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CHAPTER 3

SOIL DEPOSIT, TYPE, STRUCTURE AND CLASSIFICATION

3.1 Soil Deposits

Accumulation of soil particles forms soil deposits in a well-defined sequence of geologic processes. Depending on the type of parent materials of soil deposit, it may basically be grouped into organic and inorganic/ mineral soil deposits.

3.1.1 Soil deposits of organic origin

Organic deposits are due to the decomposition of organic matters and found usually in topsoil and marshy place. A soil deposit of organic origin is said to peat if it is at the higher end of the organic content scale (75% or more according to some authors), organic soil at the low end, and muck in between. Peat soil deposit is usually formed of fossilized plant materials and characterized by fibre content and lower decomposition or humification.

However, there are many other criteria existed to classify the organic deposits and it remains still as a controversial issue with numerous approaches available for varying purposes of classification. Soil from organic deposit refers to a distinct mode of behaviour different than traditional soil mechanics in certain respects. American Society for Testing and Materials considered one of the possible approaches for classifying soils in terms of organic contents (OC) that may be stated as follows (Edil, 1997).

- OC<5%, little effect on behaviour; considered inorganic soil.
- OC in between 6-20%; affects the properties but behaviour is still like mineral soils; organic silts and clays.

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3. Classificatn based on cohesion —

1. cohesionless — gravel sand ⇒ no cohesive force
2. cohesive — clay ⇒ has cohesive force

Table 3.2 Identification of Fine Grained Soil Fractions from Manual Tests

Soil Type	Dry Strength	Dilatancy	Plasticity	Dispersion
Sandy silt	None to very low	Rapid	Weak to friable	30 sec to 60 min
Silt	Very low to low	Rapid	Weak to friable	15 min to 60 min
Clayey silt	Low to medium	Rapid to slow	Medium	15 min to several hours
Sandy clay	Low to high	Slow to none	Medium	30 sec to several hours
Silty clay	Medium to high	Slow to none	Medium	15 min to several hours
Clay	High to very high	None	Tough	Several hours to days
Organic silt	Low to medium	Slow	Weak to friable	15 min to several hours
Organic clay	Medium to very high	None	Tough	Several hours to days

3.2.1 Structure and Fabric of Soil

Soil structure may be defined as the geometric and skeletal arrangement of the particles and inter-particle forces that may act on them. Soil structure includes gradation, arrangement of particles, void ratio, bonding agents, and associated electrical forces. Structure of the soil is the property which produces a response to external changes in the environment, such as loads, water and temperature, and other factors. Structural composition of soil influences engineering properties of soil such as permeability, compressibility and shear strength.

Common name → Soil structure
for clay → Soil fabric
→ macro  → crack on a soil mass
→ micro → crack in the molecules of soil ⇓ needs microscope

Soil fabric is a term used to describe the structure of clays and denotes the geometric arrangement of the mineral particles in a clay mass as observed by optical and electron microscope. The geometric arrangement includes particle spacing and pore size distribution. The following are the common soil structures:

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5 structures ——

- Single grain structure
- Honeycomb structure
- Flocculated structure
- Dispersed structure
- Packet or ped

The first two types of structure are usually common in coarse grained soil, the rest being for the fine grained. The different types of soil structures for coarse and fine grained soils are shown in Figs. 3.9 and 3.10 respectively.

3.4.1 Soil Structure of coarse grained soil

(a) **Single grain structure:** The primary structure of a coarse soil is typically single grained. Individual particles may assume relatively stable or unstable positions according to their mode of deposition. The dense configuration occurs in deposits built in an active water environment. Loose deposits are typically formed in quiet water or may result when a dense deposit is disturbed, by landslides, for example.

(b) **Honeycomb structure:** Very fine sand and silt may assume a honeycomb configuration of very open structure. If the gravitational forces during deposition of these materials are not sufficient to overcome inter particle attractive forces this structure results. The term metastable is sometimes used to describe this condition because of its inherent sensitivity to even the most minor disturbance. Soil deposits of this nature often appear to be firm and strong but become wet and unworkable as the excavation process breaks down the primary structure.

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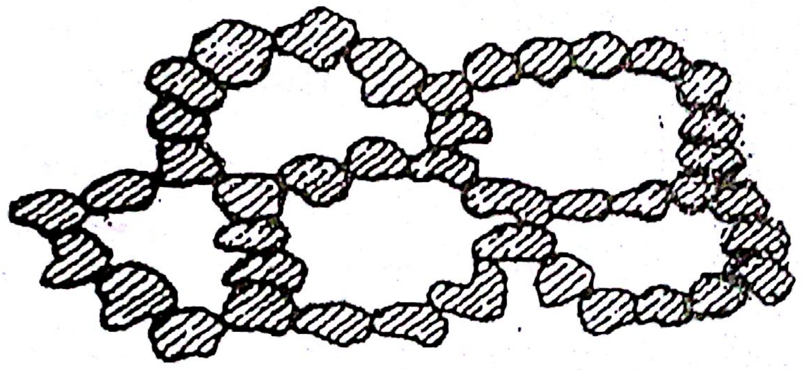
(a) Loose (b) Dense

Single (or) grained



(b) Honeycomb · (Loess soil)

Fig. 3.9 Soil Structures of coarse grained soils; (a) Single grained; (b) Honeycomb (after Schroeder and Dickenson, 1996)

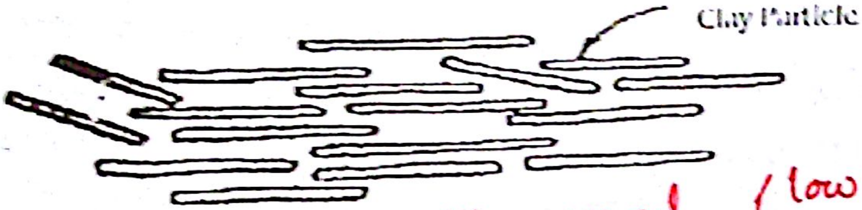
3.4.2 Soil structure of clay soils

Clay soil exhibit a structure very strongly influenced by the chemical environment existing during deposition and by stress history thereafter.

