LECTURE NOTES

ON

GEOTECHNICAL ENGINEERING (TH. 2)

FOR

DIPLOMA IN CIVIL ENGINEERING

(3RD SEMESTER STUDENTS)

AS PER SCTE&VT SYLLABUS

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3.0 Determination Of Index Properties :—

Index properties are those properties which helps in identification and classification of soil.

3.1 Water Content

There are various methods of determining the water content of soil sample, and they are :—

i) Oven-drying method
ii) Sand bath method
iii) Alcohol Method
iv) Calcium Carbide method
v) Pycnometer method
vi) Radiation method
vii) Torsion-balance method.

i) Oven-drying Method

↳ Most accurate method of determining water content.

↳ A soil sample is kept in a clean container & put in an oven to maintain a temp. between 105°C to 110°C

↳ The sample is kept for 24 hrs for complete drying.

↳ Mass of clean non-corrodible container is taken with it's lid (M_1)

↳ Mass of moist soil in container with lid is taken as M_2.

↳ Then, container with moist soil is placed in oven after removing the lid.

↳ After drying, container is removed from oven & allowed to cool in a desiccator.

↳ The mass of dry soil, the container & lid is taken as M_3.

The water content is calculated from following expression :-

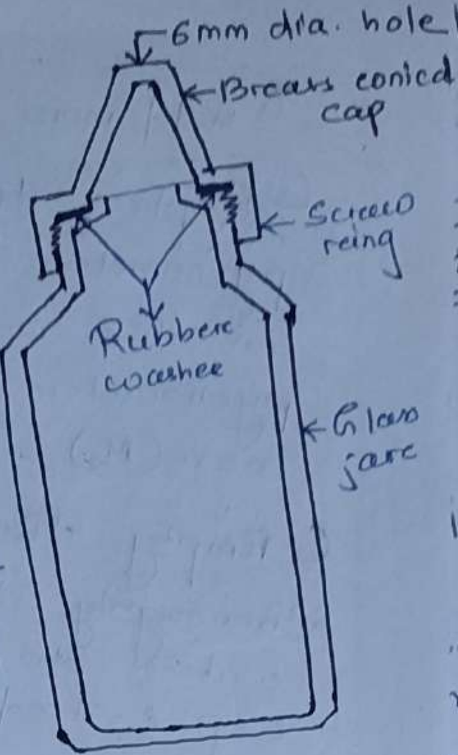
$$w = \frac{M_2 - M_3}{M_3 - M_1} \times 100$$

M_1 = Mass of container + lid

M_2 = mass of container + lid + moist soil

M_3 = mass of container + lid + dry soil.

ii) Pycnometer Method



- Quick method of determining water content.
- Pycnometer is a large density bottle of about 900ml capacity.
- Conical brass cap, having 6mm dia hole at it's top is screwed to the open end of pycnometer.
- Rubber washer is placed between conical cap and the rim of the bottle so no leakage of water takes place.

Procedure:-

- Take clean, dry pycnometer and find the mass with it's cap & washer (M_1).
- Put 200 gm to 400 gm of wet soil sample in the pycnometer and find its mass with its cap & washer (M_2).
- Fill the pycnometer to half it's height with water & mix with glass rod.

→ Vs -

↳ Add more water & stir it.

↳ Replace the screw top & fill the pycnometer flush with the hole in the conical cap. Dry the pycnometer from outside. Find the mass (M_3).

↳ Empty the pycnometer, clean it thoroughly & fill it with clean water to the hole of conical cap & find mass M_4.

Water content is calculated using following formula:–

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

3.2 Specific Gravity

Specific gravity of soil is determined by:–

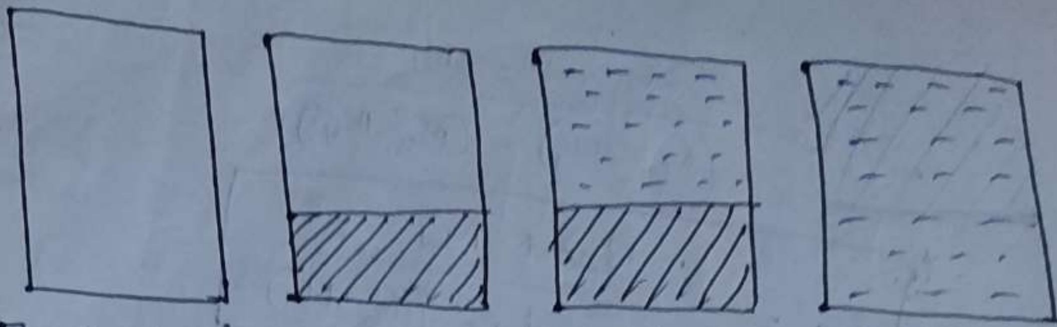
i> 50ml density bottle

ii> 500ml flask

iii> Pycnometer.

→ Density bottle is most accurate & suitable for all type of soils.

→ Flask or pycnometer is used only for coarse grained soil.



Empty Bottle (M_1) | Bottle + Dry soil (M_2) | Bottle + soil + water (M_3) | Bottle + water (M_4)

- Mass of empty dry bottle is taken as M_1.
- Sample of oven dried soil, cooled in desiccator, placed in bottle M_2.
- Bottle is filled with distilled water (or Kerosene) gradually, removing the entrapped air either by applying vaccum or by shaking the bottle. The mass is taken as M_3.
- The bottle is emptied completely & thoroughly washed & clean water (or Kerosene) is filled to top. The mass is taken as M_4.

Dry mass of soil $= M_2 - M_1 = M_d$.

$$G = \frac{\text{Dry mass of soil}}{\text{Mass of water of equal volume}}$$

$$= \frac{M_2 - M_1}{(M_4 - M_1) - (M_3 - M_2)}$$

$$\Rightarrow G = \frac{M_2 - M_1}{(M_2 - M_1) - (M_3 - M_4)}$$

$$G = \frac{M_d}{M_d - (M_3 - M_4)}$$

3.3. Particle Size Distribution

- Percentage of various sizes of particles in a dry soil sample is found by particle size analysis or mechanical analysis.
- Mechanical analysis is meant for the separation of soil into different size fractions.
- Mechanical analysis is performed in two stages :-
  - i) Sieve analysis.
  - ii) Sedimentation analysis or wet mechanical analysis.
- Sieve analysis is meant for coarse grained soil & sedimentation analysis for fine grained soil.

Sieve Analysis :-

⇒ In IS, the sieves are designated by the size of aperture (openings) in mm.

⇒ Sieve analysis can be divided into two parts :-

i) Coarse analysis

ii) Fine analysis.

⇒ Soil sample is separated into two fractions by sieving through 4.75 mm IS sieve.

⇒ The sample is retained on 4.75 mm IS sieve is termed as gravel fractions & used for coarse analysis.

⇒ Set of sieves used for coarse analysis are :- 100, 63, 20, 10 & 4.75 mm

⇒ Set of sieves used for fine sieve analysis are :- 2mm, 1mm, 600μ, 425μ, 300μ, 212μ, 150μ & 75μ IS sieves.

⇒ The sieves are arranged one over the other by keeping the largest opening sieve at the top & smallest opening sieve at the bottom.

⇒ A receiver is kept at bottom &

a cover is kept at the top.

⇒ The sample is placed on the top sieve, and the whole assembly is fitted on sieve shaking machine.

⇒ 10 mins. of shaking is desirable for soils with small particles.

⇒ The soil sample retained on each sieve is weighed.

⇒ Percentage of soil retained on each sieve is calculated on the basis of total mass of soil sample taken.

⇒ It is advisable to wash the soil portion passing through 4.75mm sieve over 75μ sieve so that silt & clay particles sticking to sand will be washed off.

⇒ The fraction retained on 75μ sieve is dried in the oven.

⇒ The dried portion is then re-sieved through 2mm, 1mm, 600μ, 424μ, 300μ, 212μ, 150μ & 75μ.

⇒ If the portion passing 75μ size is substantial or considerable, wet analysis is done for further sub-division of particle size distribution

Sedimentation Analysis

In wet mechanical analysis or sedimentation analysis, the soil fraction finer than 75μ size is kept in suspension in a liquid medium.

→ This analysis is based on Stoke's law.

→ Stoke's law states that the velocity at which the grains settle out of suspension, all other factors being equal, is dependent upon the shape, weight & size of grains.

→ It is assumed that the soil particles are spherical & have same sp. gravity.

→ The coarser particles settle more quickly than the finer ones.

so $$v = \frac{2}{9} r^2 \frac{\gamma_s - \gamma_w}{\eta}$$

$$= \frac{1}{18} D^2 \frac{\gamma_s - \gamma_w}{\eta}$$

v = terminal velocity of sinking spherical particle (m/s)

r = radius of spherical particle (m)

D = diameter " " " (m)

γ_s = unit wt of particles (kN/m³)

γ_w = " " " water/liquid (kN/m³)

η = viscosity of water/liquid ($KN \cdot s/m^2$)

$$\eta = \frac{\mu}{g}$$

μ = viscosity in poise.

g = acceleration due to gravity.

⇒ Sedimentation analysis is done by :-

i) Hydrometer ii) Pipette.

→ In both the methods, a suitable amount of oven dried soil sample, finer than 75 μ size, is mixed with a given volume v of distilled water.

→ The analysis is based on the assumption that :-

i) Soil particles are spherical

ii) Particles settle independent of other particles.

iii) Wall of jar, in which suspension is kept, also donot affect the settlement.

⇒ Hydrometer Method

→ ~~The principle of the test is same in both hydrometer & pipette method.~~

i) **Pipette Method**

→ It is a standard sedimentation method used in laboratory.

→ The equipment consists of a pipette, jar and a no. of sampling bottles.

→ A boiling tube of 500ml capacity is used in place of a jar.

→ The pipette consists of

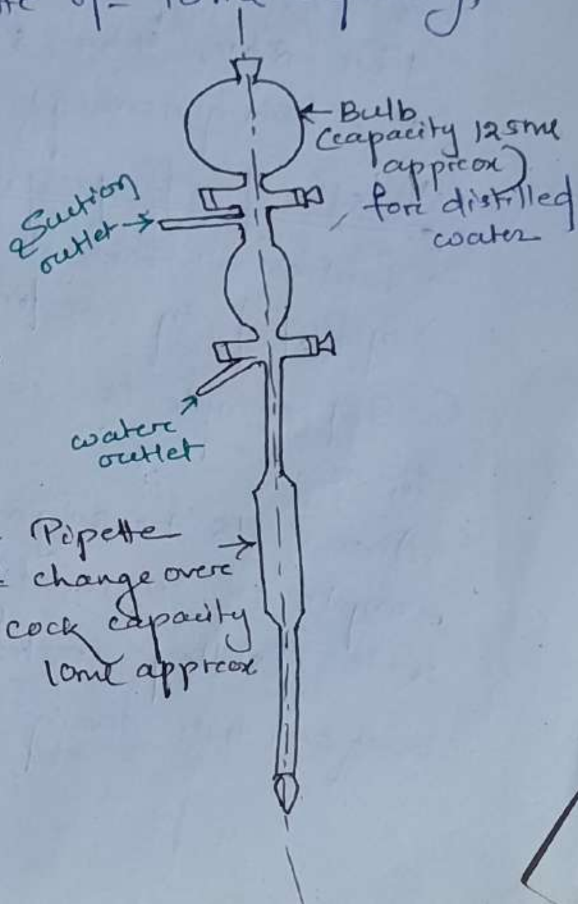
i) a 125ml bulb with stop cock, for keeping distilled water.

ii) a three way stop cock.

iii) Suction & waste water outlet.

iv) Sampling pipette of 10ml capacity.

→ The method consist in drawing off samples of soil suspension, 10ml in volume, by means of pipette from a depth of 10 cm at various time intervals after commencement of test.



→ The pipette should be inserted in the boiling tube about 25 sec before the selected time interval & time taken for sucking the sample should not be more than 10 to 20 sec.

→ Each sample is transferred into suitable sampling bottle & dried in an oven.

⇒ M_D (mass of solids) per ml of suspension is then found by taking the dry mass & dividing it by 10.

⇒ The time intervals are ½ min, 1 min, 2 min, 4 min, 8 min, 15 min & 30 min & 1 hr, 2 hr, 4 hr, 8 hr, 16 hr, 24 hr from the commencement of the test.

(ii) Method of preparing soil suspension

↳ Particles finer than 75 μ size are included in sedimentation analysis.

↳ Soil sample is washed through 75 μ sieve.

↳ About 12 to 30 gm of oven dried sample is accurately weighed & mixed with distilled water in a dish or beaker to form smooth paste

→ For proper dispersion of soil, a dispersing agent is added to the soil.

→ Some dispersing agents are sodium oxalate, sodium silicate & sodium polyphosphate compounds.

→ A dispersing solution containing 33 gm of sodium hexametaphosphate & 7 gm of sodium carbonate in distilled water to make 1 lit of solution.

→ 25 ml of this solution is added to the dish (~~&~~ mix) having soil & distilled water & mixture is warmed up gently for 10 mins.

→ The contents are transferred to a mechanical mixer.

→ Soil suspension is mixed or stirred for 15 mins or longer for highly clayey soil.

→ The suspension is washed through 75 μ sieve & suspension which has passed through the sieve is transferred to 500 ml capacity boiling tube.

→ The tube is then put in a constant temperature water bath.

-> when temp. in the tube has been stabilized to the temp. of the bath, the soil suspension is thoroughly shaken by inverting the tube & replacing in the bath.

-> Stop watch is started & soil sample are collected at various intervals with help of pipette.

-> The soil which contains organic matter & calcium compounds are pretreated before dispersing agents are mixed since these contents act as cementing agent & cause particles to settle as aggregation of particles instead of individuals.

-> Process of removing these organic matters & calcium compounds is known as pretreatment.

-> Soil is first treated with hydrogen peroxide solution to remove the organic matter by oxidation. The mixture of soil & hydrogen peroxide is kept warm at a temp. not exceeding 60°c, till no further evolution of gas takes place

→ The remaining hydrogen peroxide in the solution is then decomposed by boiling the solution.

→ To remove calcium compounds, the cooled mixture of soil is then treated with 0.2 N hydrocloric acid.

HYDROMETER METHOD

→ It is another method of sedimentation analysis.

→ The principle of the test is same in both the pipette & hydrometer method.

→ In pipette method, Mass M_D per ml of suspension is found directly by collecting 10 ml sample of soil suspension from sampling depth He.

→ In hydrometer method, M_D is computed indirectly by reading the density of soil suspension at the depth He at various time interval.

→ In pipette method, sampling depth (He) is constant (10 cm) but in hydrometer method, the sampling depth increases as the particle settle with the increase in the time interval.

→ Calibration of the hydrometer and sedimentation jar is required before starting sedimentation test.

→ In the hydrometer, the reading on the stem gives the density of soil suspension situated at center of bulb at any time.

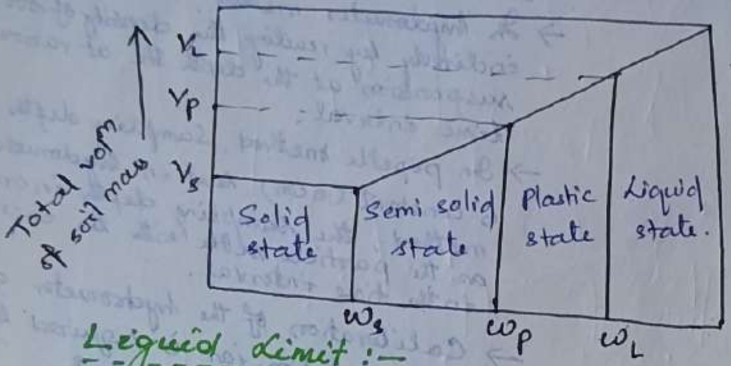
→ Hydrometer reading are recorded after subtracting & multiplying the remaining digits by 1000. It is designated as R_h.

→ The hydrometer reading R_h increases in the downward direction towards the hydrometer bulb.

→ Let H be the ht in cm between any hydrometer reading R_h and the neck & the h as ht of bulb.

→ Sedimentation jar contain soil suspension.
→ When hydrometer is immersed in the jar water level aa raises to a,a, the rise is equal to V_h of the hydrometer divided by interval area of cross section A of jar.
→ Similarly the level bb raises to b_1b_1, where bb is the level, situated at a depth He below the top level aa.

3.4 CONSISTENCY OF SOIL :-
→ Consistency is the relative ease with which soil can be deformed.
→ Atterberg divided various stages of consistency from liquid to solid state in to 4 stages :-
i) Liquid state
ii) plastic state.
iii) Semi solid state.
→ There are certain limits known as consistency limit & atterberg limit depending up to water content
→ For Engg. purpose the atterberg limit's are :- Liquid limit, plastic limit & Srinkage limit.



Liquid Limit :-
→ It is the water content corresponding to the limit between liquid & plastic state of consistency of soil.
→ It is the minimum water content at which the soil is still in the liquid state, but has shearing strength against flowing
→ It is the minimum water content at which a part of soil is cut by a groove of standard dimension will flow together from a distance of 12mm under an impact of 25 blows in the device.

Plastic Limit (W_P) :-

→ It is the water content corresponding to the limit between plastic & semi solid state of consistency of soil.

→ It is the min water content at which a soil will just begin to crumble when rolled in to a thread of 3mm in dia.

Shrinkage Limit (W_s) :-

It is the maximum water content at which a reduction in water content will not causes a decrease in the volume of a soil mass.

Plastic Index (I_P) :-

It is defined as the numerical difference between the liquid limit and the plastic limit of the soil.

$$I_P = W_L - W_P.$$

Plasticity :-

It is the property of a soil which allows it to deform rapidly with out vol. change.

Consistency Index (I_c) :-

It is the ratio of the liquid limit minus the natural water content to the plasticity index of soil.

$$I_c = \frac{W_L - W}{I_P}.$$

Liquidity Index (I_L) :-

It is the ratio of natural water content of the soil minus its plastic limit to its plasticity index.

$$I_L = \frac{W - W_P}{I_P}.$$

Determination of Liquid Limit :-

→ Liquid limit is determined by an apparatus designed by casagranda named as casagranda liquid limit apparatus.

→ It is consist of hard rubber base, over which a brass cup is placed which can be raised and lowered with the help of a handle.

→ Height of fall can be adjusted with the help of adjusting screw & before conducting the test, ht. of fall is adjusted to 1 cm.

→ Two types of grooving tools are used :-
i) Casagranda tool.
ii) ASTM tool.

