Geotechnical Engg :-

Date
Page

Introduction :-

It is the branch of civil Engg. concerned with the engineering behaviour of earth material.

→ The engineer determines & designs the type of foundations, earthworks, & pavement subgrades required for the man made structures to the building.

Soil & soil Engg. :-

Soil engineering is a specialized engineering discipline that helps in understanding the behaviours of soil.

→ Soil mechanics is the branch of soil physics & applied mechanics that describes the behaviour of soils.

Scope of soil mechanics :-

⇒ Soil mechanics is used to analyze the deformation of & flow of fluids within natural & man-made structures.

→ That are supported on made of soil or structures. Ex:- Applications are building & bridge foundations, retaining walls, dams, & pipeline systems.

Origin & formation of soil :-

→ Soils are formed from hard rock masses, loose unconsolidated transported materials & organic residue.

→ The term soil is derived from the Latin word "Solum".

→ According to soil scientists, soil means that part of the earth's crust that have been changed as a result of soil formation processes.

~~Origin of soil~~ :-

Formation of soil

Each environment has its own combination of soil-forming factors such as climate, parent material, topograpy, organisms.

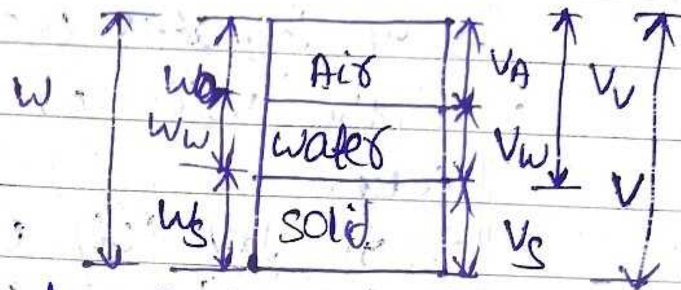
Soil forming factor:-

(1) Geologic factor-
(2) Climatic factor
(3) Topographic factor
(4) Biological factor :-

Chapter - 2

Preliminary Definations & Relationship

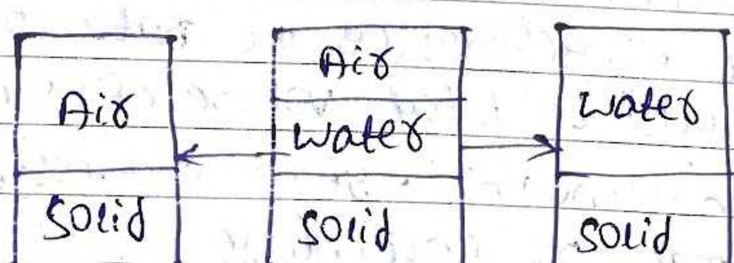
Soil as a three Phase system:-



→ Soil deposits consists of the soil particles & the void space betn the particles.

→ It consists of solid, liquid & gas particles.

→ V_A = volume of Air
V_W = volume of water
V_S = volume of solid
V_V = volume of void
W_a = weight of air
W_W = weight of water
W_S = weight of solid
W = Total weight
V = Total volume



Summer Rainy season

Conversion from 3 Phase system.

Water content (w):-

It is defined as ratio of weight of water to weight of solid.

→ It is also called moisture content
→ It has no unit. It is expressed in percentage
→ It indicates the amount of water present in the voids in comparision with weight of solids.
→ In dry soil water content $w = 0$.
→ $\boxed{w = \frac{W_w}{W_s}}$

Void Ratio (e)

→ It is defined as the ratio of volume of voids to volume of solids.
→ It has no unit. It is expressed in decimals
→ It indicates the amount of voids present in a soil mass in comparision with the amount of solids.
→ $\boxed{e = \frac{V_v}{V_s}}$

Porosity (n) :-

→ It is defined as the ratio of volume of voids to total volume of soil mass.
→ It has no unit. It is expressed in decimals or percentage.
→ Its value ranges from 0 to 100%. $(0 < n < 1)$.
→ i.e. $\boxed{n = \frac{V_v}{V}}$

→ It is denoted by the letter 'w'

→ mathematically $\boxed{w = \frac{W_w}{W_s}}$

→ It is expressed as percentage

$$w = \frac{W_w}{W_s} \times 100$$

→ sometime it became more than 100%

$w > 100\%$

→ The water content is also known as moisture content

AIR content (a_c) :-

It is the ratio betn volume of air to the volume of void

→ It is denoted by the letter 'a_c'

→ mathematically $\boxed{a_c = \frac{V_a}{V_v}}$

percentage of air void (n_a) :-

→ It is the ratio betn volume of air to the total volume

→ It is denoted by the letter 'n_a'

→ mathematically $\boxed{n_a = \frac{V_a}{V}}$

→ It is expressed as percentage

Degree of saturation (S or S_r) :-

It is the ratio betn volume of ~~void~~ water to volume of vo

→ It is denoted by the letter S or S_r

mathematically $\boxed{S \text{ or } S_r = \frac{V_w}{V_v}}$

→ It is expressed as percentage

→ But in eqn it is used as decimal

i) when S = 100% (fully saturated)

ii) when S = 0 (Dry)

Density of soil :-

It is defined as the mass of soil per unit volume.

Bulk density (ρ) or ρ :-

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

→ It is denoted by the letter 'ρ'

mathematically $\boxed{\rho = \frac{m}{V}}$

It is expressed in terms 'S'

C.G.S. gram/cm^3 (C.G.S)

M.K.S Kg/m^3

Dry density :-

→ It is the mass of solid per unit of total volume of soil mass

→ It is denoted by the letter 'ρ_d'

mathematically $\boxed{\rho_d \text{ or } \rho_d = \frac{m_d}{V}}$

Density of solid :-

→ It is denoted by the letter 'ρ_s'

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

mathematically $\boxed{\rho_s = \frac{m_d}{V_s}}$

23.1.13

Q(1)

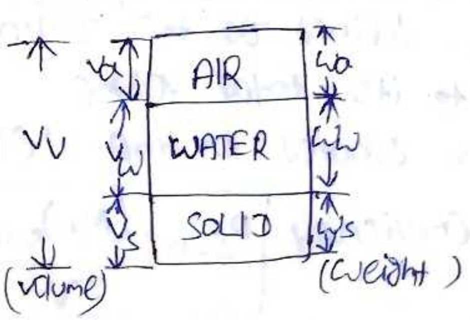
Relationship betn e and n :-

soln :- $e = \frac{V_v}{V_s}$

$n = \frac{V_v}{V}$

we know that

$e = \frac{V_v}{V_s}$ & $n = \frac{V_v}{V}$



$\frac{1}{n} = \frac{1}{\frac{V_v}{V}}$

$\Rightarrow \frac{1}{n} = \frac{1}{V_v} \times \frac{V}{1}$

$\Rightarrow \frac{1}{n} = \frac{V}{V_v} \Rightarrow \frac{1}{n} = \frac{V_v + V_s}{V_v}$ $(\because V = V_v + V_s)$

Dividing right hand V_s in Numerator & Denominator

$$\Rightarrow \frac{1}{n} = \frac{\frac{V_v+V_s}{V_s}}{\frac{V_v}{V_s}}$$

$$\Rightarrow \frac{1}{n} = \frac{\frac{V_v}{V_s}+\frac{V_s}{V_s}}{\frac{V_v}{V_s}}$$

$$\Rightarrow \frac{1}{n} = \frac{\frac{V_v}{V_s}+1}{\frac{V_v}{V_s}}$$

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

$$\cancel{\Rightarrow ne+en=e}$$

$$\Rightarrow e = n(e+1)$$

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

$$\Rightarrow n(e+1) = e$$

$$\Rightarrow ne+n = e$$

$$\Rightarrow n = e - ne$$

$$\Rightarrow n = e(1-n)$$

$$\Rightarrow \boxed{e = \frac{n}{1-n}}$$

Saturated density :-

→ It is the ratio betn saturated mass of soil, solid to its total volume

→ It is denoted by the letter 'ρ_{sat}'.

→ mathematically $\boxed{\rho_{sat} = \frac{(M_d)_{sat}}{V}}$

Submerged density (ρ_{sub}) or ρ' :-

It is defined as the ratio betn submerged mass of soil solid to its total volume

→ It is denoted by the letter 'ρ_{sub}' or ρ

→ mathematically $\boxed{\rho_{sub} = \frac{(M_d)_{sub}}{V}}$

& $\boxed{\rho' = \rho_{sat} - \rho_w}$

→ when P_w = Density of water in one gram per cm^3 or gm/cm^3

$$\boxed{P_w = 1\ gm/cm^3}$$

Bulk unit weight / Total unit of weight / weight unit weight :-

→ It is defined as the ratio betn total weight to total volume

→ It is denoted by the letter 'γ'

→ $$\boxed{\gamma = \frac{W}{V}}$$

→ It is expressed in terms of KN/m^3

Dry unit weight :-

It is the ratio betn weight of solid to its total volume.

→ It is denoted by the letter γ_d

→ Mathematically $$\boxed{\gamma_d = \frac{W_d}{V}}$$

or $$\boxed{\gamma_s = \frac{W_s}{V}}$$

Saturated unit weight :-

→ It is the ratio betn saturated weight of soil solid to its total volume.

→ It is denoted by the letter 'γ_{sat}'

→ Mathematically $$\boxed{\gamma_{sat} = \frac{(W_d)_{sat}}{V}}$$

Submerged unit weight :-

→ It is the ratio betn submerged weight of soil solid to its total volume.

