

V = Total volume of soil suspension

W = Weight of soil mass dissolved

Corrections to Hydrometer Reading

1. ***Meniscus correction : (C_m)***

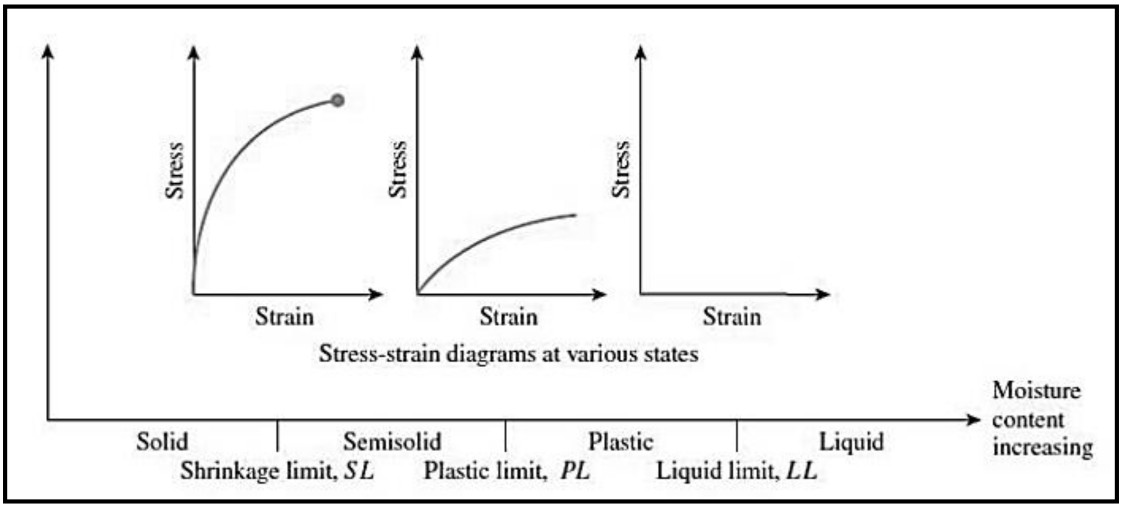
- Hydrometer reading is always correspondding to the upper level of meniscus.
- Therefore, meniscus correction is always positive (+ C_m).

2. ***Temperature correction : (C_t)***

- Hydrometers are generally calibrated at 27^0C.If the test temperature is above the standard (27^0C) the correction is added and, if below, it is subtracted.

3. **Dispersing/Defloculating agent correction: (C_d)**

- The correction due to rise in specific gravity of the suspension on account of the addition of the deflocculating agent is called Dispersing agent correction (C_d).
- C_d is always negative.
- The corrected hydrometer reading is given by (R_H) = $R_H + C_m \pm C_t - C_d$



W_L = Liquid limit water content

W_P = Plastic limit water content

W_S = Shrinkage limit water content

V_L = Volume of soil at liquid limit

V_P = Volume of soil at plastic limit

V_{dry} = Volume of soil at shrinkage limit

From the above figure

$$\frac{V_L - V_P}{W_L - W_P} = \frac{V_P - V_{dry}}{W_P - W_S}$$

LIQUID LIMIT

- Min water content at which soil has tendency to flow is called liquid limit water content.
- All soils at liquid limit will have similar shear strength (approx 2.7 kN/m^2) which is negligible.
- Liquid limit is found out using

  (a) Casagrande's tool

  (b) Cone penetration

EXAM ACHARYA
EDUCATION REDEFINED

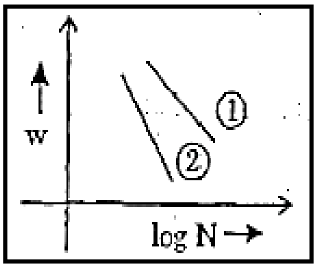
GPSC - CIVIL

Building Material and Construction

Dream is not that which you see while sleeping it is something that does not let you sleep.

A.P.J. Abdul Kalam

The content of this book covers all PSC exam syllabus such as MPSC, RPSC, UPPSC, MPPSC, OPSC etc.



- Curve (1) has larger shearing strength

  Curve (2) has lower shearing strength

- Calculation of liquid limit using fig. require two measurements however liquid limit can also be calculated from one-point method as discussed below

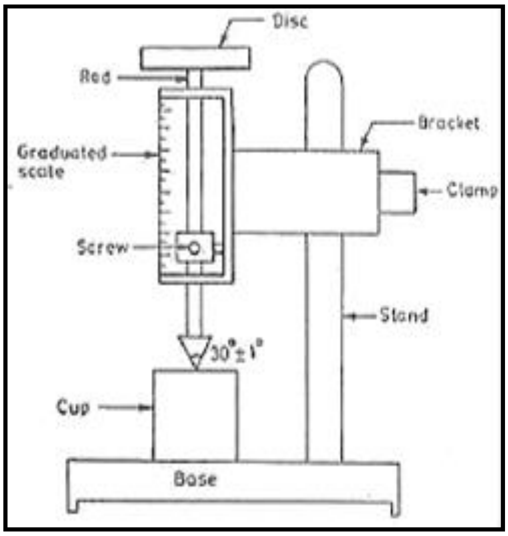
$$W_L = W_N\left(\frac{N}{25}\right)^x$$

W_N = Water content at which N blows are required to close the groove.

x varies between 0.086 – 0.121.

Cone Penetrometer Method

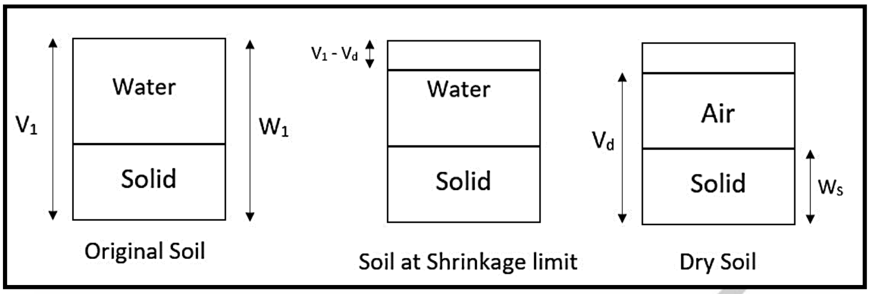
The cup is placed below the cone, and the cone is gradually lowered so as to just touch the surface of the soil in the cup. The graduated scale is adjusted to zero. The cone is released, and allowed to penetrate the soil for 30 seconds. The water content at which the penetration is 25mm is the liquid limit.



SHRINKAGE LIMIT

- That max water content at which further reduction in water content does not cause any reduction in the volume of soil sample is called shrinkage limit.
- When water content is reduced below w_s the particles are so closely spaced that volume reduction will not take place and the void space starts getting occupied by air instead of water.
- It is the min water content at which soil is saturated.

Determination of Shrinkage Limit



V_1 = Original soil volume

W_1 = Original wt of soil

W_s = dry wt of soil

V_d = dry volume of soil

- $(V_1), (W_1), (W_S)$ and (V_d) are known experimentally

$$\text{Water content} = \frac{W_W}{W_S}$$

- At shrinkage limit, $W_w = W_1 - W_s - (V_1 - V_d)\gamma_w$
- Shrinkage limit $= \frac{(W_1 - W_S) - (V_1 - V_d)\gamma_w}{W_S}$
- Shrinkage limit test can be used to determine sp. gravity of solids

$$G_s = \frac{\gamma_s}{\gamma_w} = \frac{W_s}{V_s \gamma_w} = \frac{W_s}{\left[V_1 - \left(\frac{W_1 - W_s}{\gamma_w}\right)\right]\gamma_w} \qquad G_s = \frac{W_s}{V_1\gamma_w - (W_1 - W_s)}$$

- Once G_s is known, shrinkage limit can also be determined as follows

Soil type	W_L	W_P	I_P
Sand	-	-	NP = non plastic
Silt	30-40	20-25	10-15
Clay	40-50	25-50	15-100

If plasticity index = 0, soil is non plastic.

I_P	Consistency
0	Non plastic
<7	Low plastic
7-17	Medium plastic
>17	Highly plastic

Low plastic soil is used for embankment because it is easy to compact.

SHRINKAGE INDEX

$$I_s = w_p - w_s$$

Where,

W_p = Plastic limit water content

w_s = Shrinkage limit water content

CONSISTENCY INDEX (I_C) OR RELATIVE CONSISTENCY

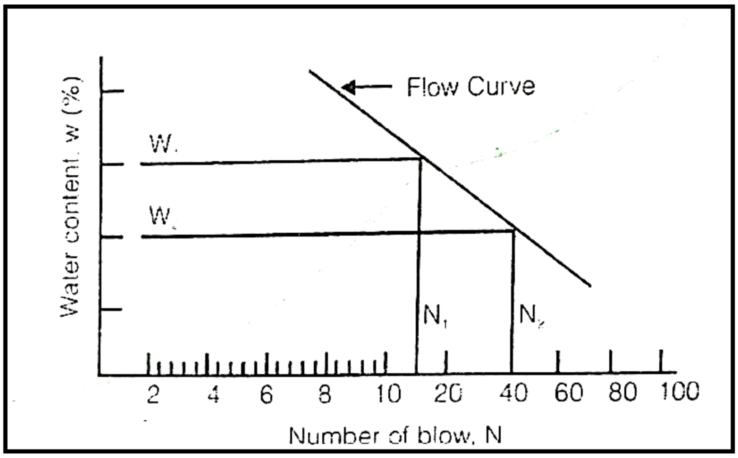
$$I_C = \frac{w_L - w}{w_L - w_P}$$

Where,

w = Natural Water Content

w_L= Liquid limit water content

w_P= plastic limit water content



TOUGHNESS INDEX (I_t)

$$\mathbf{I_t = \frac{I_P}{I_F} = \frac{Plasticity\ index}{Flow\ index}}$$

- It indicates the loss of shear strength with increase in moisture content
- It also indicates the shear strength of soil at plastic limit

We know that,

$$w = I_f \log_{10} N + C$$

N = No. of blows and also N $\propto$ shear strength

$W_L = -I_F \log_{10} N_L + C$

$W_P = -I_F \log_{10} N_p + C$

$W_L - W_P = I_F \log \dfrac{N_P}{N_L}$

$W_L - W_P = I_F \log \dfrac{S_P}{S_L}$

$$\mathbf{\frac{I_P}{I_F} = \log_{10} \frac{S_P}{S_L}}$$

N_L = No. of blows corresponding to liquid limit

N_P = No. of blows corresponding to plastic limit

We know that

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

$$\Rightarrow e = \frac{G_s\gamma_w}{\gamma_d} - 1$$

$$\Rightarrow D_r = \frac{\left(\frac{G_s\gamma_w}{\gamma_d}-1\right)_{max} - \left(\frac{G_s\gamma_w}{\gamma_d}-1\right)_{natural}}{\left(\frac{G_s\gamma_w}{\gamma_d}-1\right)_{max} - \left(\frac{G_s\gamma_w}{\gamma_d}-1\right)_{min}} \times 100$$

$$D_r = \frac{\frac{\gamma_w}{\gamma_{dmin}} - \frac{\gamma_w}{\gamma_{dnatural}}}{\frac{\gamma_w}{\gamma_{dmin}} - \frac{\gamma_w}{\gamma_{dmax}}} \times 100 = \left(\frac{\gamma_{dnatural} - \gamma_{d\,min}}{\gamma_{dmax} - \gamma_{d\,min}} \times \frac{\gamma_{dmax}}{\gamma_{dnat}} \times 100\right)$$

$$\mathbf{D_r = \frac{\gamma_{d\,max}}{\gamma_{d\,natural}}\left(\frac{\gamma_{d\,natural} - \gamma_{d\,min}}{\gamma_{d\,max} - \gamma_{d\,min}}\right) \times 100}$$

Where,

$(\gamma_d)_{max}$ = max dry unit weight

$(\gamma_d)_{min}$ = min dry unit weight

γ_d = dry unit in natural state of the deposit.

- The values of e_{max} and e_{min} for a granular soil assuming the soil grains are spherical and uniform is given as:

$$\left.\begin{aligned} e_{max} &= 91\% \\ e_{min} &= 35\% \end{aligned}\right\}$$

THIXOTROPY

- It is that property of soil due to which loss of strength (shear strength) on remoulding can be regained if left undisturbed for some time.
- Increase in strength with passage of time is due to tendency of clay soil to regain their chemical equilibrium with the reorientation of water molecule in adsorbed layer.

VC: Elementary Engineering

- Active means more prone to volume change.

Kaolinite, $A_C < 0.35 \rightarrow$ inactive

Illite, $0.38 < A_C < 0.9 \rightarrow$ Normal active

Montmorillonite, (Black cotton soil) $0.9 < A_C < 7.2$ Active.

Q.4 **The difference between maximum void and minimum void ratio of a sand sample is 0.30. If the relative density of this sample is 66.6% at a void ratio of 0.40, then the void ratio of this sample at its loosest state will be:**

(a) 0.40

(b) 0.60

(c) 0.50

(d) 0.75

Q.5 **A soil sample has natural moisture content 'w', void ratio 'e' and specific gravity of soils solids ' G_s' The bulk unit weight of soil ' g ' is given by (g'_w is unit weight of water)**

(a) $\frac{(1-w)G_s\gamma_w}{(1-e)}$

(b) $\frac{(1+w)G_s\gamma_w}{(1-e)}$

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

(d) $\frac{(1-w)G_s\gamma_w}{(1+e)}$

Q.6 **Match List-I with List-II and select the correct answer using the codes given below the lists. (Notations have their usual meaning)**

List-I

A. Fine grained soil with $w_L = 60$,

$l_p = 20, w_s = 8$

B. Fine grained soil with $w_L = 60$,

$l_p = 30, w_s = 8$

C. Fine grained soil with $w_L = 30$,

$l_p \leq 4, w_s = 20$

D. Coarse grained sand with $w_L = 40$,

$I_p = 15, w_s = 20$

TEST YOUR SELF

Q.8 In Casagrande's liquid limit device, the material of the test specimen is harder than the standard rubber. This hardness indicates that the liquid limit, plasticity index, flow index and toughness index, respectively, of the specimen, are

(a) more, less, more and same

(b) same, less, same and more

(c) less, less, same and less

(d) less, same, less and more

Q.9 Which one of the following is the water content of the mixed soil made from 1kg of soil (say A) with water content of 100% and 1kg of soil (say B) with water content of 50%?

(a) 66%

(b) 71%

(c) 75%

(d) 82%

Q.10 Given for a sample of a river sand:

Void ratio at the densest state $= 0.40$

Void ratio at the loosest state $= 1.20$

Which one of the following correctly represents the relative density of a sample prepared with a void ratio of 1.0?

(a) 12.5%

(b) 25%

(c) 75%

(d) 87.5%

Answers

1-(d), 2-(c), 3-(a), 4-(b), 5-(c), 6-(d), 7-(a), 8-(c), 9-(b), 10-(b)

- All the soils are classified into four major groups, namely, coarse grained, fine grained, organic soils and peat.

AASHTO Classification System

- American Association of State Highway and Transportation Official (AASHTO) Classification system is useful for classifying soils for highways.
- The particle size analysis and the plasticity characteristics are required to classify a soil.
- According to the AASHTO system, the soils are classified into eight groups: A-1 through A-7 with an additional group A − 8 for peat or muck.
- The system includes several sub-groups. Soils within each group are evaluated according to the group index calculated from the empirical formula.

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

Where,

a = That part of the percent passing the 75µ sieve greater than 35 and not exceeding 75 expressed as a positive whole number (range 1 to 40)

b = That part of the percent passing the 75µ sieve (−75µ) greater than 15 and not exceeding 55 expressed as a positive whole number (range 1 to 40)

c = That part of liquid limit greater than 40 and not greater than 60, expressed as a positive whole number (range 1 to 20)

d = That part of the plasticity index greater than 10 and not exceeding 30 expressed as a positive whole number (range 1 to 20)

- While calculating GI, if any term becomes negative, it is dropped. The group index should be rounded off to the nearest whole number
- If the computed value is negative, it is reported as zero.
- In general, the greater the group index value, the less desirable a soil is for highway construction within that subgroup. A group index of 0 indicates a good

Pre fix & Suffix used in soil classification

Soil type	Prefix	Sub group	Sufix
Gravel	G	Well graded	W
Sand	S	Poorly graded	P
Silt	M	Silty	M
Clay	C	Clayey	C
Organic	O	$w_L < 35$	L(i.e. low compressibility)
Peat	Pt	$35 < w_L < 50$	I (i.e. intermediate compressibility)
		$50 < w_L$	H (i.e. high compressibility)

CLASSIFICATION OF COARSE GRAINED SOIL

Classification of Coarse grained soil is done on the basis of

(1) Grain Size

(2) Gradation characteristics

(3) Percentage of fines present in soil by weight (fines means particle size $< 75\mu m$)

Case -1: When fines are less than 5%

GW Well graded gravel	GP Poorly graded gravel
1 More than half of coarse fraction > 4.75mm 2 $\dot{C}_u = \frac{D_{60}}{D_{10}} > 4$ 3 $C_c = \frac{D_{30}^2}{D_{60} \times D_{10}}$ $1 < C_C < 3$	Otherwise
SW **Well graded sand**	**SP** **Poorly graded sand**
1. More than ½ of coarse fraction < 4.75 mm 2. $C_u > 6$ 3. $1 < C_c < 3$	Otherwise

GPSC - CIVIL

Engineering Hydrology

EXAM ACHARYA
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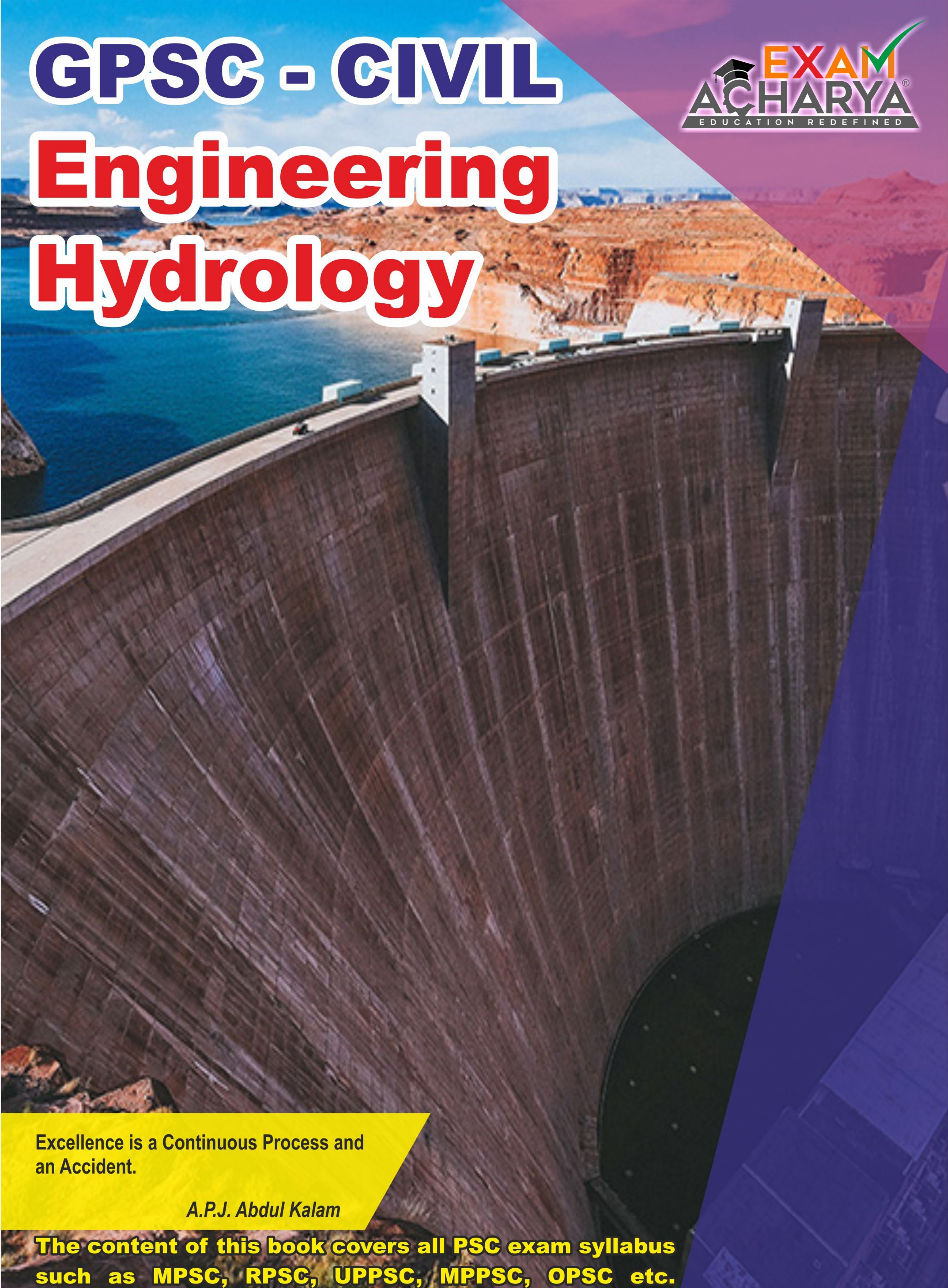


Excellence is a Continuous Process and an Accident.

A.P.J. Abdul Kalam

The content of this book covers all PSC exam syllabus such as MPSC, RPSC, UPPSC, MPPSC, OPSC etc.

CLASSIFICATION OF FINE GRAINED SOIL

Classification of Fine Grained Soil is done on the basis of Plasticity chart.