→ Casagranda tool cut a groove of 2mm wide at bottom, 11 mm wide at top and 8 mm high

→ ASTM tool cut a groove 2 mm wide at bottom 13.6 mm at top & 10 mm deep.

Procedure

↳ About 100 gm of soil sample passing through 425μ IS sieve is taken in a porcelain dish. Some quantity of water is added to it & thoroughly mixed to form a soil paste of uniform colour.

↳ The height of fall of cup of the liquid limit device is adjusted to the 1 cm.

↳ A portion of soil paste in porcelain dish is placed in the liquid limit dirce & levelled by means of spatula.

↳ Using standard grooving tool, a groove is cut in the soil.

↳ Cup is given blows by rotating the handle at 2 ~~rpm~~ revolution per second.

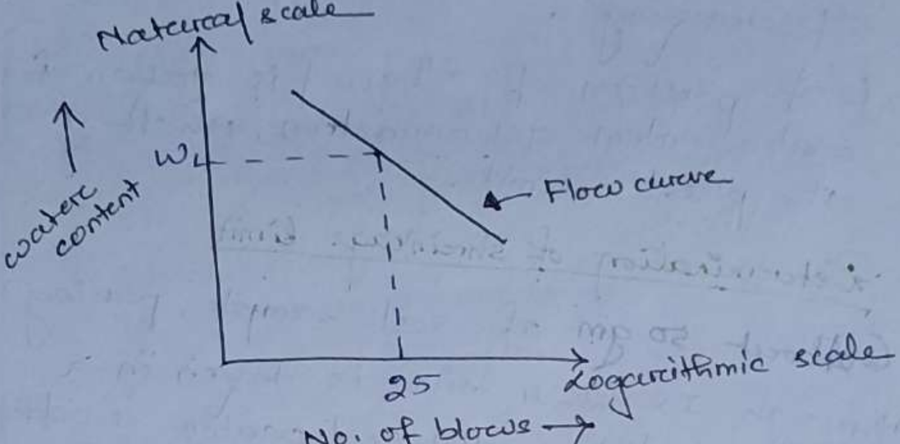
↳ The number of blows required to close the groove for a distance of 13 mm is noted down.

↳ The above steps are repeated to get at least 4 concurrent sets of number of blows & water content.

↳ It is convinient to increase the water content in successive steps and obtain blows count near about 40, 30, 20 & 10.

↳ The water content values are plotted as ordinate on natural scale against number of blows as abscissa on logarithmic scale to obtain straight line, which is known as flow curve.

↳ From this plot the liquid limit is obtained as water content corresponding to 25 blows.



Determination of plastic limit

↳ About 30 gms of soil sample passing through 425 μ IS sieve is taken & some quantity of water is added & thoroughly mixed to form a soil paste which can be rolled into balls between palm of ~~hands~~ hands.

↳ A small portion of the ball is then rolled on a smooth plate into a thread of 3mm diameter & the thread is looked for signs of cracking.

↳ If no cracks are seen; the thread is picked up and again rolled into a ball between palms to reduce water content.

↳ The ball is then rolled on smooth plate a into a thread of 3mm dia.

↳ The steps are repeated untill a 3mm diameter thread first shows signs of cracking.

↳ A portion of thread is taken for water content determination, which gives the plastic limit.

Determination of shrinkage limit

↳ About 50 gm of soil sample passing through IS 425 μ sieve is taken in a porcelain dish, distilled water is added to it, and mixed thoroughly to form a soil paste of slightly flowing consistency.

↳ The shrinkage dish (cup of 45 mm dia and 15 mm ht.) is weighed after coating inner side of the cup with a thin layer of grease or oil. ~~the~~

↳ shrinkage cup is filled with the soil paste in 3 layers, the cup ~~beigh~~ being gently tapped on a cushioned surface after filling with each layer to ensure expulsion of air bubbles.

↳ The surface of soil is levelled & outer side of cup is cleaned. The mass of shrinkage cup with wet soil pat is found & this is deducted from mass of shrinkage cup to get mass of wet soil pat (M_1).

↳ The wet soil is allowed to dry in air for some time, then kept in oven & dried for 24 hrs at 105 to 110°C.

↳ Then mass of dry soil (M_d) is found.

↳ The volume of dry soil (V_d) is found by mercury displacement method.

↳ The volume of wet soil (V_1) is equal to volume of shrinkage dish which is found by filling it with mercury & finding mass of mercury required to fill it after removing convex portion at the top by pressing with a flat plate.

↳ Volume is obtained by dividing mass by density of mercury.

4.0 Classification Of Soil

4.1 General

=> Soil is classified into various groups depending on engg. properties and characteristics.

=> From engg. point of view, soil is classified with the objective of finding the suitability of soil for construction of dam, highway, foundation & other engg. structures.

=> The soil may be classified to following systems :-

i> Particle size classification
ii> Textural classification.
iii> Highway research Board (HRB) classification.
iv> Unified soil classification.
v> Soil Classification.

i> Particle Size Classification

↳ Soil is arranged according to grain size

↳ Gravel, sand, silt & clay are used to indicate grain size.

↳ They designate the particle size,

naturally occuring soil have mixture of particles of different sizes.

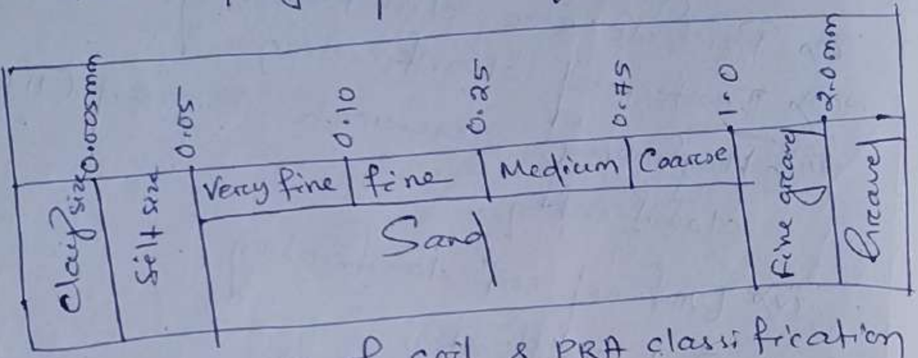
→ Silt size & clay size are mostly used word in place of simply silt or clay in this system.

→ Three such systems which have been widely used are :-

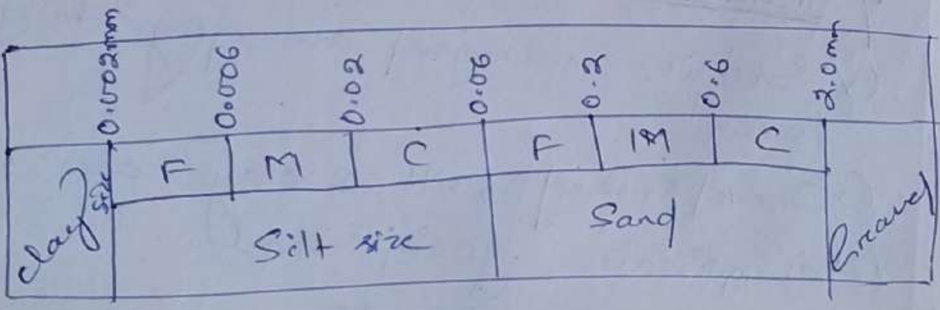
(a) U.S Bureau of soil & Public Roads Administration (PRA) classification system.

(b) M.I.T. classification system.

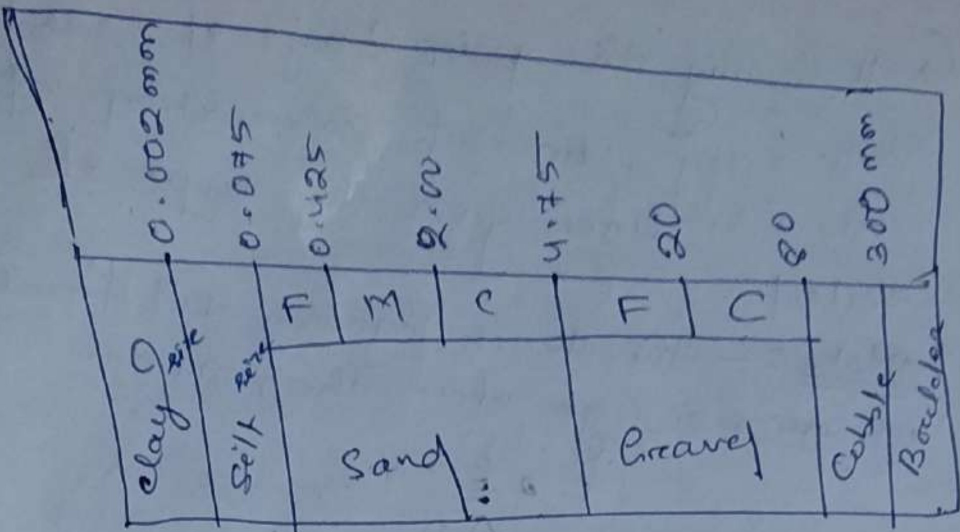
(c) Indian standard particle size classification system (based on M.I.T. system)



(a) U.S. Beaureau of soil & PRA classification system.



(b) M.I.T. classification



(c) IS classification.

ii) Textural Classification

↳ Naturally occuring soil compose of sand, silt & clay.

↳ Soil classification of composite soil based on particle size distribution is known as textural classification.

→ It is a triangular classification & it is suitable for coarse grained soil.

Example :— A soil sample is found to consist of 30% sand, 30% silt & 40% clay. Classify the soil using textural classification.

↳ The textural classification chart of U.S.P.R.A (U.S. Public Road Administrat") has been developed to classify composite soil.

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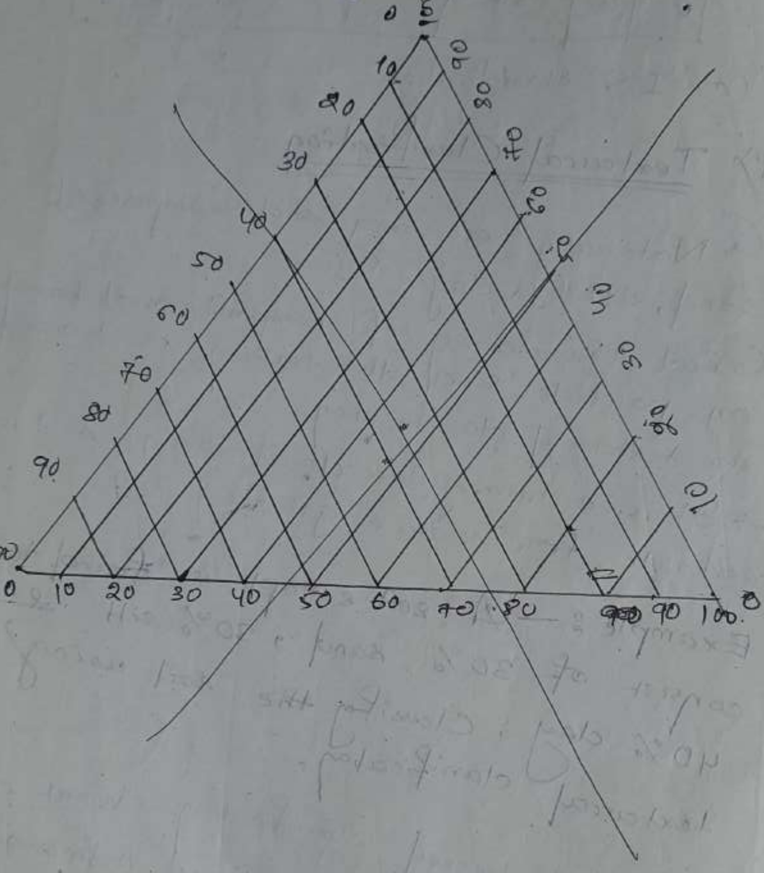
→ Knowing the percentage of sand silt & clay, lines of intersection of the 3 lines gives the type of soiled.

Note :- The chart ignores soil fractn of grr-size greater than 2.00 mm.



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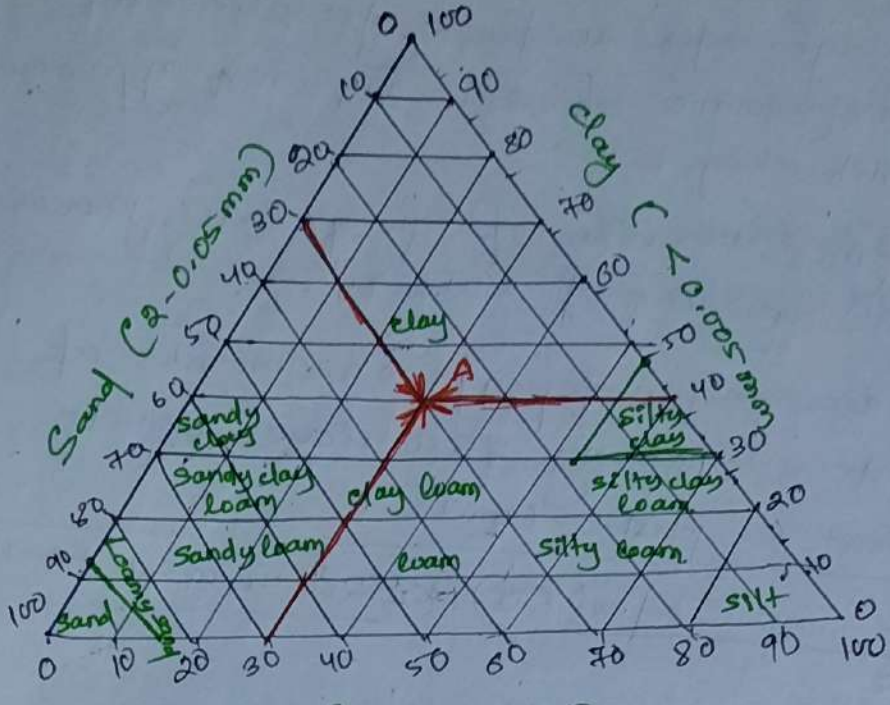


From respective points on 3 sides, intersects at point A, which lies in the region marked as clay. Hence the soil can be classified as clay.

(iii) **Highway Research Board (HRB) classif.**

- It is also known as Public Road Administration (PRA) classification.
- It is used for pavement construction.
- Soil is divided into 7 primary groups, named as A-1, A-2 - - - - A7.
- Group A-1 is divided into two sub-groups and group A-2 is divided

into 4 sub-groups.

→ Group index used to describe the performance of soil when used in pavement construction.

→ Higher the value of group index, poorer the quality of material.

→ Group index depends on amount of material passing 75μ sieve, liquid limit & plastic limit.

$$\text{Group Index (GI)} = 0.2a + 0.005ac + 0.01bd$$

where,

a = that portion of percentage passing 75μ sieve greater than 35 & not exceeding 75 expressed as whole no. (0 to 40)

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

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

q. that portion of numerical plasticity index greater than 10 & not exceeding 30 expressed as positive whole number (0 to 20).

(iv) <u>Unified Soil Classification System</u>

↳ This system is used for the construction of foundation, earth dam, canal, earth slopes etc.

↳ The coarse grained soils are classified on the basis of grain size distribution while the fine grained soils are classified on the basis of their plasticity.

↳ The soil is first classified into 2 groups

(a) Coarse grained soil

(b) Fine grained soil.

(a) <u>Coarse grained soil</u>

↳ If the soil retained on 75μ is more than 50%, then the soil is coarse grained soil.

↳ A coarse grained soil is called gravel (G) when 50% or more of coarse fraction is retained on 4.75mm sieve otherwise termed as sand (S).

↳ Coarse grained soil containing less than

5% fines, are designated as GW & SW if they are well graded and designated as GP & SP if they are poorly graded.

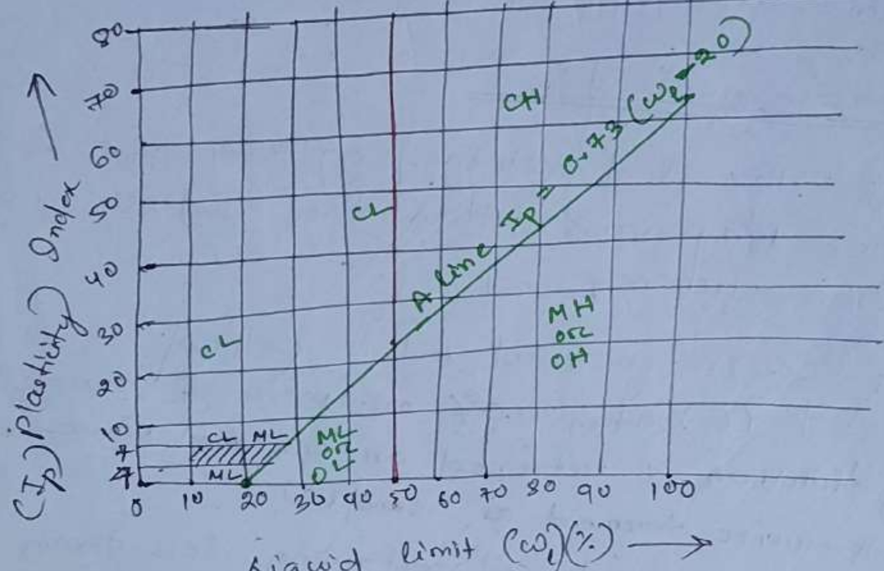
↳ If percent of fines lie between 5 to 12%, coarse grained soil are designated as GW-GM or SP-SM.

(b) **Fine grained soil**

↳ A soil is termed as fine grained if more than 50% of soil sample passes through 75μ sieve

↳ Fine grained soil is such, divided into soil (M) & clay (C) based on liquid limit & plasticity index.

↳ Organic soils are also included in this group.



75 μ IS sieve size.

ii) Fine grained soil :- when more than half of the material by mass is smaller than 75 μ IS sieve size.

iii) Highly organic soil & other miscellaneous soil material :- It contains organic matter, like peat & decomposed vegetable matter.

1) Coarse Grained Soil :-
It is divided into sub divisions :-

(a) Gravel (G) :-
when more than half of coarse fraction (75 μ) is larger than 4.75 mm IS sieve size. It is designated by G.

(b) Sands (S) :-
when more than half of coarse fraction (75 μ) is smaller than 4.75 mm IS sieve size. It includes sands & sandy soils.

⇒ Gravel & Sand are further sub-divided into 4 groups depending upon grading :-
W = well graded, clean.
C = well graded with clay binder
P = poorly graded, fairly clean
M = containing fine materials not covered in other groups.