→ It is denoted by the letter γ_{sub} or γ'

→ mathematically $$\boxed{\gamma_{sub} = \frac{(W_d)_{sub}}{V}}$$

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

where γ_w = Unit weight of water (H_2O) = $9.81\ KN/m^3$

or $1000\ KN/m^3$

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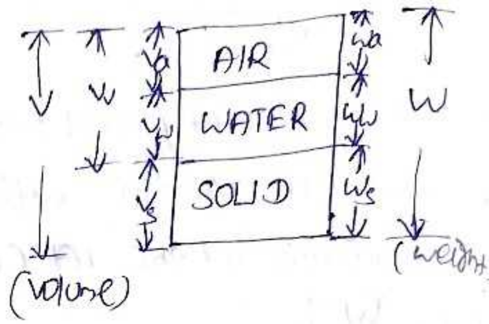
(2) Relation ship between a_c & S

Ans:- From the defination



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

$$\Rightarrow a_c = \frac{V_v - V_w}{V_v}$$

Dividing right hand side V_v on the numerator & denominator

$$\Rightarrow \cancel{a_c = \frac{\frac{V_v}{V_v} - \frac{V_w}{V_v}}{\frac{V_v}{V_v}}}$$

$$\Rightarrow a_c = \frac{V_v - V_w}{\frac{V_w}{V_v}} \Rightarrow a_c = \cancel{\frac{V_v}{V_v}} \frac{V_v - V_w}{V_v} \times \frac{V_v}{V_v}$$

$$\Rightarrow a_c = \frac{V_v}{V_v} - \frac{V_w}{V_v} \times 1$$

$$\Rightarrow \boxed{a_c = 1 - S}$$

(3) Derive the relationship betn n_a, a_c & n :?

Soln :- From the defination

percentage of air void $(n_a) = \frac{V_a}{V}$

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

Porosity $(n) = \frac{V_v}{V}$

~~Dividing~~

$$n_a = \frac{V_a}{V}$$

Dividing right hand side V_v on the numerator & denominator

$$\Rightarrow n_a = \frac{\frac{V_a}{V}}{V_v}$$

$$\Rightarrow n_a = \frac{V_a}{V_v} \times \frac{V_v}{V}$$

$$\Rightarrow n_a = \frac{V_a}{V_v} \times n$$

$$\Rightarrow n_a = a_c \times n \Rightarrow \boxed{n_a = n \times a_c}$$

~~Relation ship betn~~

Specific gravity (G) :-

Specific gravity is defined as the ratio of the weight of ~~den~~ given volume of soil at a given temp. to the weight of an equal volume of water at the same temp.

→ It is denoted by the letter 'G'

→ mathematically $\boxed{G = \frac{\gamma_s}{\gamma_w}}$

(4) Relationship between e, w, G & S :- 28.1.13

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

Ans: From the defination

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

From the defination

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

From the defination

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

From the defination

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

From the defination

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

From the defination

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

$$\frac{\gamma_s}{1} = \frac{W_s}{V_s}$$

$\Rightarrow W_s = \gamma_s \cdot V_s$

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

$\Rightarrow W_w = \gamma_w \cdot V_w$

Putting the value

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

$$\Rightarrow w = \frac{\gamma_w \cdot V_w}{\gamma_s \cdot V_s} \quad ---(1)$$

From the defination

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

$$\Rightarrow \gamma_s = G \cdot \gamma_w$$

Now putting the value of γ_s in eqn (1)

$$w = \frac{\cancel{\gamma_w} \cdot V_w}{G \cdot \cancel{\gamma_w} \cdot V_s} = \frac{V_w}{G \cdot V_s}$$

$$\Rightarrow w = \frac{1}{G} \times \frac{V_w}{V_s}$$

Dividing right hand side V_v

$$\Rightarrow w = \frac{1}{G} \times \frac{\frac{V_w}{V_v}}{\frac{V_s}{V_v}}$$

$$\Rightarrow w = \frac{1}{G} \times \frac{V_w}{V_v} \times \frac{V_v}{V_s}$$

$$\Rightarrow w = \frac{1}{G} \times S \times e$$

$$\Rightarrow w = \frac{Se}{G}$$

$$\Rightarrow Se = wG$$

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

(5) Relationship between γ_d, G, e, w

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

(6) Relationship between e, S, n_a

$$\boxed{n_a = \frac{e - Se}{e+1}}$$

(7) Relationship between γ, G, S, e & γ_w

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

(8) Relationship between γ_d, γ & w

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

④ Relationship between γ_d, G, w & n_a

$$\boxed{\gamma_d = \frac{(1-n_a)G\gamma_w}{1+wG}}$$

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⑤ Relationship betn γ_d, G, e, w :-

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

Soln:- From the defination

Dry unit weight$(\gamma_d) = \frac{W_d}{V} = \frac{W_s}{V}$ ---- (1)

or $\gamma_s = \frac{W_s}{V_s}$

$\Rightarrow W_s = \gamma_s \cdot V_s$

Specific gravity

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

$\Rightarrow \gamma_s = G\cdot\gamma_w$

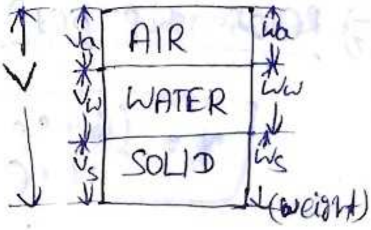
Putting the value of Ws in eqn (1) we get

$\gamma_d = \frac{\gamma_s \cdot V_s}{V}$ --- (2)

Putting the value of Ws in eqn (2) we get

$\gamma_d = \frac{G\cdot\gamma_w\cdot V_s}{V}$

Dividing right hand side Vs we get



$$\gamma_d = \frac{\frac{G\cdot\gamma_w\cdot \cancel{V_s}}{\cancel{V_s}}}{\frac{V}{V_s}} = \frac{G\cdot\gamma_w}{\frac{V}{V_s}}$$

$$\gamma_d = \frac{G\cdot\gamma_w}{\frac{V_v+V_s}{V_s}} \quad [\because V = V_v + V_s]$$

$$\gamma_d = \frac{G\cdot\gamma_w}{\frac{V_v}{V_s}+\frac{V_s}{V_s}}$$

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

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

A sample of wet soil has a volume of $0.0192\ m^3$ & a mass of 32 kg. When the sample is dry in an oven its mass is reduced to 28.5 kg. Take specific gravity of soil sample is 2.65. Determine the bulk density, dry density, water content, void ratio & γ_{sat}.

soln:- Data given

V = volume of soil mass = $0.0192 m^3$

W = weight of mass = 32 kg

W_s or W_d = weight of soil solid = 28.5 kg

G = specific gravity = 2.65

weight of water $W_w = W - W_s$

$= 32 - 28.5 = 3.5$ kg

Bulk density $\gamma = \frac{W}{V} = \frac{32}{0.0192} = 1666.667\ kg/m^3$

Dry density

$\rho_d = \frac{m_d}{V} = \frac{28.5}{0.0192} = 1484.37\ kg/m^3$

water content

$\omega = \frac{W_w}{W_s} = \frac{3.5}{28.5} \times 100 = 12.28\%$

we know the relationship

$\gamma_d = \frac{G \cdot \gamma_w}{1+e}$ (where take $\gamma_w = 1000\ kg/m^3$)

$\Rightarrow 1484.375 = \frac{2.65 \times 1000}{1+e}$

$\Rightarrow 1484.375(1+e) = 2.65 \times 1000$

$\Rightarrow 1+e = \frac{2.65 \times 1000}{1484.375} = 1.785$

$\Rightarrow e = 1.785 - 1 = 0.785$

we know the relationship

$\gamma_{sat} = \frac{\gamma_w (G+se)}{1+e}$

$\gamma_{sat} = \frac{\gamma_w (G+e)}{1+e}$ (where $s = 1$)

$= \frac{1000(2.65+0.785)}{1+0.785} = 1924.370\ kg/m^3$ (Ans)

Chapter-3 Determination of Index properties

Water content :-
It is the ratio betn weight of water to the weight of solid
→ It is denoted by the letter 'ω'
→ It is expressed by percentage
→ mathematically $\boxed{\omega = \frac{W_w}{W_s}}$

Determination of water content :-
The water content of a soil sample can be determined by following method
i) oven drying method
ii) sand bath method
iii) Pycnometer method
iv) calcium carbide method

oven drying method :-
The oven drying is a laboratory method, this is a very accurate method & quick method.
procedure :-
→ First take a empty container & clean the container properly. The container weighing balance which is 'W_1'
→ A specimen of ~~weight~~ wet soil is placed in the container & again weighing the container with wet soil which is 'W_2'
→ The container is placed in the oven for drying for 24 hours at a temp betn 105°C to 110°C.
→ When the soil is completely drying in the oven ~~putt~~ cool the dry soil & again weighing the container with soil after drying which is 'W_3'.
mathematically $\boxed{\omega = \cancel{\text{weight of water}} \frac{W_2 - W_3}{W_3 - W_1} \times 100}$
→ If the temp is lower than the 105°C may not be cause completely evaporation of water
→ If the temp. is higher than the 110°C may cause breaking down the crystaline structure of the soil particle.

→ Hence 105°-110°C temp. is choosen for most of the soil

Sand bath method :-

~~Procedure~~

→ Sand bath method is a field method for the determination of water content. This method is rapid but not accurate

Procedure :-

→ A sand bath is a large open vessel containing sand to a depth of 3cm. A tray is taken & weighted which is 'W_1'

→ The specimen of soil sample is taken in a tray & weighted which is 'W_2'

→ The tray is then placed on the sand bath. The sand bath is heated over a stove. Drying takes about 20-60 minute depending upon the type of soil. During ~~the~~ steering heating the soil specimen is stearing by a knife. When over heating occurs, the white turned brown colours. When drying ~~each~~ is complete the tray is removed from the sand bath, cooled & weighted which is W_3.

→ mathematically water content (ω) = $\frac{W_w}{W_s}$

$$\text{so } \boxed{\omega = \frac{W_2 - W_3}{W_3 - W_1} \times 100}$$

Pycnometer method :-

→ This is also a quick method of determining the water content of those soil whose specific gravity accurately ~~known~~ note.

→ A pycnometer is a glass jar of about 900ml capacity. A conical ~~&~~ brass cap having 6mm diameter hole at ~~least~~ its top. A rubber washer is placed betn the conical cap & the jar to prevent leckage of water.