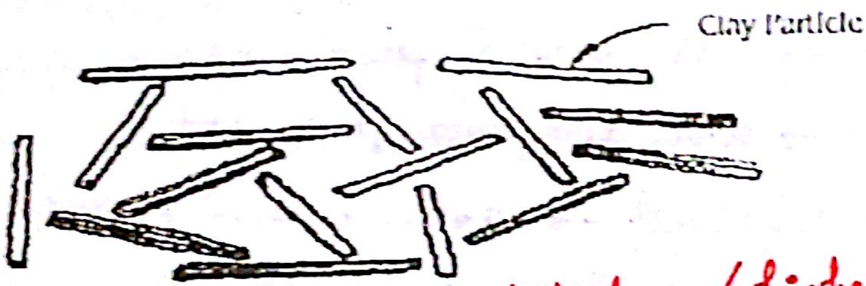
(a) **Dispersed structure:** Clay soil particles deposited as sediment in fresh water repel one another as they settle because of the like electrostatic surface charge they carry. In the resulting sediment the particles align themselves in a face to face arrangement, called dispersed structure, Fig 3.10a, as the repulsive forces from their surface charges come to an equillibrium.

Flocculated structure: If the clay is deposited in salt water, the excess of available free ions present in the water effectively neutralizes the particles' surface charges. While suspended they are, thus, free to attract each other gravitationally and form flocs or aggregates of particles as shown in Fig. 3.10b.

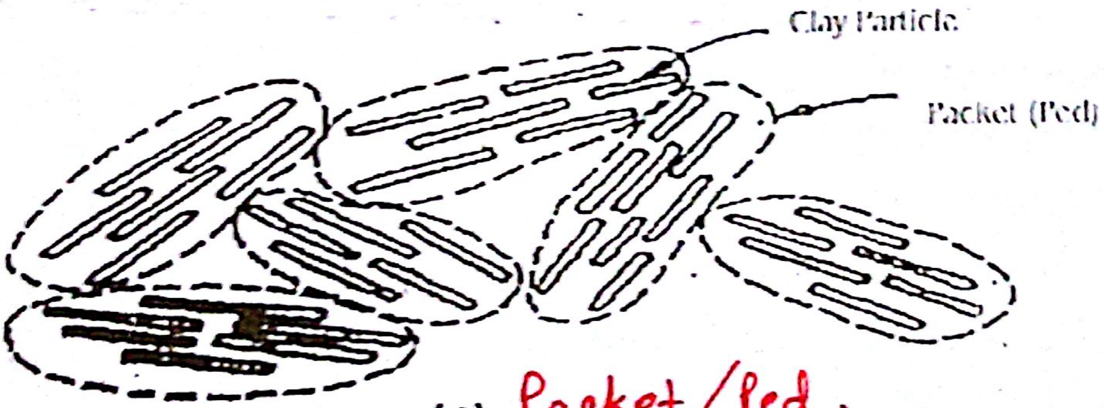
Packets or Peds: Some clay may exist in randomly arranged packets or peds, like those shown in Fig. 3.10c, which individually are made up of highly oriented individual particles.



(a) Dispersed (lower shear strength, deflection small)



(b) Flocculated (higher shear strength) (deflection higher)



(c) Packet/Ped.

3.10 Structures of clay soils; (a) Dispersed; (b) Flocculated and (c) Packet or Ped (after Schroeder and Dickenson, 1996)

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3.4.3 Fabric of Soil

The term fabric is used to define the arrangement of the particles. The term (1) micro fabric refers to those particle arrangements that require an optical microscope for their study. This is similar in sense to structure of soil. Whereas, (2) macro fabric refers to features such as stratification, fissures and voids that can be observed by naked eye. Micro and macro fabrics constitute fabric of soil, the special term used for the clay soil.

3.4.4 Shape of the particles

The behaviour of coarse grained soil is greatly influenced by the shape of the particles. However, despite of its importance, the shape of the particle has not been qualified as the criteria of classification and usually a qualitative description is given on the basis of visual classification. The particle shape can generally be divided into three major categories:

- Bulky → 3 टेर dimens^n prominent
- Flaky → 2 टेर dimension prominent.
- Needle shaped → एकटेर dimens^n prominent

Bulky particles are mostly formed by mechanical weathering of rock and minerals. The shapes of the bulky particles are usually described by the geologist as angular, subangular, subrounded and rounded depending on the roundness of a particle. Typical shapes of the bulky particles are shown in Fig. 3.11.

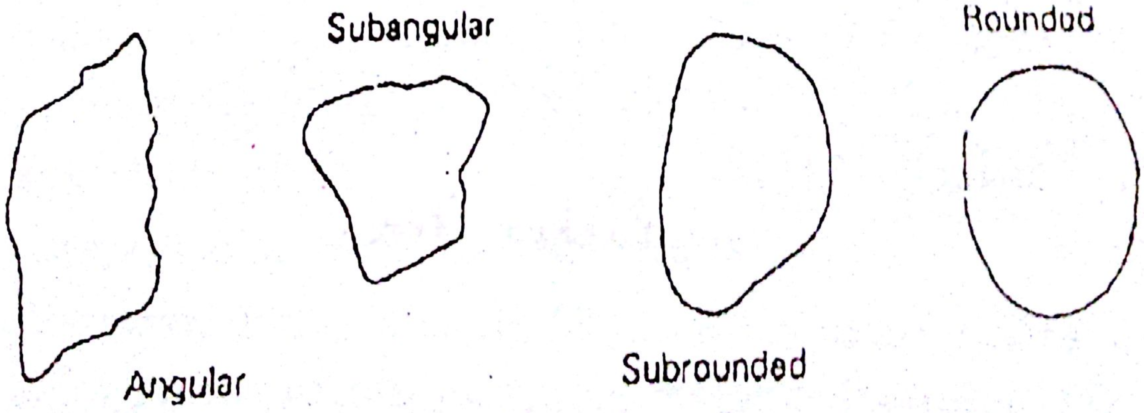

Fig. 3.11 Shapes of coarse grained soil particles (Lancellotta1995)

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The angularity of a bulky particle, A is defined as

Imp. $$A = \frac{\text{Average radius of corners and edges}}{\text{Radius of the maximum inscribed sphere}} \tag{3.1a}$$

The roundness or shericity of a particle is defined as the ratio of minimum radius of the particle edges to the inscribed radius of the entire particle.

Imp. $$S = \frac{D_e}{L_p} \tag{3.1b}$$

Where,

D_e = equivalent diameter of the particle = $\sqrt[3]{\frac{6V}{\pi}}$
V = volume of particle
L_p = length of particle → max^{m} dimens^{n}.