↳ The A-line in the chart has equation
$I_p = 0.73(\omega_L - 20)$

↳ The A-line separates clay like materials from silty & organic soil materials.

↳ Fine grained soils are further sub-divided into soil possessing low (L) or High (H) plasticity when liquid limit is less than 50% or more than 50%.

↳ when plasticity index & liquid limit plot lies in the hatched portion of plasticity chart, the soil is given dual symbol 'CL-ML'

↳ when soil having characteristics of more than one group are termed as boundary soil & gives dual group symbols.
For eg:- GW-GC means well graded gravel with clay fines.

↳ Organic silts (OL or OH) & inorganic soils (ML or MH) are also plotted on plasticity chart.

↳ Soils having liquid limit about 30% or less is known as organic (OL or OH). If liquid limit is higher, it is known as inorganic (ML or MH).

(V) Indian Standard Classification System

↳ The soil is broadly divided into 3 divisions:-

i) Coarse grained soil :- when more than half of the material by mass is larger than

These symbols are used in combination to designate the type of coarse grained soil.
ex:- GC means clayey gravel.

2) **Fine Grained Soil:-**

It is divided into 3 sub-divisions:-

i) Inorganic silts & very fine sand (M)

ii) Inorganic clays (C)

3) **Organic silts & clay & organic matter (O)**

Fine grained soil is further divided into groups depending upon liquid limit which is good index of compressibility:-

i) Silt & clay of low compressibility having liquid limit less than 35 & represented by L.

ii) Silt & clay of medium compressibility having liquid limit greater than 35 & less than 50 & represented by I.

iii) Silt & clay of high compressibility having liquid limit greater than 50 & represented by H.

Combination of these symbol indicates the type of fine grained soil. Ex:- ML means inorganic silt with low to medium compressibility.

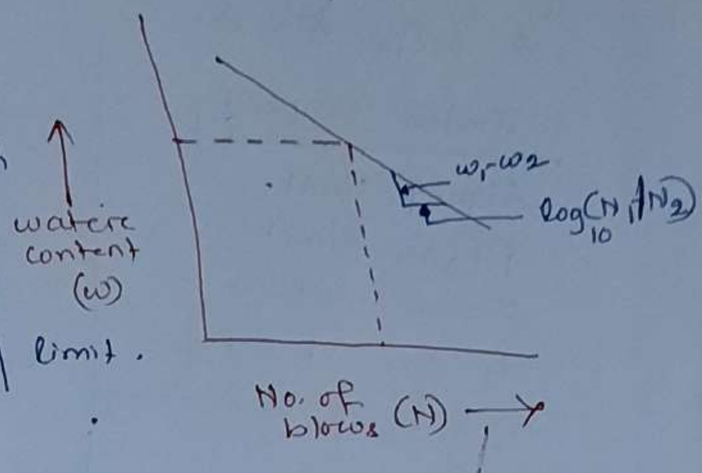
↳ A-line, dividing inorganic clay from silt & organic soil has following equation:-

$$I_p = 0.73 (\omega_L - 20)$$

P-H.12

Flow Index

Flow index (I_f) is the slope of the flow curve obtained between number of blows & the water content in Casagrande's method of determination of liquid limit.



From fig.,

$$\text{Flow index } I_f = \frac{\omega_1-\omega_2}{\log_{10}(N_2/N_1)}$$

N_1 = no. of blows at water content ω_1
N_2 = " " " " " ω_2.

$\log_{10}(N_2/N_1) = 1$ when $N_2/N_1 = 10$.

as $\log_{10} 10 = 1$.

Toughness Index

It is

$$I_t = \frac{I_p}{I_f}$$

I_p = Plasticity index $= \omega_l - \omega_p$

I_f = flow index.

Q:- Following index properties were determined for 2 soils A & B.

Index Property	A	B
Liquid limit.	65	35
Plastic limit	25	20
water content	35	25
Sp. gr. of solids	2.7	2.65
Degree of saturation	100 %	100%

which have (1) greater bulk density (2) greater dry density (3) greater void ratio.

	A	B
Plasticity index $I_p = \omega_L - \omega_P$	$65-25=40\%$	$35-20=15\%$
Void ratio $e=\omega G$	0.35×2.7 $=0.945$	0.25×2.65 $=0.663$
Dry density $\rho_d = \frac{G\rho_w}{1+e}$	$\frac{2.7\times1}{1.945}=1.388$	$\frac{2.65\times1}{1.663}$
Bulk density $\rho = \rho_d(1+\omega)$	1.388×1.35 $=1.874$ g/ml	$=1.594\times1.25$ $=1.992$ g/ml.

As plasticity index of A is more, it has more clay particles.

6.0 COMPACTION & CONSOLIDATION

6.1 Compaction

It is the process by which the soil particles are artificially arranged & packed together into a closer state of contact by mechanical means in order to decrease the porosity of soil & increase the dry density.

6.1.1 Light & Heavy Compaction Test

The test equipment consist of :—

i) Cyllindrical metal mould having internal dia of 4 inches (10.15cm), internal effective height of 4.6 inches (11.7 cm) & a capacity of 0.945 lt. (1000ml).

ii) detachable base plate

iii) Collar of 2 inches (5cm) effective height

iv) Rammer of 2.5 kg in mass.

Procedure :—

↳ 5 Kg of soil is taken & water is added to it of different percentages

↳ The mould with base plate is weighed as M_1. The exterior collar is to be attached with the mould.

↳ The moist soil in the mould is compacted by ~~letting~~ dropping the rammer ~~fall~~ through a height of 30.5cm.

↳ The above compaction is done by giv. 25 blows in layers soil layers & the cyllinder is filled by filling soil in different layers, each layer being comp. in above manner.

↳ The extension collar is then removed & the compacted soil is levelled off carey to the top of the mould by means of straight edge.

↳ Then the mould & soil is weighed as M_2.

↳ Soil is removed from the mould & a small sample of soil is taken for water content determination.

↳ This process is repeated after adding suitable amount of water to the soil in an increasing order.

↳ Then Bulk density ρ, at each compaction is calculated as follows:-

$$\rho = \frac{M_2 - M_1}{V_m}$$

$M_2 - M_1$ = Mass of compacted soil

V_m = Volume of soil
= Volume of mould.

↳ Dry density ρ_d is calculated from the relation :-

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

ω = moisture content of soil in %.

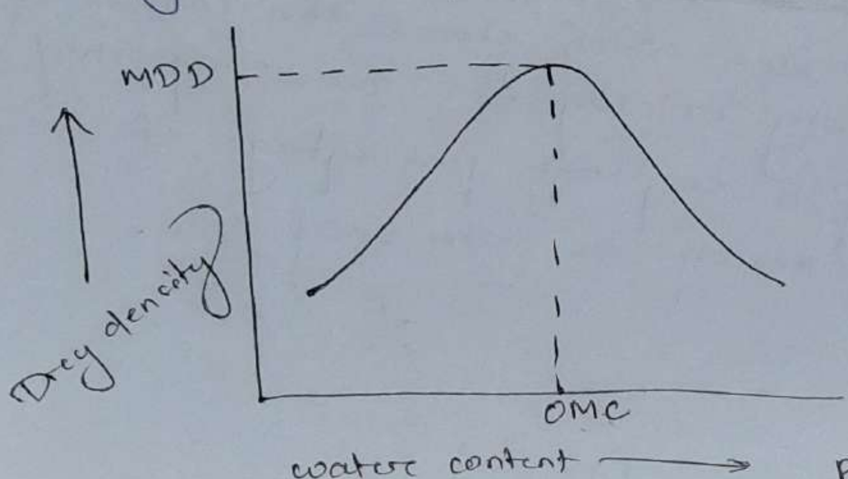
6.1.2 OMC & MDD

↳ As, a number of times, the above test is repeated, a no. of dry densities at corresponding water contents are obtained

↳ A smooth curve or compaction curve is plotted between water content as abscissa & dry densities as ordinate.

↳ The dry density goes on increasing as the water content is increased, till the maximum density is reached.

↳ The water content corresponding to max. density is called optimum moisture content (OMC)



↳ The maximum or peak point of compaction curve is called maximum dry density. (MDD)

↳ Light compaction method is also known as **Standard Proctor Test**.

↳ The equipments required for light & heavy compaction tests are same, except:—

in heavy compaction test:—

(i) rammer is of 4.9 kg & it's height of fall is 450mm.

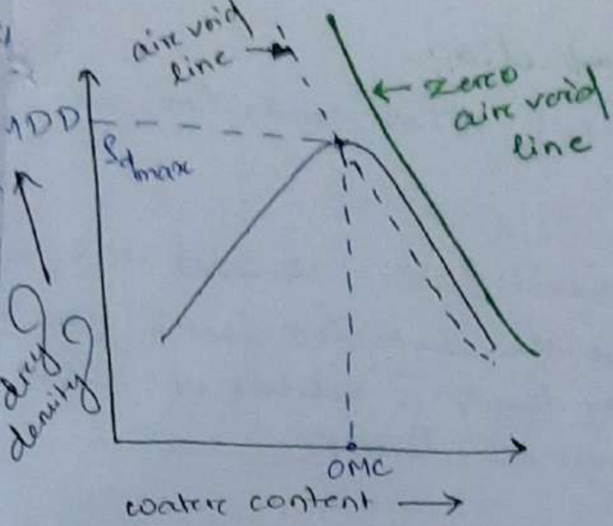
(ii) The soil is compacted in 5 equal layers instead of 3.

(iii) Each layer is given 25 blows of rammer if 1000 ml mould is used & 55 blows if 2250 ml mould is used.

↳ So, Heavy compaction method is also known as **Modified Proctor Test**.

6.1.3 Zero air void line:—

A line which shows the water content dry density relation for the compacted soil containing a constant percentage of air voids is known as air void line.



$$\rho_d = \frac{(1-n_a) G \rho_w}{1+wG}$$

n_a = % air void
ρ_d = dry density
w = water content of compacted soil
G = sp. gravity
ρ_w = density of water.

↳ The theoritical maximum compaction for any given water content corresponds to zero air voids condition ($n_a = 0$). The line showing dry density for soil containing no air voids is called zero air void line or saturation line, with eq^n

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

↳ Alternatively, a line showing the relation between water content & dry density for a constant degree of saturation S,

$$\rho_d = \frac{G \rho_w}{1+\frac{wG}{S}}$$

6.2. Factors Affecting Compaction:-

The various factors which affect compaction are as follow:-

(i) Water Content:-

From laboratory experiments, it is observed that the water content increases, the compacted density goes increasing, till a maximum dry density is achieved after which further addition of water decreases the density.

(ii) Amount of Compaction:-

Amount of compaction greatly affects the maximum dry density & optimum water content of a given soil. The effect of increasing the compactive energy results in an increase in max. dry density and decrease in optimum water con

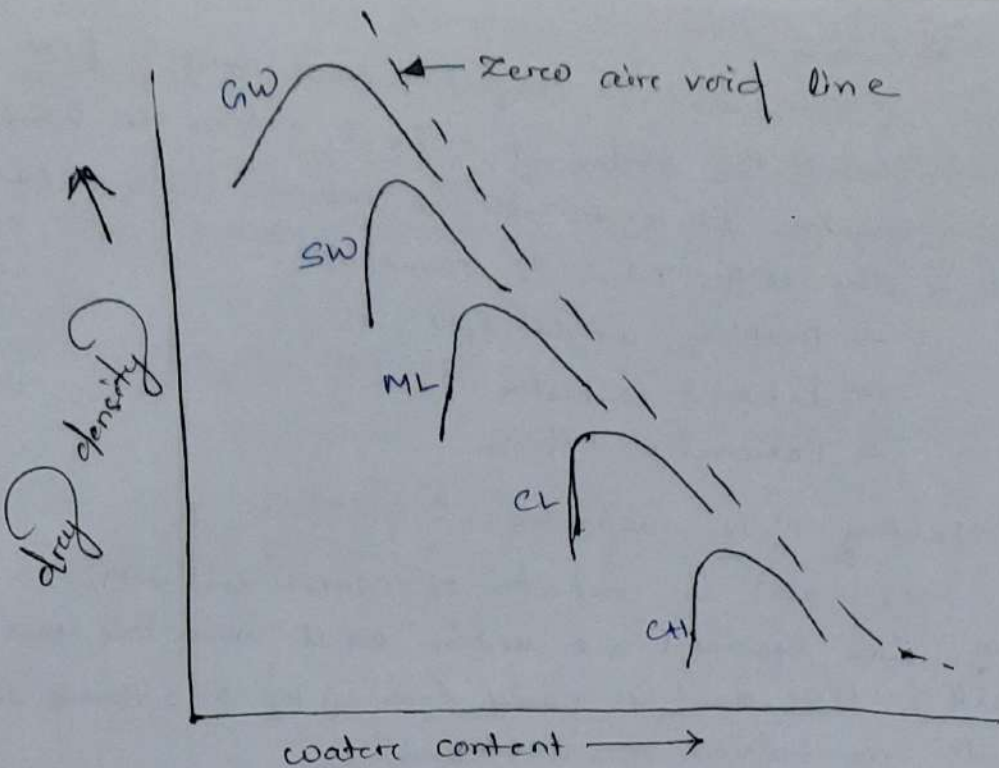
(iii) Method of compaction:-

Type of compaction or the manner in which the compactive effort is applied affects density. The weight of compacting equipment, manner of operation such as dynamic or impact, static, kneading or rolling and time & area of contact between the compacting element & soil plays role.

(iv) Types of soil:-

The maximum dry density achieved by the soil largely depends upon the type of soil. Well graded coarse grained soil attain a much higher density and lower optimum water content than fine grained soil which require more water for lubrication because of greater specific surface.

Figure shows the water content & dry density curve for a range of soil types. The coarse grained soils can be compacted to higher dry densities than fine grained soil.



(v) Addition of Admixture! -

The compaction characteristics of a soil can be modified by a number of admixtures. these admixtures can be used in construction of stabilised soil.

Field Compaction Methods! -

There are various types of compacting equipments. Use of these compacting machines depends on soil types and moisture condition.

The soil compaction equipments can be divided into two ~~conditions~~ groups:-

(i) Light soil compacting equipment.

(ii) Heavy soil compacting equipment.

(i) Light soil compacting equipment! -

These equipments are used for soil compacting of small areas only and where the compacting effort needed is less.

Some of the equipments are! -

a) Rammer

b) Vibrating equipment.

6.7

a) Rammer:-

It is used for compacting small areas by providing impact load to the soil. It is light & can be hand machine operated. It is suitable for compacting cohesive soils well as other soils. It is of three types:-

→ Dropping weight type.

→ Internal combustion type.

→ Pneumatic type.

b) Vibrating Plate compactors:-

It is used for compaction of coarse soil with 4 to 8% fines. These equipments are used for small areas. The usual weights of these machines varies from 100 kg to 2 tonnes with plate area between 0.16 m2 & 1.6 m2.

Vibro Tampers:-

It is used for compaction of small areas in confined space. This machine is suitable for compaction of all types of soil by vibration set up in a base plate through a spring activated by an engine driven reciprocating mechanism. They are usually manually guided & weigh between 50 & 100 kg.

(ii) Heavy Soil Compaction Equipments:-

These equipments are used for large areas. Different types of soils, Following are different types of these equipments-

a) Smooth wheeled Rollers:-

It is of two types:-

→ Static smooth wheeled roller

→ Vibrating smooth wheeled roller.

The most suitable soils for these roller type are well graded sand, gravel, crushed rock, asphalt etc. where crushing is required. These are used on soils which does not require great pressure for compaction. These rollers are

generally used for finishing the upper surface of soil.
these roller are used for compaction of uniform sands.

The performance of smooth wheeled rollers depend on load per cm width it transfers to the soil & diameter of the drum. the load per cm width is derived from the gross weight of drum.

The smooth wheeled roller consist of one large steel drum in front & two steel drum on the rear. the gross weight of these rollers is in the range of 8-10 tonnes. the other type of smooth wheel roller is called Tandem roller, which weighs between 6-8 tonnes.

→ Vibrating smooth wheel rollers:-

these rollers are helpful from several consideration:-

(i) Higher compaction level can be achieved with maximum work.

(ii) Compaction can be done up to greater depths.

(iii) Output is many times more than conventional rollers

These rollers are expensive but in the long run the cost becomes economical due to their outputs & improved performance. the latest work specifications for excavation recommends the use of vibratory rollers due to their advantage over static smooth wheeled rollers.

b) Sheepfoot Roller :-

→ These are used for compacting fine grained soil such as heavy clay & silty clay. These are used for compaction of soil in dams, embankments, subgrade, layer in pavement & rail road construction projects.

→ They are static & vibrating types. Vibratory types rollers are used for compaction of all fine grained soil.

→ It consist of steel drum on which projecting legs are fixed & can apply a pressure upto 19 kg/cm^2 or more. The weight of drum can be increased by ballasting with water or wet sand.

→ The compaction of soil is mainly due to foots ~~per~~ penetrating & exerting pressure on the soil. The pressure is maximum when foot is vertical.

c) Pneumatic tyred roller :-

→ These are also known as rubber tyred rollers. It is used for compaction of coarse grained soil with some fines. These are least suitable for uniform coarse soils & rocks.

→ These rollers have wheels on both axels. The wheels are staggered for ~~staggered for~~ compaction of soil layers with uniform pressure throughout the width of rollers.

d) Grid Rollers :-

It is used for compaction of weathered rocks, well graded coarse soils. It is not suitable for clayey soils, silty clays & uniform soils. The main use of these rollers are in subgrade & sub-base in road construction.

The roller have a cylindrical heavy steel surface containing a network of steel bars forming a grid with square holes. The wt. of these roller can be increased by ballasting with concrete block.

e) Pad foot / Tamping Rollers :-

It is similar to sheep foot roller with legs of larger area than sheep foot rollers. These rollers are more pre-ferred than sheepfoot rollers due to high production capacity & they are replacing sheepfoot rollers.

The degree of compaction achieved is more than sheepfoot roller. They operate at high speed & are capable to break large lumps. It is best suitable for compacting cohesive soil.

al P. . .

6.4 CONSOLIDATION

→ In a soil sample, there are voids which are either filled with air, or water or both.

→ When the voids are filled with air alone compression of soil occurs rapidly, because air is compressible & can escape easily from voids.