1 First w_L & w_p are determined for seived fraction of fine grained soil.

2 If the limit plot above A-line, classify as clay.

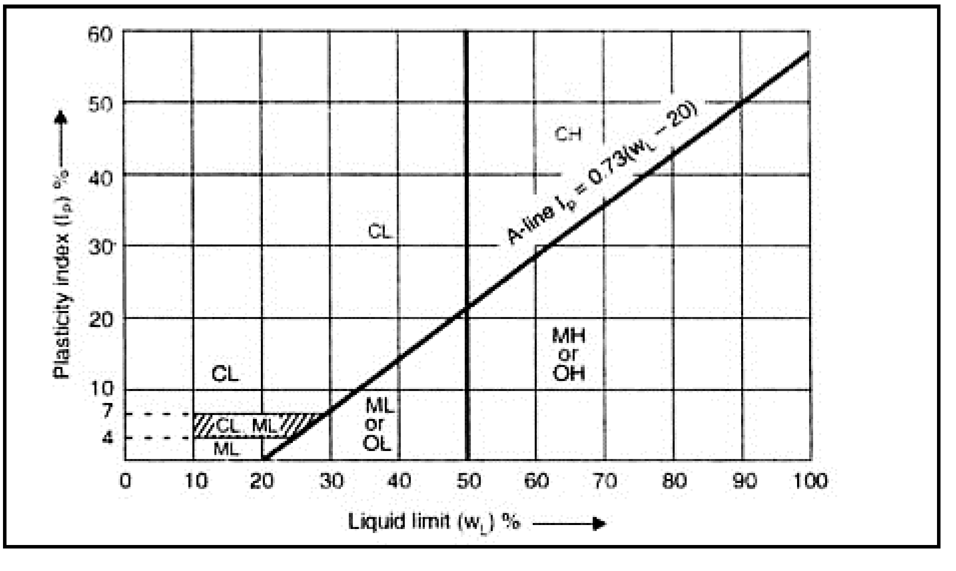
Further,

- If, $35 > w_L$ Classify as CL
- If, $35 < w_L < 50$ Classify as CI
- If, $50 < w_L$ Classify as CH

3 If the limit plots below A line, then we need to find out whether, soil is organic or inorganic. If its inorganic and below A line then it is silt (M).

Further,

- If, $35 > w_L$ Classify as ML or OL
- If, $35 < w_L < 50$ Classify as MI or OI
- If, $50 < w_L$ Classify as MH or OH
- GM or SM if fines are silty i.e. limit plots below A-line on plasticity chart.
- GC or SC - limit plots above A line



Plasticity chart

CLEAR YOUR CONCEPT

Qu1 Consider the following statements in the context of aeolian soils:

1. **The soil has low density and low compressibility**
2. **The soil is deposited by wind**
3. **The soil has large permeability**

Which of these statements are correct?

(a) 1,2 and 3

(b) 2 and 3

(c) 1 and 3

(d) 1 and 2

Qu2. If the proportion of soil passing 75 micron sieve is 50% and the liquid limit and plastic limit are 40% and 20% respectively, then the group index of the soil is

(a) 3.8

(b) 6.5

(c) 38

(d) 65

Qu3 Given that coefficient of curvature $= 1.4, D_{30} = 3mm, D_{10} = 0.6mm$

Based on this information of particle size distribution for use as subgrade, this soil will be taken to be

(a) Uniformly-graded sand

(b) Well-graded sand

(c) Very fine sand

(d) Poorly-graded sand

TEST YOUR SELF

Qu6 Consider the following statements

1. **The minimum value of group index for a soil can be taken as zero**
2. **The maximum possible value of group index for a soil is twenty.**

Which of these statements is/are correct?

(a) Both 1 and 2
(b) 1 only
(c) 2 only
(d) Neither 1 nor 2

Qu7 Match List-I (Range of particle size) with List-II (Type of soil) and select the correct answer using the codes given below the lists:

List-I	List-II
A. Less than 0.002 mm	1. Gravel
B. 0.075 mm to 0.002 mm	2. Sand
C. 80 mm to 4.75 mm	3. Cobble
D. 4.75 mm to 0.075 mm	4. Silt
	5. Clay

Codes:

	A	B	C	D
(e)	4	5	1	3
(f)	4	5	2	1
(g)	5	4	1	2
(h)	5	4	3	1

CHAPTER – 3

CLAY MINERAL AND SOIL STRUCTURE

SOIL STRUCTURE

- Soil structure means the geometrical arrangement of soil particles in a soil mass, relative to each other and the forces acting between them to hold them together in their positions.
- The concept is further extended to include the minerological composition of the grains, the electrical properties of the particle surface, the physical characteristics, ionic composition of the pore water, the interactions among the solid particles, pore water and the adsorption complex.
- In coarse grained soil structure is governed by gravity forces where as in clayey soil by surface forces are predominant.

Structure of Clay Mineral

- It has been observed that, clay soil is made up of many crystal sheets which have a repeating atomic structure.
- Atomic structure of clay minerals are built of two fundamental crystal sheets

  (a) Tetrahedron or Silica sheet

  (b) Octahedral or Alumina Sheet

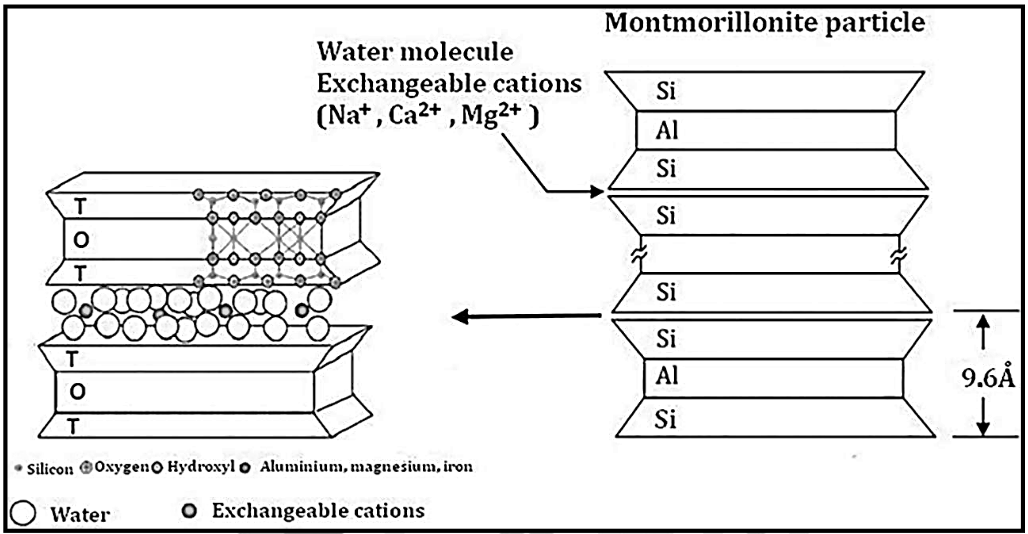
VARIOUS CLAY MINERALS

- Clay minerals Continues to change from one form to other due to weathering. Depending upon various stages of occurance they are named as follows.

VC: Elementary Engineering

- Hence although engineering properties of Halloysite is different from kaolinite in natural condition, the two may be alike when halloysite is air dried.
- Halloysite & Kaolinite clays are used for making chinaware
- Kaolinite clay is also used as an intestinal absorbant in anti-diarrhoeal medicine.

Montmorillonite



- It is also called Smectite.
- Montmorillonite is 2: 1 clay mineral as it is composed of 2 silica sheet and 1 Alumina (Gibbsite) sheet.
- Montmorillonite has largest specific surface among clay mineral.
- Montmorillonite has large amount of water and other exchangeable ions can easily enter between the layers causing the layers to be separated. Because of this property it is susceptable to substantial volume changes.
- Bentonite is a Montmorillonite clay.
- Montmorillonite is found in black cotton soil.

CHAPTER – 4

SOIL COMPACTION

INTRODUCTION

- Compaction of soil is the process of increasing the unit wt of soil by forcing the soil solids into a dense state and reducing the air voids.
- Compaction leads to increase in shear strength and helps to improve the stability and bearing capacity of soil. It also reduces the compressibility and permeability of soil.
- This is achieved by applying static or dynamic loads to the soil.
- Compaction is measured quantitatively in terms of dry unit wt (γ_d) of the soil.
- Difference between compaction and consolidation are as tabulated below.

Compaction	Consolidation
1. Instantaneous phenomenon 2. Soil always partially saturated 3. Densification due to reduction in the volume of air voids at a given water content. 4. Specific compaction techniques are used.	1. Time dependent Phenomenon 2. Soil is completely Saturated 3. Volume reduction is due to expulsion of pore water from voids. 4. Consolidation occurs on account of a static load placed on the soil.

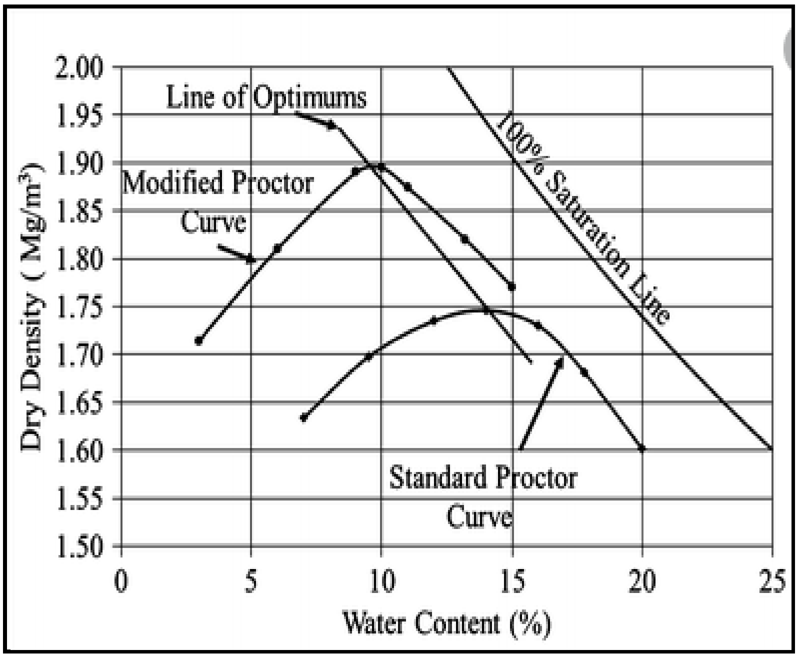
COMPACTION OF COHESIONLESS SOIL

- A cohesionless soil can be in various states like

  1 Loose angular soil

  2 Dense angular soil

  3 Honey-combed state



- Maximum dry unit weight obtained is a function of compactive effort and method of compaction for a particular type of soil.
- Compactive effort is a measure of mechanical energy applied to soil mass.
- Typical values of $\gamma_{dmax} = 16 - 20 kN/m^3$
- Typical values of $OMC = 10 - 20\%$

STANDARD PROCTOR TEST

- Weight of hammer $= 2.495 kg (5.5\ lb)$
- Height of fall $= 12'' = 304.8 mm$
- Volume of mould= 944 cc $(1/30 cft)$
- Compacted in 3 layers with 25 blows in each layer.
- Test is conducted at various moisture content and the curve as shown above is obtained.
- Compactive energy (E) applied per unit volume

$$\mathbf{E = \frac{NnWh}{V}}$$

3 Provides data on the behaviour of the material in relation to various moisture content. (i.e. how permeability etc. will be affected by m/c).

LIGHT COMPACTION TEST (IS : 2720, PART VII-1974)

- Weight of hammer $= 2.6\text{kg}$
- Height of fall $= 310\text{mm}$
- Volume of mould$= 1000$ cc
- Compacted in 3 layers with 25 blows in each layer.
- Light Compaction Test Compactive effort per unit volume

$$= \frac{25 \times 3 \times 2.6 \times 0.31 \times 9.81}{1000 \times 10^{-6}} \frac{\text{J}}{\text{m}^3}$$

$$= 593.014\text{kJ/m}^3$$

HEAVY COMPACTION TEST (IS : 2720, PART VIII-1983)

- Weight of hammer $= 4.9\text{kg}$
- Height of fall $= 450\text{mm}$
- Volume of mould$= 1000$cc
- Compacted in 5 layers with 25 blows in each layer.
- Heavy Compaction Test Compactive effort per unit volume

$$= \frac{25 \times 5 \times 4.9 \times 0.45 \times 9.81}{1000 \times 10^{-6}} \frac{\text{J}}{\text{m}^3}$$

$$= 2703.88\text{kJ/m}^3$$

(d) Method of Compaction

- Since the field compaction is essentially a kneading or rolling type compaction and the laboratory tests are dynamic-impact type compaction, therefore, laboratory compaction tests have more value of max. dry unit weight.

COMPACTION IN THE FIELD

- Laboratory compaction tests are usually utilised to specify the compacted dry unit weight to be attained in the field depending upon the size of equipment to be used in the field.
- Indian Standard (light or heavy compaction) test may be used.
- Since the control in the field cannot be as strict as in the laboratory, the specifications usually require attainment of 90 to 95 per cent of dry unit weight attained in the laboratory.
- Different types of soils in the field can be compacted by various methods, e.g., rolling, ramming (by impact) and vibrations.
- The various types of rollers are : smooth wheel rollers, pneumatic tyred rollers and sheepsfoot rollers. Ramming equipment can be the impact type, internal combustion type or the pneumatic type.
- Vibrating unit can be mounted on plates or rollers. Vibration may be induced by rotating an unbalanced mass or by a pulsating hydraulic system.
- The selection of the equipment and the procedure of compaction depends on the characteristics of the soil to be compacted. The compaction achieved in the field would depend on

    (i) Thickness of the lift (layer)

    (ii) Type of roller.

    (iii) Number of passes of the roller, and

    (iv) Intensity of pressure on the soil.

CLEAR YOUR CONCEPT

Qu1 **For conducting a Light Compaction Test, the weight of hammer (P in kg), the fall of hammer (Q in mm), the number of blows per layer (R) and the number of layers (S) required are respectively**

	P	Q	R	S
(a)	5.89	550	50	3
(b)	4.89	450	25	3
(c)	3.60	310	35	4
(d)	2.60	310	25	3

Qu2 **Sheep-foot rollers are recommended for compacting**

(a) Granular soils

(b) Cohesive soils

(c) Hard rock

(d) Any type of soil

Qu3 **In a standard proctor test, 1.8kg of moist soil was filling the mould (volume $= 944$ cc) after compaction. A soil sample weighing 23g was taken from the mould and ovendried for 24 hours at a temperature of 110°C. Weight of the dry sample was found to be 20g. Specific gravity of soil solids is $G = 2.7$. The theoretical maximum value of the dry unit weight of the soil at that water content is equal to**

(a) $\frac{4.67\text{kN}}{\text{m}^3}$

(b) $\frac{11.5\text{kN}}{\text{m}^3}$

(c) $\frac{16.26\text{kN}}{\text{m}^3}$

(d) 18.85kN/m^3

TEST YOUR SELF

Qu6 Match List-I (Equipment) with List-II (Use) and select the correct answer using the codes given below the lists:

List-I	List-II
A. Vibratory rollers	1 To compact soils in confined areas and at corners
B. Sheep foot rollers	2 To compact road and railway embank-ments of sandy soils
C. Frog hammers	3 To densify sandy soils over a large area and to a larger depth
D. Vibrofloats	4 To compact clayey soil fills

Codes:

	A	B	C	D
(a)	4	2	1	3
(b)	4	2	3	1
(c)	2	4	1	3
(d)	2	4	3	1

Qu7 Soil is compacted at which one of the following when a higher compactive effort produces highest increase in dry density?

(a) Optimum water content

(b) Dry side of the optimum moisture content

(c) Wet side of the optimum moisture.

(d) Saturation moisture content

Answer

1-(d), 2-(b), 3-(d), 4-(c), 5-(c), 6-(c), 7-(b)

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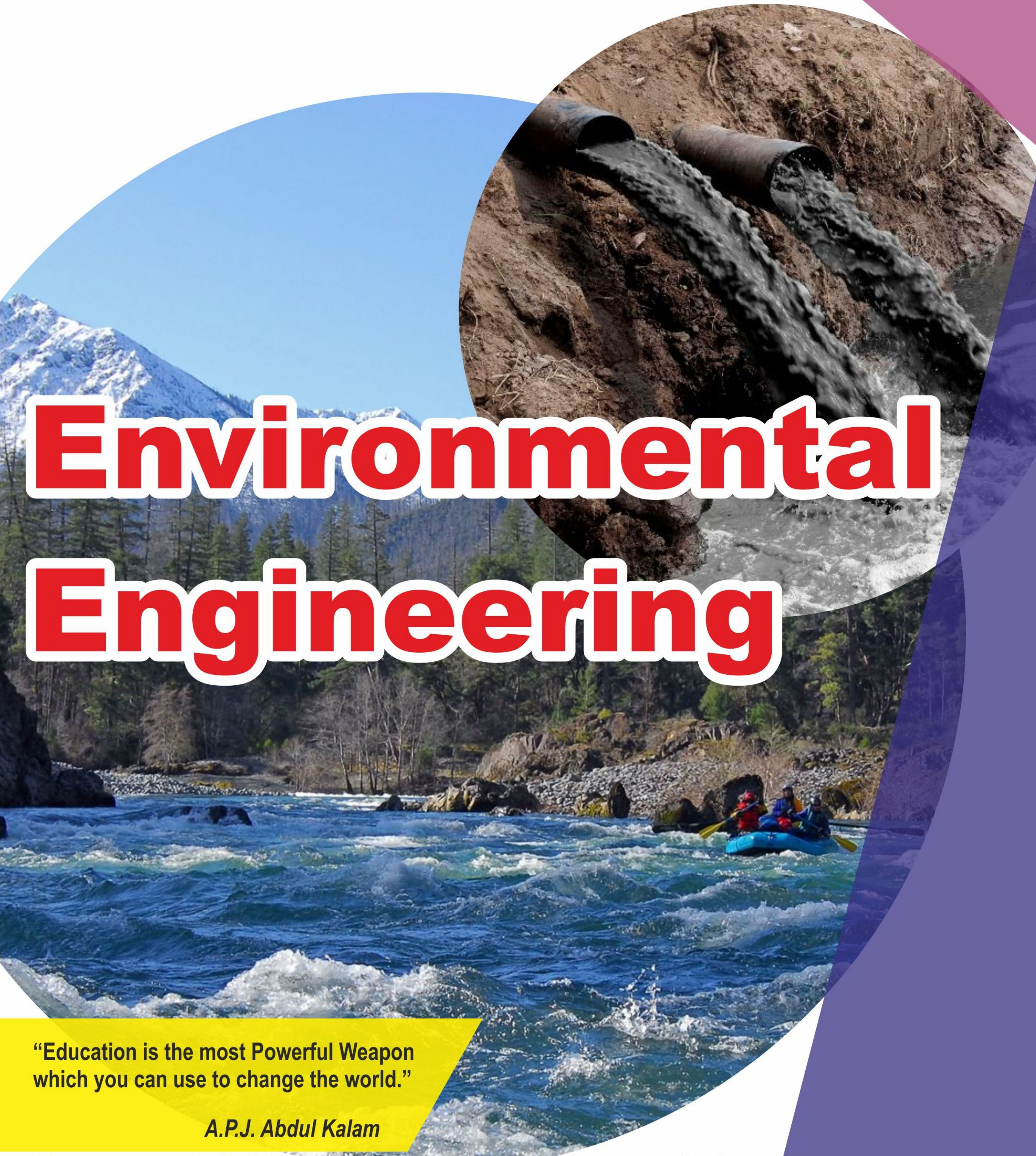


Environmental Engineering

"Education is the most Powerful Weapon which you can use to change the world."

A.P.J. Abdul Kalam

The content of this book covers all PSC exam syllabus such as MPSC, RPSC, UPPSC, MPPSC, OPSC etc.

Where,

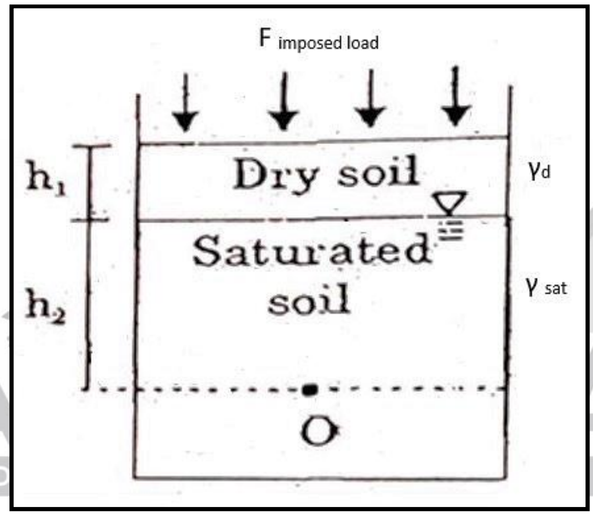
P = Force on plane X − X for weight above plane X − X

A = Area of cross section of soil mass.

- When there is imposed load on soil mass as shown below, the total stress value at point O is given by

Total stress at 'O'$= \frac{F+\gamma_d.Ah_1+\gamma_{sat}.Ah_2}{A}$

$$\boldsymbol{\sigma = \frac{F}{A} + \gamma_d h_1 + \gamma_{sat} h_2}$$



- Total stress is a physical parameter which can be measured by suitable arrangement, such as by pressure cell.

Pore Water Pressure (u)

- It is the Pressure of water filling the void space between solid particles.

Where,

σ = Total stress

U_a = Pore air pressure

U_w = Pore water pressure

X = Fraction of a unit cross section area of soil occupied by water

= 0 (for dry soil)

= 1 (for saturated soil)

$$\sigma = \sigma' + u \text{ or } \bar{\sigma} = \sigma - u$$

CAPILLARITY IN SOILS

Ground water can exist in two forms

Phreatic or Gravitational Water

- This water is subjected to gravitational forces, and saturates the voids completely. Commonly it is represented by Ground water table.
- At ground water table the pressure is atmospheric and below which pressure is in hydrostatic condition.

Capillary Water

- If the water contained in the pores of soil was only subjected to gravitational forces, then soil above ground water table would have been perfectly dry
- But in reality it is observed that soil is fully saturated upto certain height above water table and partially saturated upto some more height. This is because of capillary phenomenon in soils.
- Capillary water is held above the water table by Surface tension.
- Pressure in the capillary Zone is negative (-ve) and it does not contribute to hydrostatic pressure below Ground water table.

Where;

q = discharge

A = Cross section area of soil corresponding to flow q

k = Coefficient of permeability.

$i = \text{hydraulic gradient} = \frac{\Delta h}{L} = \frac{\text{Loss of head}}{\text{length}}$

V = superficial velocity of flow or discharge velocity.