Flaky particles have very low sphericity, usually 0.01 or less. These particles are predominantly clay minerals.

Needle shaped particles are much less common than the other two types of particles. Coral deposits and attapulgate clays are the examples of needle shaped particles.

3.5 Soil Classification and Classification Tests

For engineering purposes, soils are frequently classified into groups according to their suitability of use for a particular purpose. However, as the natural soils are infinitely varied, they do not lend themselves to separation into distinct categories. As a result, various arbitrary systems of classification have been evolved, each with a certain advantages and disadvantages for a particular purpose.

Two most common classification systems are (1) Unified Soil Classification System, used for general engineering purposes, and the (2) AASHTO system, developed by the American Association of

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Q. What is index property ?

Q. What is classificatn test ?

State Highway and Transportation Officials and often used for soils in highway and airport pavement design. Another classification system is the textural classification mainly designed to describe the appearance of the soil depending on the grain size of the soil particles. However, this is no longer in use for engineering purposes.

All these classification systems use the results of some basic tests of soil materials. These tests are known as the classification tests. The numerical results of these classification tests are often termed as index properties of soil, as because these values may be used to indicate the engineering properties of soil. The two most important types of classification tests are:

1. Grain size analysis including sieve and sedimentation analyses to measure the grain sizes. ⇒ for coarse grained

2. Consistency tests or Atterberg's limits tests to measure the soil types. It usually measures the condition or stiffness of soil due to water. ⇒ for fine grained soil

3.5.1 Grain size analysis

Grain sizes in soil samples are found by means of two tests. The (1) sieve analysis is used for coarser materials and (2) hydrometer (sedimentation) test is used for finer soils. If significant quantities of both coarse and fine grained soils are in the sample, the results of both the test will have to combine to get the grain sizes in the sample.

(1) In sieve analysis a set of sieves of various opening sizes is stacked, the sieves of largest opening size being at the top with a lid and smallest at the bottom with a pan, in order of the sizes, Fig. 3.12. Standard sieves are sized according to the space between each pair of mesh. Sizes of the opening are usually expressed in number, meaning that as many number as openings are there per linear inch of the wire mesh. This includes clear opening and the thickness of the wire. As such, as

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⊛ Sedimentation test → pipet test
→ hydrometer test

the number increases, the opening size decreases. A number 4 sieve (opening 4.75 mm) is commonly used the upper most sieve size, whereas number 200 sieve (opening 0.075 mm) is the smallest in the range. There are also some other sizes of sieves beyond these size limits. In the current practice, sieves are usually termed according to the opening in millimetres that correspond to various sieve sizes.

⊛ 10 mins horizontal shaking

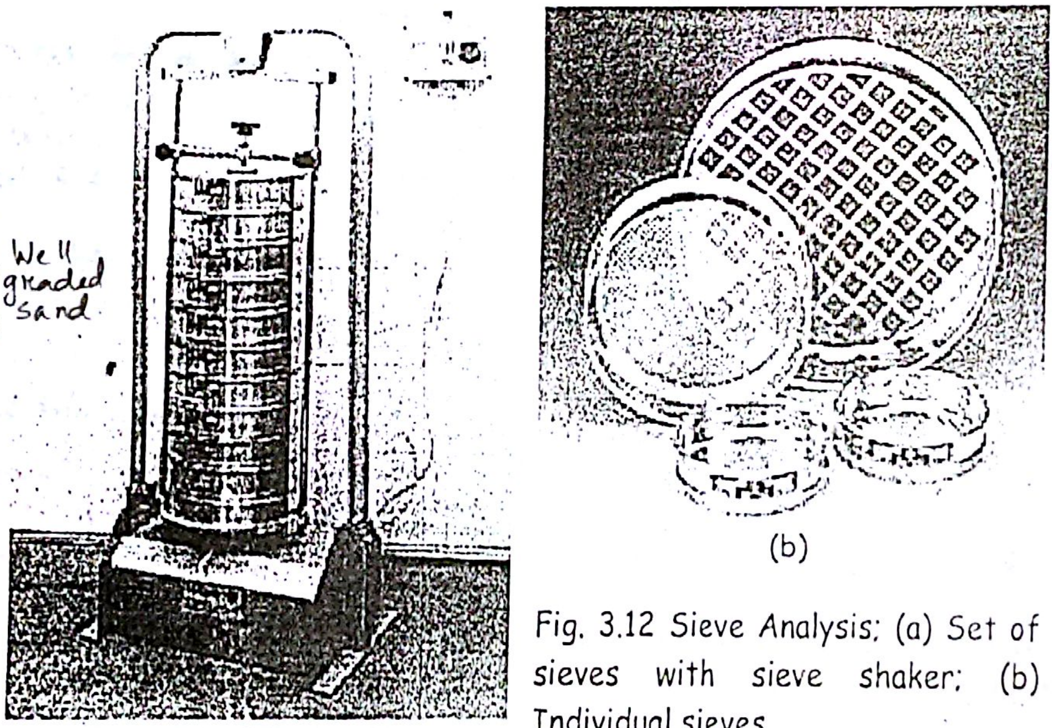
A known weight of dry powdered soil sample is placed on the top sieve and the sieves are shaken to separate the grains according to size. The mass retained in each sieve is then determined. The percent retained on each sieve, cumulative percent retained and hence the percent passing each sieve are calculated. A grain size distribution curve of sieve opening versus percent passing (often called percent finer) is plotted in a semi logarithmic graph paper to classify the soil sample.

⊛ wooden hammer must be used, not steel hammer.

- Classification (gradat)
  1. Well graded
  2. Uniform "
  3. Gap "

  → Poorly graded.

$C_u > 6$
$C_z = 1$ to 3 } Well graded sand.


(b)

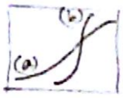
(a)

Fig. 3.12 Sieve Analysis; (a) Set of sieves with sieve shaker; (b) Individual sieves

1. effective grain size (D_{10}) ⇒ It is the size of the particle from which 10% material is finer.

2. D_{30} ⇒ - - - - - 30% material is finer.

3. D_{60} ⇒ - - - 60% " " "



$C_u = \frac{D_{60}}{D_{10}}$ ⇒ uniformity coefficient

$C_z = \frac{D_{30}^2}{D_{60} \times D_{10}}$ → Coefficient of curvature

$C_u = 1$ for uniform graded

C_u ⇒ যত greater curve-এর spread তত বেশি
for curve 'a' → C_u greater
" " 'b' → C_u smaller.