→ When saturated soil have it's voids filled with incompressible water, decrease in volume or compression can take place when water is expelled out of the voids. Such a compression resulting from long term load & escape of pore water is termed as consolidation.

→ According to Terzaghi, every process involving a decrease in water content of saturated soil without replacement of water by air is called consolidation.

Distinction Between Compaction & Consolidation

Compaction	Consolidation
→ Compaction is the compression of soil by expulsion of air from the voids of soil.	→ Consolidation is the compression of soil by expulsion of water from voids of soil.
→ It is a quick process	→ It is slow process.

8.0 EARTH PRESSURE ON RETAINING STRUCTURES

When soil mass is retained at a higher level a ...aining wall, the retained mass of soil tends to ...de which is resisted by the retaining wall. It ...erts a pressure on the retaining wall, known as ...ateral earth pressure.

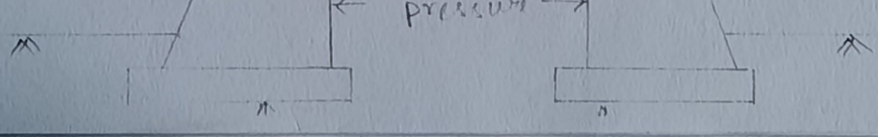
The retaining wall is constructed first & then the soil behind the wall is backfilled hence the soil ...etained is known as backfill.

The back of the wall is either vertical or slightly inclined to the vertical.

Lateral earth pressure can be grouped into 3 cate...gories, depending upon the movement of the retaining wall with respect to the soil retained.

At Rest Pressure:-

When the soil mass is not subjected to any lateral movement the Lateral earth pressure is called at-rest pressure. It occurs when the retaining wall is firmly fixed at its top & not allowed to rotate or move laterally.



The basement retaining walls which are restrai against the movement by the basement slab pro at their tops.

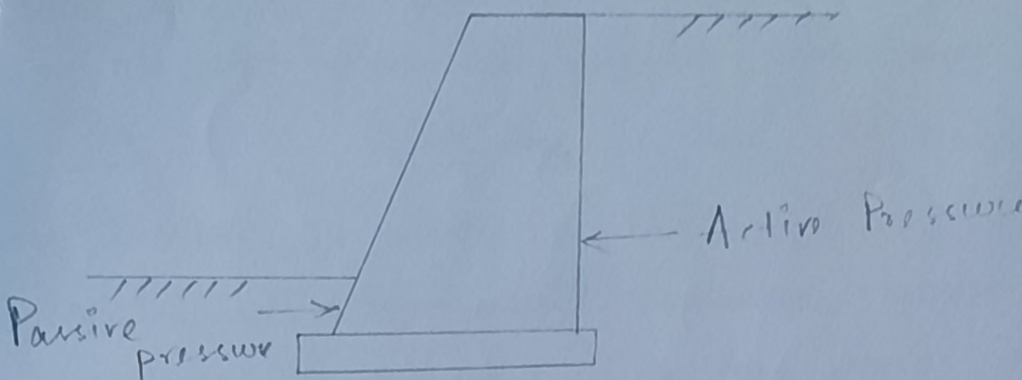
The bridge abutment wall which is restrained top by the bridge slab. The at-rest condition also known as elastic equilibrium.

Active Pressure :-

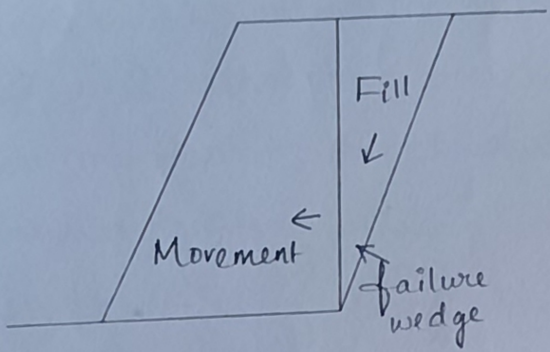
When the soil mass yields in such a way that it tends to stretch horizontally. It is a s of plastic equilibrium as the entire soil mass is verge of failure. A retaining wall when moves awa from backfill, there is a stretching of the soil mass and active state of earth pressure exists.

Passive Pressure :-

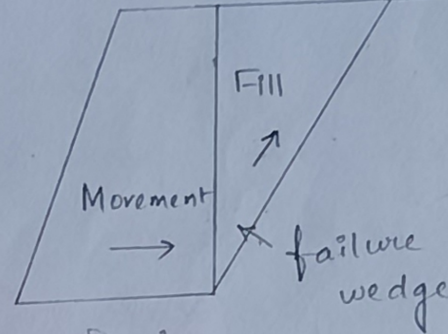
When the movement of the wall is such that the soil tends to compress horizontally. When the walls moves towards the backfill, there is an increase in the pressure on the wall & this increase continues until a max. value has reached after which there is no increase in the pressure & value will become constant. This pressure is known as passive earth pressure.



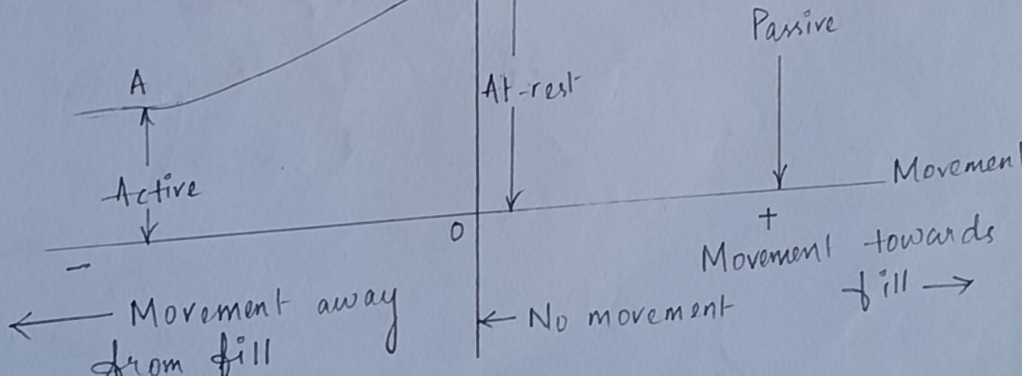
Variation of Pressure :-



Active
(i)



Passive
(ii)



(iii)

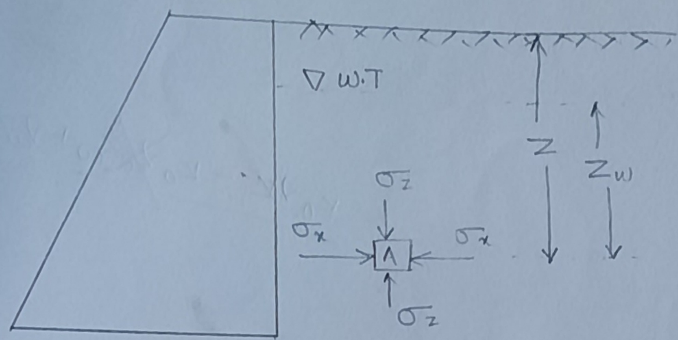
From fig.(i) we can see that the wall moves from the backfill and some portion of the back located immediately behind the wall tries to br away from the rest of soil mass.

This wedge-portion is known as failure we which moves downward & outward. The lateral pressure exerted on wall is minimum in this case. The horizontal strain required to reach the active state of plastic equilibrium is very small.

In fig.(ii) the wall moves towards the backfill & the earth pressure increases. The failure wedge moves upward & inward. The maximum value of the earth pressure is the passive earth pressure.

So, the minimum earth pressure is the active state & the maximum earth pressure is the passive state. These are two extreme conditions of plastic equilibrium. The at-rest condition is a special case of an elastic equilibrium when the state of stress corresponding to the condition where there is no movement.

Earth Pressure At Rest.



A retaining wall is considered in which no movement takes place. The vertical effective stress at point A at a depth z is given by,

$$\bar{\sigma}_z = \gamma z - \gamma_w z_w$$

The coefficient of earth pressure at-rest (K_0) is equal to the ratio of the horizontal stress to the vertical stress,

$$K_0 = \frac{\bar{\sigma}_x}{\bar{\sigma}_z}$$

$$\bar{\sigma}_x = K_0 \bar{\sigma}_z = K_0 (\gamma z - \gamma_w z_w)$$

The horizontal stress $\bar{\sigma}_x$ is represented as p_0.

$$p_0 = K_0 \bar{\sigma}_z$$

The coefficient of lateral earth pressure at rest (K_0) relates the effective stress.

The total lateral pressure (p_h) is equal to the sum of horizontal effective pressure (p_0) &

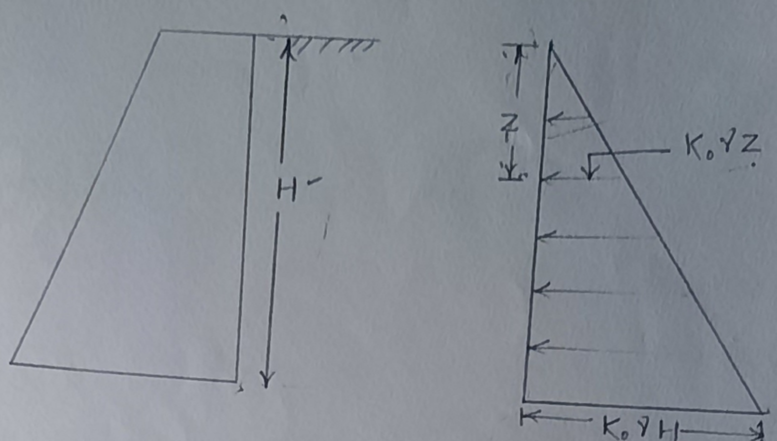
pore water pressure (u).

$p_h = p_o + u$

Lateral earth pressure at depth z, is

$p_h = K_o(\gamma z - \gamma_w z_w) + \gamma_w z_w$

$= K_o \gamma z - K_o \gamma_w z_w + \gamma_w z_w$



At rest pressure

$K_o \times \sigma$

$K_o \times \gamma D - \gamma_w$

$= K_o \gamma D$

$= K_o \gamma H.$

The pressure distribution is triangular with zero pressure at the top (z = 0), and max. pressure at the bottom of wall.

The pressure at bottom of wall is given by,

$p_h = K_o \gamma H$

Total pressure force per unit length of wall is given by,

$P = \int_0^H K_o \gamma z \, dz = K_o \gamma \left[\frac{z^2}{2}\right]_0^H$

$P = \frac{1}{2} K_o \gamma H^2$

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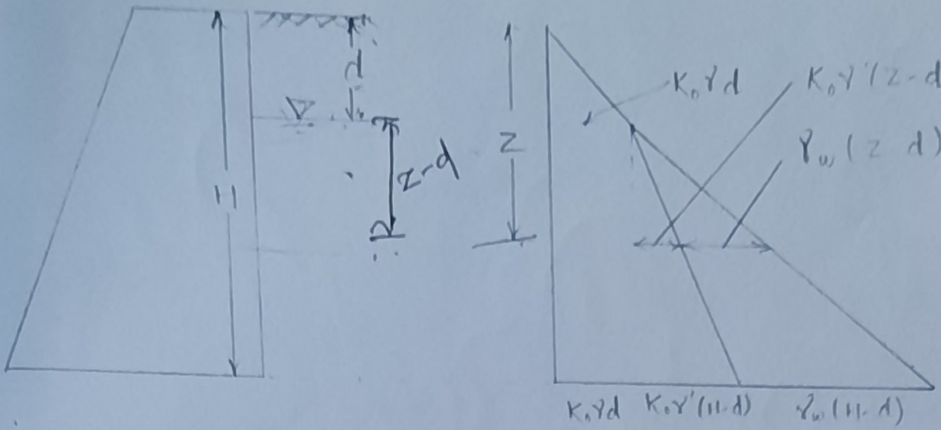


The pressure at depth $z > d$ is given by,

$$P_h = K_0 (\gamma z - \gamma_w (z-d)] + \gamma_w (z-d)$$

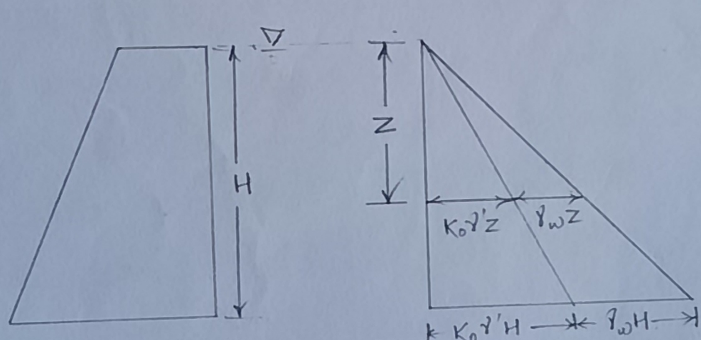
$$P_h = K_0 \gamma d + K_0 \gamma' (z-d) + \gamma_w (z-d)$$

The pressure at bottom ($z = H$) of the wall is given by,

$$P_h = K_0 \gamma d + K_0 \gamma' (H-d) + \gamma_w (H-d).$$

If water table is at the ground surface, the pressure at the bottom of wall is given by, taking $d = 0$,

$$P_h = K_0 \gamma' H + \gamma_w H.$$



The resultant pressure acts at a distance of $H/3$ from the base obtained from triangular pressure distribution diagram.

K_0 can be computed from the following expr

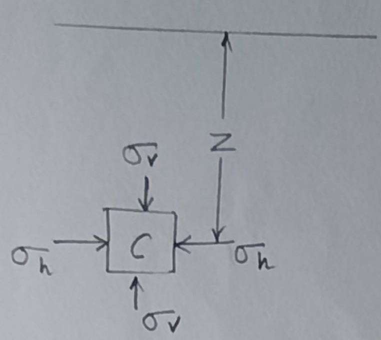
$$K_0 = 1 - \sin\phi$$

$$\frac{\bar{\sigma}_h}{\sigma_v} = K_0 = \frac{\mu}{1-\mu}$$

where,

μ = poisson's ratio.

Active Earth Pressure:-



Consider a element C at a depth z below the ground level. The horizontal pressure is,

$$\sigma_h = K_0 \sigma_v$$

where,

σ_v = vertical stress at C

$= \gamma Z$

→ The stress σ_h is minor principal stress & σ_v is major principal stress.

A retaining wall retains earth, if the retaining wall were not there, the backfill would assume a flat stable slope. When the backfill is retained, the wedge of soil above certain slope tends to slide & move away from rest of the backfill for equilibrium. It tends to push or rotate the wall away from the backfill if the wall is free to move.

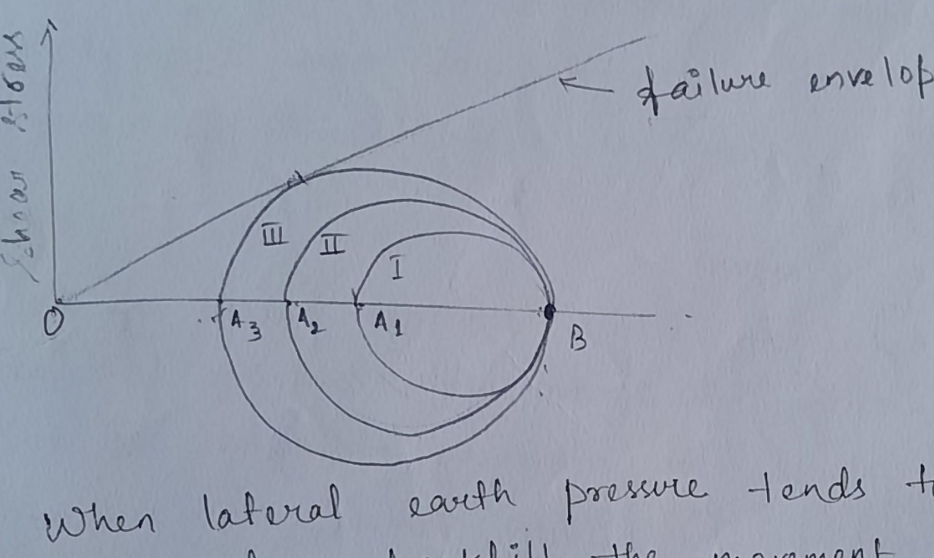
Movement of wall away from backfill causes expansion of backfill, resulting in stress release

15.15

by reducing the lateral earth pressure. Thus the movement of wall away from the backfill, is horizontal strain in backfill in form of expansion and less is lateral earth pressure. (19)

Initially the wall is in state of rest, subjected to vertical stress due to self-wt. of soil above the element & lateral earth pressure in horizontal direction.

The state of stress in soil is represented by Mohr's Circle I, where OB is vertical stress & OA_1 is lateral earth pressure at rest.



When lateral earth pressure tends to push the wall away from backfill, the movement of wall away from the backfill causes expansion of backfill resulting in stress release, thereby reducing the lateral earth pressure. Thus more the movement of wall away from the backfill, more is the

horizontal strain in the backfill in form of
sion & the less is the lateral earth pressure
The Mohr's Circle II in which $\sigma_h = \sigma_3 = OA_2$
reduced lateral earth pressure while vertical
$\sigma_v = \sigma_1 = OB$ remains constant.

The decrease in earth pressure due to mo
of wall away from the backfill & expansion & st
release continues until Mohr's Circle touches failure
envelope of backfill material & it is obtained f
Mohr's Circle (III) where backfill is on verge of fail
and no further decrease in lateral earth pressure
can takes place.

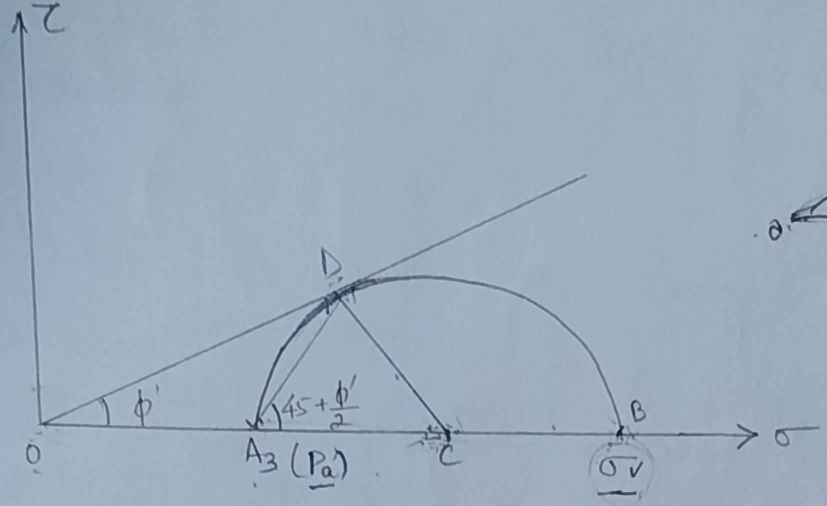
This minimum lateral earth pressure exerted on
the retaining wall, when the wall moves away from
the backfill is known as active earth pressure.