COEFFICIENT OF PERMEABILITY

- Coefficient of Permeability can be defined as Superficial velocity of flow (or velocity of flow) which would occur under a unit hydraulic gradient.
- Permeability is usually expressed in the unit of velocity
- Typical values of Permeability are as listed in the table below

Soil Type	Coefficient of Permeability cm/sec	Drainage Characteristics
Gravel	>1	Pervious
Sand	$1 - 10^{-3}$	Pervious
Silt	$10^{-3} - 10^{-6}$	Slightly Pervious
Clay	$<10^{-6}$	Impervious

- Coefficient of permeability divided by porosity is called coefficient of percolation (K_p)

$$\mathbf{K_P = \frac{K}{n}}$$

Where,

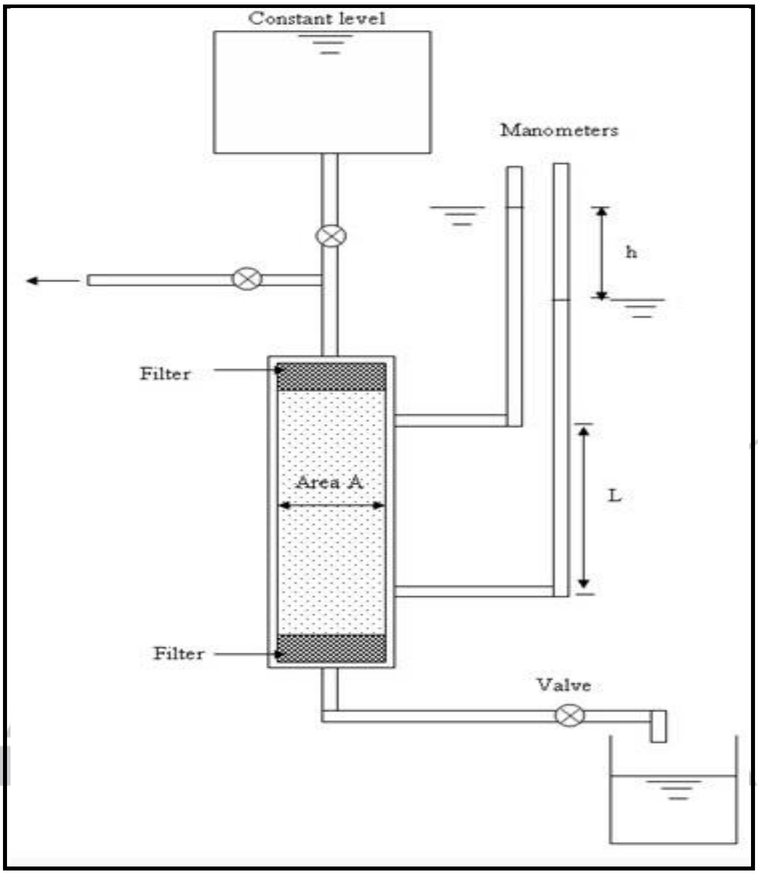
K_p = Coefficient of percolation

K = Coefficient of permeability

n = Porosity

CONSTANT HEAD PERMEABILITY TEST

- Coefficient of permeability for coarse soil is determined by means of constant head permeability test.
- Degree of saturation of soil should be 100%



We know that,

$$q = kiA$$
$$q = k\frac{h}{L}A$$
$$k = \frac{qL}{Ah}$$

Where,

q = discharge collected in time t.

L = distance between manometer taping points

- The head is measured with reference to the level of water in the constant head chamber.
- Let us consider the instant at which the head is h. For the infinitesimal small time dt, the head falls by height dh. Let the discharge through the sample be q. From continuity of flow.

$$\mathbf{-adh = qdt}$$

Where a is cross-sectional area of the standpipe.

$$\begin{aligned} \mathrm{adh} &= -(\mathrm{A} \times \mathrm{k} \times \mathrm{i}) \times \mathrm{dt} \\ \mathrm{adh} &= -\mathrm{Ak} \times \frac{\mathrm{h}}{\mathrm{L}} \times \mathrm{dt} \\ \frac{\mathrm{Akdt}}{\mathrm{aL}} &= \frac{-\mathrm{dh}}{\mathrm{h}} \end{aligned}$$

Integrating, $\frac{\mathrm{Ak}}{\mathrm{aL}} \int_{t_1}^{t_2} \mathrm{dt} = -\int_{\mathrm{h}_1}^{\mathrm{h}_2} \frac{\mathrm{dh}}{\mathrm{h}}$

$$\frac{\mathrm{Ak}}{\mathrm{aL}}(\mathrm{t}_2 - \mathrm{t}_1) = \log_e\left(\frac{\mathrm{h}_1}{\mathrm{h}_2}\right)$$

$$\mathbf{k = \frac{aL}{At} \log_e(h_1/h_2)}$$

Where, $\mathrm{t} = (\mathrm{t}_2 - \mathrm{t}_1)$, the time interval during which the head falls from h_1 to h_2.

$$\mathbf{k = \frac{2.30aL}{At} \log_{10}(h_1/h_2)}$$

FIELD TEST METHODS

- The laboratory methods for the determination of the coefficient of permeability, as discussed before do not give correct results.
- The samples used are generally disturbed and do not represents the true in-situ structure.

$$i = \frac{dh}{dr}$$

Area of flow, A = 2 π r h

From Darcy's law, q = k. i. A

$$q \quad = k\frac{dh}{dr}2\pi rh$$

$$\frac{dr}{r} \quad = 2\frac{\pi}{q}khdh$$

The expression for the coefficient of permeability can be derived making the following assumptions, known as Dupit's assumption.

(1) The flow is laminar and Darcy's law is valid.

(2) The soil mass is isotropic and homogeneous.

(3) The well penetrates the entire thickness of aquifer such that water does not come into wall from below.

(4) The flow is steady.

(5) The coefficient of permeability remains constant throughout.

(6) The flow towards the well is radial and horizontal.

(7) Natural ground water regime remains constant.

(8) Hydraulic gradient at any distance 'r' from the centre of well is assumed to be constant with depth and is equal to the slope of water table.

$$\mathbf{i = \frac{dh}{dr}}$$

$$R = 3000d\sqrt{k}$$

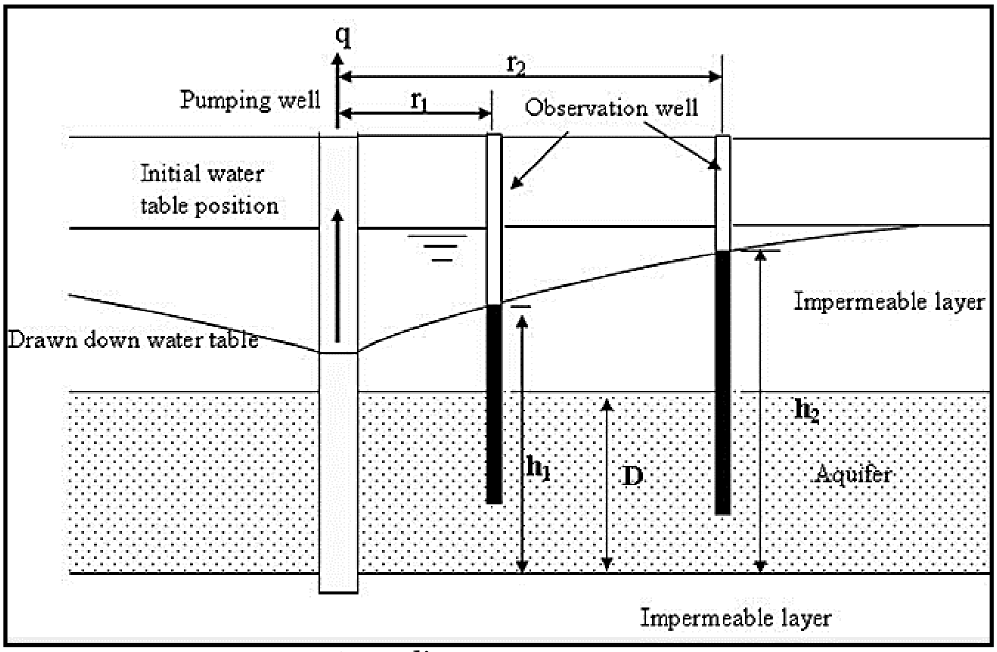
Where,

R = radius of influence (m), d = drawdown in the well (m)

k = coefficient of permeability (m/sec)

Confined Aquifer

A confined flow condition occurs when the aquifer is confined both above and below by impermeable strata. Here, the drawdown surface is for all values of r, above the upper surface of the aquifer.



According to Darcy's Low

$$q = kiA$$

$$A = 2\pi rD$$

$$q = 2\pi rD \cdot k \cdot \frac{dh}{dr}$$

$$\frac{qdr}{r} = 2\pi Dkdh$$

Integrating, $$q\int_{r_1}^{r_2} \frac{dr}{r} = 2\pi Dk\int_{h_1}^{h_2} dh$$

$$Q \log e \frac{r_2}{r_1} = 2\pi Dk\,(h_2 - h_1)$$

$$K = \frac{q \log e \frac{r_2}{r_1}}{2\pi D\,(h_2 - h_1)}$$

Allen Hazen's Formula

The coefficient of permeability of a soil is proportional to the square of a representative particle size.

$$K = CD_{10}^2$$

Loudon's Formula

According to Loudon.

$$\log_{10}(KS_A^2) = a + bn$$

Where

K = Coefficient of permeabitity

S_A = Specific Surface area

n = Porosity

a & b = Constants

Consolidation Equation

This method is suitable for fine grained soil whose $k < 10^{-7}$ cm/ sec. for which permeability test can not be conducted in laboratory.

$$K = C_v \gamma_w m_v$$

Where,

K = Permeability of soil

C_v = Coefficient of consolidation

γ_w = Unit weight of water

m_v = Coefficient of volume compressibility

Qu4 **A stratified soil deposit has three layers of thicknesses: $z_1 = 4$, $z_2 = 1$, $z_3 = 2$ units and the corresponding permeabilitiesof $k_1 = 2$, $k_2 = 1$ and $k_3 = 4$ units, respectively. The average permeability perpendicular to the bedding planes will be**

(a)4

(b)2

(c)8

(d)16

Qu5 **Consider the following statements:**

1. **Coarse sand is more than a million times permeable than a high plasticity clay.**
2. **The permeability depends on the nature of soil and not on properties of liquid flowing through soil.**
3. **If a sample of sand and a sample of clay have the same void ratio, both samples will exhibit the same permeability.**
4. **Permeability of soil decreases as the effective stress acting on the soil increases.**

Which of these statements are correct?

(a)1 and 2

(b)1 and 3

(c)1 and 4

(d)2 and 3

CHAPTER 6

SEEPAGE THROUGH SOIL

INTRODUCTION

- Seepage is a process in which liquid leaks through a porous medium from high head to towards low head

Although flow velocity of water during seepage is very small (as this flow is Laminar) but in reality itposses many problems such as

(1) Losses of water Reservoir

(2) Reduction in effective weight of soil.

(3) Uplift pressure below dam.

(4) Piping failure.

- To find the solution of above discussed problems and many more it becomes important to calculate the seepage flow.
- Seepage flow is calculated with the help of flownets. Where flownet is a graphical representation of path taken by water particle and head variation along the path.
- The concept of flow net is based on Laplace's equation of contunity.

LAPLACE EQUATION – 2 D FLOW CONDITIONS

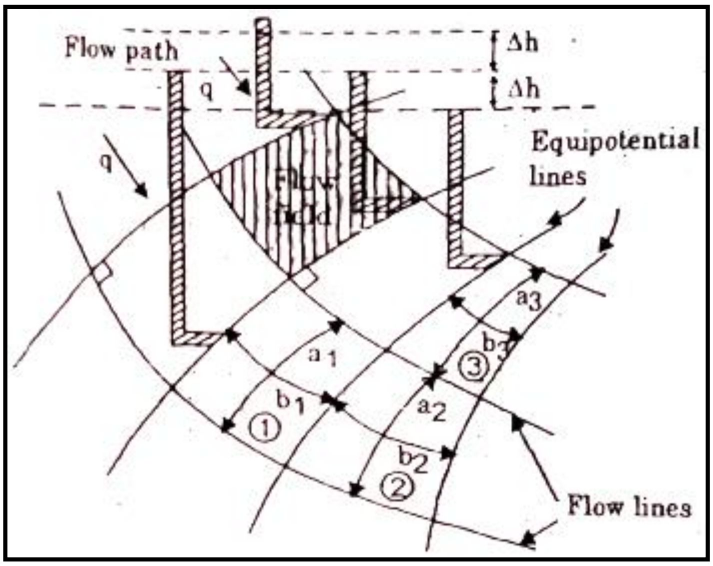
Following Assumptions are made in 2D flow.

1. Soil mass is fully saturated and darcy's law is valid.

$$v_x = K_x i_x = K_x \frac{dh}{dx}$$

$$v_y = K_y i_y = \frac{K_y dh}{dy}$$

Properties and Use of Flow Net



1. Flow lines and equipotential lines are orthogonal to each other in case of isotropic soil.

2. Space between two adjacent flowlines is called flow channel or flow path.

3. The figure formed in flow net between two adjacent flowlines and adjacent equipotential line is called flow field.

4. All flow fields are elementary squares (linear or curvilinear)

   i.e., $\frac{a_1}{b_1} = \frac{a_2}{b_2} = \frac{a_3}{b_3}$

5. Head loss through each successive equipotential line is equal

$$\Delta h_1 = \Delta h_2 = \Delta h_3 = \Delta h$$

6. Discharge through each flow channel is constant

   i.e. $\Delta q_1 = \Delta q_2 = \Delta q_3 = \Delta q$

Consider flow fields (1), (2) & (3) as shown in the diagram above.

According to darcy's law

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Fluid Mechanics and Hydraulic Machines

"Success Consists of going from Failure without Loss of Enthusiasm."

Winston Churchill

The content of this book covers all PSC exam syllabus such as MPSC, RPSC, UPPSC, MPPSC, OPSC etc.

$$\text{Then, } \Delta q = k\frac{H}{N_d} \times \frac{a}{b}$$

- If N_f is the number of flow channels in a flow net where N_f = (No. of flow lines -1)

  Then total discharge through a flow net will be

$$Q = \Delta q \times N_f$$

$$Q = k\frac{H}{N_d} \times N_f \times \frac{a}{b}$$

$$Q = KH\frac{N_f}{N_d} \times \frac{a}{b}$$

- Flow net is generally drawn such that $\frac{a}{b} \to 1$

  Hence, $q = KH\frac{N_f}{N_d}$

- From the above discussion we can conclude that rate of flow is a function of

  (i) Permeability*(k)*

  (ii) Headloss*(H)*

  (iii) Shape factor $\left(\frac{N_f}{N_d}\right)$

Salient Points About Flow Net

(i) Shape factor is only function of boundary condition.

(ii) Flow net will not change if Permeability (k) of soil changes i.e. Soil is changed.

(iii) Flow net will not change if head loss during flow is changed.

(iv) In both the above cases i.e. (ii) & (iii) the quantity of seepage will be different.

(v) Flow net is unique for a given boundary condition and if the boundary condition does not change $\frac{N_f}{N_d}$ will not change.

$$\text{F.O.S. against heave} = \frac{\text{Submerged weight of soil prism+Effective weight of filter}}{\text{Total seepage pressure}}$$

Pore water pressure measurement:Pore water pressure at any point can be calculated by working out the energy equation between the point at which pore water pressure is to be calculated and the point of known pressure and elevations.

METHODS OF OBTAINING FLOW NET

1. Analytical method
2. Electrical flow analogy
3. Capillary flow analogy
4. Sand model
5. Graphical method.

Qu4 Match List-I (Flow type) with List-II (Flow characteristics) and select the correct answer using the codes given below the lists:

List-I	List-II
A. Transient flow	**1. Seepage flow is a function of time**
B. Turbulent flow	**2. Hydraulic gradient varies with square of velocity**
C. Steady-state flow	**3. Flow at low velocity**
D. Laminar flow	**4. Governing equation in 2-D is $k_x \frac{\partial^2 h}{\partial x^2} + k_y \frac{\partial^2 h}{\partial y^2} = 0$**

Codes

	A	B	C	D
(a)	1	2	4	3
(b)	3	2	1	1
(c)	1	2	3	4
(d)	2	1	4	3

Qu5 Consider the following statements:

1. **Constant head permeameter is best suited for determination of coefficient of permeability of highly impermeable soils.**
2. **Coefficient of permeability of a soil mass decreases with increase in viscosity of the pore fluid.**
3. **Coefficient of permeability of soil mas increases in temperature of the pore fluid.**

Which of these statements are correct?

(a) 1 and 2

(b) 1 and 3

(c) 2 and 3

(d) 1, 2 and 3

Codes:

	A	B	C
(a)	2	4	5
(b)	2	3	5
(c)	1	3	5
(d)	2	3	4

Qu9 Mach List-I (Type of strata below foundation) with List-II (Type of foundation movement) and select the correct answer using the codes given below the lists:

List-I	List-II
A. Sand	**1. Practically no movements**
B. Heterogeneous landfill	**2. Immediate settlements**
C. Black cotton soil	**3. Large relative settlements**
D. Hard rock	**4. Heaving of foundations**

Codes:

	A	B	C	D
(a)	2	3	1	4
(b)	2	3	4	1
(c)	3	2	1	4
(d)	3	2	4	1

Answers

1-(c), 2-(c), 3-(b), 4-(a), 5-(c), 6-(c), 7- (c), 8-(b), 9-(b)

- After the initial compression soil reaches into fully saturated state, further reduction in volume occurs due to expulsion of pore water i.e., water present in the soils
- Immediate settlement can occur if significant lateral strain takes place. This is due to deformation of soil under undrained condition. This immediate settlement can be calculated from elastic theory

PRIMARY CONSOLIDATION

- Primary consolidation occurs due to expulsion of excess pore water pressure generated due to decrease in total stress. It is a time dependent phenomenon.

For Example:

- When water table is lowered permanently in a structure overlaying a clay layer.

Magnitude of settlement due to 1º-consolidation depends on:

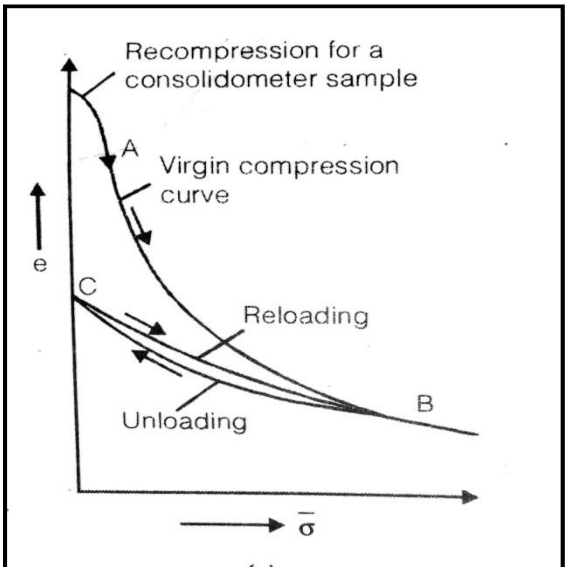
1. Compressibility of soil
2. Magnitude of stress increase
3. Thickness of soil layer
4. Permeability of soil

2º - CONSOLIDATION

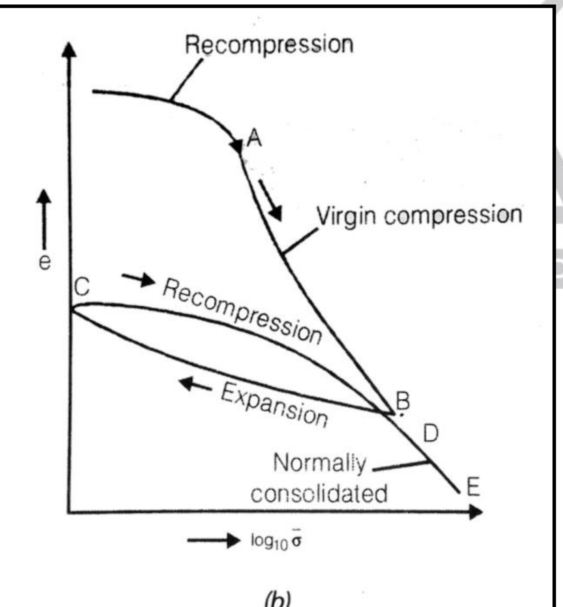
- Experimentally, it has been shown that compression of soil layer does not cease when excess pore water pressure has been completely dissipated to zero. It continues at a gradually decreasing rate under constant effective stress.
- 2°-Consolidation is thought to be due to gradual readjustment of clay particles into a more stable configuration following the structural disturbance caused by the decrease in void ratio.
- Rate of 2°-consolidation is thought to be controlled by highly viscous film of absorbed water, surrounding the clay mineral particles in soil

COMPRESSIBILITY CHARACTERISTIC

- Shape of the curve are related to the stress history of clays.



(a)



(b)

- Soil tend to retain the effect of stress changes that have taken place in their geological history in the form of their structure.
- A soil is said to be Normally-consolidated when the existing effective stress is the stress it has ever experienced in its stress history.

$$\frac{\Delta H}{H_o} = \frac{\Delta e}{1 + e_0}$$

$$\boldsymbol{\Delta H = \frac{C_\propto \cdot H_0}{1 + e_o} \log_{10} \frac{t_2}{t_1}}$$

Where, e_o = void ratio at the end of 1° consolidation.

H_0 = height at the end of 1° consolidation.

For NC soil,

$$\boldsymbol{C_\propto \text{ is } 4\% - 6\% \text{ of the value of } \frac{C_c}{1 + e_0}}$$

DETERMINATION OF C_c BY EMPIRICAL RELATIONSHIP

1 For undisturbed clay with medium sensitivity.

$$\left(\text{Sensitivity } = \frac{q_u \text{ undisturbed}}{q_u \text{ remoulded}} < 4\right)$$
$$C_c = 0.009(w_L - 10)$$

Where, w_L is in percentage (i.e. liquid limit)

2. For remoulded clay $C_c = 0.007(w_L - 7)$

3. $C_c = 1.15(e_0 - 0.35)$, where. e_o initial void ratio before consolidation.

4. $C_c = 1.15 \times 10^{-2} \times w_n$ for organic soil

   Where, w_n = natural water content in percent

5. $C_c = 0.37(e_0 + 0.003w_L + 0.0004\omega_n - 0.34)$

6. $C_c = 0.3(e_0 - 0.27)$ - inorganic cohesive soil, clayey silt, silty clay.

7. For most Normally Consolidated soil of medium sensitivity $C_c = 0.2 - 0.5$

   For organic clay $C_c > 4$

   For Peat $C_c \rightarrow 10 - 15$

These results are used for preliminary estimate of settlement.