C_z ⇒ curvature কিভাবে change হবে।

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4 sieve-এর উপর ⇒ অল্প retain করলে
100gm sample নিব

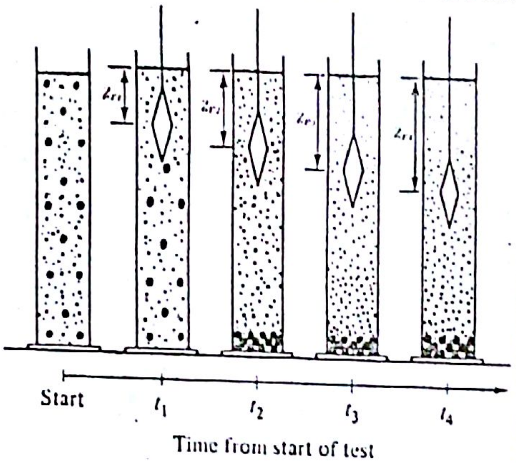
⇒ বেশি retain করলে
500 gm sample নিব।

② Sedimentation analysis is used to find the size of the smaller grains. The most common of sedimentation analysis is hydrometer test. The rate at which particles settle in a fluid media is used as an indicator of their size, in a hydrometer analysis. This is given by Stoke's law which states that particles in a suspension settle out at a rate that varies with their size. A hydrometer is used to measure the density of the water soil suspension at times intervals as the grains settle. The size of the particle that has settled along the centre of the hydrometer bulb can be calculated, and the density indicated the percentage of sample still in the suspension. Fig. 3.13 shows the depth of the centre of hydrometer changing as particles settle out over a period of time.

In the test procedure a known amount of powdered soil sample of finer soils (usually passing no. 200 sieve) is mixed with water of known temperature to form a suspension. The density of the suspension is determined using the hydrometer at different time intervals.

(*)
- $v = \frac{\rho_s - \rho_w}{18\eta} D^2$
- $D = \sqrt{\frac{30\eta}{(G_s - 1)\rho_w}} \sqrt{\frac{L}{t}}$

or, $D\,(mm) = K\sqrt{\frac{L\,(cm)}{t\,(min)}}$



⊛ #200 দিয়ে passing-এর ক্ষেত্রে % retained decimal-এর পর 1/2 ঘর রাখব।
⇓
এর থেকে বড় sieve গুলিতে full no. রাখব।

⊛ Cumilitive % retained —
⇓
যদি ঐ একটামাত্র sieve থাকে তাহলে কতটুকু retain করবে।

- Fig. 3.13 Changes of the centre of hydrometer during sedimentation analysis (after Atkins, 1997)

The following formulas are derived as using Stoke's law to determine the particle diameter and percent finer. The particle diameter, d and percent finer is given by

$$d = \left(\frac{3\mu_w}{\gamma_w(G_s - 1)}\right)^{\frac{1}{2}} \times \left(\frac{Z_r}{t}\right)^{\frac{1}{2}} \qquad (3.2)$$

mixture
Soil = 50 gm
H_2O = certain amount (almost 1000 gm)
Na Hexametaphosphate soln

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Hydrometer → 151 H → gives density reading
→ 152 H ⇒ gives exactly the amount of soil in suspension.
⇓
Suppose reading = 10 gm ⇒ i.e. 10 gm suspend and 10 gm settle down.
So 10gm is finer.

Where,

G_s = specific gravity of soil solids
γ_w = unit weight of water in kN/m^3
μ_w = viscosity of water in N-sec/m^2
Z_r = maximum distance of travel of a soil particle in cm
t = time of fall in min.

The percent of particles finer than the particle diameter given by eq. (3.2) can be calculated as

$$N = \frac{G_s \gamma_w V R_h}{10(G_s - 1)W} \qquad (3.3)$$

Where,

N = percent finer
R_h = hydrometer reading
W = weight of fine soil fraction mixed with water
V = volume of suspension

The units should be consistent in eq. (3.3). Thus for each hydrometer, R_h, a set of values for particle diameter, d and percent finer, N can be obtained. A similar grain size distribution curve of particle diameter versus percent finer may be obtained using these data. A combined grain size distribution curve may also be drawn using the data of sieve analysis and hydrometer analysis. Typical grain size distribution curves for different types of soils are shown in Fig. 3.14. The following terminologies in relation to grain size distribution curve are used to describe and classify the soil.

(ii) **Shape of the curve:** A soil composed of mainly of one size particles is called uniform or poorly graded soil. The curve lies within a narrow range, Fig. 3.14I. When a grain size distribution curve is spread over a wide range, it means to contain all sizes of the grains in the sample. Usually the curve of this type of soil is 'S' shaped and the soil is known as well graded, Fig. 3.14II. The third type of soil is a mixture of only large and small size particles with some of the intermediate sizes missing is gap graded, Fig.3.14III. The soil types are illustrated in Fig. 3.15.

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(ii) **Effective size:** is the grain size corresponding to 10 percent of the passing by weight that is 10 percent of the materials are smaller than effective size. Effective size is termed by D_{10}.

(iii) **Coefficient of curvature, C_z:** The spread of the grain size distribution curve is expressed by this parameter as

$$C_Z = \frac{(D_{30})^2}{D_{60} \times D_{10}} \qquad (3.4)$$

Where, D_{30} is the diameter corresponding to 30 percent finer. A coarse grained soil is considered well graded if the coefficient of curvature, C_Z is between 1 and 3 and C_u is greater than 4 for gravels and 6 for sands.

(iv) **Sorting coefficient of curvature, S_o:** This is another parameter to measure uniformity and is generally used by the geologist. The use of sorting coefficient is not frequent in geotechnical engineering.

$$S_o = \sqrt{\frac{D_{75}}{D_{25}}} \qquad (3.5)$$

The definition of the sizes of the particles is illustrated in Fig. 3.16.