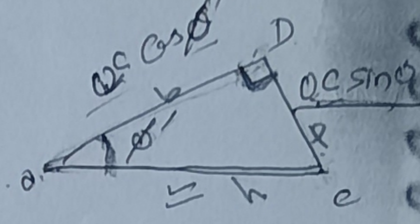
Passive Earth Pressure :-

All retaining walls are not placed on the
ground surface on the front side but are laid at
some depth. Hence, the retaining walls has soil to
some depth on its front side. When wall moves away
from the backfill due to active earth pressure, it
actually moves towards the soil on front side.

This movement is resisted by the front soil &
exerts a lateral pressure on the wall, in the
direction opposite to active earth pressure.

16.16





$\sin\phi = \frac{P}{h}$

$P = h\sin\phi$

$CD = OC\sin\phi'$

$p_a = OA_3 = OC - A_3C$

$A_3C = DC = OC\sin\phi'$

$p_a = OC - OC\sin\phi' = OC(1 - \sin\phi')$ —— (i)

$\sigma_v = OB = OC + CB = OC + OC\sin\phi'$ $OB = CD$.

$\sigma_v = OC(1 + \sin\phi')$ —— (ii)

from eqn. (i) & (ii),

$$\frac{p_a}{\sigma_v} = \frac{1 - \sin\phi'}{1 + \sin\phi'}$$

$$p_a = \left(\frac{1 - \sin\phi'}{1 + \sin\phi'}\right)\sigma_v$$

$$p_a = K_a(\gamma . z)$$ (σ_v)

$kN/m^3 \times m = kN/m^2$

$\sigma_v = \gamma z$

at rest $K_0 \gamma z$

active $K_a \gamma z$

pass $K_p \gamma z$

K_a = coefficient of active earth pressure

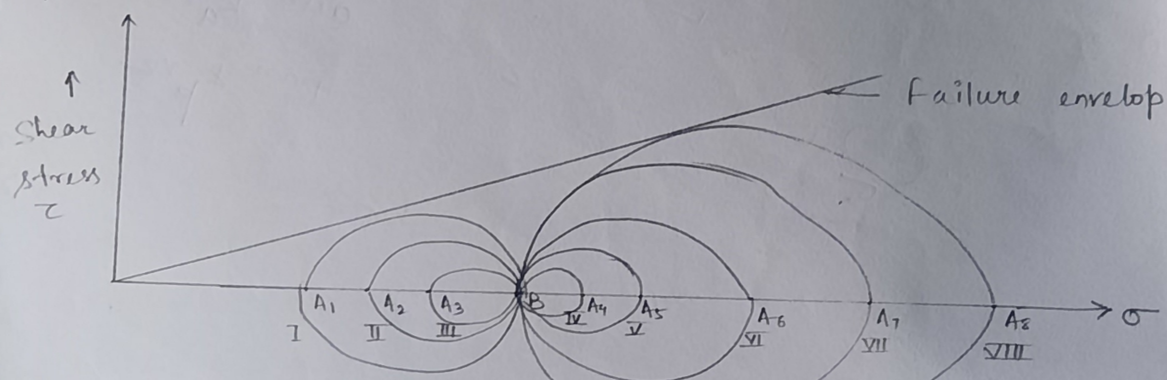
$$= \frac{1 - \sin\phi'}{1 + \sin\phi'} = \tan^2\left(45 - \frac{\phi'}{2}\right)$$

Passive Earth Pressure :-

All retaining walls are not placed on the surface on the front side but are laid at depth. Hence, the retaining walls has soil to depth on its front side. When wall moves from the backfill due to active earth pressure, it actually moves towards the soil on front side.

This movement is resisted by the front soil exerts a lateral pressure on the wall in the direction opp. to active earth pressure. The movement of wall towards the front soil causes comp. of soil which ultimately increases the lateral pressure from the front soil.

More the movement of wall towards the front soil, the more is horizontal strain in the front soil in form of paction and more is lateral earth pressure.



17:17

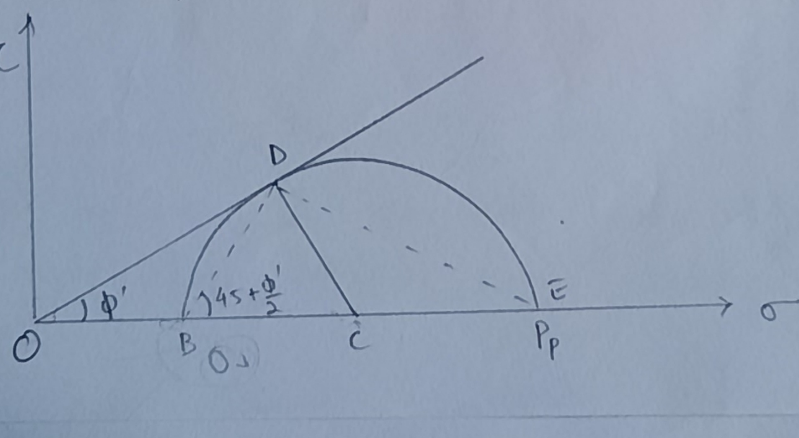
In Mohr's Circle II, $\sigma_h = \sigma_3 = OA_3$ is increased lateral earth pressure whereas vertical stress $\sigma_v = \sigma_1 = OB$ remains constant. (24)

Increase in lateral earth pressure causes decrease diameter of Mohr's Circle as II & III. Further increase in lateral earth pressure from front soil makes it higher than vertical stress. At this stage lateral earth pressure becomes major principal stress & vertical stress becomes the minor principal stress.

The increase in lateral earth pressure due to movement of wall towards the front soil & compression continues until Mohr's circle touches the failure envelope of front soil.

When Mohr's circle touches the failure envelope, the front soil is on verge of failure & no further increase in lateral earth pressure can take place.

The max. lateral earth pressure exerted on the retaining wall, when the wall moves towards the front soil, while it reaches its limiting equilibrium, is known as passive earth pressure.



$P_p = OE + CE$

$= OC + CD = OC + OC \sin\phi' = OC(1+\sin\phi')$

$OB' = OC - BC = OC - CD = OC - OC\sin\phi'$

$\sigma_v = OC(1-\sin\phi')$

From eqn. (i) & (ii),

$$\frac{P_p}{\sigma_v} = \frac{OC(1+\sin\phi')}{OC(1-\sin\phi')} = \frac{1+\sin\phi'}{1-\sin\phi'}$$

$$\Rightarrow P_p = \left(\frac{1+\sin\phi'}{1-\sin\phi'}\right)\sigma_v$$

$$P_p = K_p \gamma Z$$

K_p = coefficient of passive earth pressure,

$$K_p = \frac{1+\sin\phi'}{1-\sin\phi'} = \tan^2\left(45+\frac{\phi'}{2}\right)$$

Rankine's Theory :-

Rankine's theory of lateral earth pressure is applied to uniform cohesionless soils only. Following are the assumptions of Rankine's theory :-

i) The soil mass is semi-infinite, homogeneous, dry & cohesionless.

ii) The ground surface is plane which may be horizontal or inclined.

iii) The back of the wall is vertical & smooth.

iv) The wall yields about the base & satisfies the deformation conditions for plastic equilibrium.

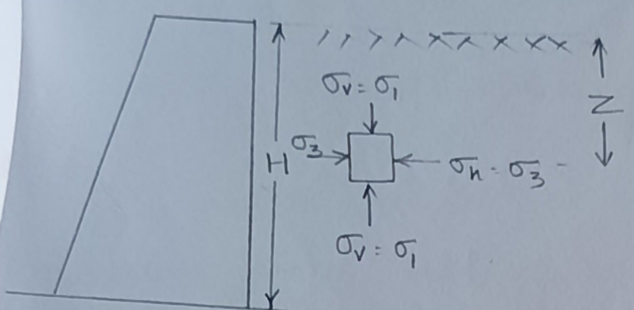
15 15

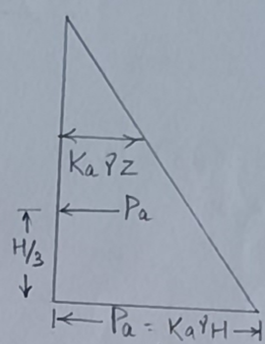
The following cases of cohesionless backfill will be considered :-

(22)

(i) Dry or moist backfill with no surcharge.
(ii) Submerged backfill.
(iii) Backfill with uniform surcharge.

Dry or moist Backfill with no Surcharge :-





Consider an element at depth z below the ground surface.

When wall is at the point of moving outward (i.e away from the backfill) active state of plastic equilibrium is established. The horizontal pressure σ_h is then min. principal stress σ_3 & vertical pressure σ_v is major principal stress σ_1.

$$\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\phi}{2}\right)$$

$$\frac{\sigma_3}{\sigma_1} = \frac{\sigma_h}{\sigma_v} = \frac{p_a}{\sigma_v} = \frac{1}{\tan^2\left(45+\frac{\phi}{2}\right)} = \cot^2\left(45+\frac{\phi}{2}\right)$$

$P_D = \sigma_C + c_F$

where,

σ_h = lateral earth pressure = p_a

σ_v = vertical pressure = $\gamma . z$

$p_a = \gamma . z . \cot^2(45^\circ + \frac{\phi}{2}) = K_a \gamma z$

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi}.$$

At $z = H$, the earth pressure is,

$p_a = K_a \gamma H$.

The total eactive earth pressure p_a,

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

acting at H/3 above base of wall.

If soil is dry, γ is dry unit wt. of soil & if wet, γ is moist unit wt.

Submerged Backfill :-

In this case, the sandfill behind retaining wall is saturated with water.

The lateral pressure is made up of two components:

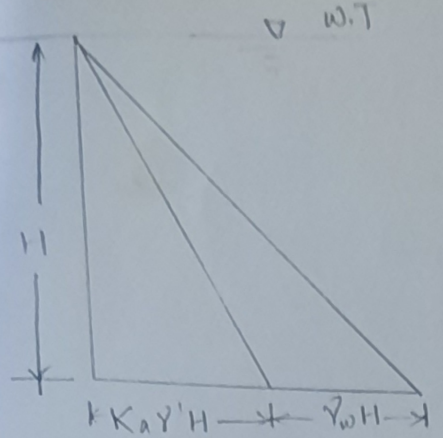
i) lateral pressure due to submerged wt. γ' of soil.

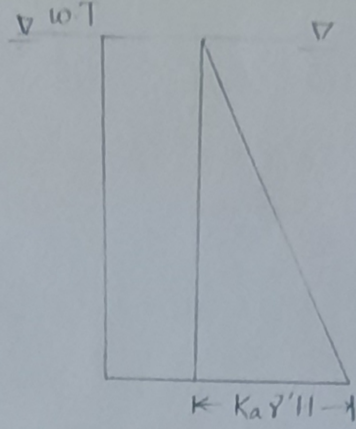
ii) lateral pressure due to water.

The pressure at a depth z below the surface,

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

19:19



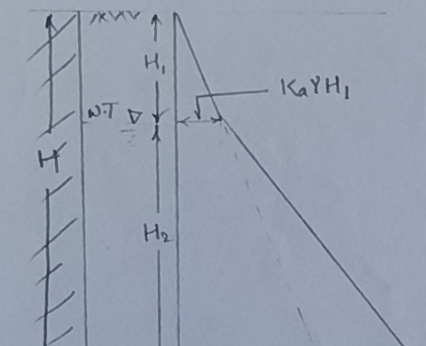


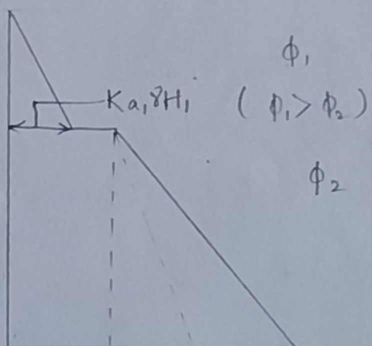
The pressure at base of retaining wall ($z = H$) is,

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

If water stands on both sides of wall the water pressure need not be considered,

$$p_a = K_a\gamma'H$$





If backfill is partly submerged i.e the backfill is moist to a depth H_1 below the ground level & then it is submerged, the lateral pressure at base is

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

It is assumed that the value of ϕ is same for moist as well as submerged soil.

If value of ϕ is different i.e ϕ_1 & ϕ_2 the earth pressure coefficient K_{a_1} & K_{a_2} will be different for both portions.

If ϕ increases, K_a decrease.

The lateral earth pressure at the base,

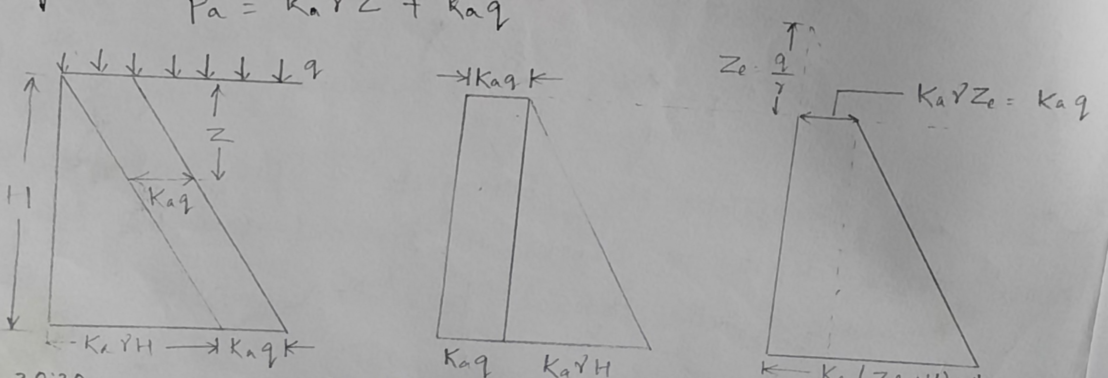
$$p_a = K_{a_1} \gamma H_1 + K_{a_2} \gamma' H_2 + \gamma_w H_2$$

Backfill with Uniform Surcharge :-

If the backfill is horizontal & carries a surcharge of uniform intensity q per unit area, the vertical pressure increment, at any depth z, will increase by q. The increase in pressure due to this will be $K_a q$.

Hence, lateral pressure at any depth z is given by,

$$p_a = K_a \gamma z + K_a q$$



At the base of wall, $p_a = K_a \gamma H + K_a q$.

9.0 FOUNDATION ENGINEERING

DATION :-

It is the lower portion of the building usually ted below ground level, which transmits the loads the super structure to the supporting soil. So, foundation is that part of the structure which in direct contact with the ground to which loads re transmitted.

FUNCTIONS OF FOUNDATION :-

i) Reduction of Load Intensity

Foundation distributes the loads of the super structure, to a larger area so that the intensity of the load at its base (total load divided by the total area) does not exceed the safe bearing capacity of the sub-soil.

) Even Distribution of Load

Foundation distribute the non-uniform load of super structure evenly to the sub-soil. For example, two columns carrying unequal loads can have a combined footing which may transmit the load to sub soil evenly with uniform soil pressure. Due to this, unequal or differential settlements are minimized.

iii) Provision of level Surface
(2) Foundation provide leveled & hard sur which the super structure can be built.

iv) Lateral Stability
It anchors the super structure to the gro thus imparting lateral stability to the supe structure. The stability of the building, again sliding & overturning due to horizontal force (such as wind, earthquake etc) is increased due foundation.

v) Safety against Underming
It provides the structural safety against underming or scouring due to burrowing anim & flood water.

vi) Protection Against Soil Movements
Special foundation measures prevents or minimizes the cracks in the superstructure, due to expansion or contraction of the sub-soil because of moisture movement in some problematic soils.

SHALLOW FOUNDATION:
A foundation is said to be shallow if its depth is equal to or less than the width of foundation.

Deep FOUNDATION:

When the depth of foundation is greater or equal to width of foundation, it is known as deep foundation.

(3)

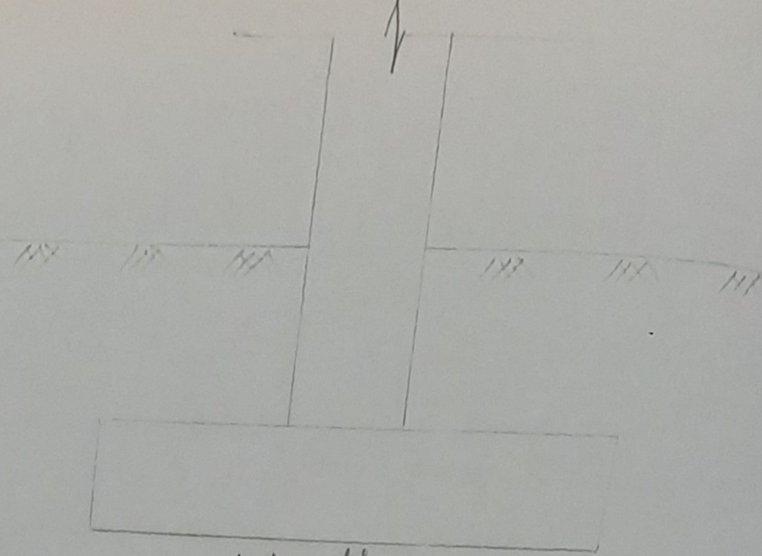
TYPES OF SHALLOW FOUNDATION:-

The various types of shallow foundations are as follows:-

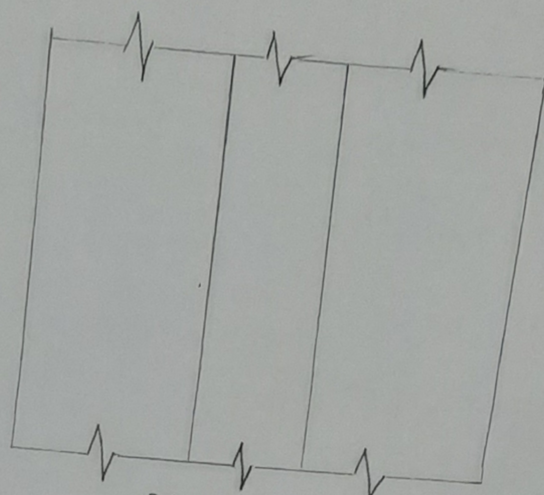
i) Strip footing
ii) Spread or Isolated footing
iii) Combined footing.
iv) Strap or Cantilever footing
v) Mat or Raft foundation

Strip Footing:-

A strip footing is provided for a load bearing wall. A strip footing is also provided for a row of columns which are so closely spaced that their spread footings overlap or nearly touch each other. In such a case, it is more economical to provide a strip footing than to proride a no. of spread footings in one line. A strip footing is also known as continuous footing.