TEST YOUR SELF

Qu3 Match List-I (Property) with List-II (Slope of the curve) and select the correct answer using codes given below the lists:

List-I	List-II
A. Coefficient of compressibility	1. Stress-deformation
B. Compression index	2. Stress-void ratio
C. Coefficient of subgrade modulus	3. Volume- pressure
	4. Log stress- void ratio

Codes:

	A	B	C
(a)	4	2	1
(b)	4	3	2
(c)	2	4	1
(d)	3	4	1

Qu4 Consider the following:

1. Initial consolidation

2. Primary consolidation

3. Secondary consolidation

The three stages which would be relevant to consolidation of a soil deposit includes

(a) 1,2 and 3

(b) 2, 3 and 4

(c) 1, 3 and 4

(d) 1, 2 and 4

Answer

1-(d), 2-(d), 3-(c), 4-(a)

- Cohesive soils may derive their strength from (ii) & (iii) i.e., "Friction" and "Cohesion".
- Highly plastic clays, however only have (iii) i.e., "Cohesion" as their source of shear strength.

STRESS AT A POINT-MOHR CIRCLE OF STRESS

- In a stressed soil mass, shear failure can occur along any plane.
- At any stressed point, there exists three mutually perpendicular planes on which there are no shearing stresses acting. These are known as principal planes. The normal stresses that act on these planes are called the principal stresses, the largest of these is called the major principal stress (σ_1), the smallest is called minor principal stress (σ_3), and the third one is called the intermediate principal stress (σ_2).

The corresponding planes are respectively designated as the major, minor and intermediate principal planes. However, the critical stress conditions occur only at σ_1 and σ_3.

- In a two dimensional stress system, the major and minor principal planes occur on horizontal and vertical directions as shown in fig. (a).
- If σ_1 and $\sigma_3$are known it can be shown that on any plane AB inclined at angle θ to the direction of major principal plane, the normal stress σ and the shear stress τ are given by:

$$\sigma = \left(\frac{\sigma_1 + \sigma_3}{2}\right) + \left(\frac{\sigma_1 - \sigma_3}{2}\right) \cos 2\theta$$

$$\tau = \left(\frac{\sigma_1 - \sigma_3}{2}\right) . \sin 2\theta$$

- Mohr demonstrated that these equations tend themselves to graphical representation. It can be shown that "the locus of stress coordinates (σ, τ) for all planes through a point is a circle, called the "Mohr circle of Stress" and,

$$\sigma = OE = OC + CE$$

$$= \frac{\sigma_{1+}\sigma_3}{2} + \frac{\sigma_{1-}\sigma_3}{2}.\cos 2\theta$$

$$\tau = DE = \frac{\sigma_1 - \sigma_3}{2}.\sin 2\theta$$

- Point A on the Mohr Circle is a unique point called the 'pole' or "Origin of planes".
- The property of pole is: "a line drawn through the pole intersects the Mohr circle at a point which represents the state of stress on a plane which has the same inclination in space as the line itself ".
- This property can be utilized in locating the pole in a situation where the state of stress (σ, τ) on a certain plane is known.

Important Relationships Obtained from the Mohr Circle

1. Maximum shearing stress occurs on planes inclined at 45° to principal planes.

$$\tau_{max} = \frac{\sigma_1 - \sigma_3}{2} (at\ \theta = 45°)$$

2. The normal stresses on plane of maximum shear are equal to each other and they are given by

$$\sigma_{1,2} = \frac{\sigma_1 - \sigma_3}{2}$$

3. The sum of normal stresses on mutually perpendicular planes is a constant i.e,

$$\sigma_1 + \sigma_3 = \sigma_{n1} + \sigma_{n2} = \text{constant}$$

4. When the principal stresses are equal to each other, the radius of the Mohr's circle becomes zero which means that shear stresses vanish on all planes. Such a point is called Isotropic point.

- The second problem is that at failure, the shear stress on the failure plane defines the shear strength of material.

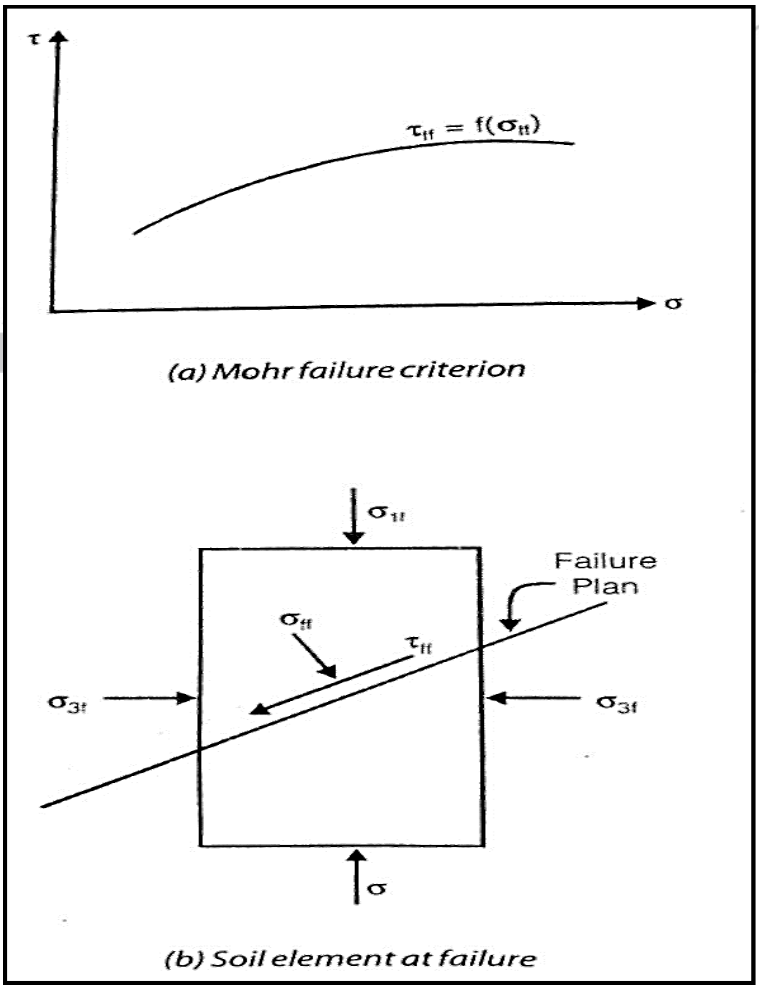
2. Mohr Failure Criterion

- Mohr failure theory is based on the hypothesis that materials fail when the shear stress on the failure plane at failure reaches a value which is unique function of the normal stress on the plane, i.e.

$$\tau_{ff} = f(\sigma_{ff})$$

Where, $\tau = shear\ stress\ and\ \sigma$ = normal stress.

The first subscript refers to ‘failure plane’ and the second subscript denotes- “at failure”.



(a) Mohr failure criterion

(b) Soil element at failure

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Reinforced Cement Concrete

Education's purpose is to
replace an empty mind with an open one.

Malcolm Forbes

The content of this book covers all PSC exam syllabus such as MPSC, RPSC, UPPSC, MPPSC, OPSC etc.

Where,

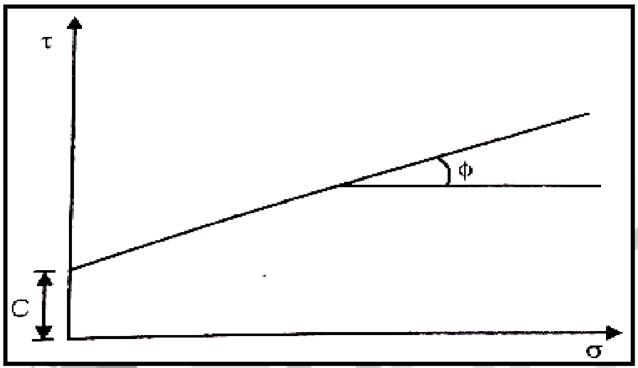
τ_f = Shear strength of soil

C = Apparent cohesion

σ = Normal stress on the plane of rupture

ϕ = Angle of internal friction

- 'c' and $'\phi'$ are also referred to as shear strength parameters of the soil. c and ϕ are not inherent properties of the soil. These are, infact, related to the type of test and the condition under which these are measured.



- Mohr coulomb failure criterion can be expressed in the form of

$$\tau_{ff} = C + \sigma_{ff} \tan\phi$$

- The above criterion affords a procedure by which the stresses on the failure plane at failure can be determined.

- Also angle of failure can be expressed in terms of angle of shearing resistance ϕ,

$$2\theta_f = 90° + \phi$$

$$\Rightarrow \theta_f = 45° + \frac{\phi}{2}$$

Field Tests

1. Vane Shear Test
2. Penetration Test

- A cohesionless or a coarse-grained soil may be tested for shearing strength either in dry condition or in saturated condition.
- A cohesive or fine grained soil is usually tested in the saturated condition.

Based on Drainage Conditions Shear Tests on Saturated Soils are Further Classified as

Unconsolidated Undrained Test (UU)

- Drainage not permitted at any stage
- 5 to 10 minutes are required for whole test, thus, called Quick Test
- Not suitable for sandy soils.
- Suitable for clayey soils.

Consolidated Undrained (CU)

- Drainage is allowed during application of normal stress
- No drainage is allowed during application of shear stress
- Also called consolidated Quick Test

Consolidated Drained (CD) Test

- Drainage is permitted at every stage.
- 4 to 6 weeks are required to complete the test, so, it is also called "slow test"
- Suitable for cohesionless soil
- Not suitable for clayey soils.

Triaxial Test

This test has several merits, like:

- Most versatile of all the shear testing methods, as it can be performed on all soils, be it sand, silt or clay.
- Pore water measurements can be done accurately
- Volume changes can also be measured.
- There is no rotation of the principal stresses during the test.
- Failure plane is not predetermined Specimens can fail on any weak plane or can simply bulge
- The stress distribution on the failure plane is fairly uniform.

Principle

- The soil specimen is subjected to three compressive stresses in mutually perpendicular directions, one of the three stresses being increased until the specimen fails in shear. The desired 3-dimensional stress system is achieved by an initial application of all-round fluid pressure or confining pressure through water. While this confining pressure is kept constant throughout the test, axial or vertical loading is increased gradually and at a uniform rate. The axial stress thus constitute the major principal stress and the confining pressure acts in the other two principal directions, the intermediate and minor principal stresses being equal to the confining pressure. The principle is shown in following figure:

$$A = A_O.\frac{(1 \pm \epsilon_V)}{(1 - \epsilon_L)}$$

Where, ϵ_V = Volumetric strain

ϵ_L = Linear strain

- In general $\epsilon_V = 3\epsilon_L$
- If drainage is not permitted, there will be no volume change, hence ϵ_V = O

$\therefore$ $A = \frac{A_O}{1-\epsilon_L}$ for CU test

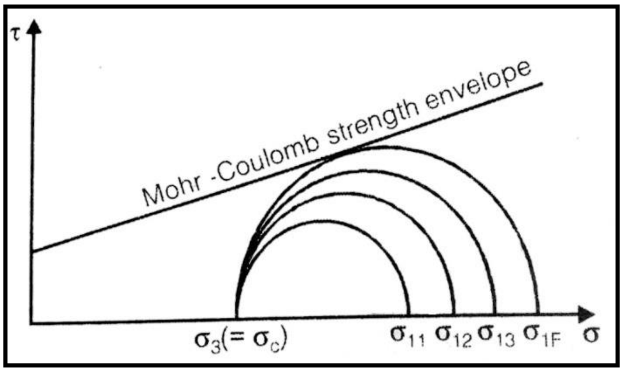
Where, $\frac{1}{1-\epsilon_L}$ = correction factor

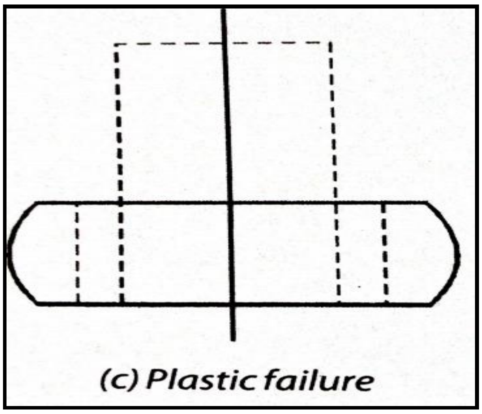
- Now there are two cases of Mohr's circle:

Case 1: When $\sigma_3$is kept constant and $\Delta\sigma$ is increased gradually till failure, we have the mohr circle for C - ϕ soil will be like:

The cell pressure, σ_c, which is also the minor principal stress (σ_3) is constant and σ_{11}, σ_{12}, ...σ_{1f} are the major principal stresses at different stages of loading and at failure. The Mohr circle at failure is tangential to Mohr coulomb strength envelope.

Now important relationships between major and minor principal stresses at failure can be obtained from Mohr circle:





(a) Brittle failure

- Found in dense sands & in over consolidated clays
- It has well defined shear plane.

(b) Semi plastic or semi-brittle failure

- Found in silts and in C - Ø soils
- It has shear cones and some lateral bulging.

(c) Plastic failure

- In normally consolidated clays and in loose sands
- It has well expressed lateral bulging
- It has no specific stage to pin-point failure, so failure is assumed at 20% strain

Unconfined Compression Test

- This is a special case of a triaxial compression test where confining pressure (σ_3) is zero
- Since the specimen is laterally unconfined, the test is known as unconfined compression test.
- The axial or vertical compressive stress is the major principal stress and the other two principal stresses are zero.

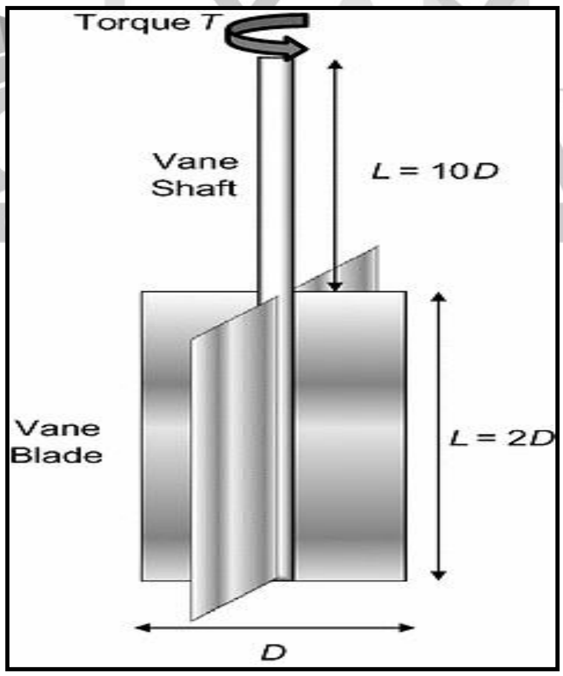
Hence shear strength,

$$S = c + \sigma \tan \emptyset = \frac{q_U}{2} + 0$$

$$\Rightarrow \qquad S = \frac{q_U}{2} \text{ for clays}$$

Vane Shear Test

- Suitable for sensitive clays.
- The vane is pushed gently into the soil upto the required depth at the bottom of a borehole and torque is applied gradually to the upper end of the rod until the soil fails in shear, due to the rotation of the vane. The torque is measured by the angle of twist. Shear failure occurs over the surface and the ends of a cylinder having a diameter 'd' equal to the diameter of the vane.
- The vane and shear stress distribution are as shown below:



(a) Shear Vane

PORE PRESSURE PARAMETERS

- Pore pressure parameters are empirical coefficients which are used to express the response of pore pressure to changes in total stress under "Undrained conditions".
- This understanding of changes in total stress is important because shear strength of soil is a function of effective stress rather than total stress and effective stress is related directly to pore pressures developed in the soil.
- These were introduced by skempton.
- These parameters are very useful in field problems where pore pressures that are induced consequent to change in total stress may have to be computed for e.g., in case of construction of earth embankment over a soft clay deposit.
- Parameter B is defined as,

$$B = \frac{\text{Pore water pressure developed in soil}}{\text{Increase in cell pressure}}$$

$$= \frac{\Delta U_c}{\Delta \sigma_3} \text{ (in case of triaxial test)}$$

B is also given by, $B = \frac{1}{1+n\cdot\frac{C_v}{C_c}}$

Where,

n = porosity

C_v= Coefficient of consolidation

C_c = Coefficient of compression

∴ B = 1 for saturated soils

and B = 0 for dry soils

- B varies from 0 to 1 depending on the degree of saturation S. The relationship between S and B is not linear.

CLEAR YOUR CONCEPT

Qu1 Which one of the following statement provides the best argument that direct shear tests are not suited for determining shear parameters of a clay soil?

(a) Failure plane is not the weakest plane.

(b) Pore pressures developed cannot be measured.

(c) Satisfactory strain levels cannot be maintained.

(d) Adequate consolidation cannot be ensured.

Qu2 Consider the following statements related to tri-axial test:

1. Failure occurs along predetermined plane.

2. Intermediate and minor principal stresses are equal

3. Volume change can be measured

4. Field conditions can be simulated.

Which of these statements are correct?

(a) 1, 2 and 3

(b) 1, 2 and 4

(c) 1, 3 and 4

(d) 2, 3 and 4

Qu3 For a fully saturated clay Skempton's pore pressure parameter B is:

(a) Zero

(b) Between zero & 1

(c) 1

(d) More than 1

Qu6 A vane 20 cm long and 10 cm in diameter was pressed into a soft marine clay at the bottom of a bore hole. Torque was applied gradually and failure occurred at 1000 kg cm. The cohesion of the clay in kg / cm^2 is:

(a) $\frac{1}{\pi} \times \frac{6}{7}$

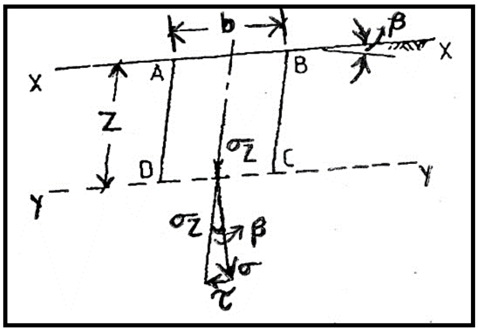
(b) $\frac{1}{\pi} \times \frac{5}{7}$

(c) $\frac{1}{\pi} \times \frac{4}{7}$

(d) $\frac{1}{\pi} \times \frac{3}{7}$

Answers

1-(a), 2-(d), 3-(c), 4-(b), 5-(c), 6-(a)



Considering unit thickness Volume of the prism V = z b cosβ

and weight of the prism W = γz h cosβ

Vertical stress on YY due to self-weight,

$$\boldsymbol{\sigma_z = \frac{W}{b} = \gamma z \cos\beta} \qquad \text{...(i)}$$

This vertical stress can be resolved into the following two components:

$$\boldsymbol{\sigma = \sigma_z \cos\beta = \gamma z\, \mathrm{b}\cos^2\beta} \qquad \text{...(ii)}$$

and,

$$\boldsymbol{\tau = \sigma_z \sin\beta = \gamma z\ \cos\beta \sin\beta} \qquad \text{...(iii)}$$

Failure will occur if the shear stress τ exceeds the shear strength τ_f of the soil. The factor of safety against such failure is given by.

$$\boldsymbol{F = \frac{\tau_f}{\tau}} \qquad \text{...(iv)}$$

(1) Cohesionless Soils

From Coulomb's equation, we have

$$\tau_f = \mathrm{c} + \sigma \tan \phi$$

For a cohesionless soil, c = 0.

$$\tau_f = \sigma \tan \phi$$

Where,

S_n, is a dimensionless quantity known as the stability number and is given by

$$\boldsymbol{S_n} = \frac{\mathbf{c}}{\boldsymbol{\gamma H_c}} \qquad \ldots(\text{x})$$

If a factor of safety F_c, is applied to the cohesion such that the mobilised cohesion at a depth H is,

$$\boldsymbol{c_m} = \frac{\mathbf{c}}{\boldsymbol{F_c}} \qquad \ldots(\text{xi})$$

Then,

$$\boldsymbol{S_n} = \frac{\boldsymbol{c_m}}{\boldsymbol{\gamma H}} = \frac{\mathbf{c}}{\boldsymbol{F_c \gamma H}} \qquad \ldots(\text{xii})$$

From eqns. (x) and (xii), we get,

$$\frac{c}{\gamma H_c} = \frac{c}{F_c \gamma H}$$

$$\text{or,} \boldsymbol{F_c} = \frac{\boldsymbol{H_c}}{\boldsymbol{H}} = \boldsymbol{F_H}$$

Hence, the factor of safety against cohesion, F_c, is the same as the factor of safety with respect to height, F_H.

STABILITY OF FINITE SLOPES

- In case of slopes of limited extent, three types of failure may occur. These are: face failure, toe failure and base failure.
- Various methods of analysing the failure of finite slopes are discussed below.

Swedish Circle Method

In this method, the surface of sliding is assumed to be an arc of a circle.

EXAM ACHARYA

GPSC - CIVIL

Solid Mechanics

"Education is the most Powerful Weapon which you can use to change the world."

A.P.J. Abdul Kalam

The content of this book covers all PSC exam syllabus such as MPSC, RPSC, UPPSC, MPPSC, OPSC etc.

GPSC - CIVIL Solid Mechanics

Considering unit thickness of the soil mass,

$$W = A\times 1\times \gamma = A\,\gamma$$

Where,

γ = Unit weight of the soil

A = Cross-sectional area of the sector AS_1CB.

- The area A can be determined either by using a planimeter or by drawing the figure to a proper scale on a graph paper and counting the number of divisions of the graph paper covered by the area.

Now, disturbing moment, $M_d = W\times d$

Where,

d = lever arm of W with respect to O.

- The distance d may be determined by dividing the area into an arbitrary number of segments of small width and taking moments of all these segments about O.

Thus, the factor of safety against slope failure,

or,

$$\mathbf{F}= \frac{M_R}{M_D} = \frac{cR^2\theta}{Wd} \qquad \ldots(\text{xiv})$$

- A number of trial ship circles are chosen and the factor of safety with respect to each of them is computed. A curve is then plotted to show the variation of factor of safety with various slip circles. The slip circle corresponding to the minimum factor of safety is identified from this curve. This is the potential slip surface, and the corresponding factor of safety is the factor of safety against failure of the slope AB

(iii) Lateral thrust from adjacent slices. E_L and E_R. In simplified analysis it is assumed that $E_L = E_R$. Hence the effects of these two forces are neglected.

(iv) Soil reaction R across the arc. According to the laws of friction, when the soil is about to slide, R will be inclined to the normal at an angle ϕ.

(v) The vertical stresses, V_L and V_R, which are equal and opposite to each other and hence need not be considered.