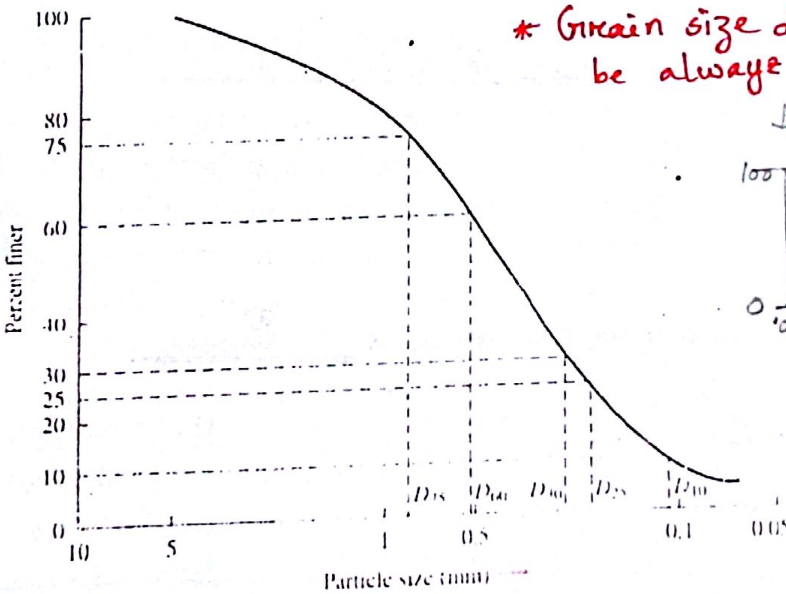
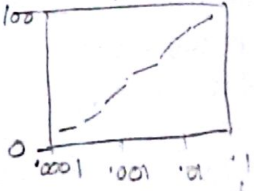


Fig. 3.16 Definitions of the sizes of the particles

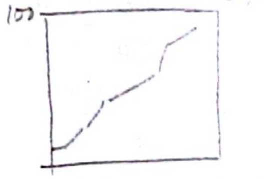
* Grain size distribⁿ curve will be always straight line.

Suppose 15%



Hydrometer

85%



sieve analysis

Combined curve

Hydrometer ৮০% আসে sieve-এ তা 15%. So 0.15 দিয়ে এর সব value গুণ করে then combine curve হবে।

Combined is needed to calculate C_u, C_z, D_{10}, D_{60}, D_{30} etc. . . . ⇐

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3.5.2 Atterberg's limits

The behaviour fine grained cohesive soil depends on its mineral composition, water content, degree of saturation and structure. In particular, the water content has always been considered an important and reliable indication of the behaviour of cohesive soils since the beginning of soil mechanics (Lancellotta, 1995). Swedish soil scientist Atterberg, in the early 1900's, first identified that a gradual decrease in water content of a clay soil slurry causes the soil to pass through four states or conditions (consistency): ① liquid, ② plastic, ③ semi-solid and ④ solid.

① Liquid state is the condition of a fine grained soil at which the soil will flow on its own weight.

Liquid limit

② The plastic state is that condition at which the soil can be remoulded to any shape without any development of cracks.

Plastic limit

③ Semi-solid is the condition at which the soil can remoulded but only with the development of cracks.

Shrinkage limit

④ At solid state the soil cannot be remoulded at all; if done the soil specimen would get broken.

[Atterberg also identified three limiting water contents, in between the soil states, that are commonly known as Atterberg's limits.] However, their potential use in soil mechanics was first indicated by Terzaghi (1925). The upper and lower limits of water content within which a clay element exhibits plastic behaviour are defined as ① liquid and ② plastic limits respectively. Similarly, the limiting water content between semi-soilid and soilid states is the ③ shrinkage limit.

The liquid limit (w_L or LL), is the water content at the transition of liquid state to plastic state, whereby it gains a certain small shearing strength. A study by Youssef et. al. (1965) on clays over the range of 30 - 200% indicates that the range of undrained shear strength at liquid limit is from 1.3 to 2.4 kPa with an average value of 1.7 kPa.

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The plastic limit (w_P or PL) is the minimum moisture content at which the soil can be deformed plastically. As standardized, it can be taken as the smallest water content at which the soil begins to crumble when rolled out into thin threads, approximately 3 mm in diameter. That is at plastic limit the soil must gain some minimum stiffness or strength. According to Skempton and Northey (1953) the shear strength at plastic limit is about 100 times that at liquid limit. As such, 170 kPa shear strength may be considered as the requisite strength of the soil at plastic limit.

The shrinkage limit (w_S or SL) is the smallest water content below which a soil sample will not reduce its volume any, that is, it will not shrink any further with further drying. An illustration of the states and limits of the soils are shown in Fig. 3.17

no shear strength

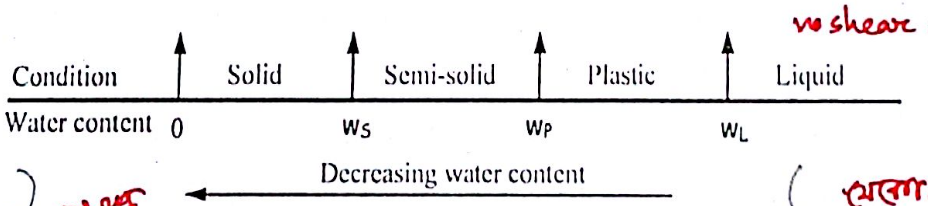

Fig. 3.17 States and limits of fine grained soils

\#4 – 4.75 mm
\#10 – 2 mm
\#40 – .425 mm
\#200 – 0.075 mm

গুরুত্বপূর্ণ

যেকোন limit determination এর জন্য sample কে specially ready করব

1. dried in the air, not in the oven
2. grind it with wooden hammer.

3.5.2.1 Determination of liquid limit

Liquid limit of a soil can be determined by using either of the two methods; Cassagrande's method and Cone penetration method.

(1) Cassagrande's Method

Cassagrande (1932) developed a standard device for the determination of liquid limit suggesting that at the water content of liquid limit a clay soil has a shearing strength of approximately 2.5 kPa. The device consists of a metal cup seated onto a hard rubber base and fixed to a crank shaft arrangement with a handle, so that the cup can be alternately raised and dropped.

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* cup-এ sample paste দিব সমানভাবে so that it will make a horizontal surface.