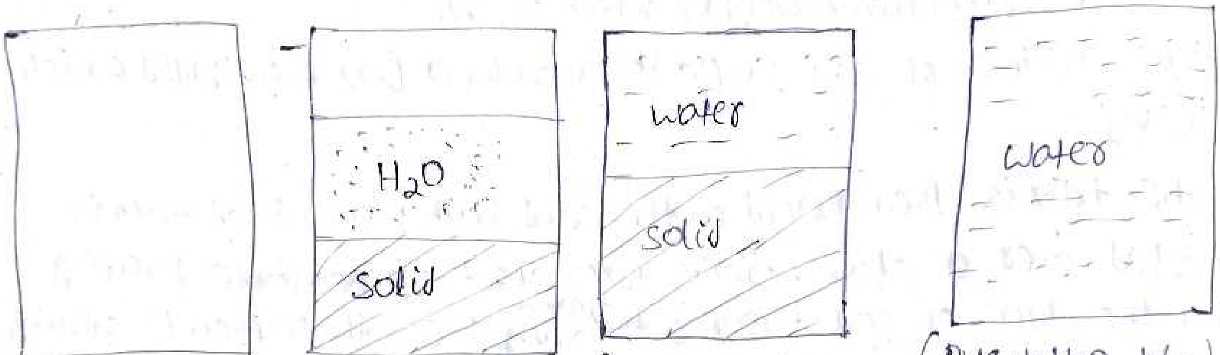
Procedure :-

→ Take a clean, dry pycnometer & find its mass with its ~~which is~~ cap and washer which is 'W_1'

→ A sample of wet soil about 200gm. to 400gm. is taken in the pycnometer & weighted which is 'W_2'

→ ~~Fee~~ Fill the pycnometer to half of its height & mix it throughly with the glass rod & more water is added & stears it. Replace the skew top & fill the pycnometer with the hole in the conical cap. ~~Dry~~ the pycnometer from out side & weighted which is 'W_3'

→ Empty the pycnometer clean it ~~slowly~~ thoroughly & fill it with clean water to the ~~un~~ whole of the conical cap & wet which is 'W_4'



(Empty PYC. W_1) (PYC + wet soil W_2) (PYC + Solid + H_2O, W_3) (PYC + H_2O, W_4)

→ mathematically

$$\text{water content}(\omega) = \left[\left(\frac{W_2 - W_1}{W_3 - W_4}\right)\left(\frac{G-1}{G}\right) - 1\right] \times 100$$

problem

A pycnometer test for the determination of water content of a soil sample having sp. gravity 2.70. The weight of moist soil equal to 230.75 gm. weight of pycnometer with full of water is ~~equal~~ 2965.20gm. weight of pycnometer with soil and water 302~~8~~.85 gm. Calculate the water content of the soil.

A:- weight of moist / wet soil = 230.75 gm

weight of pycnometer full of water = 2965.20gm = W_4

Full soil + H_2O = W_3 = 3092.85 gm

wet of PYC. + moist of soil = W_2

$\Rightarrow W_1 + 230.75 = W_2$

$\Rightarrow W_1 - W_2 = 230.75$gm

$$\text{water content } \omega = \left[\left(\frac{W_2 - W_1}{W_3 - W_4}\right)\left(\frac{G-1}{G}\right) - 1\right] \times 100$$

$$\Rightarrow w = \left[\left(\frac{230.75}{3092.85 - 2965.20}\right)\left(\frac{2.70-1}{2.70}\right) - 1\right] \times 100$$

$\Rightarrow w = 13.82\%$

Particle size Analysis :-

The method of separation of soil into different fraction based on the particle size known as particle size analysis. It is also known as mechanical analysis.

Type of Particle size Analysis :-

mechanical analysis are two type
- → sieve Analysis
- → sedimentation Analysis

sieve Analysis :-

- → sieves are deginated by the size of square opening in mm or micron
- → sieves are various size ranging from 80mm to 70M are available
- → The diameter sieves is generally 15cm to 20cm
- → It is meant for coarse grain soil.
- → The coarse grains soil is divided into two types
  - i) Gravel Fraction
  - ii) sand Fraction.

Gravel Fraction :-

- → The sieves size more than 4.75mm is known as gravel fraction.
- → A set up coarse grain consisting of the sieves of size 80mm, 40mm 20mm, 10mm, 4.47mm required for gravel fraction.

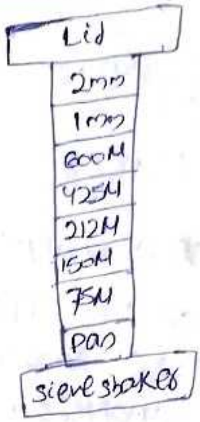
sand Fraction :-

- → The sieves size less than 4.47mm is known as sand Fraction.
- → A set up size consisting of ~~20mm~~, 2mm, 1mm, 600M, 425M, 212M, 150M, 75M is used for sand fraction.

Types of sieves Analysis :-

There are two types of sieves analysis
- → Dry sieves Analysis
- → weight sieves Analysis

Dry sieves Analysis :-



→ stepped are ~~one~~ over the other with decrease size top to bottom.

→ The sieve are stepped one over the other with decreasing size from top to bottom.

→ The sample is placed in top sieve.

→ The sieve of the largest opening is kept at the top

→ A lid is placed at the top of the largest sieve.

→ A pan which ~~is~~ has no opening is placed at the bottom of the smallest sieve.

→ The set up sieve is kept on the sieve shaker or hand sieving is done for 15 minutes generally.

→ The mass of soil retaining on its each sieves & pan is weighted with the help of weighting balancing machine.

→ The mass of the retain soil is checked against original mass.

Fines modules :-

It is ratio of comulative % of weight retain to the hundred or percentage

$$\text{Fines modules} = \frac{\text{comulative \% of weight retain}}{100}$$

→ It has no unit.

Take total weight - W

In sieve particle size	Weight retained	% of retain	comulative % at reaction	percentage of fines
400 mm	W_1	$\frac{W_1}{W} \times 100$	$C_1 = P_1$	$100 - C_1$
80 mm	W_2	$\frac{W_2}{W} \times 100$	$C_2 = P_1 + P_2$	$100 - C_2$
40 mm	W_3	$\frac{W_3}{W} \times 100$	$C_3 = P_1 + P_2 + P_3$	$100 - C_3$
20 mm	W_4	$\frac{W_4}{W} \times 100$	$C_4 = P_1 + P_2 + P_3 + P_4$	$100 - C_4$
10 mm	W_5	$\frac{W_5}{W} \times 100$	$C_5 = P_1 + P_2 + P_3 + P_4 + P_5$	$100 - C_5$
4.75 mm	W_6	$\frac{W_6}{W} \times 100$	$C_6 = P_1 + P_2 + P_3 + P_4 + P_5 + P_6$	$100 - C_6$
2.36 mm	W_7	$\frac{W_7}{W} \times 100$	$C_7 = P_1 + P_2 + P_3 + P_4 + P_5 + P_6 + P_7$	$100 - C_7$

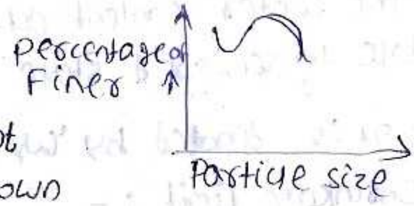
~~weight~~ sieve Analysis :-

It is the soil contain substatial wantity (say more than 5%) of Fine Particles, a ~~weight~~ analysis rewired.

Particle size distribution curve :-

- → It indicates the distribution of particle different sizes of soil mass.
- → It gives idea about the typend gradation of soil
- → It is also known as graduation of curve.
- → The Particle size distribution curve is plotted on a semilog graph paper. The percentage of Finally plotted as ordinate on a natural ~~fancal~~ scale & the particle size is plotted as absisa on logarithm scale. A soil sample may be either well graded on poorly graded.

well graded soil :-



If a soil contents the particle of different sizes in good proposition such a soil is known as well graded soil or uniformly graded soil.

Poorly graded soil :-

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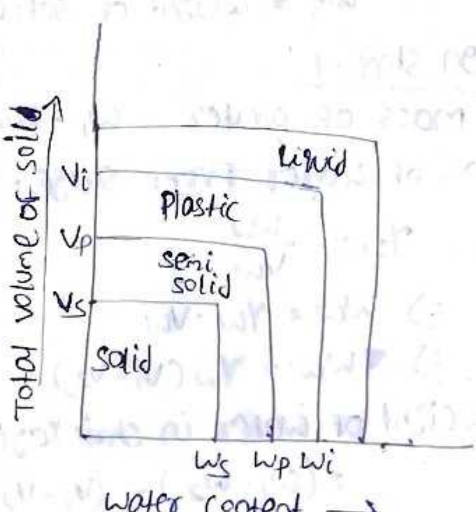
when the soil has an excess ~~access~~ of certain particle & deficiency or other is known as poorly graded soil.

consistency of soil :-

- consistency of soil means degree of firmness of the soil which may be termed as soft stiff or hard.
- → As ~~Per~~ swedish aggricultural Engineer Atter berg's find out that a Fine grain soil enisting in 4 steps
  1. (1) Liquid state
  2. (2) Plastic state
  3. (3) semi solid state
  4. (4) Solid state

consistency Limit :-



- → The water content at which the soil changes from one step to other is known as consistency limit.
- → The graph is plotted in betn Total volume of soil and water content.
- → The total volume of soil is plotted as ordinate & the water content is plotted as absisa.

Liquid state / Liquid Limit :-

The minimum water content at which the soil remaining liquid state is known as liquid state or liquid limit.

or

→ The water content at which the soil changes from liquid state to plastic state is known as liquid limit.

→ It is denoted by 'W_L'.