- The weight W is resolved into a normal component N and a tangential component T.
- For some of the slices T will enhance the failure, for the others it will resist the failure. The algebraic sum of the normal and tangential components are obtained from

$$\sum \mathbf{T} = \sum(\mathbf{W}\ \sin\alpha)$$

And,

$$\sum \mathbf{N} = \sum(\mathbf{W}\ \cos\alpha)$$

Now, driving moment,

$$M_D = \mathbf{R}\sum\mathbf{T}$$

and, Restoring moment, $M_R = R[c\sum\Delta l + \sum N\tan\phi]$

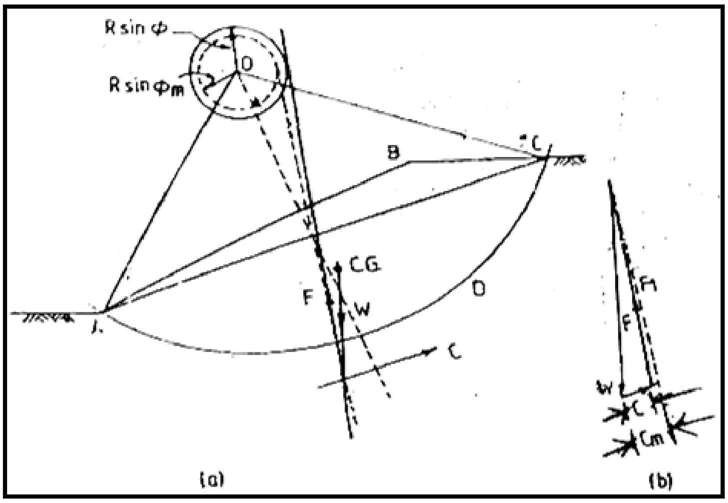
But, $\sum\Delta l$ = total length of arc $AS_1C = R\theta$

$$M_R = R[cR\theta + \sum N\tan\phi]$$

Factor of safety,

$$\mathbf{F} = \frac{M_R}{M_D} = \frac{\mathbf{R}[\mathbf{cR}\theta + \sum \mathbf{N}\tan\phi]}{\mathbf{R}\sum\mathbf{T}}$$

or,



$$\Sigma T = \Sigma\ (W \sin \alpha)$$

$$C'_m = \frac{c'}{F_c}$$

Where,

F_c = factor of safety with respect to cohesion.

- The cohesive force is given by,

$$C = c'_m L_c = \frac{c' L_c}{F_c}$$

- But, summing up the moments of all forces about O and equating to zero, we get,

$$C \times L_a \times R = C \times L_a \times a$$

Where,

a = Perpendicular distance of line of action of C from the centre of the slip circle.

$$a = \frac{L_a}{L_c} \times R$$

(iii) The other force is the soil reaction F, which is assumed to be tangential to the friction circle

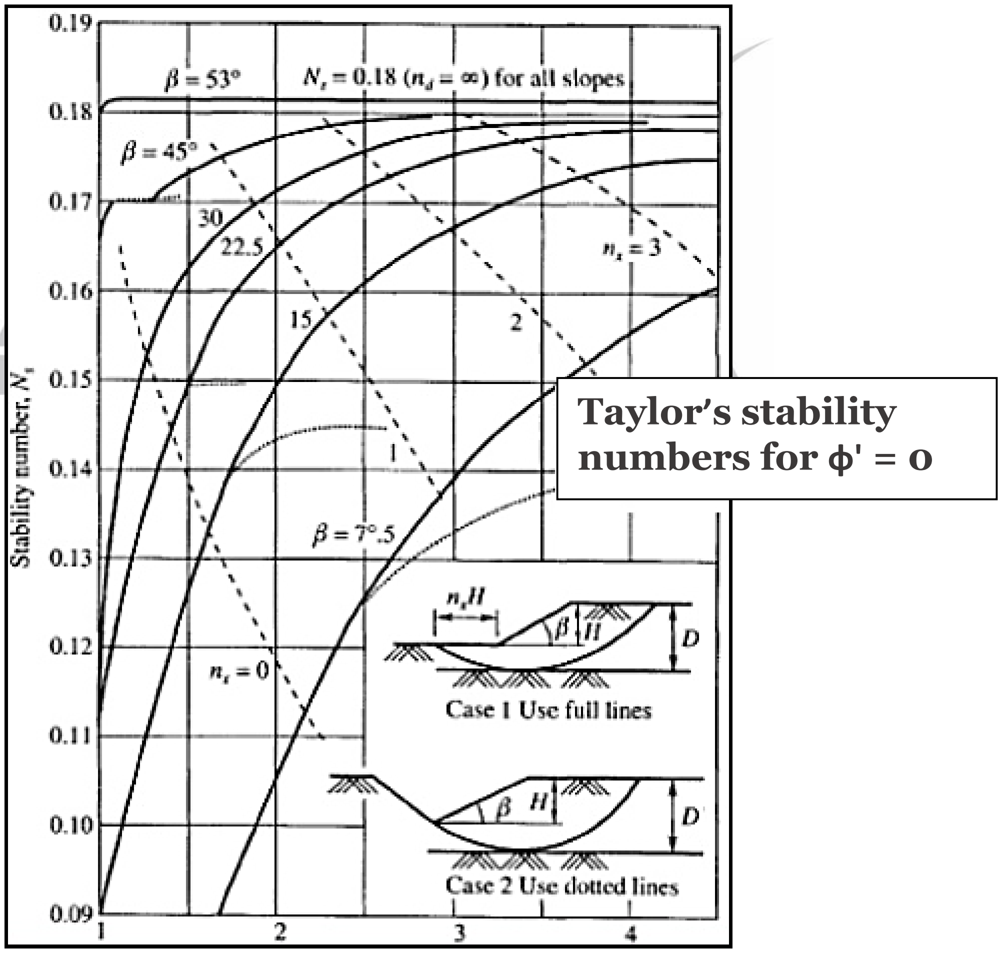
Taylor's Stability Number

- Taylor carried out stability analysis of a large number of slopes having various heights, slope angles and soil properties. On the basis of the results, he proposed a simple method by which the factor of safety of a given finite slope can be easily determined with reasonable accuracy.

- Taylor introduced a dimensionless parameter, called Taylor's Stability Number, which is given by

$$\mathbf{S_n} = \frac{\mathbf{c}}{F_c \gamma H}$$

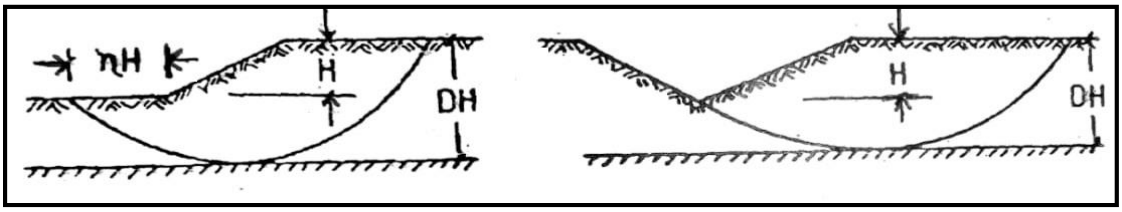
The value of S_n may be obtained from Figure below.



- The stability numbers are obtained for factor of safety w.r.t cohesion, while the factor of safety w.r.t. friction, F_ϕ is initially taken as unity.

- The figure also consists of a third set of curves, shown with broken lines, for various values of n, where n represents the distances x of the rupture circle from the toe, as illustrated in Fig. and is given by,

$$\mathbf{n} = \frac{x}{H}$$



3. Stability of U/S and D/S slopes during construction

Which of these statements are correct?

(a) 1 and 2

(b) 1 and 3

(c) 2 and 3

(d) 1, 2 and 3

Qu5 For stability analysis of slopes of purely cohesive soils, the critical centre is taken to lie at the intersection of:

(a) The perpendicular bisector of the slope and the locus of the centre.

(b) The perpendicular drawn at one-third slope from the toe and the locus of the centre.

(c) The perpendicular drawn at two-third slope from the toe and the locus of the centre

(d) Directional angles

Qu6 A slope is to be constructed at an angle of 30° to the horizontal from a soil having the properties, C = 15 kN/m^2, ∅ = 22.5°, γ = 19KN/ m^3. Taylor's stability number is 0.046. If a factor of safety (with respect to cohesion) of 1.5 is required, then the safe height of the slope will be:

(a) 25.8 m

(b) 19.1 m

(c) 17.2m

(d) 11.5 m

Qu9 Consider the following statements:

1. **Method of slices overestimates the value of factor of safety**
2. **Exact value of factor of safety is obtained in $\emptyset_r = 0$ analysis.**
3. **Reduction in shearing resistance in embankment is caused by tension crack at the top of it.**
4. **Plane surface of failure never occurs in saturated soils.**

Which of the statements given above are correct?

(a) 1 and 4 only

(b) 2 and 3 only

(c) 3 and 4 only

(d) 1 and 2 only

Qu10 The factor of safety with respect to cohesion of a clay slope laid at 1 in 2 to a height of 10 m, if the angle of internal friction $\emptyset = 10°$, C = 25 kN/m^2 and γ = 19 kN/m^3

(a) 1.06

(b) 2.06

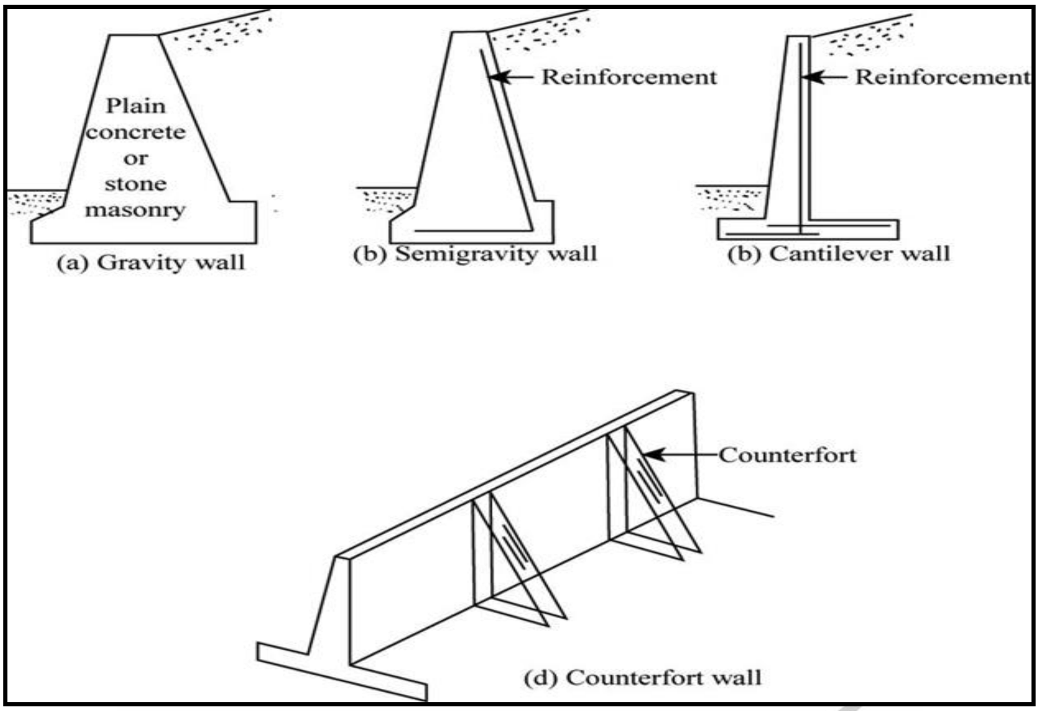
(c) 3.06

(d) 4

Answers

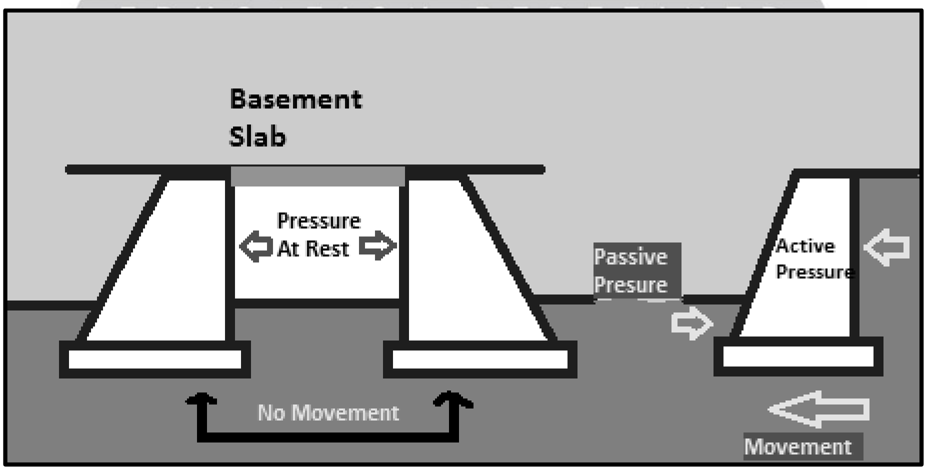
1-(a), 2-(c), 3-(c), 4-(b), 5-(d), 6(d), 7-(c), 8-(c), 9-(c), 10-(b)

Some Types of Retaining Walls



TYPES OF LATERAL EARTH PRESSURE

- Lateral earth pressure can be divided into 3 categories, depending upon the movement of retaining wall with respect to back fill soil.



1 Earth Pressure At Rest - wall does not move at all

2 Active Earth Pressure - wall moves s away from the backfill soil.

3 Passive Earth Pressure - wall moves towards the backfill soil.

Note

- Soil is not a perfectly elastic material. Hence, the above expression will not give the correct result.

$$\frac{\sigma_X}{E} - \mu\left(\frac{-\sigma_Y}{E}\right) - \mu\left(\frac{-\sigma_Z}{E}\right) = 0$$

We know that,

$$\sigma_X = \sigma_Y$$

$$\frac{\sigma_X}{E}(1-\mu) = \frac{\mu\sigma_Z}{E}$$

$$\boldsymbol{\sigma_X = \frac{\mu}{(1-\mu)}\sigma_z}$$

$$\boldsymbol{\sigma_X = K_o.\sigma_z}$$

Where, k_o = Earth pressure coefficient at Rest.

- For a perfectly cohesion-less soil (c = 0)

$$\boldsymbol{K_0 = (1 - \sin\emptyset)}$$

- If the soil is normally consolidated, (N.C. soil):

$$\boldsymbol{K_0 = 0.19 + 0.233\log_{10}(I_P)}$$

Where, I_P= Plasticity index

- For over consolidated soil (OC soil)

$$\boldsymbol{K_{0(OC)} = K_{0(NC)}\sqrt{O.C.R}}$$

$$\textbf{O.C.R.}= \left(\frac{\sigma}{\sigma_0}\right)$$

Where,

σ_0 = Pre consolidation stress

Rest condition) and this pressure developed on the wall when the soil is on the verge of failure is called Active Earth pressure.

Passive Earth Pressure

- Pressure developed an wall when wall moves towards the soil, a block of soil has the tendency to move up and inside. Hence, shear stress is mobilised on the soil.
- Because of the component of mobilised shear in the direction of wall, pressure on the wall increases beyond the pressure at rest condition and this is called Passive Earth Pressure.

EARTH PRESSURE THEORIES

- There are two classical theories of Earth Pressure

(a) Rankine's theory (1857)

(b) Coulomb's theory (1776)

Rankine theory came later and is considered to be simpler than Coulomb's theory.

Rankine Theory

- Rankine's theory considered stress in soil mass when it attains plastic equilibrium. Here, by plastic equilibrium we infer that every point in the soil mass experience shear failure, under the effect of shear stress developed.

Assumptions in Rankine's Theory

1. Soil is semi-infinite, homogeneous, isotropic, dry and cohesion less.
2. Soil is in a state of plastic condition at the time of active and passive pressure generation.
3. The backfill soil is horizontal.
4. Back of wall is vertical and smooth.

Where,

$$K_a = \frac{1-\sin\emptyset}{1+\sin\emptyset}$$

K_a = Coefficient of active earth pressure

$$P_a = K_a\sigma_z - 2C\sqrt{K_a}$$

But for cohesionless soil,

$$C = 0$$

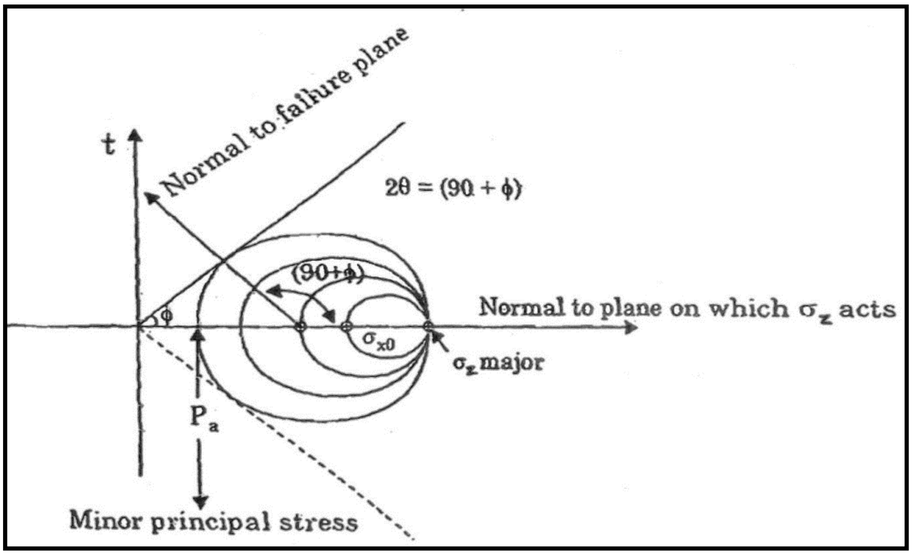
$$P_a = K_a\sigma_z$$

also,

$$K_a = \left(\frac{1-\sin\emptyset}{1+\sin\emptyset}\right) = \tan^2\left(45 - \frac{\emptyset}{2}\right)$$

After drawing the Mohr circle for two state of stress develope i.e., σ_1 & σ_3 we can show that the two plane of failures are inclined at

1. $45° + \frac{\emptyset}{2}$ with the horizontal (major principal plane)



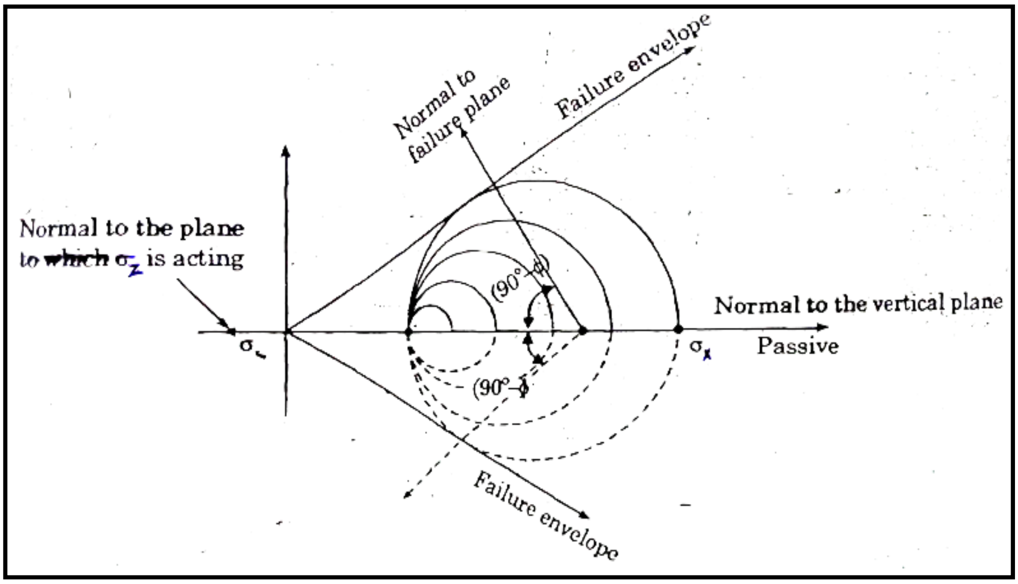
GPSC - CIVIL

Structural Analysis

"All of us do not have Equal Talent.
But, all of us have an Equal Opportunity
to Develop our Talents."
A.P.J. Abdul Kalam

The content of this book covers all PSC exam syllabus such as MPSC, RPSC, UPPSC, MPPSC, OPSC etc.

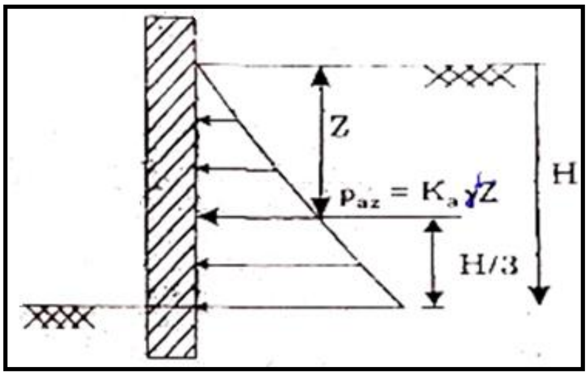
(1) Failure plane inclined at (45º – $\emptyset/2$) with the horizontal (minor principal plane)



Note:
$K_a = \frac{1-\sin\emptyset}{1+\sin\emptyset} = \tan^2(45 - \emptyset/2)$
$K_p = \frac{1+\sin\emptyset}{1-\sin\emptyset} = \tan^2(45 + \emptyset/2)$
$K_a = \frac{1}{K_P}$

VARIOUS CASES OF EARTH PRESSURES

Case I: cohesionless soil on a vertical smooth wall



Similarly passive earth pressure, per unit length of wall

$$P_{pz} = K_p\gamma z$$

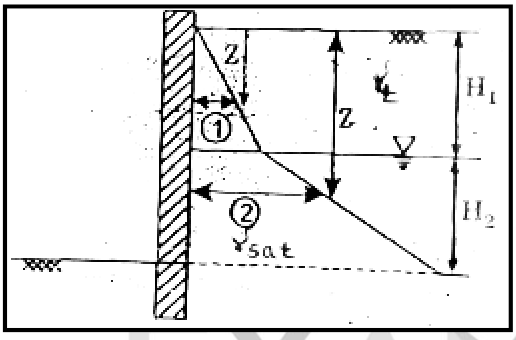
- Similarly

$$\mathbf{F_p} = \left(\frac{K_P.\gamma_{sub}.H^2}{2} + \frac{\gamma_w.H^2}{2}\right)$$

Line of action of F_a & F_p will be at H/3 distance from base of wall.