⊛ blows limit $15 < \text{blows} < 35$

* [illegible] হাতের ছাঁকনি থেকে soil sample নিব

⇓

must ছাকনে থেকে নিতে হবে।

The soil is first dried sufficiently to be broken up by a mortar and pestle. It is then sieved using a 425 μm (No. 40) sieve and materials passing are taken for the limit tests. The soil sample is mixed thoroughly with water into a smooth thick paste, and stored in an air tight contained for 24 hours to allow the water to mix thoroughly with all the soil grains. At the time of testing, the soil is remixed and portion of it placed in the metal cup. The soil paste is leveled off in the cup to give a maximum depth of 8 mm. A groove is formed in the soil along the mid section of the cup, using a standard grooving tool.

The handle is turned at two revolutions per seconds and the number of blows (drops of the cup) is counted until the two parts of the soil come in continuous contact at the bottom of the groove along a distance of about 12 mm. A sample is taken from the closed portion of the groove for moisture content determination. The remaining soil is removed from the cup to mix with the original paste adding further water, and the test is repeated for several times. The number of blows and moisture content in each trial are recorded. It is desirable to have the tests in the blow range of 15 to 35. The liquid limit, according to this method, is defined as the moisture content at which 25 blows are required to close the groove. A schematic diagram of Cassagrande's apparatus is shown in Fig. 3.18.

A semilog plot of the points is obtained with number of blows as abscissa in logarithmic scale, as against moisture content in linear scale. A best fitting straight line is drawn which is termed as flow curve, Fig. 3.19. [The moisture content corresponding to 25 blows is read from the flow curve as the liquid limit of soil. The slope of the flow curve is termed as flow index, I_F and may be expressed as

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[Lambe (1951) suggested that flow curves plotted on a double log scale should give parallel straight lines represented by the following general equation

$$w_L = w\left(\frac{n}{25}\right)^e \qquad (3.6a)$$

Where, w = moisture content in percent at n blows. The value of exponent e is suggested as 0.10 by Kapre and Kulkarni (1972). Equation (3.6) is commonly known as the one point method of determining the liquid limit by Cassagrande's apparatus. Nagaraj and Jayadeva (1981) proposed an alternative relation for one point method of determining liquid limit. According to them,

$$w_L = \frac{w}{1.3215 - 0.23\log n} \qquad (3.6b)$$

Where, w = moisture content in percent at blows n, ranging in between 15 to 35.

Flow index (I_F)

↑

Slope of the flow curve $= \frac{w_1 - w_2}{\log_{10} N_2 - \log_{10} N_1}$

$= \frac{w_1 - w_2}{\log\left(\frac{N_2}{N_1}\right)}$

(semilog)

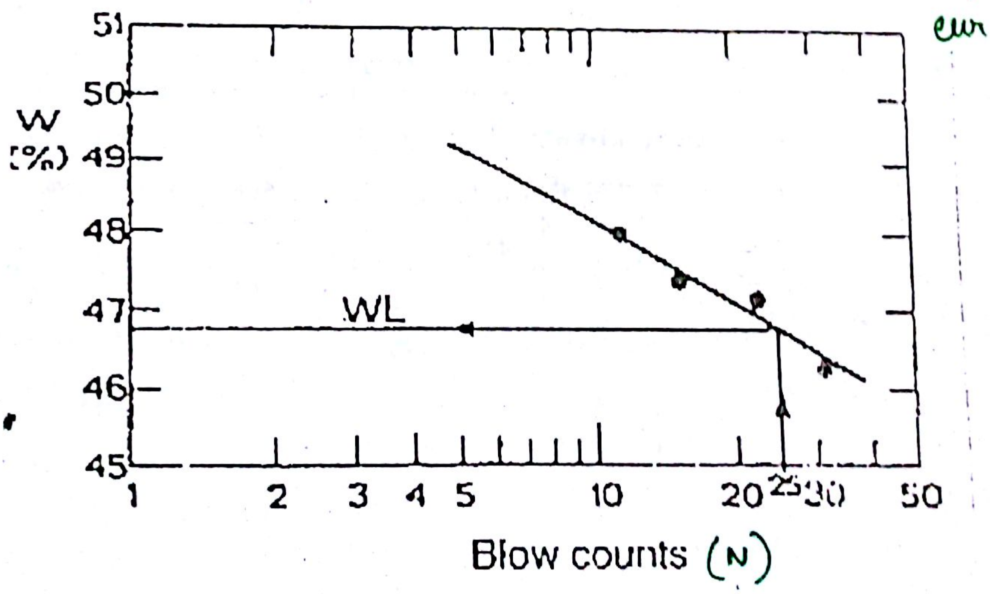

Fig. 3.19 Determination of liquid limit from flow curve (after Lancellotta, 1995)

↓

(<10 হলে decimal-এর পর 1 digit
>10 " whole number হবে।)

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(2) Cone penetrometer method

⇒ cone must be touched at the center.
⇓
we release the cone for 5 seconds

The cone penetrometer apparatus mainly consists of a stainless steel cone 35 mm long with an apex angle of 30° and having a mass including its connecting shaft of 80 gms (0.78 N). The cone is mounted on a stand which will allow it to be dropped and then held in position while its vertical movement is measured. The assembly is mounted on a stand having a base, locking device, dial gauge and other adjustment screws. A cylindrical metal cup, approximately 55 mm in diameter and 40 mm deep is used to contain the test soil sample.

The preparation of soil sample is similar to that of Cassagrande's method. The soil paste is filled in the metal cup and the top surface is struck off level. After placing the cup on the base of the stand (Fig. 3.20), the cone is lowered to just touch the surface of the soil paste. The dial gauge is then set and the reading noted. The cone is released to penetrate the soil paste for 5 seconds and relocked in its new position. The second dial gauge reading is then taken. The difference between the first and the second dial gauge readings gives the amount of cone penetration. A small portion of soil is taken from the cup for moisture content determination.

The soil paste is taken out from the cup, remixed with the original sample paste adding further water. Thus the penetration and moisture content test procedures are repeated several times. A plot is drawn of cone penetration against moisture content. The liquid limit of the soil is taken as the water content corresponding to a penetration of 20 mm. A typical plot is presented in Fig. 3.21. Nagaraj and Jayadeva (1981) also proposed for cone penetration method an expression for determining liquid limit as:

$$w_L = \frac{w}{0.65 + 0.0175D} \qquad (3.7)$$

where,

w : moisture content corresponding to penetration D (ranging in between 16 to 26 mm.)