Plastic Limit :-

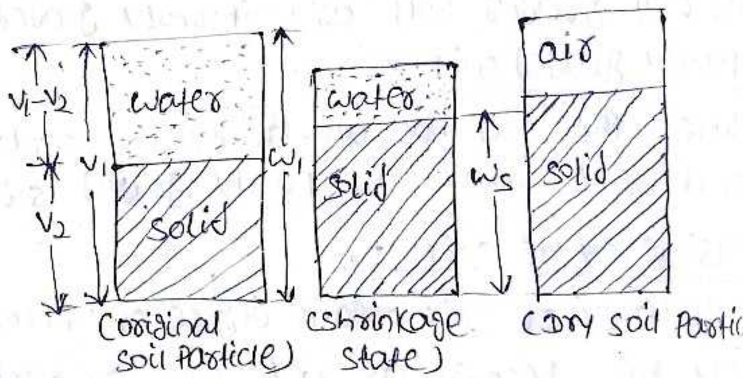
The minimum water content at which the soil remain in plastic state is known as Plastic limit.

or

The water content at which the soil changes from plastic state to semisolid state is known as plastic limit.

→ It is denoted by 'W_P'.

Shrinkage Limit :-



Here V_1 = volume of soil sample

V_2 = volume of soil at shrinkage state

W_1 = Total weight of soil

W_S = weight of solid

In stage-1

mass of water $= W_1 - W_S$

loss of water from stage-1 to stage-2

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

$$\Rightarrow W_w = \gamma_w \cdot V_w$$

$$\Rightarrow W_w = \gamma_w (V_1 - V_2)$$

weight of water in shrinkage state

$$= (W_1 - W_2) - (V_1 - V_2)\gamma_w$$

Shrinkage limit = $\frac{\text{weight of water in shrinkage state}}{\text{weight of solid in shrinkage state}}$

Mathematically

$$\frac{(W_1 - W_s) - (V_1 - V_2)\gamma_w}{W_s}$$

Plasticity Index (I_P):-

⇒ The difference betn liquid and plastic limit is known as plasticity index.

→ It is denoted by I_P

⇒ mathematically $I_P = W_L - W_P$

where W_L = Liquid limit

W_P = Plastic limit.

Shrinkage Index (I_S):-

⇒ The difference betn plastic limit & shrinkage limit is known as shrinkage index.

→ It is denoted by 'I_S'

→ mathematically $I_S = W_P - W_S$

where W_P = Plastic limit

W_S = Shrinkage limit.

Liquidity Index (I_L):-

⇒ It is denoted by 'I_L'

→ mathematically $I_L = \left(\frac{\omega - \omega_P}{I_P}\right) \times 100$

where I_P = Plastic index

ω_P = plastic limit

ω = natural water content.

→ when ω becomes ω_P then the liquidity index becomes zero

so $\omega = \omega_P$

$$I_L = \left(\frac{\omega_P - \omega_P}{I_P}\right) \times 0$$

$$I_L = 0$$

⇒ Then the soil is in hard state. sometimes it is also known as water plasticity ratio.

→ when ω becomes ω_L then the liquidity index is 100% or 1

Hence the soil is in liquid state.

→ mathematically $I_L = \left(\frac{\omega_L - \omega_P}{I_P}\right) \times 100$

$$I_L = \left(\frac{I_P}{I_P}\right) \times 100$$

$$I_L = 100\% \text{ or } 1$$

Volumetric shrinkage (V_s) :-

→ It is denoted by the letter 'V_s'

mathematically $V_s = \left(\frac{V_1 - V_d}{V_d}\right) \times 100$

where V_1 = volume of soil mass with respect to water content ω_1,

$V_d = V_2$ (volume of soil mass with respect to water content ω_2

Shrinkage ratio (S_R) :-

→ It is denoted by S_R.

→ mathematically $S_R = \left(\frac{\frac{V_1 - V_d}{V_d}}{\omega_1 - \omega_2}\right) \times 100$

Consistency Index (I_C) :-

→ It is denoted by I_C.

mathematically $I_C = \left(\frac{\omega_L - \omega}{I_P}\right) \times 100$

when ω becomes ω_P the consistency index is 100% or 1

$I_C = \left(\frac{\omega_L - \omega_P}{I_P}\right) \times 100$

$= \left(\frac{I_P}{I_P}\right) \times 100 = 100\%$ (Then the soil is hard or strong)

→ The soil is relatively strong or hard

→ when ω becomes ω_L then the soil is soft

mathematically $I_C = \left(\frac{\omega_L - \omega_L}{I_P}\right) \times 100$

$I_C = 0$ (Then the soil is soft)

Flow index (I_F) :-

→ It is denoted by $I_F = \frac{\omega_1 - \omega_2}{\log_{10}\left(\frac{N_2}{N_1}\right)}$

when N_1 = Number of blows

ω_1 = Corresponding water content

N_2 = number of blows

ω_2 = corresponding water content.

Toughness Index (I_T or I_t) :-

mathematically $I_t = \frac{I_P}{I_F}$

when I_P = Plastic index

I_F = Flow index.

* A soil has liquid limit 30% & a flow index 15.5%. If the plastic limit is 15%. Determine the plasticity index, toughness index. If the water content of the soil at its natural condition in the fill is 25% then find the liquidity index & consistency index.

Soln :-

Data is given

liquid limit (w_L) = 30%

Flow index (I_f) = 15.5%

plastic limit (w_P) = 15%

Natural water content (w) = 25%

plastic index (I_P) = $w_L - w_P$

$= 30 - 15 = 15$

Toughness index (I_t) = $\frac{I_P}{I_f} = \frac{15}{15.5} = 0.967 \times 100 = 96.77\%$

liquidity index (I_L) = $\left(\frac{w - w_P}{I_P}\right) \times 100$

$= \left(\frac{25-15}{15}\right) \times 100 = \left(\frac{10}{15}\right) \times 100 = 66.67\%$

Consistency index (I_C) = $\left(\frac{w_L - w}{I_P}\right) \times 100$

$= \left(\frac{30-25}{15}\right) \times 100$

$= \frac{5}{15} \times 100 = 33.33\%$

Specific Gravity (G) (G) :-

→ It is denoted by the letter 'G'.

$$G = \frac{1}{\left(\frac{1}{SR} - \frac{w_s}{100}\right)}$$

* The atterberg limits of a soil sample are $w_L = 52\%$, $w_P = 35\%$. $w_S = 14\%$. If the specimen of the soil shrink from a volume 16 cc at liquid limit 8.2 cc when it is over dried. Calculate shrinkage ratio & specific gravity of soil solid.

Soln :- Data given :-

$w_L = 52\% = w_1$

$w_P = 35\% = w_2$

$w_S = 14\%$

Find out shrinkage ratio (SR) = ?

specific gravity (G) = ?

Shrinkage ratio (SR) = $\left(\dfrac{\dfrac{V_1 - V_2}{V_2}}{W_1 - W_2}\right) \times 100$

$= \left(\dfrac{\dfrac{16 - 8.2}{8.2}}{52 - 35}\right) \times 100 \quad = 2.5$

specific gravity (G) = $\dfrac{1}{\left(\dfrac{1}{SR} - \dfrac{W_s}{100}\right)}$

$= \dfrac{1}{\left(\dfrac{1}{2.5} - \dfrac{14}{100}\right)} = 3.85$

Chapter-4
permeability

*What is permeability?

A:- The property of material or soil which permits the passage or seapage of water (other fluids) through its interconnecting voids is known as permeability.

-> A material having contineous voids is called permeable.

-> Gravel are highly permeable

-> stiff clay is least permeable & hence such a clay may be termed as impermeable

20.3.13

Darcy's Law

-> This law state that "for a laminar flow in a homogeneous soil velocity of flow it is directly proposnal to the hydraulic gradiant.

mathematically

$v \propto i$

where v = velocity of flow

i = hydraulic gradient

$v = ki$

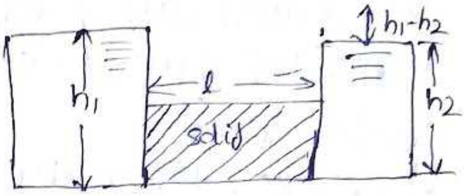
... co-efficient of permeability

Rate of discharge (q) :-

$$q = A \cdot V$$

where A = cross sectional area of soil mass

V = velocity



so $q = \cancel{K}\, AKi$

$q = KiA$ --- (1)

If the soil sample of the lane 'L' and cross sectional area 'A' is subjected to differential head of water $h_1 - h_2$

Then the hydraulic gradient $i = \frac{h_1 - h_2}{L}$

Now putting the value in eqn (1) we get

$$q = K \times \frac{h_1 - h_2}{L} \times A$$

Co-efficient of permeability :-

When the hydraulic gradient is unity (means 1) then

$$V = K \times i$$

$$V = K \quad \text{or} \quad K = V$$

Hence the co-efficient of permeability is defined as the average velocity of flow of water through the homogeneous soil mass under unit hydraulic gradient.

Unit of permeability or co-efficient of permeability :-

Unit of permeability is ~~seem~~ same as velocity i.e. meter/second, m/day or c.m/second

Factor affecting permeability :- Imp

The factors affecting permeability can be easily understood from the expression for the co-efficient of permeability.

$$\boxed{K = DS^2 \times \frac{\gamma_w}{n} \times \frac{e^3}{1+e} \times C}$$

where DS = Diameter of spherical grain

γ_w = unit weight of water

n = viscosity

e = void ratio

C = constant known as new shape constant

The factor affecting permeability are,

(i) Grain size
(ii) properties of pore fluid
(iii) Effect of void ratio
(iv) structural arrangement of soil particle
(v) Entrapped air & foreign matter
(vi) Adsorbed water in clayey soil

Grain size :-

→ It indicates, the permeability is directly proposnal to grain size.

→ mathematically

$$K \propto Ds^2$$

$$\Rightarrow K = CDs^2$$

where C = constant

Ds = Diameter of grain

Generally, D_{10} is used here

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

where D_{10} = Effective size

Properties of ~~Por~~ pore fluid :-

→ It indicates, the permeability is directly proposnal to the unit weight of water & inversely proposnal to the viscosity

mathematically

$$K \propto \gamma_w$$

$$K \propto \frac{1}{\eta}$$

$$\Rightarrow K \propto \frac{\gamma_w}{\eta}$$

$$\Rightarrow K = K_P \cdot \frac{\gamma_w}{\eta}$$

$$\Rightarrow K_P = K \cdot \frac{\gamma_w}{\eta}$$

where K_P = constant known as Physical permeability constant.