Case III—Partialy Submerged cohession less soil on vertical smooth wall

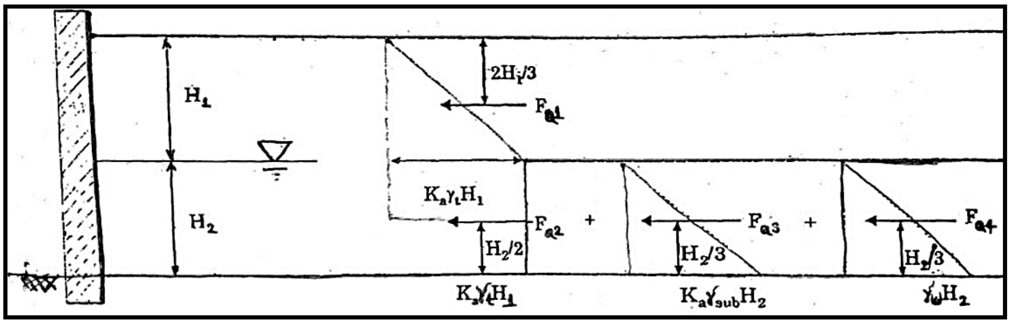
If a soil is partially submerged as in the case shown below, then active earth pressure is calculate as shown below



At Point (1), $P_a = K_a\gamma_t Z$

At Point (2), $P_a = K_a\underbrace{[\gamma_t H_1 + \gamma_{sub}(Z - H_1)]}_{\text{effective stress}} + \gamma_w\,(Z - H_1)$

Calculation of Force due to active earth pressure



$F_{a1} = \left(\frac{1}{2}.K_a.\gamma_t.H_1^2\right)$ acting at $2H_1/3$ from top

$F_{a2} = (K_a.\gamma_t.H_1.H_2)$ acting at $H_2/2$ from base of wall

$F_{a3} = \left(\frac{K_a.\gamma_{sub}.H_2^2}{2}\right)$ acting at $H_2/3$ from base of wall

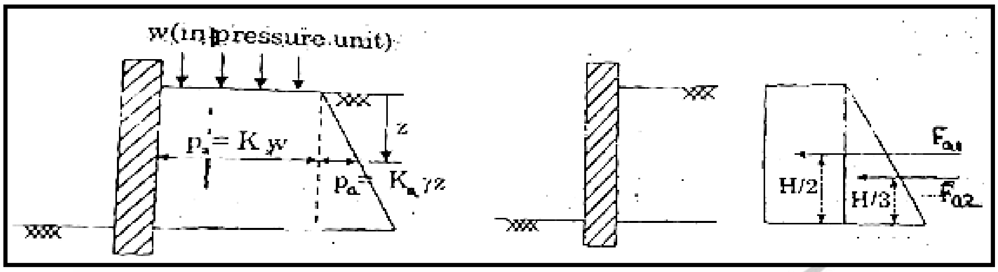
$F_{a_1} = \frac{1}{2}.K_{a_1}.\gamma_1.H_1^2$

$F_{a_2} = (K_{a_2}.\gamma_1.H_1.H_2)$

$F_{a_3} = \frac{1}{2}.K_{a_2}.\gamma_2.H_2^2$

Resultant force location from bottom = $\left[\frac{Fa_1\left(H_1+H_2-\frac{2H_1}{3}\right)+Fa_2\left(\frac{H_2}{2}\right)+Fa_3\left(\frac{H_2}{3}\right)}{(Fa_1+Fa_2+Fa_3)}\right]$

Case V- Soil with surcharge load:



$F_{a_1} = K_{a_1} wH$

$F_{a_2} = K_{a_1} \frac{\gamma H^2}{2}$

Resultant force location from bottom = $\frac{Fa_1 \times \frac{H}{2} + Fa_2 \times \frac{H}{3}}{Fa_1 + Fa_2}$

COULOMB'S THEORY OF EARTH PRESSURE

Assumptions

1. The backfill is dry, cohesionless, Isotropic.
2. Back of wall can be inclined.
3. Backfill can be inclined.
4. There would be friction between the wall and the soil.
5. Failure plane in assumed to be a plane surfaces (actually curved).
6. Sliding wedge is assumed to be a rigid body

- By assuming various trial wedges at different trial angle (λ) the value of P will be calculated.
- For Active Earth Pressure, P is the highest value obtained for various trial wedges.
- For Passive Earth Pressure, P will be the minimum value that will cause the wedge to move.

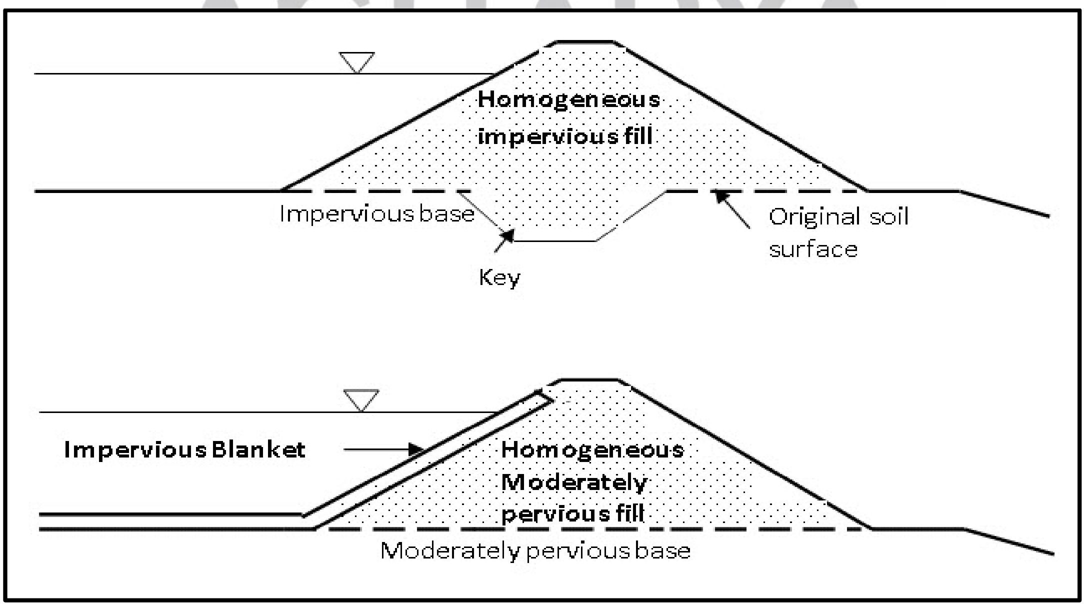
COFFER DAM

A Coffer Dam is a temporary structure constructed usually in a river, lake. etc., to keep the working area dry for construction of other structures. After the construction of coffer dam, the area is dewatered by pumping

Types of Coffer Dam

Earth Embankments

- This the simplest type of coffer dam. This type of coffer dam is made by placing the fill at a suitable location at a stable slope.

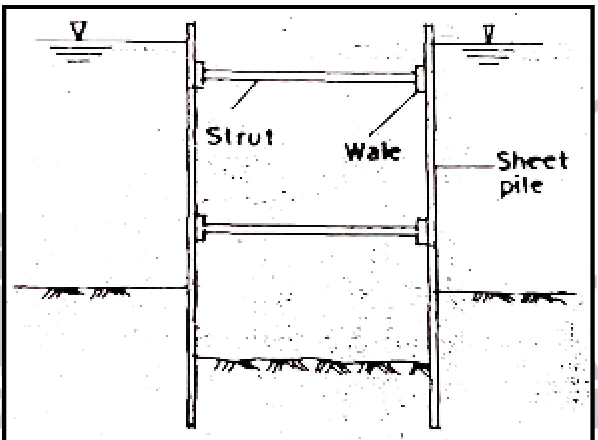


Double Wall Coffer Dams

- These coffer dams consist of two lines of sheeting, tied together with the space between the sheeting filled with soil.
- Two rows of sheeting may be connected to each other by a combination of wales and tie rods.
- Double-walled coffer dams are suitable for moderate heights.

Braced Coffer Dams

- A braced coffer dam is formed by driving vertical sheeting into the ground. The Vertical sheeting is held in position through horizontal beams called wales.



Braced Coffer Dam

- Braced coffer dams are more commonly used as land coffer dams for supporting soil during excavation.
- They are economical for small to moderate heights.

Cellular Coffer Dams

- A cellular coffer dam is made by driving sheet piles to form a series of cells which are later filled with a suitable soil.
- The cells are interconnected for water-tightness and are self stabilising against lateral pressure of water and soil

CLEAR YOUR CONCEPT

Qu1 A retaining wall with a vertical smooth back is 8m high. It supports a cohensionless soil (γ = 19kN/m^3, ∅ = 30°). The surface of the soil is horizontal. The thrust on the wall is:

(a) 202.7 kN/m

(b) 192.7 kN/m

(c) 182.7 kN/m

(d) 172 kN/m

Qu2 Match List-I with List-II and select the correct answer using the codes given below the lists:

List-1	List-11
A. Active Pressure	**1. Wall moves towards backfill**
B. Passive Pressure	**2. No movement of wall**
C. Earth Pressure	**3. Wall moves away from backfill**

Codes:

	A	B	C
(a)	1	2	3
(b)	2	3	1
(c)	3	2	1
(d)	3	1	2

Qu6 Given that for a soil backfill, K_A = coefficient of active earth pressure, K_P = coefficient of passive earth pressure and K_o = coefficient of earth pressure at rest, which one of the following represents the correct relationship between K_A, K_O and K_P?

(a) $K_o = K_P / 2$

(b) $K_o = (K_A+K_P)/2$

(c) $K_o = (K_P - K_A)/2$

(d) None of the above

Qu7 A vertical retaining wall retains a c-∅ backfill and carries a surcharge of uniform intensity 'q' and unit area. The depth Z_0 from the top of the wall where the active earth pressure is zero is given by ($\alpha = 45° + \emptyset/2$ and γ = unit weight of the soil)

(a) q/γ

(b) $\frac{2c}{\gamma}\tan\alpha - \frac{q}{\gamma}$

(c) $\frac{2c}{\gamma}\tan\alpha + \frac{q}{\gamma}$

(d) $\frac{2c}{\gamma}\tan\alpha$

<u>TEST YOUR SELF</u>

Qu8 A vertical retaining wall retains a horizontal backfill of dry, homogeneous and isotropic cohesionless soil. If the angle of wall friction 'δ' equals the angle of internal friction '∅', then the coefficient of active earth pressure 'K_a' as per Coulomb's wedge theory would be

(a) $\frac{\cos\emptyset}{1+\sin\emptyset}$

(b) $\frac{\cos\emptyset}{(1+\sin\emptyset)^2}$

(c) $\frac{\cos\emptyset}{(1+\sqrt{2}\sin\emptyset)}$

(d) $\frac{\cos\emptyset}{(1+\sqrt{2}\sin\emptyset)^2}$

CHAPTER – 11

BEARING CAPACITY

MODES OF FAILURE OF A STRUCTURE

A structure when loaded may fail in the following two ways

(1) ***Failure due to shear:*** When the supporting power of soil is less than structural load at the foundation level

(2) ***Failure due to Excessive Settlement:***The settlement of a foundation, especially the differential settlement, must be within the permissible limit. Excessive settlement may affect the utility of the structure, spoil the appearance of the structure and in some cases may even cause damage to the structure.

BEARING CAPACITY

The load carrying capacity of foundation soil or rock which enables it to bear and transmit loads from a structure

OR

The load supporting power of a soil without shear failure is known as bearing capacity.

Important Definitions

(a) ***Gross pressure intensity*** (q_g) :

It is the total pressure at the base of the footing due to the weight of the weight of the superstructure, self weight of the footing and weight of the earth fill.

(b) ***Net pressure Intensity*** (q_{net}) :

The difference between gross pressure and overburden pressure, γD_1 at the base of the footing is called net pressure intensity.

$$q_{net} = q_g - \bar{\sigma}$$

$$= q_g - \gamma' D_f$$

$$q_s = q_{ns} + \gamma' D_f$$

$$\downarrow$$

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

FACTORS AFFECTING BEARING CAPACITY

1. Nature of soil and its physical and engineering properties.
2. Nature of foundation and other details such as the size, shape, depth and rigidity of the structure.
3. Location of the ground water table relative to the foundation level.
4. The total and differential settlements that the structure can withstand without functional failure.
5. Initial stresses on the soil if any.

METHODS FOR DETERMINATION OF BEARING CAPACITY

1. Bearing capacity from various building codes
2. Analytical methods
3. Plate bearing tests
4. Penetration tests
5. Model tests and prototypes tests (Housel's approach)
6. Laboratory tests.

Analytical Methods

These are based on

(a) Theory of elasticity

(b) Classical earth pressure theory

(c) Theory of plasticity.

- If 'S$_1$' and 'S$_2$' are settlements brought about by two bearing areas of similar shape of different sizes, 'A$_1$' and 'A$_2$' respectively, with equal contact pressure.

Then, $$\frac{S_1}{S_2} = \sqrt{\frac{A_1}{A_2}}$$

K = Shape coefficient or influence value, depending on size, shape and rigidity of the slab

q = Net pressure applied from the slab on to the soil

A = Area of the bearing slab

E = Modulus of elasticity of soil

μ = Poisson's ratio for the soil

Rankine's Earth Pressure Theory

(only for Ø soils)

- Rankine used the relationship between principal stresses at limiting equilibrium conditions of soil elements

Rankine's method for bearing capacity of a footing

- When the load on the footing increases and approaches to a value 'q_u' a state of plastic equilibrium is reached under the footing. The value of 'q_u' is given by

$$q_u = \gamma D_f \left(\frac{1+\sin\emptyset}{1-\sin\emptyset}\right)^2$$

$$D_{f\ min} = \frac{q_u}{\gamma}\left(\frac{1-\sin\emptyset}{1+\sin\emptyset}\right)^2$$

Limitations

(i) Base of the footing is considered smooth.

(ii) Size of the foundation is not accounted.

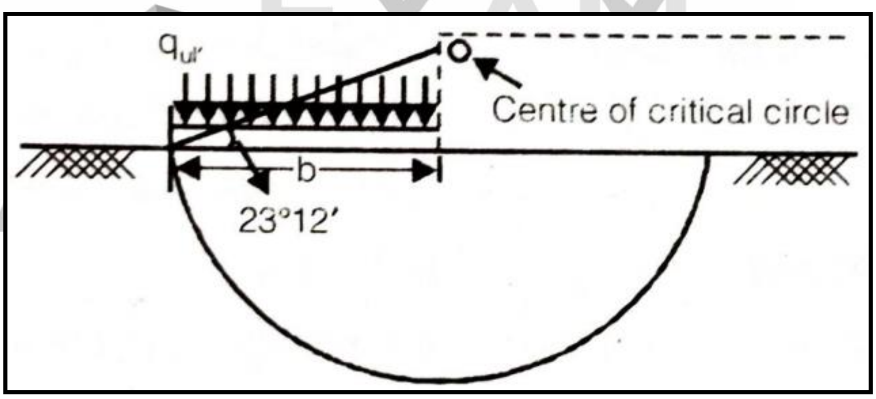
Fellenius' Method (For c-soils)

- This method may be used to determine the ultimate bearing capacity of highly cohesive soils.
- The failure is assumed to take place by slip and the consequent heaving of a mass of soil is on one side.

$$q_{ult} = \frac{W.l_r + CR}{b.l_O}$$

$$q_{ult} = 5.5C$$

- Location of Critical circle



Location of Critical circle for surface footing in Fellenius' method

Prandtl's Method (for c-∅ soils)

- This theory is applicable for c-∅ soil both Prandtl considered the base of the footing to be smooth which is not practicable. Though he accounted for the size of footing. The ultimate bearing capacity is given by

$$q_u = CN_C + \gamma D_f N_q + 0.5\gamma B N_\gamma$$

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EXAM ACHARYA
EDUCATION REDEFINED

Surveying

The best Brains of the Nation may be found on the last Benches of the Classroom.

A.P.J. Abdul Kalam

The content of this book covers all PSC exam syllabus such as MPSC, RPSC, UPPSC, MPPSC, OPSC etc.

Assumptions

1. Footing is a strip footing (L >> B)
2. Soil is Homogenous
3. 2-D plane strain condition prevails.
4. Base of footing is rough
5. The base of footing is laid down at shallow depth $\left(\frac{D_f}{B}\right) \leq 1$ i.e., shallow foundation
6. Loading is vertical and symmetric i.e. (moment = 0)
7. General shear failure occurs.
8. Ground is Horizontal
9. Shearing resistance of soil between the ground surface and base of footing is neglected Thus, footing considered as a surface footing with uniform surcharge = ($\gamma . D_f$) at the base of footing
10. Shear strength of soil is goverened by Mohr's coulmb criteria.

(A) Bearing Capacity for Strip Footing is Given By

$$q_{ult} = cN_c + \gamma D_f N_q + \frac{1}{2}\gamma B N_\gamma$$

- This equation is called Terzaghi's general bearing capacity formula.
- N_c, N_q, N_γ are called "bearing capacity factors" for shallow continuous footing. The parameters are in terms of effective stresses.
- For purely cohesive soils, $\emptyset = 0$

  $N_c = \frac{3\pi}{2}+1= 5.7$ for rough footing however this value was $N_c = 5.14$ for smooth footing as given by Prandtl. Nq = 1 and $N_\gamma = 0$

  The bearing capacity of a strip footing with a rough base $q_{ult} = 5.7C + \gamma D_f$

Zone III

- Zone of Linear shear
- Zone of Passive Rankine state
- Makes an angle of (45° - Ø/2) with horizontal

Note

- In Terzaghi theory failure zones are not assumed to extend up to ground level. Whereas in Meyerhoft's, Brinchhansen and in Vesic's method the failure zone is assumed to extend upto ground level.
- The Failure surface is log spiral for C-Ø soil and is circular for purely cohesive soils.
- Before the load is applied, beneath the base of the footing is in a state of elastic equilibrium, when the load is increased beyond a certain critical value the soil gradually passes into a state of plastic equilibrium.
- Prandtl equation gives lower ultimate bearing capacity and hence must be considered for conservative design, especially when one is not sure of the roughness of the footing and its true impact.
- A factor of safety of 3, for important structures is generally adopted. It may however, bereduced to 2.5 or 2 when the soil properties are known with confidence and the structure is less important.

Bearing capacity factors as used in Terzaghi's Equation

$$N_{\emptyset} = \tan^2 (45° + \emptyset/2)$$

$$N_q = N_{\emptyset} e^{\pi \tan \emptyset}$$

$$N_{\gamma} = 1.8 \tan\emptyset \, (N_q - 1)$$

$$N_c = \cot\emptyset \, (N_q - 1)$$

For purely cohesive soils, Ø = 0

$$N_{\emptyset} = 1$$

$$N_q = 1$$

$$N_{\gamma} = 0$$

$$N_c = 5.7$$

TYPES OF SHEAR FAILURES

The above discussion of Terzaghi's theory is based on the general shear failure however the following three distinct modes of shear failure may exist.

b. Local Shear Failure

In case of very soft clays large deformations may occur below the footing before the failure zones are fully developed. The plastic equilibrium develops only in part of the soil below the footing, the failure surface therefore do not reach the ground and only slight heaving occurs, there would be no tilting of the footing, this type of failure mostly occurs in loose or soft soils of high compressibility. For local shear failure c' and Ø' are used such that c = $\frac{2}{3}$c and tanØ' = $\frac{2}{3}$tanØ.

The corresponding values of

$N_{\emptyset}'$, N_c', N_{γ}' and N_q' are used.

c. Punching Shear Failure

When there is compression of the soil under the footing, accompanied by shearing in vertical direction around the edge of the footing, there is noheaving of the ground surface away from the edges of the footing and there is no question of any tilting of the footing. Relatively large settlements do occur in this type of failure and ultimate bearing capacity is not well defined. This type of failure occurs in deep footings in soils of low compressibility. B/D_f Ratio is quite important for such failure.

Though it is difficult to define the limiting conditions for general and local shear failure.

Comparative Analysis

1. Stress-strain test in C - Ø soil

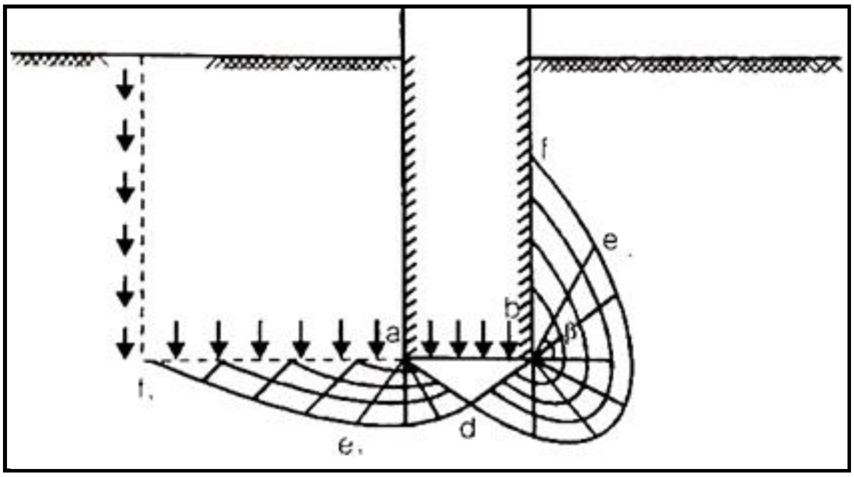
(a) General shear failure occurs at low strain say < 5%

(b) Local shear failure occurs at large strain 10 to 20%

2. Angle of shear friction

(a) For General shear failure Ø > 36°

(b) For Local shear failure Ø < 28°



Deep foundation

Bearing capacity factors N_c, N_q, N_γ depend not only on $\emptyset$, but also on the depth and shape of the foundation and roughness of the base.

Therefore

$$q_n = cN_c.S_c.d_c.i_c + \gamma D_f \cdot N_q \cdot S_q \cdot d_q \cdot i_q + \frac{1}{2} \gamma B N_\gamma . S_\gamma . d_\gamma . i_\gamma$$

Where, S, d and i are shape, depth and inclination correction factors.

<u>Skemptons Method (For c-soil only)</u>

- N_c is a function of depth and shape. The net ultimate bearing capacity is given by

$$q_{nu} = cN_c$$

Where N_c is given as follows.

(a) Maximum Value of N_c is limited to

$N_c = 9$ for square or circular footing

$N_c = 7.5$ for strip footing.