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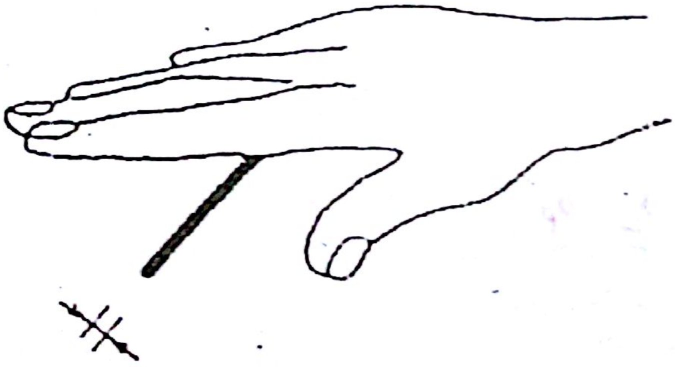
3.5.2.2 Determination of Plastic Limit

The preparation of soil sample is similar to that of liquid limit. Approximately 20 gms of the soil paste is moulded in the hand until it dries sufficiently for slight cracks to appear. The sample is then divided into two approximately equal portions and these divided into four sub-samples. One of the sub-samples is taken and rolled into a ball, and then rolled on a glass sheet to form a thread of soil, Fig. 3.22. The rolling, using the palm and fingers with light pressure, is continued until the diameter of the tread reaches approximately 3 mm. The soil is again reformed into a ball; the action of handling the soil has the effect of drying it. It is then re-rolled on the glass sheet until the thread starts to crumble just as the diameter reached approximately 3 mm. At this stage the thread fragments are taken into an air tight container.

The process is repeated for the other three sub-samples and the thread fragments are collected in the same container, and their combined water content determined. The procedure is also repeated for other half portion of the original sample and moisture content is also determined. The average of the two water contents is reported as the plastic limit, w_p.

(গুঁড়া হবে)

* crumble হবে, not cracked



(3·2 mm)

Fig. 3.22 Determination of plastic limit (after Lancellotta, 1995)

② Wroth and Wood (1978) suggested a method of determining the plastic limit using cone penetration apparatus, as used for liquid limit. This can be done by using a cone of similar geometry but with a mass

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2) Liquidity Index, LI or $I_L = \frac{\overset{\nearrow \text{natural } H_2O \text{ content}}{w_n} - PL}{LL - PL} = \frac{w_n - PL}{PI}$

If $w_n = LL$, $I_L = 1$ and $w_n = PL$ then $I_L = 0$

Q. Whether $I_L < 0$ or $I_L > 1$?? — I_L can be less than zero. → in dry soil condition i.e. $w_n = 0$

Table 3.3 Typical values of liquid and plastic limits(after Atkins, 1997)

Soil	Liquid limit, w_L	Plastic limit, w_P
Silt clay mixture	25-40	20-30
Kaolinite clay	40-70	20-40
Montmorillonite clay	300-600	100-200

I_L can be >1 where $w_n > LL$ in special condit's where soil can't flow.

Plasticity index is sometimes used to describe the soils. Table 3.4 gives a description of soils in terms of plasticity shows its relation with dry strength. The relevant field tests for the soils are also mentioned in the Table.

Table 3.4 Plasticity of soils (after Atkins, 1997)

Plasticity index, I_P	Term used for the soil	Dry strength	Field test
0 - 3	Nonplastic	Very low	Grains fall apart easily
4 - 6	Slightly plastic	Low	Easily crushed by fingers
7 - 15	Moderately plastic	Low to medium	Slight pressure required to crush
16 - 35	Plastic	Medium to high	Difficult to crush
Over 35	Highly plastic	High	Impossible to crush with fingers

↘ lot of deformation ⇒ যত plastic হবে তত deformatn হবে বেশি
⇓
So non-plastic is the best for foundation. e.g. (Sand)

[The current state of a soil, in terms of Atterberg limits, is defined by the liquidity index (I_L or LI) which can be expressed as]

$$I_L = \frac{w_n - w_P}{w_L - w_P} = \frac{w_n - w_P}{I_P} \qquad (3.9)$$

Soft soils have liquidity index near to unity, whereas, stiff clays may have values near to zero. Quick clays have a liquidity index greater than 1.0.

[Toughness index. I_T is defined as the ratio of plasticity index (I_P) to the flow index (I_F). That is,

3) Consistancy Index, I_c or CI = $\frac{LL - w_n}{LL - PL}$ ⇒ It also can be >1 or <0.

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All the indexes are determined the con

$$I_T = \frac{I_P}{I_F} \qquad (3.10)$$

The shear strength of a fine grained soil at water content near to plastic limit is a measure of its toughness. Toughness of two fine grained soils with same plasticity index is inversely proportional to the flow indices. For clay, toughness index is usually less than 3.

The presence of even small amounts of certain clay mineral may have significant effect on the properties of the soil. An indirect method of obtaining information on the type and effect of clay mineral in a soil is to relate plasticity to the quantity of clay size particles. This is defined by the term activity, which is the ratio of plasticity index to the percentage of clay sizes material in the soil. Thus,

$$Activity, A_C = \frac{Plasticity\ Index, I_P}{\%of\ clay\,size\ particles\,(\leq 0.002\,mm)} \qquad (3.11)$$

Activity of a soil qualitatively signifies the behaviour of the soil as active, normal or inactive. For example, clay with kaolinite (a stable clay mineral) will have a low activity value (approximately 0.4). Whereas, clay with montmorillonite (expansive clay mineral) will have a high activity value (may be even 7.0). A relative activity classification is shown in Table 3.4.

Table 3.4 Activity classification of soils

Activity, A_C	Classification	Probable clay mineral
< 0.75	Inactive	Kaolinite
0.75 - 1.25	Normal	Kaolinite and Illite
> 1.25	active	Montmorillonite

1. S.G of soil solid is known

2. Sp Gravity of soil solid is known.

2 procedures

3.5.2.3 Determination of shrinkage limit

⇒ saturated H_2O content of soil at its minm volm in the process of drying.

The shrinkage characteristics of a soil can be estimated in many ways. Of which shrinkage limit and linear shrinkage tests are most common. The apparatus determining the shrinkage limit mainly consists

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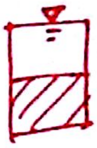
As it is saturated for this test ⇒ there will be only soil solid & water



⇓

After drying, vol^n change

⇓

solid can't change vol^m.

of a stainless steel or porcelain shrinkage dish, a mercury cup and a glass plate with prongs. Fig 3.24 shows a schematic diagram of the apparatus for determination of shrinkage limit.