~~Effect of void ratio~~ :-

→ Through the unit weight of water does not change with the temp, there is a great variation in viscosity with temp.

Effect of void ratio :-

→ It indicates that the effect of void ratio on the values of permeability

→ Mathematically

$$\frac{1}{K_2} = \left[\frac{C_1 e_1^3}{1+e_1}\right] \div \left[\frac{C_2 e_2^3}{1+e_2}\right]$$

→ If the void space in a soil particle is more than permeability increases.

Structural arrangement of soil particle :-

→ The fine grain soil with floculated structural more permeable then the fine grain soil ~~th~~ with this dispersed structure.

Entrapped air & foreign matter :-

→ Entrapp air in the void of soil mass decreases the permeability

→ organic ~~void~~ foreign matter present in the voids mass decreases the permeability.

• Effect adsorbed water in claye soil :-

The adsorb water surrounding the fine soil particle is not free ~~m~~ to move & requires pore space or void space available for passage of water or flow of water. So this presence in soil decreases the permeability

Chapter - 5 COMPACTION

Defination :- It is a mechanical method by which pressing the soil particle close to each other

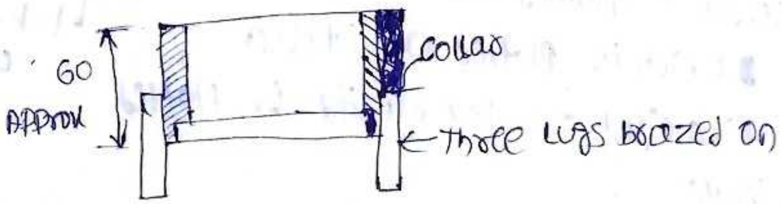
→ Due to compaction, the stability & bearing capacity of soil increases

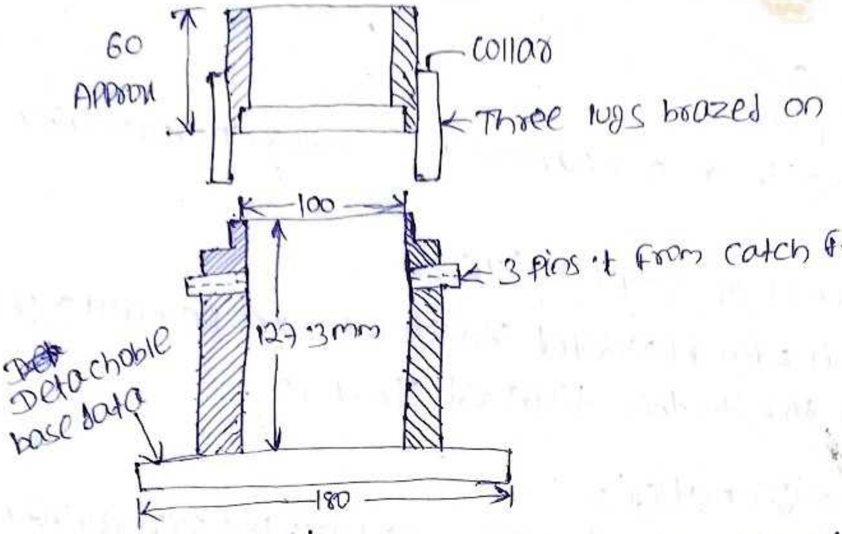
→ Due to compaction, the permeability decreases.

→ It is required for the construction of earth dam, ~~highity~~ highway & railway.

→ The amount of compaction & water content can be find out by standa[rd] proctor test.

Standard - proctor test :-





→ It consists of a mold of 100mm internal diameter, 127.3mm height & 1000ml capacity.

→ The mold has a base plate & removal collar of height 60mm

→ The required amount of soil is taken in the mold

→ The soil was compacted in the mold 3 equal layer

→ Each layer has given 25 blows

→ A rammer compacted is of weight 2.6kg mass with a free of fall of height 310 mm of collar 60mm.

→ If the percentage of soil return on 4.75mm sieve is more than 20%.

large mould should be used.

→ The size of mold of diameter of 150 mm height of 127.3 mm & capacity 2250ml & no. of blows required 28

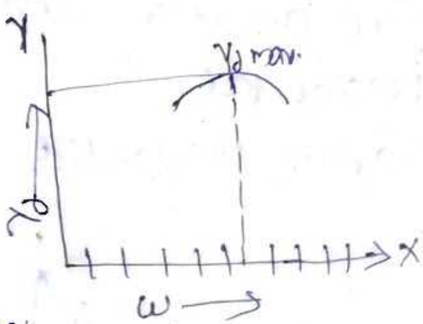
→ mathematically $\boxed{\gamma_d = \frac{\gamma}{1+\omega}}$

where γ_d = dry unit weight

γ = Bulk unit weight

ω = water content

Compaction Curve :-



→ It is a curve which is plotted on a graph paper

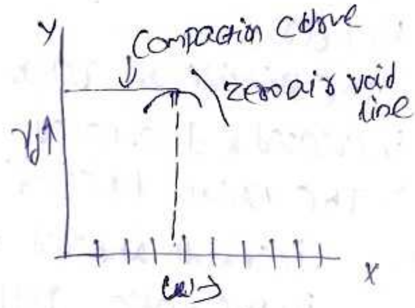
→ This curve is plotted betn the water content which is plotted as absisa & dry unit weight or dry density is plotted as ordinate.

→ There is increase in water content $w \propto \gamma_d$, γ_d also increases.
→ A stage will occur given we will increase the water content then γ_d cannot increases. Its value decreases.
→ The water content for solding to the maximum dry density is known as obtimum moisture content or OMC

25·3·13

Zero Air void line :-

→ When there is no air void all degree of saturation is=100%. at that instant we can found γ_d



→ In this condition the soil became saturated by reduction γ_d in air void to zero but with no changes with water content.
→ The zero air void line is also known as saturation line
→ The zero air void line curve is plotted betn water content & dry density on a graph paper.
→ The water content as absaisa & the dry density is plotted as ordinate
→ It is more or less parallel to the right limb of compaction curve.
→ The soil could also become saturated by increasing the water content such that all air voids are filled
→ The dry density is calculated by following formula

$$\gamma_d = \frac{(1-n_a) G \gamma_w}{1+wG}$$

where w = water content
G = specific gravity
γ_w = unit weight
n_a = percentage of air void

Imp

Method of compaction / Field compaction method / Method of compaction used in Field :-

→ Several methods are used for compaction of soil in field are

i) Rammer or Tamper
ii) Roller
iii) vibrator

Rammer / Tamper :-

→ A hand operated rammer consists of a block of iron or stone about 3-5 kg is mass attach to a wooden rod

→ The rammer or tamper is lifted by about ·3 mt. & drop on the soil to be compacted.

→ It can be used for all types of the soil

→ It is not used where large quantities of soil are involved

Rollar :-

→ The principle of rollars is the application of pressure, which is slowly increased & decreased

→ The various types of rollars which are used for the compaction are

i) Smooth wheeled rollar

ii) Pneumatic rollar

iii) Sheeps foot rollar

Smooth wheeled rollar :-

→ The compacting efficiency of the smooth ~~field~~ rollar are suitable to roll a wide range of soil, such as pavement materials for the various layer.

Pneumatic rollar :-

→ This type of rollars are pulled by tractors.

→ This type of rollar is more suitable to compact the fine sand

→ This type of rollar

Sheeps foot rollar :-

→ This type of rollar is consists of a ~~w~~hole of steel cylinder

→ The weight of this type of rollar can be increased by filling the drum with wet soil

→ This rollars may be pulled ~~Gooted~~ by tractors.

→ The efficiency of this type of rollars depends on the weight of the rollar.

→ This type of rollar gives ~~gets~~ adequate compaction

→ However the top layer of the sub grade may be compacted using smooth wheel rollar so as to get a properly finished surface vibrator:-

→ The vibrator consists of vibrating unit

→ These are use to remove void space

→ The vibrators are available a variet form in the market.

Consolidation :-

Defination :- The compression of saturated soil under steady static pressure is termed as consolidation which is termed as consolidation. It is completely due to expulsion of water from the voids.

→ It is generally related to fine grained soils such as silts & clays. saturated clay consolidate at a much slower rate due to their low permeability.

Distinction betn compaction & consolidation :-

Compaction	Consolidation
(1) Mechanical process by which unit weight of soil increased at a particular water content	(1) Natural process under sustained load due to which volume of soil decreases gradually.
(2) Explusion of pore air	(2) occurs due to expel of pore water.
(3) Always partially saturated soil	(3) Always full saturated soil.
(4) Instantaneous process	(4) Time taking process
(5) Mechanical equipment are used e.g. rammers & rollers.	(5) Natural process that occurs over long period of time.
(6) Leads to increase in shear strength of soil, stability & bearing capacity.	(6) No such benefits.

Shear strength

Concept of shear strength :-

The shear strength of soil is the resistance to deformation by continuous shear displacement of soil particles or on masses upon the action of a shear stress.

18.3. MOHR-COULOMB FAILURE THEORY

Of the many theories of failure that have been proposed, only that formulated by Mohr (1900) has been useful in case of soils. The following are essential points of Mohr's strength theory :

1. Material fails essentially by shear. The critical shear stress causing failure depends upon the properties of the material as well as on normal stress on the failure plane.

2. The utlimate strength of the material is determined by the stresses on the potential failure plane (or plane of shear).

3. When the material is subjected to three dimensional principal stress (i.e. σ_1, σ_2, σ_3) the intermediate principal stress does not have any influence on the strength of material. In other words, the failure criterion is independent of the intermediate principal stress.

Note. For detailed discussions on various theories of failure, see Chapter 19, where the effect of the intermediate principal stress has also been disussed.