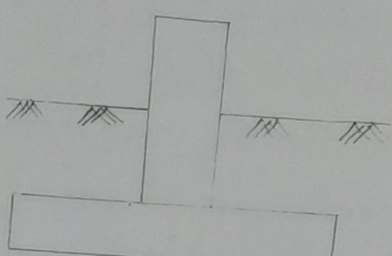


Elevation



Plan

Spread or Isolated Footing :-



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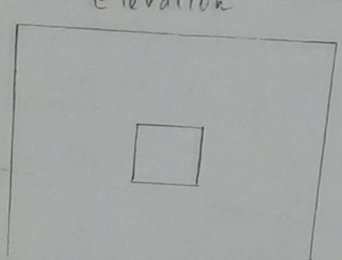
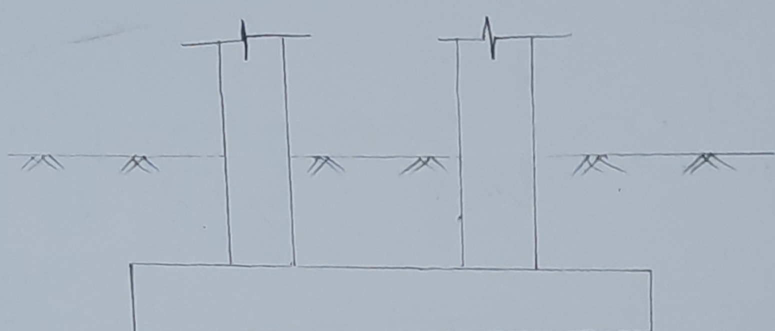


Plan

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

ined Footing :-

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



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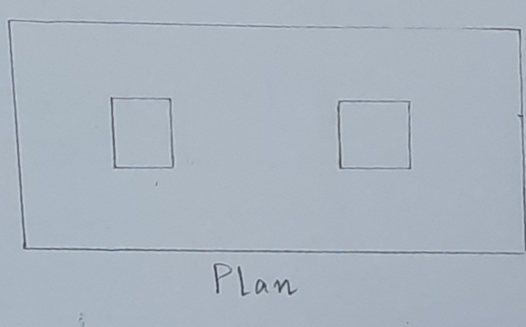


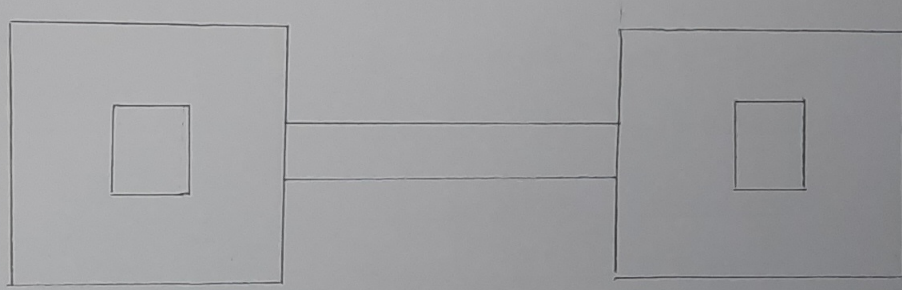
Plan

iii) Provision of level ...

Strip or Cantilever Footing :

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

Elevation

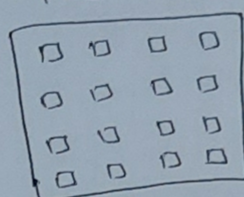


Plan

Raft Foundation :-

A mat or raft foundation is a large slab supp-
-orting a no. of columns & walls under the entire
structure or a large part of the structure. A mat
is required when the allowable soil pressure is
low or where the columns & walls are so close
that individual footings would overlap or nearly
touch each other.

Mat foundations are useful in reducing the
differential settlements on non-homogeneous soil or
where there is a large variation in the loads on
individual columns.



TYPES OF DEEP FOUNDATION :-

Deep foundation is classified into the following
three types :-

> Pile foundation
> Pier foundation
> Well (Caissons) foundation

Pile Foundation :-

Pile is a slender member with small area of
cross-section relative to its length. They can transfer
load either by friction or by bearing. Pile foundation

iii) Provision ... are used when :-

→ The load is to be transferred to stronger or compressible stratum, preferably rock.

→ The granular soil need to be compacted.

→ The horizontal & inclined forces need to be car... from the bridge abutments & retaining walls.

Classification of Pile Foundation :-

The pile foundation can be further classified i... following types on various basis such as function, material, method of installation which as listed below

Based on function :-

→ Bearing pile

→ Friction pile

→ Combined pile (both bearing & friction)

Based on material :-

→ Timber piles

→ Concrete piles

→ Steel piles

Based on method of installation :-

→ Large displacement piles

→ Small displacement piles

→ Non-displacement piles

oundation :-

hese are underground cylindrical structural
that support heavier load of the structure
h shallow foundation cannot resist. Pier found
n can only transfer load by bearing. Pier
ndation are shallower in depth than the pile
oundation. Pier foundation are used when :-

The top strata is a decomposed rock underlying
as sound rock strata.

The soil is stiff clay that occurs large resistance
for driving the bearing pile.

Well (Caissons) Foundation :-

Caissons refer to box or a case. These are
hollow inside & are usually constructed at the site
& sunk in place into a hard bearing strata. As
they are expensive in construction, they are restri
cted to major foundation works. Well foundations
are suitable when the soil contains large boulders
obstructing the penetration during installation
of pier or pile foundations. Caissons are used for
bridge piers, abutments in rivers & lakes & other

are used when :-

shore protection works. They are used to heavy vertical loads & horizontal loads & in the construction of large water front as pump houses.

Classification of Well Foundation :-

→ Open caissons

→ Pneumatic caissons

→ Box caissons

OF FAILURE :-

When load of the structure is more than bearing capacity of soil, the footing fails. There are three types of failures, they are :-

i) General shear failure.

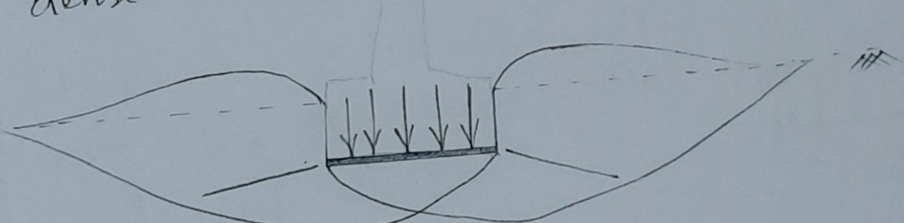
ii) Local shear failure

iii) Punching shear failure.

General Shear Failure :-

→ Continuous failure surfaces develop between the edge of footing & ground surface.

- When pressure on footing reaches ultimate bearing capacity, the soil around edge of footing gradually spreads downwards & outwards.
- In this type of failure considerable bulging of soil takes place.
- It occurs in soil of low compressibility i.e dense or stiff soil.



are used when :

Characteristics of general shear failure :

i) It has well defined failure surface, re... upto ground surface.

ii) There is considerable bulging of soil mass a... to the footing.

iii) Failure is accompanied by tilting of footing.

iv) Failure is sudden.

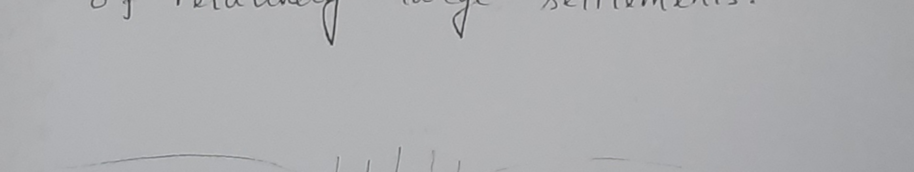
v) The ultimate bearing capacity is well defined.

Local Shear Failure :-

→ In this failure significant compression of soil under the footing takes place due to this reason the failure surfaces do not reach the ground surf... and only slight heaving occurs.

→ In this failure tilting of foundation is not expected.

→ Failure is not sudden & characterised by occurrance of relatively large settlements.



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..h failure occurs in soil of high compressi- ... and in sand having relative density lying ..ween 35 & 70%.

..ltimate bearing capacity in such type of failure ... not well defined.

..haracteristics of Local shear failure :-

- Failure surfaces do not reach ground surface.
- There is only slight bulging of soil around the footing.
- Failure is not sudden & no tilting of footing.
- Failure is defined by large settlements.
- Ultimate bearing capacity is not well defined.

..unching Shear Failure :-

- It occurs in relatively high compressible soil under the footing along with shearing in vertical direction around the edge of footing.
- It occurs in relatively loose sand with relative density less than 35%. It also occurs in soil of low compressibility.

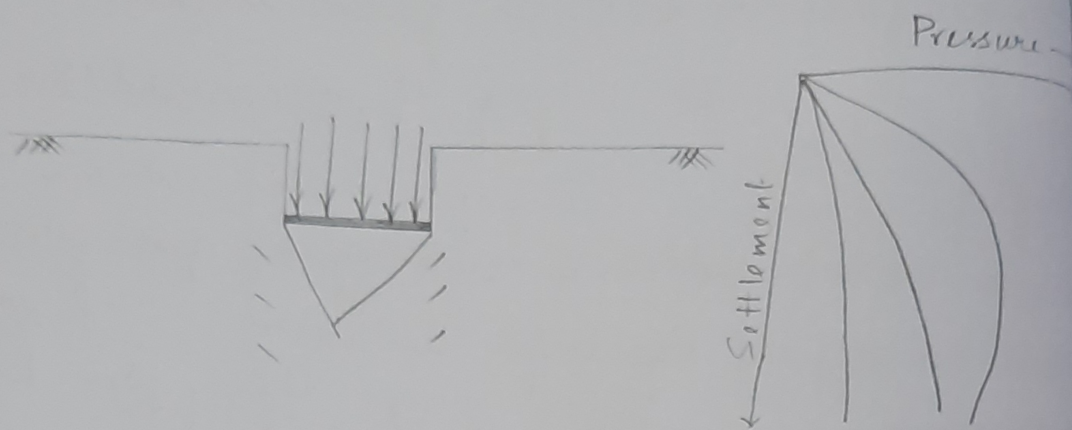
are used when :

→ The failure surface is vertical or slight[ly]
and follow the perimeter of base & never [reach]
the ground surface.

→ There is no heaving of the ground surface
from the edges & no tilting of footing

→ Relatively large settlements takes place.

→ The ultimate bearing capacity is not well defin[ed]



Characteristics of Punching shear failure :–

i) No failure pattern is observed.

ii) The failure surface which is vertical, follows the perimeter of base.

iii) No bulging of soil around the footing.

iv) No tilting of footing.

v) Ultimate bearing capacity is not well defined.

vi) Large settlement takes place.

2020

At the base of

BEARING CAPACITY OF SOIL :-

(11)

Foundation Soil :- It is the upper part of the earth carrying the load of the structure.

Bearing Capacity :- It is the capacity of soil to support the loads applied to the ground.

Gross Pressure Intensity (q) :- It is the total pressure at the base of the footing due to weight of the superstructure, self-weight of footing & wt. of earth fill, if any.

* Net pressure intensity (q_n) :- It is the excess pressure, or the difference in intensities of the gross pressure after the construction of the structure and original overburden pressure.

$$q_n = q - \gamma D$$

where, D = depth of footing

γ = avg. unit wt. of soil above foundation base.

* Ultimate Bearing Capacity (q_f) :-

It is the minimum gross pressure intensity at the base of foundation at which the soil fails

* **Net Ultimate Bearing Capacity (q_{nf}):-** It is minimum net pressure intensity causing the of soil.

$$q_f = q_{nf} + \bar{\sigma} \quad \text{or} \quad q_{nf} = q_f - \bar{\sigma}$$

where,
$\bar{\sigma}$ = effective surcharge at the base level of foundation.

* **Effective surcharge at the base level of found ($\bar{\sigma}$):-** It is the intensity of vertical pressure at the base level of foundation, computing assuming total unit wt. of the portion of the soil above the water table & submerged unit wt. for the portion below the water table.

* **Net safe bearing Capacity (q_{ns}):-** It is the net ultimate bearing capacity divided by a factor of safety F.

$$q_{ns} = \frac{q_{nf}}{F}$$

* **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 the net safe bearing capacity plus original overburden pressure.

$q_s = q_{ns} + \gamma D = \frac{q_{ns}}{F} + \gamma D$

is also known as ultimate bearing capacity divided by a factor of safety F.

Safe Bearing Pressure or Net Soil Pressure for specified settlement:- It is the intensity of loading that will cause a permissible settlement or specified settlement for the structure.

Allowable Bearing Capacity or pressure (q_a):-
It is the net loading intensity at which neither the soil fails in shear nor there is excessive detrimental to the structure.
(harmful or damaging)

Minimum Depth:-
IS codes specifies that all foundations should extend to a depth of at least 50 cm below the natural ground surface. But in case of rocks, only the top soil should be removed & the surface should be cleaned.
Sometimes, the minimum depth of foundation is determined from Rankines formula,

$$D_{min} = \frac{q}{\gamma}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)$$

where,

q : intensity of loading

γ : unit wt of soil.

Bearing Capacity of Soil using Terzaghi

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

$$\Rightarrow q_{nf} = cN_c + \bar{\sigma}(N_q - 1) + 0.5\gamma B N_\gamma$$

where,

$c.N_c$ = effect of cohesion

$\bar{\sigma} N_q$ = effect of over burden

$\gamma B N_\gamma$ = effect of soil in shearing zone.

N_c, N_q & N_γ = Bearing capacity factors.

Terzaghi gave the following eqn. for bearing capacity factors.

$$N_c = (N_q - 1)\cdot\cot\phi$$

$$N_q = \frac{a^2}{2\cdot\cos^2\left(45+\frac{\phi}{2}\right)}$$

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

$K_{p\gamma}$ = passive earth pressure coefficient

$(3\pi/4 - \phi/2)\tan\phi$

case of taking account the shape of the footing (strip, round, square etc) Terzaghi used only shape factors with cohesion (S_c) & base (S_γ) terms.

Taking into account these factors, Terzaghi's original eqn. can be written as :-

$$q_f = cN_cS_c + \bar{\sigma}N_q + 0.5\gamma BN_\gamma S_\gamma$$

Shape	Strip	Round	Square	Rectangle
S_c	1.0	1.3	1.3	$1+0.3\frac{B}{L}$
S_γ	1.0	0.6	0.8	0.8 or $1-0.2\frac{B}{L}$

2) Frictional cohesive soil ($c-\phi$ soil)

For circular footing,

$$q_f = 1.3cN_c + \bar{\sigma}N_q + 0.3\gamma BN_\gamma$$

where, B = dia. of footing

For square footing,

$$q_f = 1.3cN_c + \bar{\sigma}N_q + 0.4\gamma BN_\gamma$$

where, B = width or length of footing

For strip footing,

$$q_f = cN_c + \bar{\sigma}N_q + 0.5\gamma BN_\gamma$$

1) Cohesive Soil ($\phi = 0$, $c > 0$)

For circular footing,

$$q_f = 1.3 c N_c + \bar{\sigma} = 7.4c + \bar{\sigma}$$

For strip footing,

$$q_f = 5.7c + \bar{\sigma}$$

For rectangular & square footing,

$$q_f = c N_c \left(1 + 0.3 \frac{B}{L}\right) + \bar{\sigma}$$

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

For strip footing,

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

For rectangular & square footing,

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

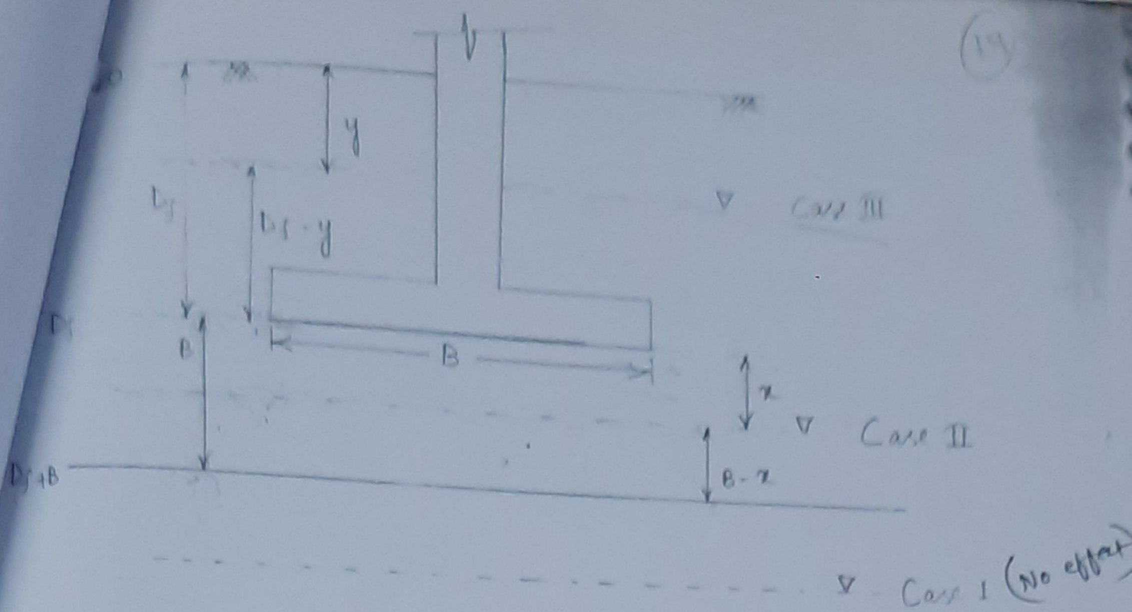
For circular footing,

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

Terzaghi's Equation for Location of Water Table :-

Terzaghi's eqn. for strip footing,

$$q_{nf} = c N_c + \bar{\sigma}(N_q - 1) + 0.5 \gamma B N_\gamma$$



Case I - when water table is below depth $D_f + B$,

$$q_{nf} = cN_c + \gamma D (N_q - 1) + 0.5 \gamma B N_\gamma$$

where,
γ = Bulk unit wt.