(b) 1. When $= \frac{D_f}{B} = 0$

$N_c = 5.14 \approx 5.0$ for strip footing

Brinch Hansen's Method

- According to Hansen, the ultimate bearing capacity is given by

$$q_u = c.N_C.s_c.d_c.i_c.g_c b_c$$

$$+\sigma_O.N_q.s_q.d_q.i_q.g_q.b_q$$

$$+\frac{1}{2}\gamma.B.N_\gamma.s_\gamma.d_\gamma.i_\gamma.g_\gamma.b_\gamma$$

Where,

σ_o = $\bar{\sigma}$ effective overburden pressure at foundation level.

s = Shape factor, to account for the effect of the shape of the foundation in developing a failure surface

d = Depth factor to account for the embedment depth and the additional shearing resistance in the top soil.

i = Inclination factor to account for both horizontal & vertical component of foundation loads

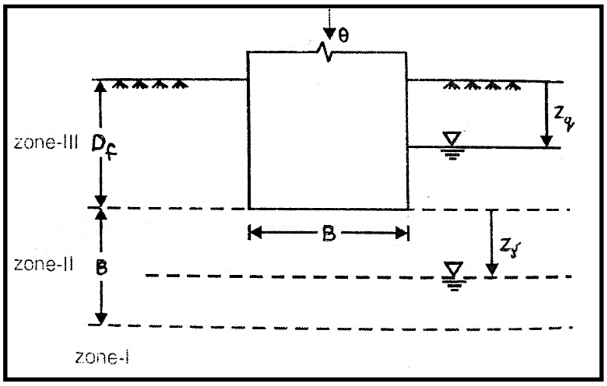
g = Ground factor

b = Base factor

g = Density of soil below the foundation level

EFFECT OF WATER TABLE ON BEARING CAPACITY

- For any position of water table the ultimate bearing capacity equation may be modified by



Note

- Due to water table c and ϕ are also affected up change is small and may be neglected. In case of pure sands (c = 0) if water table rises to ground level then, q_u will be reduced by 50 % But if soil is pure clay then N_γ = 0 and N_q = 1. Hence net ultimate bearing capacity will be nearly unaffected due to rise of ground water table.

SETTLEMENT CONSIDERATIONS IN DESIGN OF FOUNDATIONS

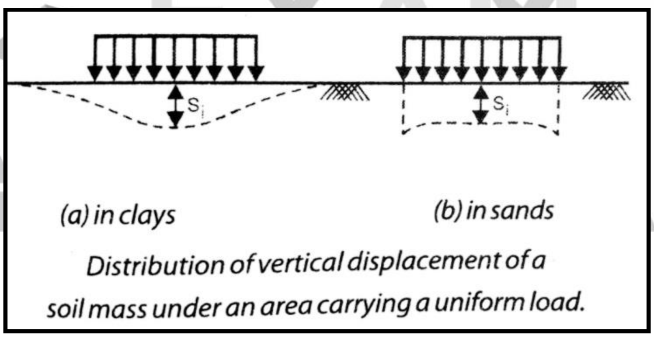
Allowable bearing capacity for the design of a footing is governed by two factors:

1. Bearing capacity of soils
2. Permissible settlement.

- Settlement in soils are of two types

  (a) Immediate settlement (S_i) or elastic settlement

  (b) Consolidation settlement (S_c) due to expulsion of water



Total settlement S = S_i + S_c

Immediate settlement or elastic settlement

$$S_i = qB\frac{(1-\mu^2)}{E}I_f$$

I_f – Influence factor for settlement

q - Uniform load intensity

μ - Poisson's ratio

E - Young's modulus.

Soil type	Permissible Total Settlement		Permissible diff Settlement	
	For Isolated Footing	For Raft Foundation	For Isolated Footing	For Raft Foundation
Sandy Soil Clays	40 mm 65 mm	40-65 mm 65-100 mm	25 mm 40 mm	25 mm 40 mm

BEARING CAPACITY CALCULATION METHODS

1. Plate Load Test

- Generally performed on uniform sandy soils: (though can be used for cohesive soils also) this test is used for determining the ultimate B.C. as well as the probable settlement of the soil for a given loading and for a given depth. This test can also be used to find modulus of subgrade reaction.
- The rigid plate at the foundation level is loaded with gradually increasing load and the settlements are measured from each increment of load.
- The ultimate B.C. is then taken as load at which the plate starts sinking at a rapid rate.
- The Rigid bearing plate to be used is square in section of minimum size 30 cm × 30 cm and maximum size 75 cm × 75 cm. Plate thickness should be sufficient to provide rigidity and is ≈ 25 mm (≮10 mm)
- The size of the pit in which the plate is placed is generally not less than 5 times the size of the plate. A small square hole is of the size as that of the test plate and the depth of hole being such that,

$$\frac{D_f}{B_f} = \frac{d_p}{B_p}$$

B_f = width of foundation

B_p = width of plate

D_f = depth of foundation

d_p= depth of plate hole

- The settlement of Footing is found to vary with its size. Terzaghi and peck has suggested the following relation

$$\frac{S_i}{S_p} = \left(\frac{B_{f(B_p + 0.3)}}{B_{p(B_f + 0.3)}}\right)^2 \longrightarrow \text{for sandy soils}$$

S_i = Permissible settlement of the proposed foundation (in mm)

S_p - Settlement of the test plate (in mm)

B_f - Size of the proposed foundation (in mm)

B_p - Size of the test plate (in mm).

- For clays E_s (mod. of elasticity) for clays is relatively constant.

$$\frac{S_i}{S_p} = \frac{B_f}{B_p} \longrightarrow \text{for clays}$$

- For c $-\phi$

$$\frac{S_f}{S_p} = \left(\frac{B_f}{B_p}\right)^{n+1}$$

Where,

n = Coeff. (depends on type of soil).

n = 0 for clay

n = 0.5 for sand.

Limitations

1. Effect of size is very important, since the size of the test plate and the size of the prototype foundations is different, the result of the plate load test does not directly reflect the Bearing Capacity of foundation.

   The Bearing Capacity of footings in sands varies with the size of footing, thus the scale effect gives rather misleading results in this case. However, this effect is not pronounced in cohesive soils, as the B.C. is essentially

Bearing Capacity from Penetration Test

- Standard Penetration
  - Uses dynamic approach
- Cone Penetration
  - Static
  - Dynamic

Standard Penetration Test

$$N_1 = N_{obs} \times \frac{350}{\bar{\sigma} + 70}$$

N_{obs} – Observed value of SPT

N_1 –Corrected Value for over burn pressure

$\bar{\sigma}$ – effective over burden pressure(in kN/m^2) $\neq$ 280 kN/m^2

if $\bar{\sigma}$ > 280 then No correction is required.

Dilatancy Correction

In case of fine sand or silt below water table apparently high values may be noted for N_2 in such cases the following correction is applied.

The value of SPT, corrected for over burden pressure ie., N_1 is further corrected for dilatancy

$$N_2 = 15 + \frac{1}{2}(N_1 - 15)$$

N_1 - The observed SPT value after correction for over burden

N_2 - Finally corrected value

- Above correction is applied only when N_1> 15. "The fine sands and silt below the water table offer higher resistance to driving due to the development of the excess pore pressure (which could not be dissipated immediately), leading to increased apparent soil resistance, giving higher observed value of N during SPT."

B = Width of foundation

N = Avg. corrected S.P.T. number.

S = Permissible settlement of foundation

C_w = Water table correction factor

$q_{a\,net}$ = Net allowable bearing pressure,

Teng's Equations

$$q_{ns} = 1.4(N-3)\left(\frac{B-0.3}{2B}\right)^2 S C_w C_D \text{ KN}/m^2$$

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

$$C_D = \left(1+\frac{D_f}{B}\right) \leq 2$$

Where,

C_w = Water table correction factor

D_w = Depth of water table from base of footing

B = Width of foundation

C_d = Depth correction factor = $1+\frac{D_f}{B} \leq 2$

S = Permissible settlement in 'mm'.

I.S. Code Method

$$q_{ns} = 1.38\,(N-3)\left(\frac{B-0.3}{2B}\right)^2 S C_w$$

q_{ns} = Net safe bearing pressure in kN / m^2

B = Width in meter.

S = Settlement in 'mm'.

I.S. Code Formula for Raft:

Cone Penetration Test

$$C = 1.5\left[\frac{q_c}{\bar{\sigma}_0}\right]$$

where,

q_c = Static cone resistance in kg / cm^2

c = Compressibility coefficient

$\bar{\sigma}_0$ = Initial effective over burden pressure in kg / cm^2.

$$S = 2.3\,\frac{Ho}{C}\log_{10}\left[\frac{\overline{\sigma_0} + \overline{\sigma\Delta}}{\overline{\sigma_0}}\right]$$

where, 'S' = Settlement.

$$q_{ns} = 3.6 q_s R_w \text{ when B < 1.2 m.}$$

where, q_{ns} = Net safe bearing pressure in kN / m^2

$$q_{ns} = 2.7 q_c R_w \text{ when B > 1.2 m}$$

where, R_w = Water table correction factor.

For Cohesive Soils

The allowable soil pressure for cohesive soils is generally governed by net safe bearing capacity, although settlement criterion may control for soft clays. Generally a factor of safety of 3 against shear also insures safety against settlement.

GPSC - CIVIL

Transportation Engineering

END is not the end if fact E.N.D. means
" Effort Never dies"

A.P.J. Abdul Kalam

The content of this book covers all PSC exam syllabus such as MPSC, RPSC, UPPSC, MPPSC, OPSC etc.

Qu4 In Terzaghi's bearing capacity analysis, the soil wedge immediately below the footing remains in a state of

(a) Plastic equilibrium

(b) Radial shear

(c) Elastic equilibrium

(d) Linear shear

Qu5 The static cone penetrating test and a standard penetrating test are performed on a soil at a certain depth. The value of static cone penetration resistance is 8 MPa and the N value is 20. The soil met with at that depth is:

(a) Sandy silt

(b) Clay-silt

(c) Sand and gravel mixture

(d) Medium dense sand

Qu6 What is the allowable soil pressure for a raft (10m × 10m) if the depth is 5m and the undrained cohesion is 40KN/m^2. Take the factor of safety as 2.5

(a) 100.6 kN / m^2

(b) 105.6 kN / m^2

(c) 110.6 kN / m^2

(d) 115.6 kN / m^2

Qu7 The allowable bearing capacity at 25mm allowable settlement for a footing in a sandy soil is 15t /m^2. The allowable bearing capacity for the same footing permitting a settlement of 40mm is:

(a) 24t /m^2

(b) 30t /m^2

(c) 35t /m^2

(d) 40t /m^2

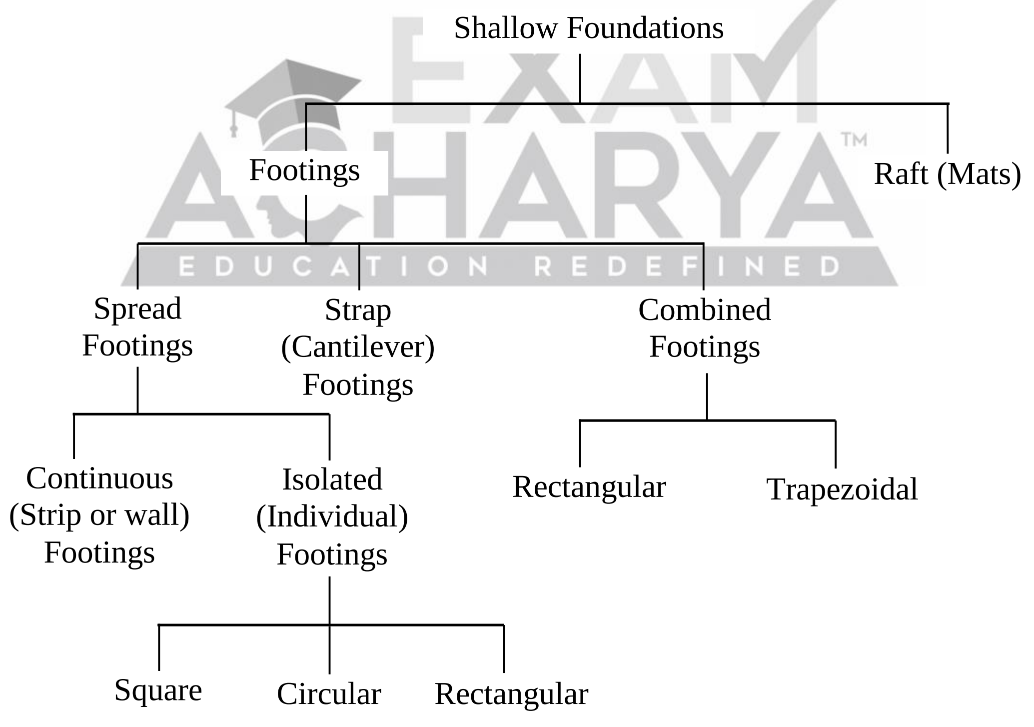
CHAPTER 12

TYPES OF FOUNDATION & SELECTION CRITERIA

Footings are generally the lowermost supporting part of the structure known as sub - structure and are last structural elements through which load is transferred to foundation comprising soil/rock.

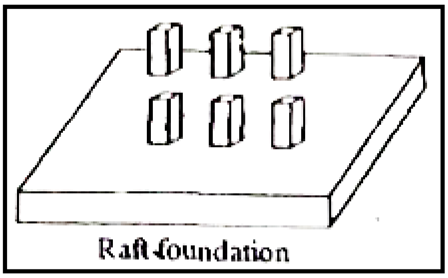
- Structural elements transfer the applied loads from one part of the building to the other. These are in turn transmitted to the foundation which transfers it to the underlying soil / rock.

Classification of Shallow Foundations



Raft / Mat Foundation

These type of foundations are large continuous footing which support all columns and walls of a structure and are constructed when soil is weak



- **Combined footings :** A combined footing supports two or more columns in a row when the areas required for individual footings are such that they come very near to each other,

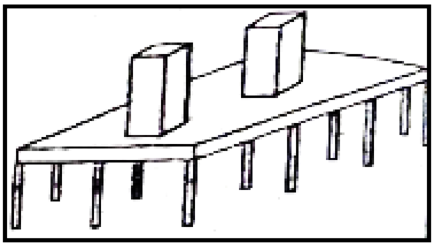
Pile Foundation

These are used to transmit heavy column loads to a group of piles joined at top by a pile cap. The piles transmit the structural loads to the underlying soil through friction and bearing.

Such type of foundation systems is usually adopted when the material below footing is too weak to support the structure and it becomes essential to transfer loads to better strata underlying weaker strata. These foundations are very expensive.

The choice of a particular type of foundation depends on

(a) Magnitude of loads (b) Nature of subsoil strata (c) Nature of Superstructure



- **Pier Foundations:** Pier foundations are somewhat similar to pile foundations but are typically larger in area than piles. An opening is drilled to the desired depth and concrete is poured to make a pier foundation.

  Usually, pier foundations are used for bridges.

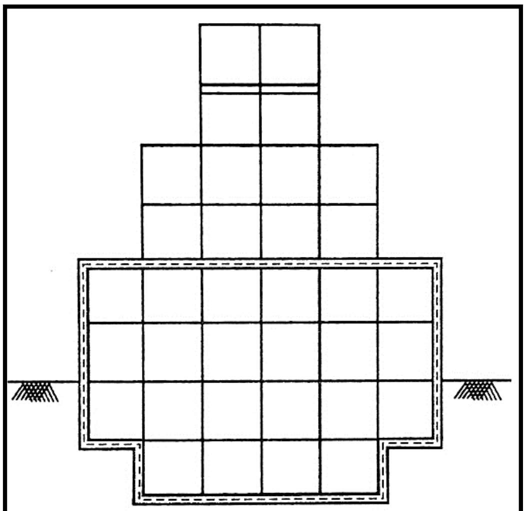
CAISSONS (WELLS)

- A caisson is a structural box or chamber that is sunk into place or built in place by systematic excavation below the bottom. Caissons are classified as ‘open’ caissons, ‘pneumatic’ caissons, and ‘box’ or ‘floating’ caisson. Open caissons may be box - type or pile – type.

- The top and bottom are open during installation for open caissons. The bottom may be finally sealed with concrete or may be anchored into rock.

- Pneumatic caisson is one in which compressed air is used to keep water from entering the working chamber, the top of the caisson is closed. Excavation and concreting is facilitated to be carried out in the dry The caisson is sunk deeper as the excavation proceeds and on reaching the final position, the working chamber is filled with concrete.

- Box or floating caisson is one in which the bottom is closed. It is cast on land and towed to the site and launched in water, after the concrete has got cured. It is sunk into position by filling the inside with sand, gravel, concrete or water. False bottoms or temporary bases of timber are sometimes used for floating the Caisson to the site. The various types of caissons are shown in Figure.



Rigid box caisson foundation using floating foundation concept (McCarthy, 1977)

CHOICE OF FOUNDATION TYPE AND PRELIMINARY SELECTION

Depends Upon Following Factors

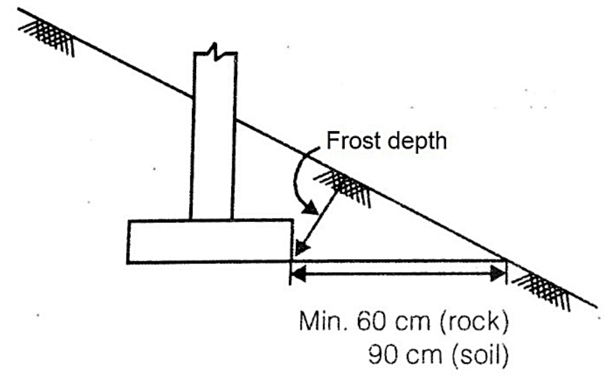
1. The function of the structure and the loads it must carry,
2. The subsurface conditions,
3. The cost of the foundation in comparison with the cost of the superstructure.

The final choice of the type of foundation require following steps:

1. Information regarding the nature of the superstructure and the probable loading is required.
2. The approximate subsurface conditions or soil profile is to be ascertained.
3. Suitability of all possible types of foundation is judged
4. More detailed studies, including tentative designs, of the more promising types are made in the next phase.

5 Final selection of the type of foundation is made based on the cost - the most acceptable compromise between cost and performance.

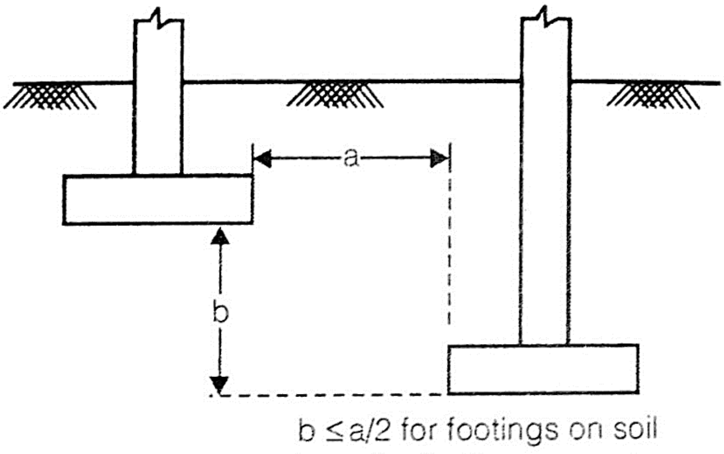
4. Footings on sloping ground should be constructed with a sufficient edge distance (minimum 60 cm to 90 cm) for protecting against erosion.



Edge distance for footing on sloping ground

5. The difference in elevation between footings should not be so great as to introduce undesirable overlapping of stresses in soil. The guideline used for this is that the maximum difference in elevation should be maintained equal to the clear distance between two footing in the case of rock and equal to half the clear distance between two footings in the case of soil. This is also necessary to prevent disturbance of soil under the higher footing due to the excavation for the lower footing

- The density index of granular soils in situ is generally determined by standard penetration tests



Footings at different elevations - restrictions

Qu4 The maximum settlement for raft foundation on clay is limited to

(a) 40 to 65 mm

(b) 65 to 100 mm

(c) 30 to 65 mm

(d) 0 to 20 mm

Qu5 The dependence of the settlement of a footing on sand on the width of the footing is

(a) Directly proportional

(b) Indirectly proportional

(c) Logarithmically proportional

(d) none of the above

Qu6 The major problem with settlement analysis is

1. Obtaining reliable value of the elastic parameters;

2. Obtaining a reliable stress profile from the applied load

Which of these statement is / are correct?

(a) 1 alone

(b) 2 alone

(c) 1 and 2

(d) None of these

CHAPTER – 13

PILE FOUNDATIONS

CLASSIFICATION OF PILES

Pile may be classified in a number of ways based on different criteria:

1. Function or action
2. Composition and material
3. Installation

Classification based on Function or Action

- Pipe may be classified as follows based on the function or action:
- ***End - bearing piles:*** Used to transfer load through the pile tip to a suitable bearing stratum, passing soft soil or water.
- ***Friction piles:*** Used to transfer loads to a depth in a frictional material by means of skin friction along the surface area of the pile.
- ***Tension or uplift piles:*** Used to anchor structures subjected to uplifts due to hydrostatic pressure or to overturning moment due to horizontal forces
- ***Compaction piles:*** Used to compact loose granular soils in order to increase the bearing capacity.
- ***Anchor piles:*** Used to provide anchorage against horizontal pull from sheet piling or water.
- ***Fender piles:*** Used to protect water front structures against the ships or other floating objects. Sheet piles: Commonly used as bulkheads, or as impervious cut - offs to reduce seepage and uplift under hydraulic structures.
- ***Batter piles:*** Used to resist large horizontal and inclined forces, especially in water front structures,

BEARING CAPACITY OF PILES

- The ultimate bearing capacity of a pile is the maximum load which it can carry without failure or excessive settlement of the ground. The bearing capacity also depends on the methods of installation

METHODS OF DETERMINING PILE CAPACITY

1. ***Static Analysis or Analytical Methods:*** Suitable for friction pile in cohesive soil.
2. ***Dynamic Analysis:*** Suitable for friction piles in cohesionless soils or in dense sands.
3. ***Pile load Tests (field approach):*** Best and accurate method for piles.
4. ***Penetration tests (field approach)***

1. Static Analysis

The ultimate bearing load of the pile $Q_{up}$is given by

Q_{up} = Load taken by base + Load by skin friction

$= Q_{up} + Q_f = q_b A_b + f_s A_s$

In case of pile foundation the type of failure is punching shear failure, irrespective of the density index of the soil, so long the depth to width ratio is greater than 4.

In above formula

q_b = Bearing capacity in point - bearing for the pile.

f_s = Unit skin friction for the pile - soil system.

A_b =Bearing area of the base of the pile

A_s = Surface area of the pile in contact with soil.

$$q_b = \mathrm{C}N_c + \gamma D_f N_q + \frac{1}{2}\gamma\, \mathrm{B}N_\gamma \text{ (In general)}$$

SKIN FRICTION CAPACITY

(a) For Piles in Sands

$$f_s = \sigma_h \tan\delta = K_o \bar{q}_0 \tan\delta$$

where,

q_0 = Average effective overburden pressure acting along embedded length

$= \frac{O + \gamma D_f}{2}$

δ = angle of friction between piles and soil

= 0.75ϕ for concrete piles and 20° for steel piles

K_0= Avg. coefficient of lateral earth pressure.