The theory was first expressed by Coulomb (1776) and later generalised by Mohr. The theory can be expressed algebraically by the equation. $\tau_f = s = F(\sigma)$ ...(18.6)

where $\tau_f = s =$ shear stress on failure plane, at failure = shear resistance of material

$F(\sigma) =$ function of normal stress

If the normal and shear stress corresponding to failure are plotted, then a curve is obtained. The plot or the curve is called the *strength envelope*. Coulomb defined the function $F(\sigma)$ as a linear function of σ and gave the following strength equation :

$$s = c + \sigma \tan \phi \qquad \text{...(18.7)}$$

where the empirical constants c and ϕ represent repectively, the intercepts on the shear axis, and the slope of the straight line of Eq. 18.7 [Fig 18.3 (*a*)]. These parameters are usually termed as *cohesion* and *angle of internal friction* or *shearing resistance* respectively.

...h has the maxi...

MEASUREMENT OF SHEAR STRENGTH

The measurement of shear strength of soil involves certain test obsevations at failure ...h the help of which the *failure envelope* or *strength envelope* can be plotted corresponding ...a given set of conditions (specially the drainage conditions). Shearing resistance can be ...ined in the laboratory by the following four methods :

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

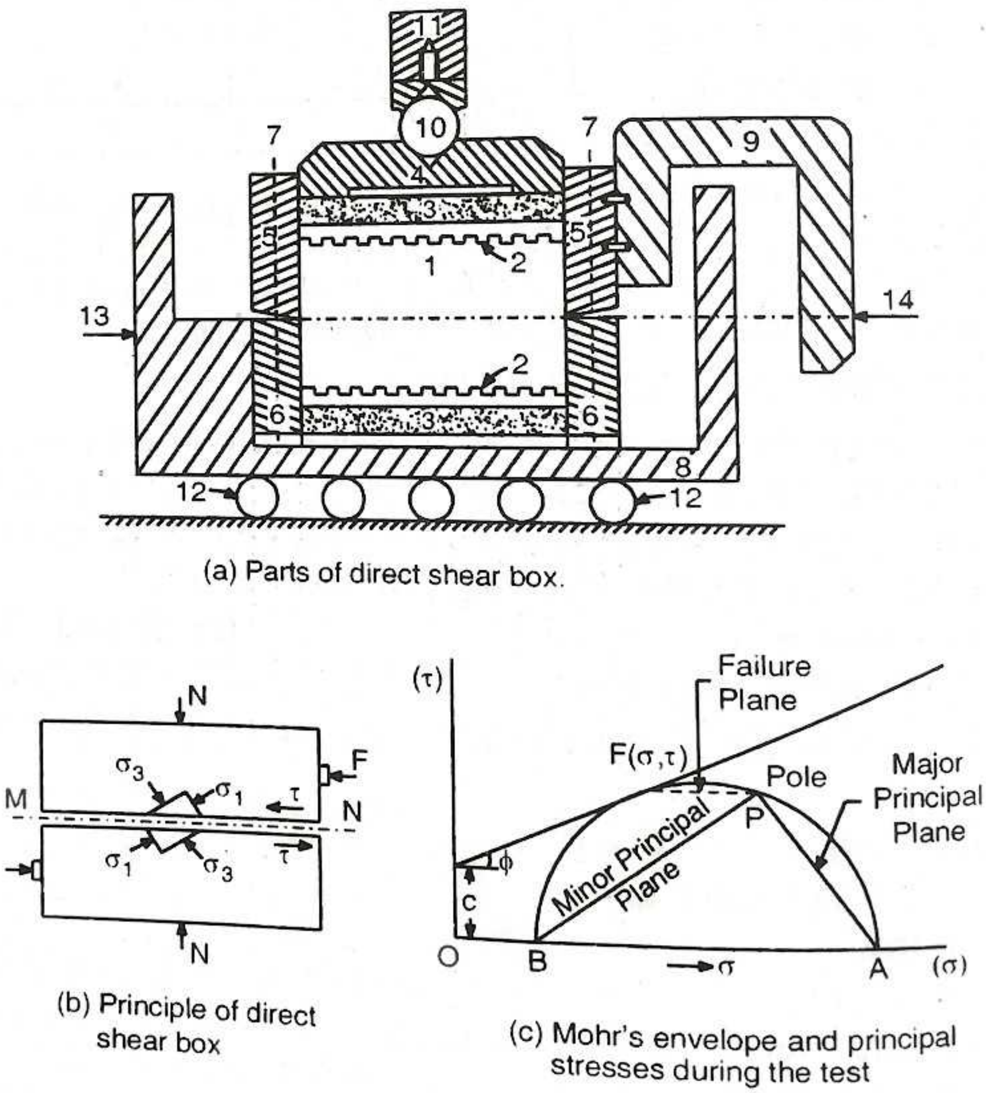
Again depending upon the *drainage conditions*, three types of shear tests have been ...oped :

(*a*) Undrained test or quick test
(*b*) Consolidated undrained test
(*c*) Drained test.

In the *undrained* or *quick test*, no drainage of water is permitted. Hence there ...no dissipation of pore pressure during the entire test. In the *drained test*, drainage is ...itted throughout the test during the application of both normal and shear stresses, so ...full consolidation occurs and no excess pore pressure is set up at any stage of the ... . In the consolidated-undrained test, drainage is permitted under the initally applied ...al stress only and full primary consolidation or softening is allowed to take place. ...drainage is allowed afterwards. The parameters c and ϕ are not fundamental properties ...the soil; they may simply be considered merely coefficients derived from the geometry ...the graph obtained by the plotting shear stress at failure against normal stress. They ...ary with drainage conditions of the test.

18.6. DIRECT SHEAR TEST

This is a simple and commonly used test and is performed in a shear-box appa (Fig. 18.6). The apparatus consists of a two piece shear box of square or circular cross-se The lower half of the box is rigidly held in position in a container which rests slides or rollers and which can be pushed forward at a constant rate by geared driven either by electric motor or by hand. The upper half of the box butts aga proving ring. The soil sample is compacted in the shear box, and is held between grids and porous stones (or plates). As shown in Fig. 18.6 (*a*), the upper half of specimen is held in the upper box and the lower half in the lower box, and the between the two parts of the box is at the level of the centre of the specimen. load is applied on the specimen from a loading yoke bearing upon steel ball of pr



(a) Parts of direct shear box.

(b) Principle of direct shear box

(c) Mohr's envelope and principal stresses during the test

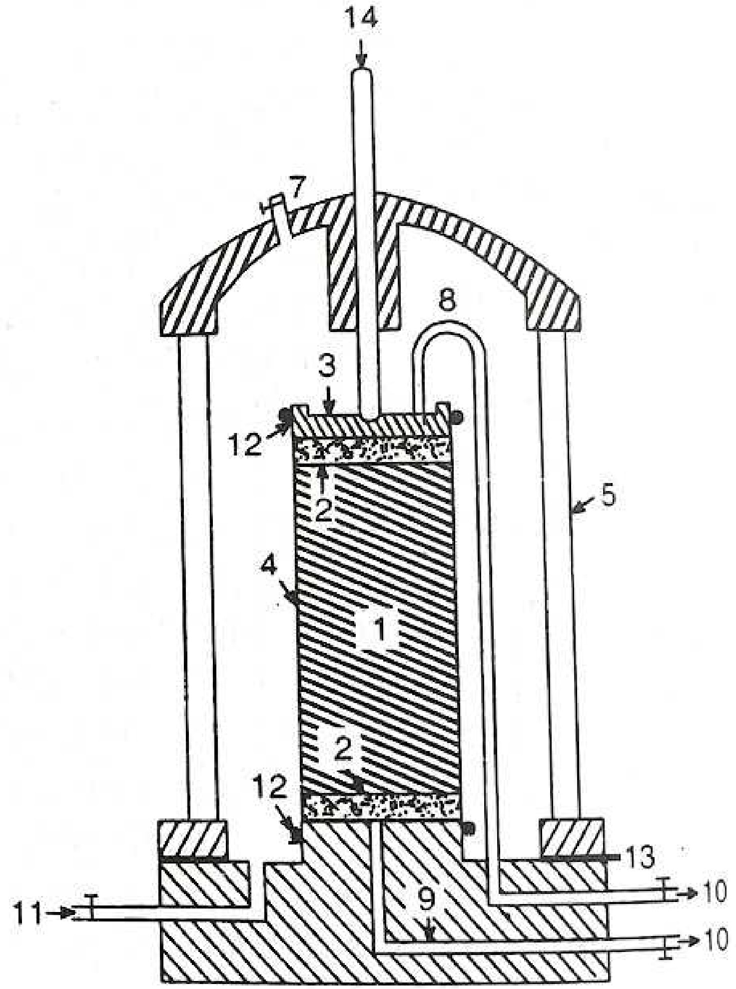
1. SOIL SPECIMEN
2. METAL GRIDS
3. POROUS STONES
4. LOADING PAD
5. UPPER PART
6. LOWER PART
7. SCREWS TO FIX TWO HALVES OF SHEAR BOX
8. CONTAINER FOR SHEAR BOX
9. U-ARM
10. STEEL BALL
11. LOADING YOKE
12. ROLLERS
13. SHEAR FORCE APPLIED BY JACK
14. SHEAR RESISTANCE MEASURED PROVING RING

FIG. 18.6. SHEAR BOX TEST.

18.7. TRIAXIAL COMPRESSION TEST

The strength test more commonly used in a research laboratory today is the triaxial compression test, first introduced in the U.S.A. by A. Casagrande and Karl Terzaghi in 1936-37. The solid specimen, cylindrical in shape, is subjected to direct stresses acting in three mutually perpendicular directions. In the common solid cylindrical specimen test, the major principal stress σ_1 is applied in the vertical direction, and the other two principal stresses σ_2 and σ_3 ($\sigma_2 = \sigma_3$) are applied in the horizontal direction by the fluid pressure round the specimen.