Case II - when water table is between D_f & $D_f + B$,

$$q_{nf} = cN_c + \gamma D (N_q - 1) + 0.5 [x\gamma + (B - x)\gamma_{sub}] N_\gamma$$

Case III - when water table is between 0 to D_f,

$$q_{nf} = cN_c + [\gamma y + (D_f - y)\gamma_{sub}](N_q - 1) + 0.5 \gamma_{sub} B N_\gamma$$

q_u $(cN_c + \gamma D N_q + 0 \ldots \quad) - \gamma D$

$q_n = q_u - \gamma D \qquad \gamma D (N_q - 1)$

Another Method,

$$q_{nf} = c N_c + \gamma D_f (N_q - 1) R_{w_1} + 0.5 \gamma B N_\gamma R_{w_2}$$

R_{w_1} & R_{w_2} = correction factors for the terms in N_q & N_γ for the effect of water table

$$R_{w_1} = 0.5 \left(1 + \frac{D_w}{D_f}\right) \text{ when } 0 < \frac{D_w}{B} \leq 1$$

where D_w = depth of water table below the ground level limited to a depth of D_f.

& $$R_{w_2} = 0.5 \left(1 + \frac{D_w'}{B}\right) \text{ when } 0 < \frac{D_w'}{B} < 1$$

where,

D_w' = depth of water table measured from the base of footing with a limiting value equal to width of footing.

7.0 Shear Strength.

It is the maximum resistance to shear stress just before failure.

- Soils are not much subjected to direct shear, they are subjected to direct compression from which shear stress develops.

→ Shear failure of soil occurs when shear stress induced due to the applied compressive load exceeds the shear strength of the soil.

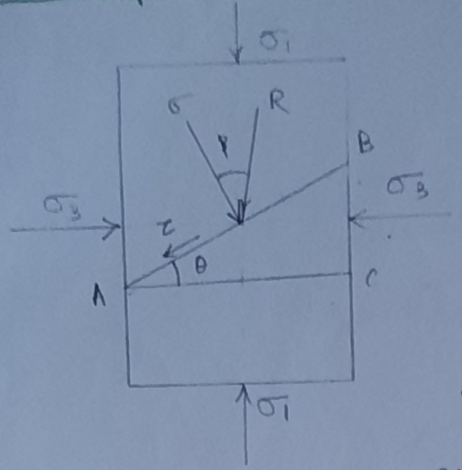
→ Failure in soil occurs by relative movement of the particles and not by breaking of particles.

→ A every point on a stressed body, there are three planes on which the shear stresses are zero. These planes are known as principal planes. The plane with max. compressive stress (σ_1) is called major principal plane and with minimum compressive stress (σ_3) as minor principal plane. The third principal plane is subjected to a stress which has the value between σ_1 & σ_3, known as intermediate principal plane.

- The stresses on intermediate principal plane are not much relevant only major principal stress (σ_1) & minor principal stress (σ_3) are generally important.

In soil mechanics compressive stresses are taken as positive & tensile stresses as negative.

7.1 Concept of shear strength



Consider a plane. The major & minor principal stresses acts on this plane. The major principal plane is horizontal & minor principal plane is vertical.

The plane AB is inclined at an angle θ to the major principal plane AC.

Resolving forces acting on wedge ABC in horizontal,

$$\sigma_3 BC = \sigma AB \sin\theta - \tau AB \cos\theta$$

where, σ = normal stress on AB.

τ = shear stress on AB.

The above eqn. is simplified as,

$$\sigma_3 \frac{BC}{AB} = \sigma \sin\theta - \tau \cos\theta$$

$$\Rightarrow \sigma_3 \sin\theta = \sigma \sin\theta - \tau \cos\theta \quad \text{——— (i)}$$

Resolving forces vertical,

$$\sigma_1 AC = \sigma AB \cos\theta + \tau AB \sin\theta$$

$$\Rightarrow \sigma_1 \cos\theta = \sigma \cos\theta + \tau \sin\theta \quad \text{——— (ii)}$$

Multiplying eqn. (i) by $\cos\theta$ & eqn. (ii) by $\sin\theta$ & ... ing,

$$(\sigma_1 - \sigma_3)\sin\theta\cos\theta = \sigma(\cos\theta\sin\theta - \cos\theta\sin\theta) + \tau(\sin^2\theta + ...)$$

$$\Rightarrow (\sigma_1 - \sigma_3)\sin\theta \cdot \cos\theta = \tau$$

$$\Rightarrow \boxed{\tau = \frac{1}{2}(\sigma_1 - \sigma_3)\sin 2\theta} \quad \text{——— (a)}$$

Substituting the value of τ in eqn. (i),

$$\sigma_3 \sin\theta = \sigma\sin\theta - \frac{\sigma_1 - \sigma_3}{2}\sin 2\theta \cdot \cos\theta$$

$$\Rightarrow \sigma_3 = \sigma - (\sigma_1 - \sigma_3)\cos^2\theta$$

$$\Rightarrow \sigma = \sigma_3 + (\sigma_1 - \sigma_3)\left(\frac{1 + \cos 2\theta}{2}\right)$$

$$\Rightarrow \boxed{\sigma = \frac{\sigma_1 + \sigma_3}{2} + \frac{(\sigma_1 - \sigma_3)}{2}\cos 2\theta} \quad \text{——— (b)}$$

Eqn. (a) & (b) give the stresses on the inclined plane AB, making an angle θ with the major principal plane AC.

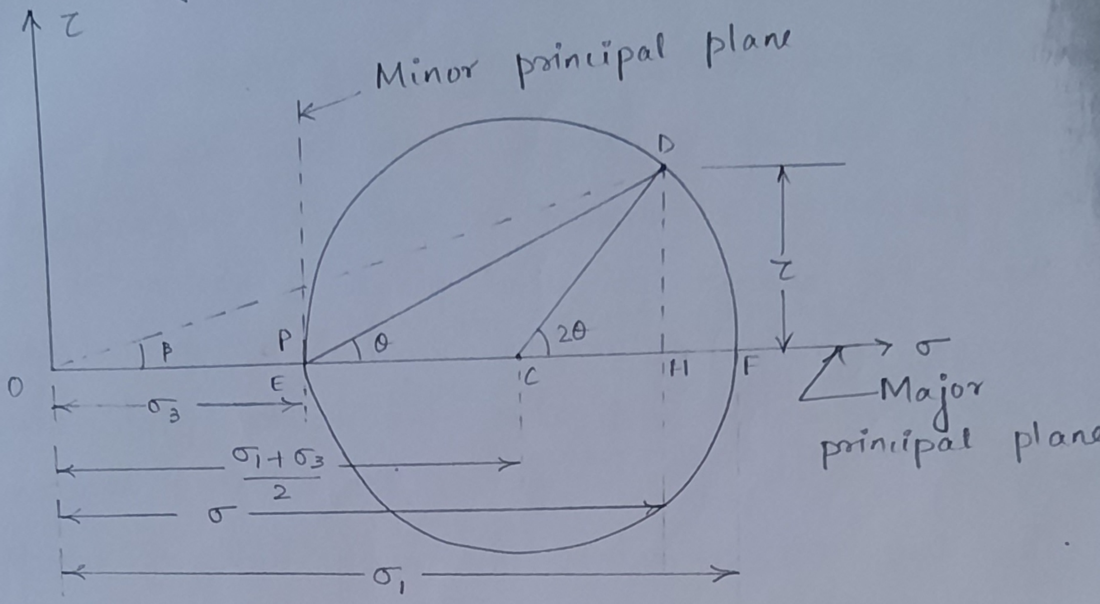
7.2 Mohr's Circle :-

→ Otto Mohr, a german scientist desired a graphical method for determination of stresses on a plane inclined to principal planes. The graphical construction is known as Mohr's Circle.

→ In this method, an origin O is selected & normal stresses are plotted along horizontal axis shear stresses on vertical axis.

As comp. stress are taken positive, they are plotted towards right of origin.

The positive shear stresses are plotted upward from origin



Point E represents the minor principal stress σ_3 & point F, the major principal stress σ_1. Point C is the mid-point with the normal stress coordinate equal to $\left(\frac{\sigma_1 + \sigma_3}{2}\right)$. The circle is drawn with C as center and EF as diameter.

This circle is known as Mohr's circle. Each point on the circle gives the stresses σ & τ on a particular plane.

The point D on the circle gives the stresses on the plane AB inclined at an angle θ to the major principal plane. The line DE makes an angle with σ-axis. The angle DCF subtended at the centre is twice the angle DEC.

$$OH = OC + CH$$

$$\Rightarrow OH = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2} \cos 2\theta = \sigma$$

$$\Rightarrow DH = CD \sin 2\theta = \frac{\sigma_1 - \sigma_3}{2} \sin 2\theta = \tau$$

The point E is known as pole (P).

When the major principal plane is horizontal & the minor principal plane is vertical & pole lies at the point E which indicates the minor principal stress.

The line OD represents the magnitude of resultant stress on inclined plane AB. The angle of the obliquity of the resultant with the normal of the plane AB is equal to the angle β.

7.3 Mohr-Coulomb Failure Theory :-

Many theories of failure have been proposed, but of them that formulated by Mohr has been useful in case of soil.

Following are essential points of Mohr's strength theory :-

i) Material fails essentially by shear. The critical shear stress causing failure depends upon the

23 : 23

properties of material as well as normal stress on failure plane.

(24)

The ultimate strength of the material is determined by the stresses on the failure plane.

When the material is subjected to three dimensional principal stress (i.e $\sigma_1, \sigma_2, \sigma_3$) 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.

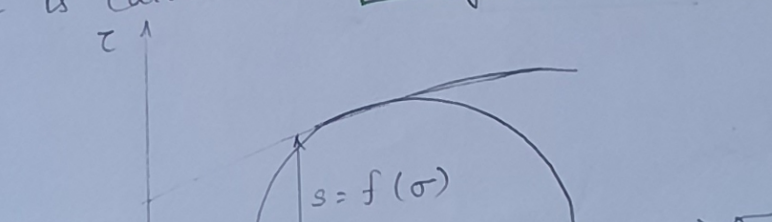
The soil fails when the shear stress (τ_f) on the failure plane at failure is a unique function of the normal stress (σ) acting on that plane.

$$\tau_f = f(\sigma)$$

Since the shear stress on the failure plane at failure is defined as shear strength (s), the eqn. can be written as,

$$s = f(\sigma)$$

If the normal & shear stress corresponding to failure are plotted, then a curve is obtained. The plot or the curve is called the **strength envelope**.



Failure of the material occurs when the circle of the stresses touches the strength Mohr circle represents all possible combinations of & normal stresses at the stressed point.

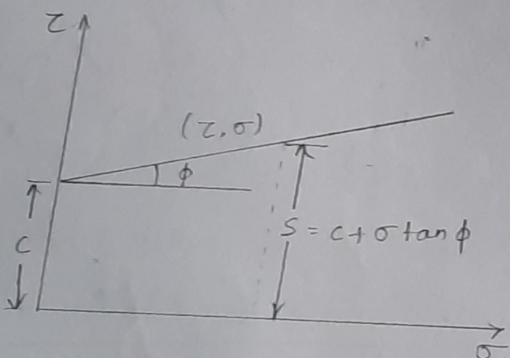
Any Mohr's circle which does not cross the failure envelope & lies below the envelope represents a no failure or stable condition.

The Mohr circle cannot cross the Mohr envelope as the failure would have already occured as the Mohr circle touches the envelope.

Coulomb defined the function $F(\sigma)$ as a linear function of σ & gave the following strength equation

$$s = c + \sigma \tan\phi$$

Mohr envelope is replaced by a straight line by Coulomb.



c is the cohesion. Cohesion holds the particles of the soil together in a soil mass, and is independent of normal stress.

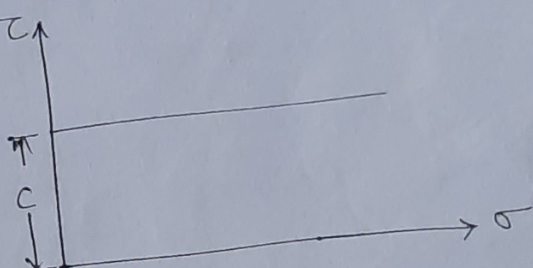
24: 24

gle ϕ is called the angle of internal friction represents the frictional resistance between the icles, which is directly proportional to the normal ress.

Coulomb considered the relationship between shear rength & normal stress can be represented by a traight line.

Mohr theory recognises that the shear strength epends on normal stress, but indicates that the elation is not linear.

For a ideal pure friction material, the straight line passes through the origin. However, dense sand exhibit a slightly curved straight line, indicated by dashed line.



In case of purely cohesive (plastic) material, the straight line is parallel to σ axis. The strength of such soil is independent of normal stress acting on the plane of failure.

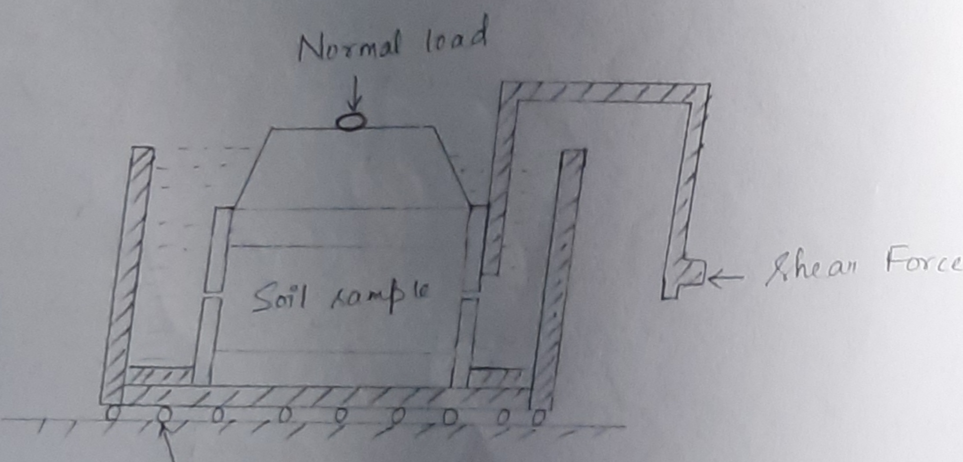
Failure of the

So, the Mohr envelope can be considered straight if the angle of internal friction ϕ assumed to be constant. Depending upon the of the material the failure envelope may be straight or curved, & it may pass through the origin or it may intersect the shear stress axis.

7.4 Measurement of Shear strength

Direct Shear Test :-

The test is carried out on a soil sample confined in a metal box of square cross-section which is split horizontally at mid-height. A small clearance is maintained between the two halves of the box. The soil is sheared along a predetermined plane by moving the top half of the box relative to the bottom half. The box is usually square in plane of size 60mm × 60mm. A typical shear box is given below.



the sample is fully or partially saturated, perforated metal plates & porous stones are placed below & abone the sample to allow free drainage. the sample is dry, solid metal plates are used. load normal to the plane of shearing can be (29) applied to the soil sample through the lid of the box.

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

As a vertical normal load is applied to the sample, shearing stress is gradually applied horizontally, by causing the two halves of the box to more relative to each other. The shear load is measured together with the corresponding shear displacement. The change of thickness of the sample is also measured.

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

Advantages of the test :-

→ It is easy to test sands & gravels.

→ Large samples can be tested in large shear box, as small samples can give misleading results due to imperfections such as fractures & fissures, ... may not be truly representative.

→ Samples can be sheared along predetermined plane when the shear strength along fissures or other selected planes are needed.

Disadvantages of the test :-

→ The failure plane is always horizontal in the test, & this may not be the weakest plane in the sample. Failure of the soil occurs progressively from the edges towards center of sample.

→ There is no provision for measuring pore water pressure in the shear box & so it is not possible to determine effective stresses from undrained test.

26:26

ar box apparatus cannot give reliable undrained strengths because it is impossible to prevent localised drainage away from shear plane.

(30)

Unconfined Compression Test :-

It is a special form of triaxial test in which the confining pressure is zero. The test can be conducted only on clayey soil which can stand without confinement.

Although the test can be conducted in a triaxial test apparatus as a U-U test, it is more convenient to ~~for~~ perform it in an unconfined compression testing machine.

There are two types of machines :-

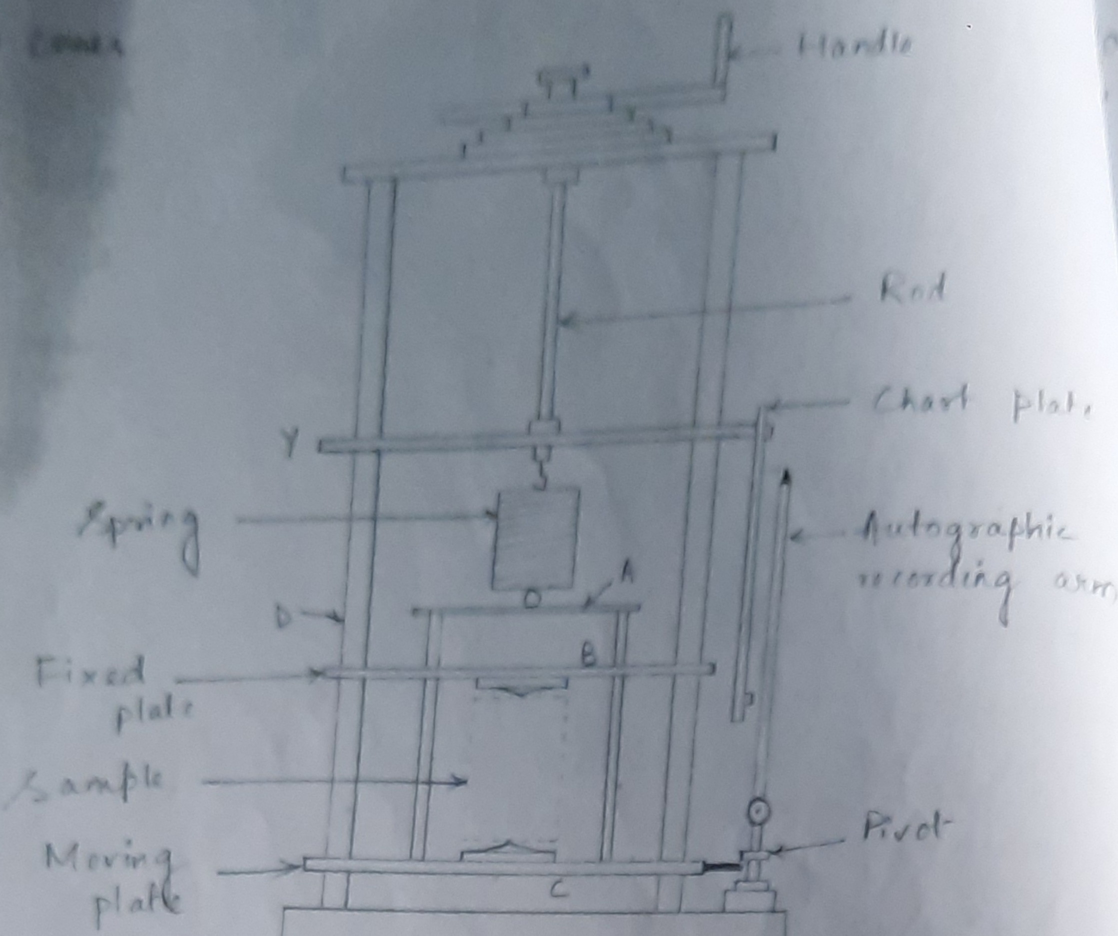
i) Machine with spring

ii) Machine with proving ring.