= 1.0 for loose sand

= 2.0 for dense sand

(b) For Piles in Clays

$$f_s = C_a = \alpha.c$$

where α is adhesion factor depending upon consistency of clay

2. Dynamic Analysis

- Dynamic methods are suitable for dense cohesionless soil only. These methods should not be used for loose saturated sands because liquefaction may arise which will not give true results.
- The total energy supplied by the hammer in driving a pile in to a sand stratum by a certain distance S is utilized by the bearing resistance offered by the pile and in covering losses caused by friction, heat, hammer rebound, vibration and elastic compression of the pile.

(b) Hiley's Formula: (I.S. Formula)

This formula accounts the energy losses for

(1) Elastic compression of pile, pile cap and soil.

(2) Energy loss during impact of the pile and hammer. It is more realistic.

The Ultimate load carrying capacity of pile is given by

$$Q_{up} = \frac{\eta_h . \eta_b . WH}{S + \frac{C}{2}}$$

$$Q_{ap} = \frac{Q_{up}}{F_S}$$

Where,

f_s = Factor of safety = 3

n_h = Efficiency of hammer

n_b = Efficiency of blow.

$$\boldsymbol{n_h} = \mathbf{0.75 \text{ to } 0.85}$$

for single acting steam hammer

$$\boldsymbol{n_h} = \mathbf{0.75 \text{ to } 0.80}$$

for double acting steam hammer

$$\boldsymbol{n_h = 1} \text{ for drop hammer}$$

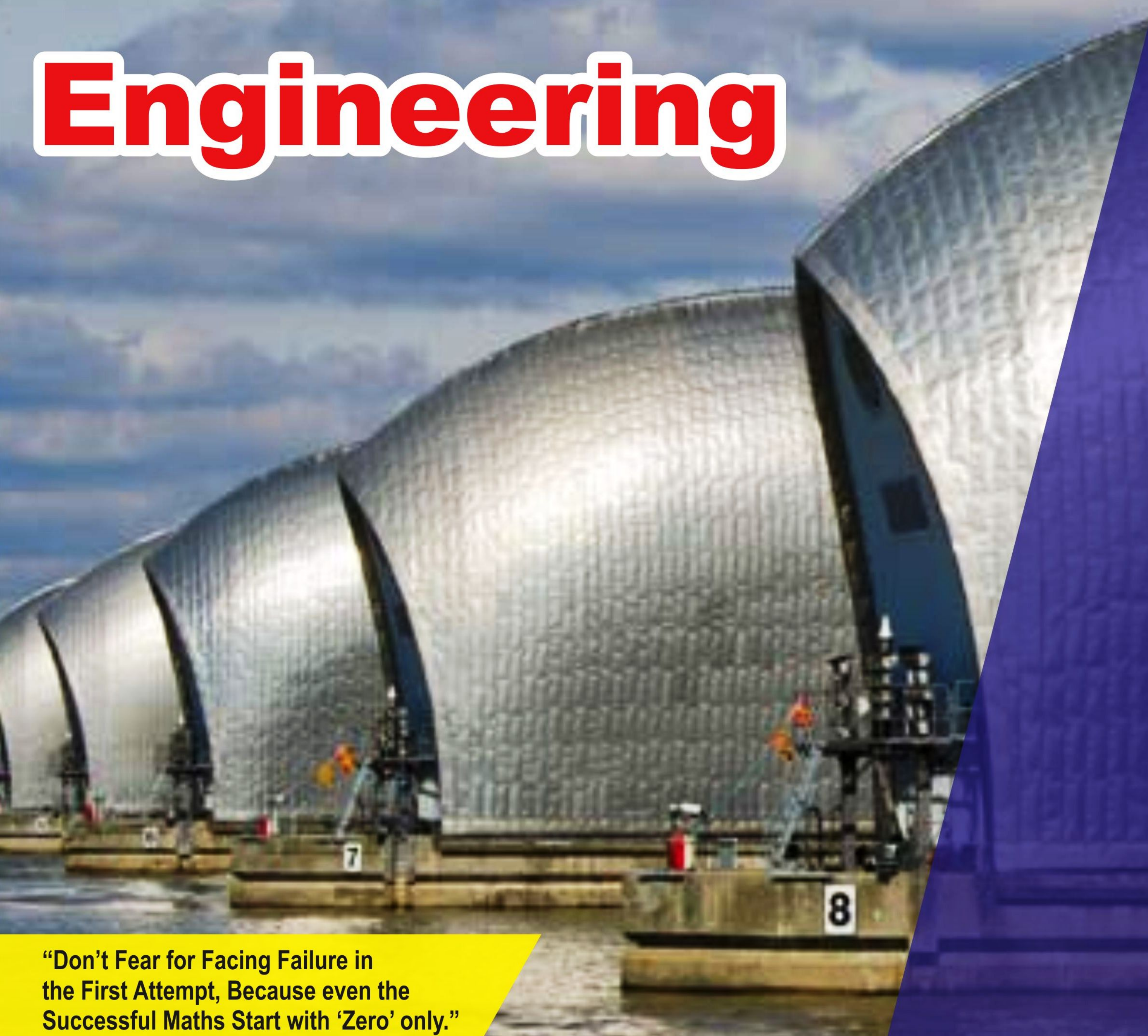
$$\boldsymbol{n_b} = \frac{\textbf{Energy of hammer after impact}}{\textbf{Energy of hammer just before Impact}}$$

$$\boldsymbol{n_b} = \frac{\boldsymbol{W + e^2 P}}{\boldsymbol{W + P}}, \textbf{ when W} > \boldsymbol{p_e}$$

EXAM ACHARYA
EDUCATION REDEFINED

GPSC - CIVIL

Water Resource Engineering



"Don't Fear for Facing Failure in the First Attempt, Because even the Successful Maths Start with 'Zero' only."

A.P.J. Abdul Kalam

The content of this book covers all PSC exam syllabus such as MPSC, RPSC, UPPSC, MPPSC, OPSC etc.

3. Penetration Test

(To determine load carrying capacity of piles in sandy soil)

The SPT - N value can be used to determine load carrying capacity of pile

$$Q_{up} = q_b A_b + f_s A_s$$

q_b (end bearing resistance in kN / m^2) for piles driven in sands is given by

$$q_b = 40.N.\frac{D_f}{B} \Uparrow 400N$$

Where, B = width or diameter of pile.

f_s (unit skin friction in kN / m^2) for piles in sands is given by

$$f_s = 2\overline{N}$$

Where $\overline{N}$ = SPT - N value average over the embedded depth D_f of the pile.

- The above value of f_s should be multiplied by 1/2 for small displacement piles such as H section piles
- For bored piles above value of Q_{pu} should be multiplied by 1/3 and value f_s by 1/2

4. Load Tests on Pile

(Best method especially for clayey soils)

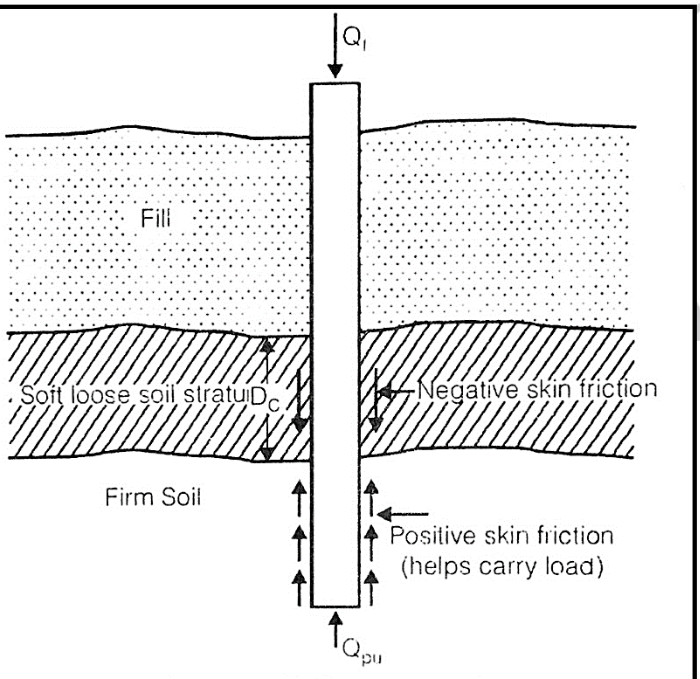
- Due to remolding, softening and changes in shear strength parameters of clays due to pile installation, it is very difficult to precisely estimate the load carrying capacity, analytically. Pile driving formulas (Dynamic formulas) are also not applicable to piles in clays.
- The actual load tests, can, however, be used to determine load carrying capacity of clayey and sandy soils both.

NEGATIVE SKIN FRICTION

'Negative skin friction' or 'downward drag' is a phenomenon which occurs when a soil layer surrounding a portion of the pile settles more than the pile. Such relative motion may occur when the clay stratum undergoes consolidation due to

1 A fill recently placed over the clay stratum.
2 Lowering of the ground water table.
3 Reconsolidation occurring due to disturbance caused by pile driving in sensitive clay stratum, etc.

Negative skin friction increases gradually as the consolidation of the clay layer proceeds since the effective overburden pressure gradually increases due to dissipation of excess pore pressure.



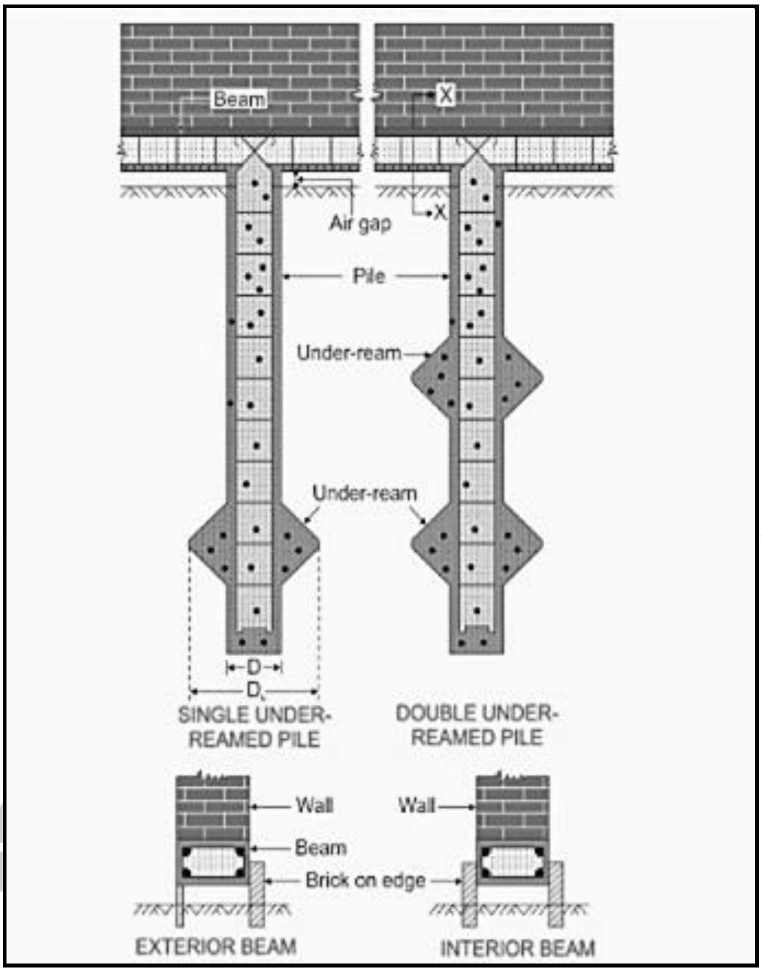
Negative skin friction on a pile

Factor of Safety

$$= \frac{\textbf{Ultimate load capacity}}{\textbf{Working load + Negative skin friction force}}$$

UNDER REAMED PILES

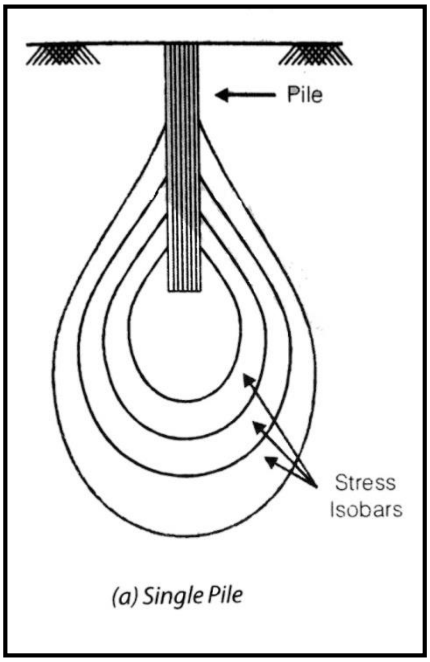
- An 'under-reamed' pile is one with an enlarged base or a bulb; the bulb is called 'under-ream'.



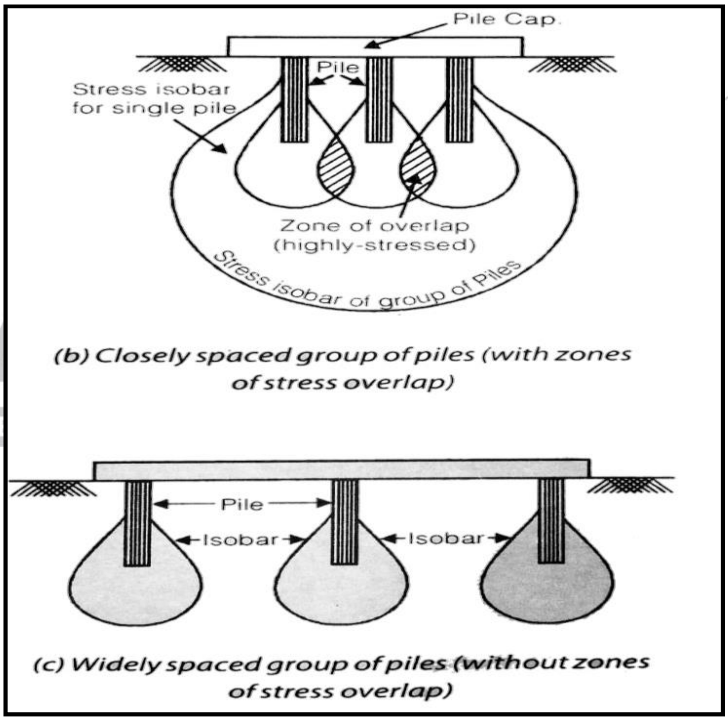
- Under-reamed piles are cast-in-situ piles, which may be installed both in sandy and in clayey soils. The ratio of bulb size to the pile shaft size may be 2 to 3, usually a value of 2.5 is used. The bearing capacity of the pile increases because of the increased base area. Field tests indicate that an under-reamed pile is more economical than a straight bored pile for given load.

The load capacity of an under - reamed pile may be found in much the same way as for driven piles.

$$Q_{up} = Q_{eb} + Q_{sf} = q_b \cdot A_b + f_s \cdot A_s$$



(a) Single Pile



(b) Closely spaced group of piles (with zones of stress overlap)

(c) Widely spaced group of piles (without zones of stress overlap)

EFFICIENCY OF THE PILE GROUP

$$\eta_g = \frac{Q_{ug}}{n \cdot Q_{up}}$$

$$= \frac{\text{Ultimate loadcap of pile group}}{'n'\,times\ of\ ultimate\ load\ cap\ of\ single\ pile}$$

Load Carrying of Pile Group

$$Q_{ug} = q_{eb}A_{bg} + f_sA_{sg}$$

where, A_{bg} = Base area of the group as a whole

$= \mathrm{B}^2$ (B is size of the group)

A_{sg} = Area of group in contact of soil to soil for friction

$= (4B) \cdot D_n (D_n =$ depth of pile group)

$q_{eb} =$ end bearing resistance

$$= \mathrm{CN_c} + \gamma \mathrm{D_f N_q} + \frac{1}{2}\gamma \mathrm{BN_\gamma}$$

$\mathrm{f_s}$ = unit friction resistance

$= \mathrm{C_u} + \bar{\sigma} \tan\delta$ for clays

$f_s = \alpha . C$ for pile and soil

$= C$ for soil and soil ($\therefore \alpha = 1$)

Where ' α ' is adhesion factor.

SETTLEMENT OF PILE GROUPS

- Settlement of the pile group is always found tobe greater corresponding single pile, mainly due to the overlapping of the individual influence zones of piles, while in a group. The ratio of the settlement of a pile group to the settlement of a single pile, when both are carrying the same proportion of their ultimate load, is called the settlement ratio of the group.

  The computation of settlement of pile group is done by assuming the entire pile group as an equivalent raft, located at depth, $\frac{2}{3}D_f$, asshown in figure. The load

CLEAR YOUR CONCEPT

Qu1 Consider the following field tests

1 Vertical pile load test

2 Cyclic pile load test

3 Lateral pile load test

4 Instrumented test pile

While estimating the load carrying capacity of a pile, the tests that can be used for separating the skin resistance from point resistance, would include:

(a) 1 and 3

(b) 1 and 4

(c) 2 and 3

(d) 2 and 4

Qu2 Match List-I (Load Case) with List-II (Expression for slope/Deflection) and select the correct answer using the codes given below the lists (Flexural rigidity = EI):

List-I	List-II
A. Slope for tip load of *W*	**1.** $\frac{WL^3}{8EI}$
B. Deflection for tip load of W	**2.** $\frac{WL^3}{6EI}$
C. Slope for total UDL of W	**3.** $\frac{WL^3}{3EI}$
D. Deflection for total UDL of W	**4.** $\frac{WL^2}{2EI}$

Codes:

	A	B	C	D
(a)	4	2	3	1
(b)	1	3	2	4
(c)	4	3	2	1
(d)	1	2	3	4

Qu6 Dolphin is a type of which one of the following?

(a) Pile foundation

(b) Isolated footing

(c) Raft foundation

(d) Caisson

TEST YOUR SELF

Qu7 Consider the following statements pertaining to a pile group and a single pile at failure:

1 In loose and medium dense sands, the failure load per pile in a group will generally be greater than the failure load for a single pile.

2 In cohesive soils, the failure load per pile in a group will be greater than failure load for a single pile.

3 For piles driven in dense sands, the failure load per pile in group is greater than the failure load for a single pile.

4 When the pile spacing is greater than 10 times the pile diameter, the failure load per pile in a group and for the single pile are practically same in both sands and clays.

Which of these statements are correct?

(a) 1 and 4

(b) 2 and 4

(c) 1,2,3 and 4

(d) 2,3 and 4

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CHAPTER – 14

SOIL EXPLORATION

- The field and laboratory studies carried out for obtaining the necessary information about the soil characteristics including the position of ground water table is termed as Soil Exploration.
- The elements of soil exploration depend mostly on the importance and magnitude of the project, but generally should provide the following.

  (1) Information to determine the type of foundation required such as a shallow or deep foundation.

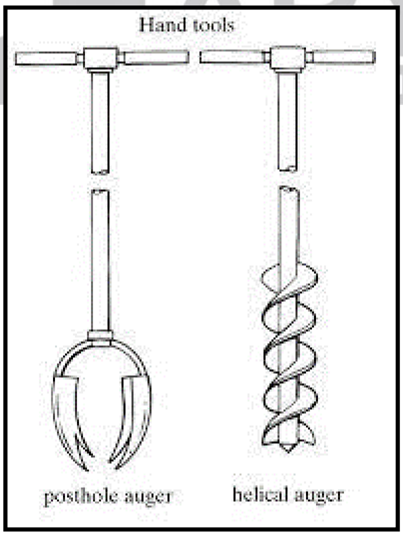
  (2) Necessary information with regards to the strength and compressibility characteristics of the subsoil to allow the design consultant to make recommendation on the safe bearing pressure or pile load capacity.

- Soil exploration involves broadly the following.

  1) Planning of a program for soil exploration.
  2) Collection of disturbed and undisturbed soil or rock samples from the holes drilled in the field. The number and depths of holes depend upon the project.
  3) Conducting all necessary in-situ tests for obtaining the strength and compressibility characteristic the soil or rock directly or indirectly.
  4) Study of ground water conditions and collection of water samples for chemical analysis
  5) Geophysical exploration, if required.
  6) Conducting all the necessary test on the samples of soil/rock and water collected.
  7) Preparation of drawing, charts etc.
  8) Analysis of the data collected.
  9) Report preparation

Auger Boring

- In this type of boring is carried by an equipment called Auger.
- Auger is held vertically and pressed down while rotating it.
- Soil around the auger gets sheared and fills the annular space.
- As the annular space is filled, the auger is withdrawn and cleaned.
- The process is repeated by inserting the cleaned auger in the hole.
- Sample obtained from Auger boring are highly disturbed and are used for identification purpose only.
- Hand operated augers are used for boring holes upto a depth of 6m.
- Power driven augers are used for greater boring depths or where hard or stiff soil strata are encountered.
- Auger boring is done in partially saturated sands, silts and medium to stiff clays.
- Auger boring is usually prefered from small depth of exploration of example shallow foundation, Highways and borrow pits.



Rotatory Boring

- Rotatory Boring is of two types

  (a) ***Mud Rotatory Boring***→ A hole is drilled with the help of a rotating bit. Bentonite soil with some admixture is used as a drilling mud: Drill mud comes upwards through the anular space between the drill rod and the side of the hole.

  (b) ***Core Drilling***→ In this method core barrels with diamond bits are used in place of drilling bit, A solid core of rock formed inside the cylinder is taken out without jerk. Hence in this method samples obtained are with least disturbance.

- Rotatory boring is useful in the soil which are highly resistant to auger and wash boring. eg. dense sand and dense clay.

SOIL SAMPLES

- Soil samples can be basically classified into two samples disturbed samples & Undisturbed samples.
- Disturbed sample are those in which natural soil structure gets modified or destroyed during the sampling operation.
- But with suitable precautions, we can preserve the natural moisture content and the proportion of mineral constituents which is called as Representative samples, even though they are disturbed samples,
- Representative samples are being used for identification purpose whereas Non representative samples are of no use.
- Undisturbed samples are those in which original soil structure is preserved as well as mineral properties have not undergone any change. Although its impossible to obtain such samples in field but they play a major role in identification of soil properties such as soil structure, water content, shear strength, consolidation characteristics and etc.
- The extent of disturbance of sample due to the sampler depends upon.