The test equipment specially consists of a high pressure cylindrical cell, made of perspex or other transparent material, fitted between the base and the top cap. Three outlet connections are generally provided through the base : cell fluid inlet, pore water out let from the bottom of the specimen and the drainage



1. SOIL SPECIMEN
2. POROUS DISC
3. TOP CAP
4. RUBBER MEMBRANE
5. PERSPEX CYLINDER
6. LOADING RAM
7. AIR RELEASE VALVE
8. TOP DRAINAGE TUBE
9. BOTTOM DRAINAGE TUBE
10. CONNECTIONS FOR DRAINAGE OR PORE PRESSURE MEASUREMEN
11. CELL FLUID INLET
12. RUBBER RINGS
13. SEALING RING
14. AXIAL LOAD THROUGH PROVING RING

FIG. 18.7. THE TRIAXIAL CELL.

18.8. UNCONFINED COMPRESSION TEST

The unconfined compression test is a special case of triaxial compression test in $\sigma_2 = \sigma_3 = 0$. The cell pressure in the triaxial cell is also called the confining pressure. Due to the absence of such a confining pressure, the uniaxial test is called the unconfined compression test. The cylindrical specimen of soil is subjected to major principal stress σ_1 till the specimen fails due to shearing along a critical plane of failure.

In its simplest form, the apparatus consists of a small load frame fitted with a proving ring to measure the vertical stress applied to the soil specimen. Fig. 18.12 (*a*) shows an unconfined compression tester (Goyal and Singh, 1958). The deformation of the sample is measured with the help of a separate dial gauge. The ends of the cylindrical specimen are hollowed in the form of cone. The cone seatings reduce the tendency of the specimen to become barrel shaped by reducing end-restraints. During the test, load versus deformation readings are taken and a graph is plotted. When a brittle failure occurs, the proving ring dial indicates a definite maximum load which drops rapidly with the further increase of strain. In the plastic failure, no definite maximum load is indicated. In such a case, the load corresponding to 20% strain is arbitrarily taken as the failure load.

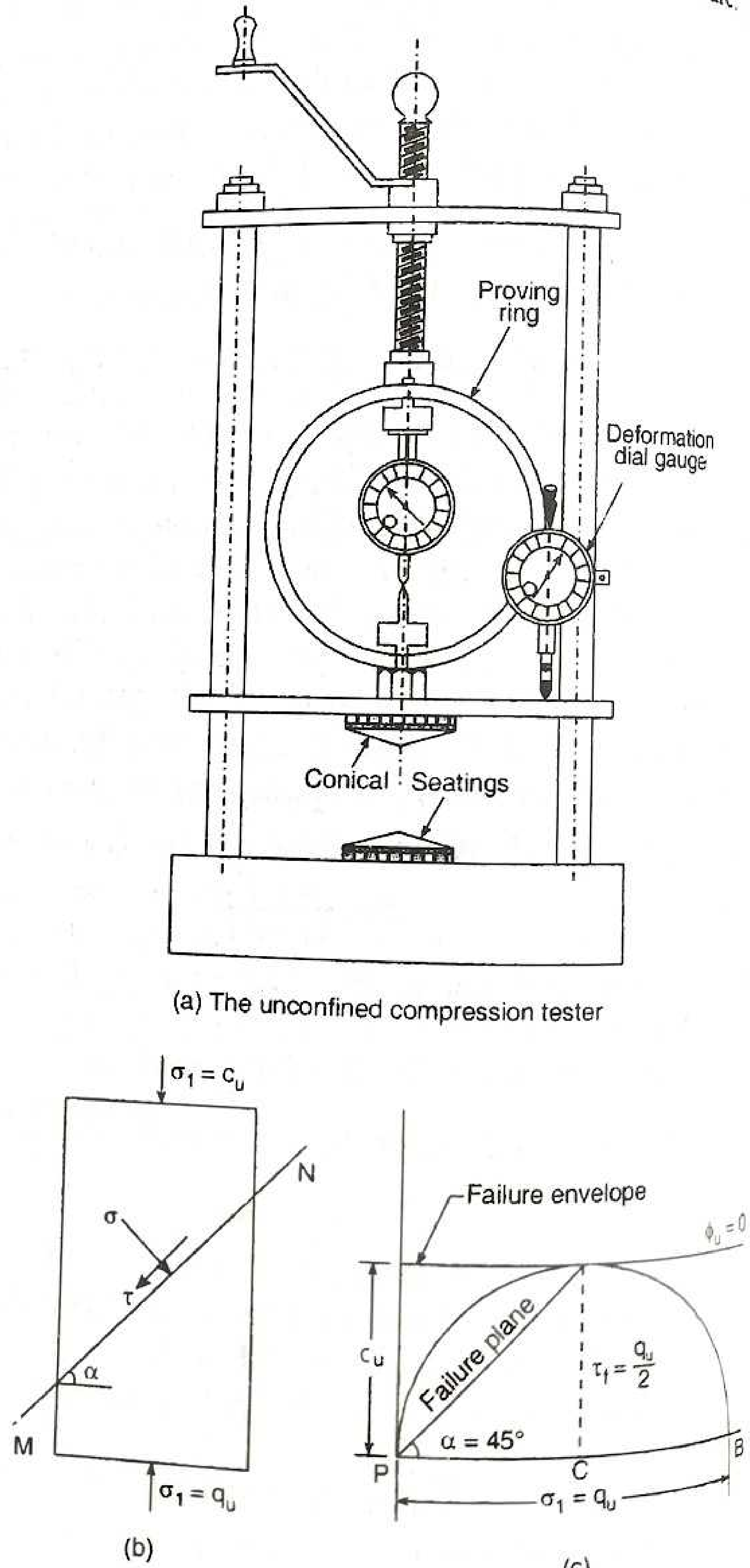


FIG. 18.12. UNCONFINED COMPRESSION TEST.

20.3. EARTH PRESSURE AT REST*

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

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

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

Hence $\quad \sigma_h = \mu\,(\sigma_h + \sigma_v) \quad$ or $\quad \dfrac{\sigma_h}{\sigma_v} = K_0 = \dfrac{\mu}{1-\mu} \quad$ ...(20.10)

where K_0 is *coefficient of the earth pressure at rest.*

Designating the lateral earth pressure (σ_h) at rest by p_0 and substituting $\sigma_v = \gamma z$, we have

$$p_0 = K_0\,\gamma z$$

The pressure distribution diagram is thus triangular with zero intensity at $z = 0$ and an intensity of $K_0\,\gamma H$ at the base of the wall, where $z = H$. The total pressure P_0 per unit length for the vertical height H is given by

$$P_0 = \int_0^H K_0\,\gamma z\,.\,dz = \tfrac{1}{2} K_0\,\gamma H^2 \qquad \text{...(20.12)}$$

The behaviour of soil is not in accordance with the elastic theory and do not have a well-defined value of the Poission's ratio. Table 20.1 gives some value of K_0 available from experience.

$P_0 = \frac{1}{2} \times 0.3610$

20.4. ACTIVE EARTH PRESSURE : RANKINE'S THEORY

As originally proposed, Rankine's theory of lateral earth pressure is applied to uniform cohesionless soils only. Later, it was extended to include cohesive soils, by Resal (1910) and by Bell (1915). The theory has also been extended to stratified, partially immersed and submerged soils. Following are the assumptions of the Rankine theory :

1. The soil mass is semi-infinite, homogeneous, dry and cohesionless.
2. The ground surface is a plane which may be horizontal or inclined.
3. The back of the wall is vertical and smooth. In other words, there are no shearing stresses between the wall and the soil and the stress relationship for any element adjacent to the wall is the same as for any other element far away from the wall.
4. The wall yields about the base and thus satisfies the deformation condition for plastic equilibrium.

However, the retaining walls are constructed of masonry or concrete, and hence the back of the wall is never smooth. Due to this, frictional forces develop. As a consequence of Rankine's assumption of no-existence of frictional forces at the wall face, the resultant pressure must be parallel to the surface of the backfill. The existence of the friction makes the resultant pressure inclined to the normal to the wall at an angle that approaches the friction angle between the soil and the wall.

The following cases of cohesionless backfill will now be considered :

1. Dry or moist backfill with no surcharge.
2. Submerged backfill
3. Backfill with uniform surcharge.

Bearing Capacity

24.1. DEFINITIONS

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

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

3. **Foundation soil**. It is the upper part of the earth mass carrying the load of the structure.

4. **Bearing capacity**. The supporting power of a soil or rock is referred to as its bearing capacity. The term bearing capacity is defined after attaching certain qualifying prefixes, as defined below.

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

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

$$q_n = q - \overline{\sigma} = q - \gamma D \qquad \text{...(24.1)}$$

where γ is the average unit weight of soil above the foundation base.

7. **Ultimate bearing capacity** (q_f). The ultimate bearing capacity is defined as the minimum gross pressure intensity at the base of the foundation at which the soil fails in shear. (When the term bearing capacity is used in this book without any prefix, it may be understood to be ultimate bearing capacity).

8. **Net ultimate bearing capacity** (q_{nf}). It is the minimum net pressure intensity causing shear failure of soil. The *ultimate bearing capactiy* q_f and the *net ultimate capacity* are evidently connected by the following relation :

$$q_f = q_{nf} + \overline{\sigma} \qquad \text{...(24.2)} \qquad \text{or} \qquad q_{nf} = q_f - \overline{\sigma} \qquad \text{...(24.2 } a)$$

where $\overline{\sigma}$ is the *effective surcharge* at the base level of the foundation.

9. **Effective surcharge at the base level of foundation** ($\overline{\sigma}$). It is the intensity of vertical pressure at the base level of foundation, computed assuming total unit weight for

the portion of the soil above the water table and submerged unit weight for the po[r]
below the water table.