Machine with Spring :-

In this test, the unconfined compression testing machine is loaded with spring. It consist of two metal cones which are fixed on horizontal which are fixed on horizontal loading plates B & C supported on vertical post. The upper loading plate B is fixed in position, whereas the lower plate C can slide on the vertical post.

The unit specimen is placed between the



When handle is turned, the plate A is lifted upward. As the plate A is attached to the plate C, the latter plate is also lifted. When the handle is slowly, a compressive force acts on the specimen. Eventually, the specimen fails in shear.

The compressive force is proportional to the extension of the spring.

The strain in specimen is indicated on a chart fixed to the machine. As lower plate C moves upward, the pen attached to this plate swings sideways.

ateral movement of the pen is proportional to strain in the specimen.

(2)

The chart plate is attached to the yoke Y. As yoke moves upward when the handle is rotated, the chart plate moves upward. The pivot of the arm of the pen also moves upward with the lower plate. The vertical movement of the pen relative to the chart is equal to the extension of the spring & hence the compressive force.

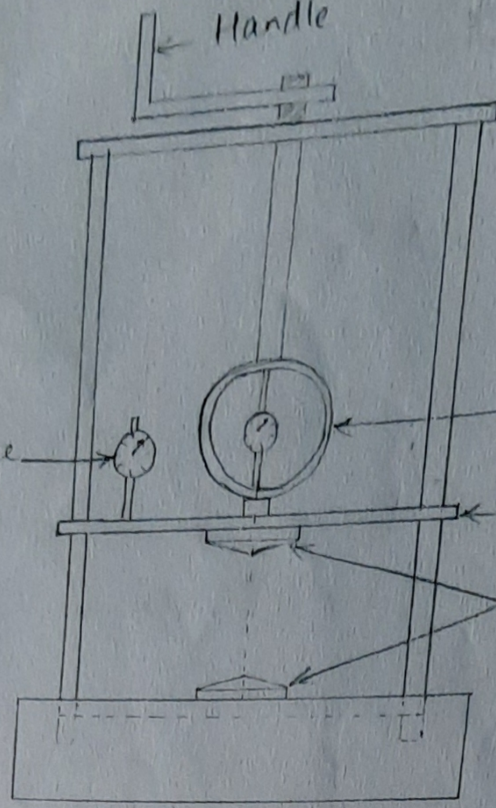
Thus the chart gives a plot between the deformation & compressive force. Spring of different stiffness can be used depending upon the expected compressive strength of the specimen.

Machine with a Proving Ring :-

In this machine a proving ring is used to measure the compressive force. There are two plates have cone seatings for the specimen. The specimen is placed on the bottom plate so that it makes contact with the upper plate. The dial gauge & proving ring are set to zero.

The compressive load is applied to the specimen by turning the handle. As the handle is turned the upper plate moves downward & causes compression.

Failure ...



As the handle is turned the shearing is continued till the specimen fails or till 20% of axial stra[in] occurs, whichever is earlier.

The compressive force is determined from the proving ring reading & the axial strain is found from the dial gauge reading.

In an unconfined compression test, the minor principal stress (σ_3) is zero. The Mohr Circle passes through the origin which is also the pole.

The major principal stress (σ_1) is equal to the deviator stress,

$$\sigma_1 = \frac{P}{A}$$

25.28

P = axial load

A = cross-sectional area.

The axial stress at which the soil fails is known as unconfined compressive strength (q_u).

The unconfined compression test is generally applicable to saturated clays for which the apparent angle of shearing resistance ϕ_u is zero.

$$\sigma_1 = 2C_u$$

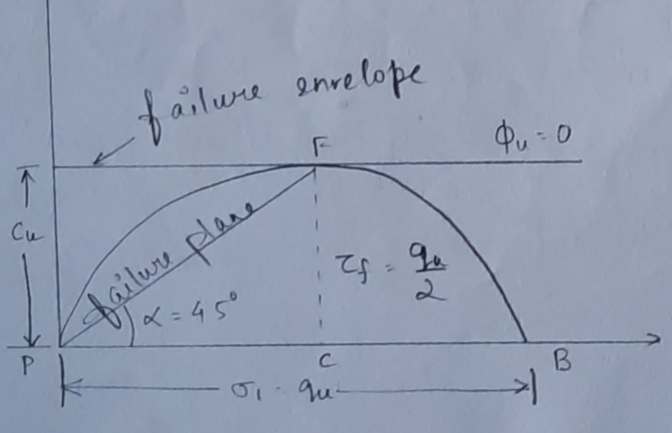
when Mohr circle is drawn, its radius is equal to $\frac{\sigma_1}{2} = C_u$. The failure envelope is horizontal. PF is the failure plane, & the stresses on the failure plane are

$$\sigma = \frac{\sigma_1}{2} = \frac{q_u}{2}$$

$$\tau_f = \frac{\sigma_1}{2} = \frac{q_u}{2} = C_u$$

where,

q_u = unconfined compressive strength at failure.



failure of the material

The compressive strength is calculated on the basis of changed cross sectional area A_2 at failure,

$$A_2 = \frac{V}{L_1 - \Delta L} = \frac{A_1}{1 - \frac{\Delta L}{L_1}}$$

where,

V = initial vol. of the specimen

L_1 = initial length of specimen

ΔL = change in length at failure.

Merits :-

i) The test is convenient, simple & quick.

ii) It is suited for measuring the unconsolidated-undrained shear strength of intact, saturated clay.

iii) The sensitivity of the soil may be easily determined by conducting the test on an undisturbed sample & then on the remoulded sample.

Demerits :-

i) The test cannot be conducted on fissured clay.

ii) The test may be misleading for soils for which the angle of shearing resistance is not zero. For such soils, the shear strength is not equal to half the compressive strength.

..e Shear Test :-

Undrained shear strength of soft clay is determin.. laboratory of by vane shear test. The test can be ..nducted in the field on the soil at the bottom of ..ore hole.

The test can be performed even without drilling a bore hole by direct penetration of the vane from ground surface if it is provided with strong shoe to protect it.

The apparatus consist of a vertical steel rod having four thin stainless steel blades fixed at its bottom end. The IS code recommends that the height H of the vane should be equal to ~~the~~ twice of the overall diameter D. The diameter & length of the rod are recommended as 2.5 mm & 60 mm.

For conducting the test in laboratory, a specimen of size 38 mm diameter & 75 mm height is taken in a container which is fixed securely at bottom.

Failure " " lowered into the specimen

The vane is gradually lowered into the specimen vane is at a depth of the top of the specimen. The reading of mm below the top of specimen. The reading of strain indicator & torque indicator are taken.

Torque is applied gradually to upper end of rod at the rate of about 6° per minute. The torque acting on the specimen is indicated by a pointer fixed to the spring. The torque is continued till the soil fails in shear. The shear strength of soil is given by,

$$s = \frac{T}{\pi\left(D^2\frac{H}{2} + \frac{D^3}{6}\right)}$$

where,

T = torque ~~in N-cm & s = N/~~

$= T_1 + T_2$

T_1 = resisting torque at the sides

T_2 = resisting torque at the top & bottom.

If top of the vane is above the soil surface & depth of the vane inside the sample is H_1.

$$s = \frac{T}{\pi\left(D^2\frac{H_1}{2} + \frac{D^3}{12}\right)}$$

30130

Vane shear test can be used to determine the sensitivity of soil. After the initial test, the vane rotated rapidly through several revolutions such that the soil becomes remoulded. The test is repeated on the remoulded soils & shear strength in remoulded state is determined,

$$\text{Sensitivity} = \frac{(s)\ \text{undisturbed}}{(s)\ \text{remoulded}}.$$

Merits :-

i) Test is simple & quick.

ii) Ideally suited for determination of in-situ undrained shear strength of non-fissured, fully saturated clay.

iii) The test can be conveniently used to determine the sensitivity of the soil.

Demerits :-

i) Test cannot be conducted on fissured clay or clay containing sand or silt.

ii) Test doesnot give accurate results when the failure envelope is not horizontal.

Machine Foundation:-

› Soil Dynamics :-

It is that part of soil mechanics which deals with soil under dynamic conditions. It studies the effect of forces on soil in any way associated with causing motion in soil.

In some cases foundations are subjected to dynamic loads. These loads may result from various causes such as vibratory motion of machine, movement of vehicles, impact of hammer, earthquake, winds, waves, nuclear blast & pile driving. The dynamic loads transmitted to the foundation & their effect on the strata below can be determined using the principles of soil dynamics and theory of vibrations.

Machine foundations are subjected to dynamic forces caused by the machine. The dynamic forces are transmitted to the foundation supporting the machine. Although the moving parts of the machines are generally balanced, there is always some unbalance in practice which causes eccentricity of rotating parts.

Various Terms :-

i) Vibration :- It is the time-dependent, repeated motion of translational or rotational type.

31:31

riodic motion :- It is the motion which repeats itself periodically in equal time intervals. (35)

Period (T) :- The time period in which the motion itself is called the period of motion or simply period.

iv) Cycle :- The motion completed in the period is called the cycle of motion.

v) Frequency :- (f) - The no. of cycles of motion in a unit of time is known as the frequency of the vibration. It is usually expressed in hertz (ie cycles per second).

$$T = \frac{1}{f}$$

vi) Free Vibration :- It occurs under the influence of forces inherent in the system itself, without any external force. However, to start free vibrations, some external force or natural disturbance is required. Once started, the vibration continue without an external force.

vii) Forced Vibration :- It occurs under the influence of a continuous external force.

viii) Natural frequency :- The system under free vibrations vibrates at the frequency known as

Failure of the [illegible] natural frequency

natural frequency. The natural frequency is characteristics of the system. A system may have more than one natural frequency.

ix) Resonance :- When the frequency of the exciting force is equal to one of the natural frequencies of the system, the amplitudes of motion become excessively large. This condition is known as resonance.

x) Damping :- The resistance to motion which develops due to friction & other causes is known as damping.

xi) Degree of freedom :- The no. of independent co-ordinates required to describe the motion of a system is called degree of freedom.

Types of Machines :-

Basically, there are three types of machines :-

i) Machines which produce a periodic unbalanced force, such as reciprocating engines & compressors. The speed of such machines is generally less than 600 r.p.m. In these machines, the rotary motion of the crank is converted into the translatory motion.

achine which produce impact loads, such as forge hammers & punch presses. In these machines, the dynamic force attains a peak value in a very short time & then dies out gradually. The response is a pulsating curve. It vanishes before the next pulse. The speed is usually between 60 to 150 blows per minute.

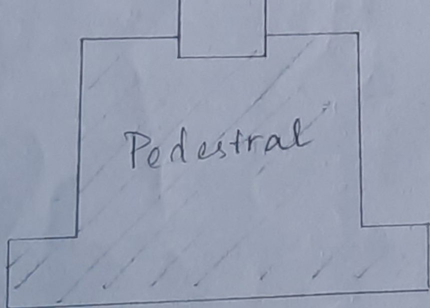
36

iii) High speed machines, such as turbines & rotary compressors. The speed of such machines is very high, sometimes, it is even more than 3000 rpm.

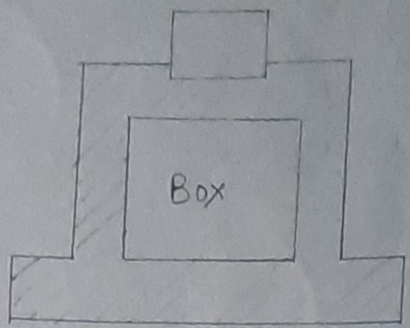
Types of Machine foundations :-

The following 4 types of machine foundations are commonly used:-

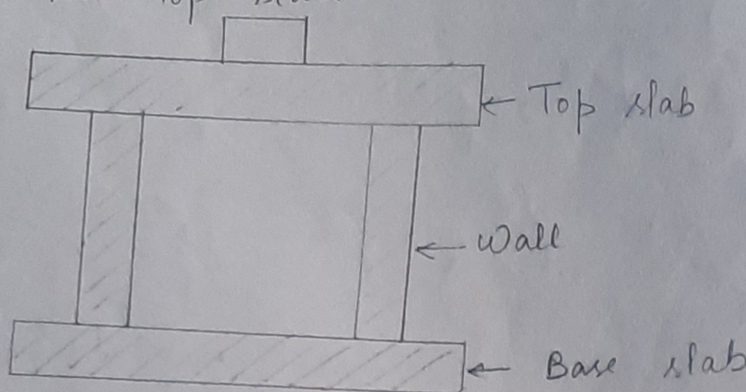
i) Block type - This type of machine foundation consists of a pedestral resting on a footing. The foundation has a large mass & a small natural frequency.



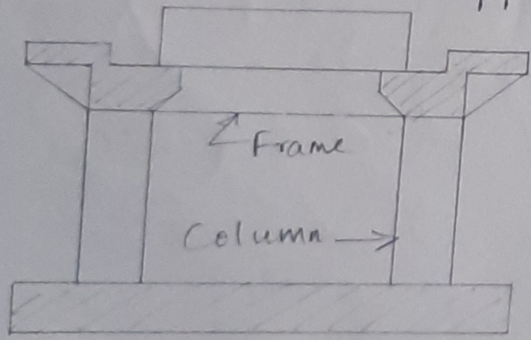
ii> Box type :- The foundation consist of a hollow block. The mass of the foundation is less than in the block type & the natural frequency is incre...



iii> Wall type :- A wall type of foundation consists a pair of walls having a top slab. The machine rests on the top slab.



iv> Framed type :- This type of foundation consists of a vertical columns having a horizontal frame at their tops. The machine is supported on the frame.



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General Requirements :-

i) Like ordinary foundations, it should be safe against shear failure caused by superimposed loads, and also the settlements should be within the safe limits

The soil pressure should normally not exceed 80% of the allowable pressure for static loading.

ii) There should be no possibility of resonance. The natural frequency of the foundation should be either greater than or smaller than the operating frequency of the machine.

iii) The amplitudes under service conditions should be within the permissible limits for the machine.

iv) The combined center of gravity of the machine & the foundation should be on the vertical line passing through the center of gravity of the base plane.

v) Machine foundation should be taken to a level lower than the level of the foundation of the adjacent buildings & should be properly separated.

vi) The vibrations induced should neither be annoying to the persons nor detrimental to other structures.

vii) The depth of ground water table should be atleast one-~~forth~~ fourth of the width of the foundation

Failure of the material
below the base plane

Design Criteria for Reciprocating type Machines

→ The size of the foundation block (in plan) should be larger than the bed plate of the machine with minimum allround clearance of 15cm.

→ The width of the foundation should be at least equal to the distance of the center of gravity of the crank shaft to the bottom of foundation in all vertical machines.

→ The depth of the foundation should be such as to rest the foundation on a good bearing strata and to ensure stability against rotation in a vertical plane.

→ Whenever possible, the operating frequency should be lower than the natural frequency of the foundation soil system & the frequency ratio should be less than 0.5. When the operating frequency is higher than the natural frequency of the foundation soil system of the machine, the frequency ratio should be more than 2 for important machines & more than 1.5 for others.

→ The combined center of gravity of machine & the foundation block should be as much below the top of the foundation as possible.

24.3.24

Requirements for Impact Type

The design requirements of the impact type machines, as drop & forge hammers, are different than of the reciprocating type machine

The stresses produced at the time of impact in the foundation base (soil, timber, sleeper, spring elements or pile) should be within 0.8 times allowable static stresses.

The design of entire foundation system should be such that the centers of gravity of the anvil (a base block for hammer on which material is forged into shape by repeated striking) & of the foundation block, as well as the joints at which the resultant of forces in the elastic joints act, coincide with the line of fall of the hammer tup (weighing block which strikes the material being forged)

→ The max. vertical vibrational amplitude of the foundation block should not be more than 1.2 mm. In case of foundations on sand below the ground water, the permissible amplitude should not be more than 0.8 mm.

→ For the anvil, the permissible amplitude, which depends upon the weight of the tup should be taken from the following table :-

Weight of tup	Max. permissible [illegible]
upto 1 t	1mm
2 t	2mm
More than 3 t	3 to 4 mm.

→ The area of foundation block should be such the safe loading intensity of soil is never exceed during the operation of the hammer. The depth of foundation block should be so designed that the b is safe both in punching shear & bending. However the following minimum thickness of foundation block should be provided.

Weight of tup (tonnes)	Max. depth of foundation block
upto 1.0	1mm
1.0 to 2.0	1.25 mm
2.0 to 4.0	1.75 mm
4.0 to 6.0	2.25 mm
over 6.0	2.5 mm.

→ The wt. of anvil may be generally kept at 25 times the weight of tup. The wt. of the foundation block generally varies from 66 to 120 times the wt. of the tup. Where the foundation rest on stiff clays or compact sandy deposits, the wt. should be from 75 to 80 times the wt of tup. For moderately firm to soft clays & for medium dense to loose sandy deposits, the wt. of the block should be

35:35

m 90 to 120 times the wt of tup. (3)

The foundation block should be made of reinforced concrete & reinforcement should be arranged along the three axes & also diagonally to prevent shear. More reinforcement should be provided at the top side of the foundation block than at the other side. Reinforcement at the top may be provided in the form of layers of grills made of 16mm dia. bars suitably spaced to allow easy pouring of concrete. The reinforcement should be provided at least 25 kg per m^3 of concrete.