10. Net safe bearing capacity (q_{ns}). The net safe bearing capacity is the net ulti[mate]
bearing capacity divided by a factor of safety F

$$q_{ns} = \frac{q_{nf}}{F} \qquad \ldots(2$$

11 Safe bearing capacity (q_s). The maximum pressure which the soil can [carry]
safely without risk of shear failure is called the safe bearing capacity. It is equal to
net safe bearing capacity plus original overburden pressure:

$$q_s = q_{ns} + \gamma D = \frac{q_{nf}}{F} + \gamma D \qquad \ldots(24$$

Sometimes, the safe bearing capacity is also referred to as the ultimate bearing cap[acity]
q_f divided by a factor of safety F.

12. Safe bearing pressure or Net soil pressure for specified settlement. It i[s]
the intensity of loading that will cause a permissible settlement or specified settlement fo[r]
the structure.

13. Allowable bearing capacity or pressure (q_a). It is the net loading intensity a[t]
which neither the soil fails in shear nor there is excessive settlement detrimental to the
structure.

24.2. MINIMUM DEPTH OF FOUNDATION : RANKINE'S ANALYSIS

Rankine considered the equilibrium of two soil elements, one immediately below
foundation (element I), and the other just beyond the edge of the footing (element II),
but adjacent to element I. When the load on the footing increases, and approaches a value
q_f, a state of plastic equilibrium is reached under the footing. For the shear failure
element I, element II must also fail by lateral thrust from element I. During the stat[e]
of shear failure (plastic equilibrium), the following principal stress relationsip exists

$$\sigma_1 = \sigma_3 \tan^2 \alpha + 2c \tan \alpha \qquad \ldots(24.5$$

For cohesionless soil $\qquad \sigma_1 = \sigma_3 \tan^2 \alpha \qquad \ldots(24.5$

For element II : $\qquad \sigma_3 = \sigma_v = \gamma D$

$\therefore \qquad \sigma_1 = \sigma_h = \gamma D \tan^2 \alpha \qquad \ldots(1)$

For element I :

$\therefore \qquad \sigma_3 = \sigma_h = \sigma_1$ of element II

$\qquad = \gamma D \tan^2 \alpha$

$\qquad \sigma_1 = \sigma_3 \tan^2 \alpha = \gamma D \tan^4 \alpha$

But $\qquad \sigma_1 = q_f$

$$\therefore \qquad q_f = \gamma D \tan^4 \alpha = \gamma D \left[\frac{1 + \sin \phi}{1 - \sin \phi} \right]^2 \qquad \ldots(24.6)$$

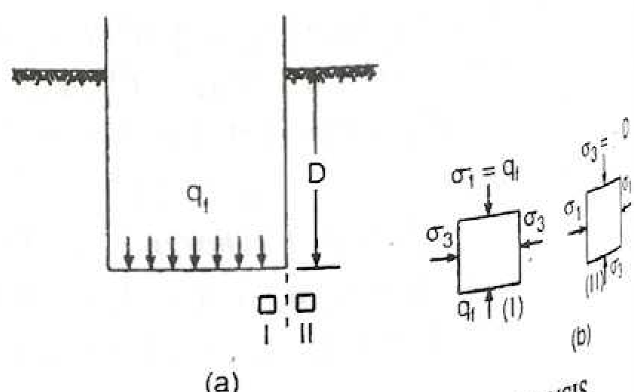


FIG. 24.1. RANKINE'S ANALYSIS.

In the effective stress analysis the above expression reduces to

$$q_f = \gamma D \left[\frac{1 + \sin \phi'}{1 - \sin \phi'} \right]^2 \qquad \ldots(24.7)$$

Eq. 24.6 gives the bearing capacity of cohesionless soils as zero at the ground surface. This is not consistent with the general experience. Eq. 24.6 may be used in the following form to get the minimum depth of foundation

$$D_{min} = \frac{q}{\gamma} \left[\frac{1 - \sin \phi}{1 + \sin \phi} \right]^2 \qquad \ldots(24.8)$$

where q = intensity of loading.

24.3. TYPES OF BEARING CAPACITY FAILURES

Experimental investigations have indicated that when a footing fails due to insufficient bearing capacity, distinct failure patterns are developed, depending upon type of failure mechanism. Failure is accompanied by appearance of *failure surfaces* and by building of sheared mass of soil. Vesic (1963) observed three types of bearing capacity failures (Fig. 24.2).

1. General shear failure, 2. Local shear failure and 3. Punching shear failure.

1. General shear failure : In the case of *general shear failure*, continuous failure surfaces develop between the edges of the footing and the ground surface, as shown in Fig. 24.2 (*a*). When the pressure approaches the value of q_f, the state of *plastic equilibrium* is reached initially in the soil around the edges of the footing, and it then gradually spreads downwards and out wards. Ultimately, the state of plastic equilibrium is fully developed throughout the soil above the failure surfaces. The failure is accompanied by appearance of failure surfaces and by considerable bulging of sheared mass of soil. However, the final slip movement would occur only on one side, accompanied by *tilting* of the footing. Such a failure occurs in soils of *low compressibility*, i.e. dense or stiff soil, and the pressure-settlement curve is of the general form as shown is curve *a* of Fig. 24.2 (*d*). Following are the typical *characteristics* of general shear failure.

(*i*) It has well defined failure surfaces, reaching upto ground surface

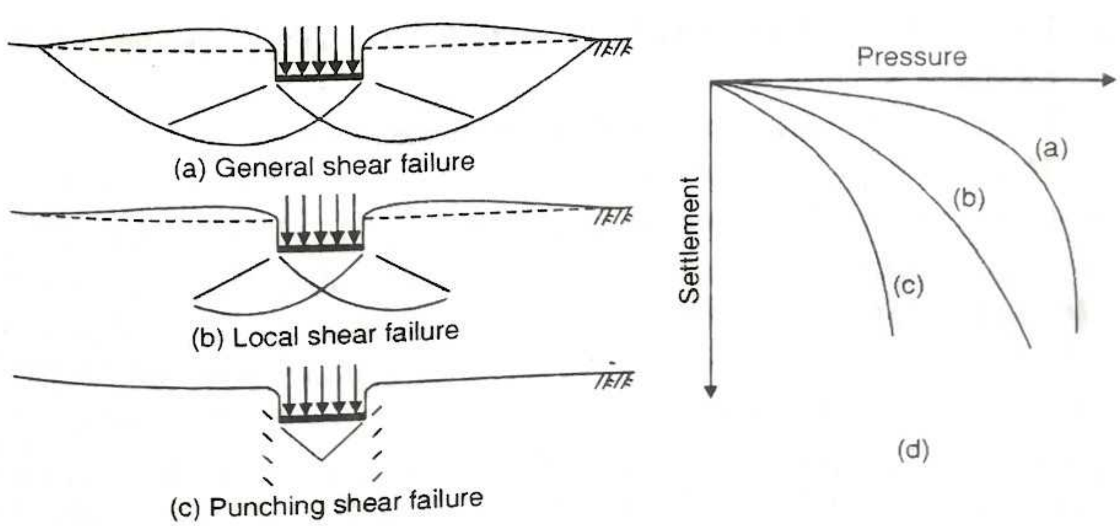


FIG 24.2 MODES OF BEARING CAPACITY FAILURES

(ii) There is considerable bulging of sheared mass of soil adjacent to the footing
(iii) Failure is accompanied by tilting of the footing
(iv) Failure is sudden, with pronounced peak resistance
(v) The ultimate bearing capacity is well defined

2. **Local shear failure** : In *local shear failure*, there is significant compression of the soil under the footing and only partial development of state of plastic equilibrium. Due to this reason, the failure surfaces do not reach the ground surface and only slight heaving occurs. The pressure-settlement curve is represented by curve b of Fig. 24.2(d), where the peak of the base resistance may never be reached. In such a failure, tilting of foundation is not expected. Local shear failure is associated with soils of high compressibility and in sands having relative density lying between 35 and 70 percent. The failure is not sudden, and it is characterised by occurrence of relatively large settlements which would not be acceptable in practice. Also, ultimate bearing capacity in such a failure is not well defined. Following are typical *characteristics* of local shear failure :

1. Failure pattern is clearly defined only immediately below the footing
2. The failure surfaces do not reach ground surface
3. There is only slight bulging of soil around the footing
4. Failure is not sudden and there is no tilting of footing
5. Failure is defined by large settlements
6. Ultimate bearing capacity is not well defined.

3. **Punching shear failure** : *Punching shear failure* occurs where there is relatively high compression of soil under the footing, accompanied by shearing in the vertical direction around the edges of the footing. Punching shear may occur in relatively loose sand with relative density less than 35%. Punching shear failure may also occur in a soil of low compressibility if the foundation is located at considerable depth. The failure surface, which is vertical or slightly inclined and follows the perimeter of the base, never reaches the ground surface. There is no heaving of the ground surface away from the edges and no tilting of the footing. Relatively large settlements occur in this mode. The ultimate bearing capacity is not well defined. Following the *characteristics* of punching shear failure:

(i) No failure pattern is observed.
(ii) The failure surface, which is vertical or slightly inclined, follows the perimeter of the base.
(iii) There is no bulging of soil around the footing
(iv) There is no tilting of footing
(v) Failure is characterised in terms of very large settlements
(vi) The ultimate bearing capacity is not well defined.

Conditions for typical mode of failure :

Table 24.1 gives the conditions under which a typical mode of failure may occur.

Modes of failure of footings in sand

Fig. 24.3 shows the modes of failure of footings in sand. As the relative depth/width ratio increases, the limiting relative densities at which failure type change increases.

Assumptions in Terzaghi's Analysis

1. The soil is homogeneous and isotropic and its shear strength is represented by Coulomb's equation.

2. The strip footing has a rough base, and the problem is essentially two dimensional.

3. The elastic zone has straight boundaries inclined at $\psi = \phi$ to the horizontal, and the plastic zones fully develop.,

4. P_p consists of three components which can be calculated separately and added, although the critical surface for these components are not identical.

5. Failure zones do not extend above the horizontal plane through the base of the footing, *i.e.* the shear resistance of soil above the base is neglected and the effect of soil around the footing is considered equivalent to a surcharge $\sigma = \gamma D$.