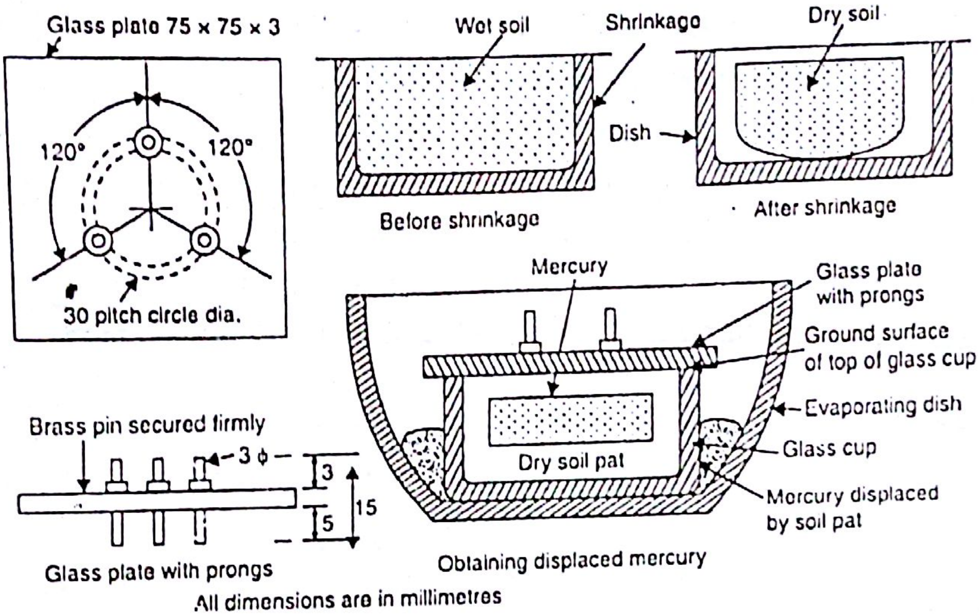
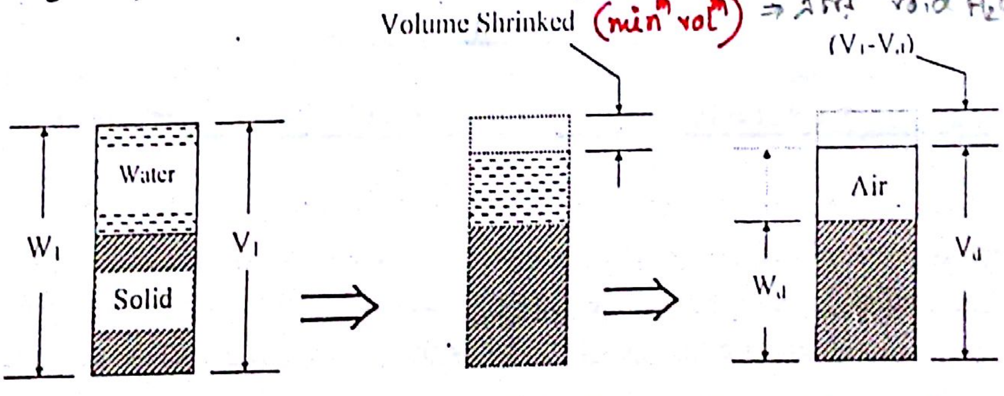


Fig 3.24 Determination shrinkage limit (after Venkatramaiah, 2006)

Soil sample passing 425 μm sieve is mixed with water to form a creamy paste which is filled in shrinkage dish in level with the rim. The weight of the soil paste is taken. It is dried gradually first in air and then in oven to form a soil pat. The dry weight of the soil pat is taken and volume is determined by displacement of mercury using mercury cup and glass plate. Phase diagrams of drying stages are shown in Fig. 3.25.



(a) Soil slurry (b) Soil at shrinkage limit (c) Dry soil pat

(min^n vol^n) ⇒ এখানে void H_2O দ্বারা fill up থাকে এর moisture content (অর্থাৎ shrinkage limit

Fig. 3.25 Phase diagrams of the soil sample at different stages of shrinkage limit test

After shrinking, vol^n can't be determined for irregular shape ⇒ So we have to submerge it in some liquid ⇒ In this case MERCURY is used.

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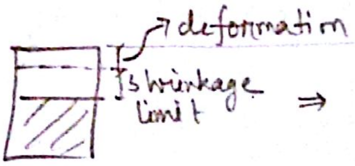
Q. Derive the SL for both the cases?



⇒ So SL ↑, Deformⁿ ↓ & vise versa.

⇓

So as SL greater, as better.

Therefore,

$$Shrinkage Limit, W_L \text{ or } SL = \frac{Weight\ of\ Water\ at\ Shrinkage\ Limit}{Weight\ of\ Soil\ Solid}$$

$$= \frac{\left(V_d - \frac{W_d}{G_s}\right)\gamma_w}{W_d} = \left(\frac{V_d}{W_d} - \frac{1}{G_s}\right)\gamma_w \qquad (3.13)$$

Linear shrinkage test Dia = 1" Length = 140 mm

The linear shrinkage is a measure of degree of decrease of a soil sample in one dimension. The apparatus, Fig. 3.26, mainly consists of a brass mould having a length, L_m, and semi-circular cross section, where the soil paste (slurry) is placed taking care not to entrap air. The surface of the soil paste struck off level. Similar to that of shrinkage limit test, the sample is initially dried in air until it shrunk clear of the mould and then placed it in an oven to complete the drying. After cooling, the length of the sample, L_d is measured and the linear shrinkage is obtained as follows:

$$Linear\ Shrinkage, LS(\%) = \left(1 - \frac{Final\ length}{Initial\ length}\right) \times 100$$

$$= \left(\frac{L_m - L_d}{L_m}\right) \times 100 \qquad (3.14)$$

For soils with very small clay content the liquid and plastic limits may not produce reliable results. An approximation of the plasticity index may be obtained by measuring the linear shrinkage and using the following expression (Whitlow, 1996).

$$I_P = 2.13 LS \qquad (3.15)$$

Shrinkage limit and linear shrinkage, along with the other soil test results, can provide indication of the swelling properties of the soil. However, shrinkage test results are not directly used in any classification scheme. Table 3.5 presents qualitative swelling potential of soils against shrinkage limit, linear shrinkage and other soil properties.

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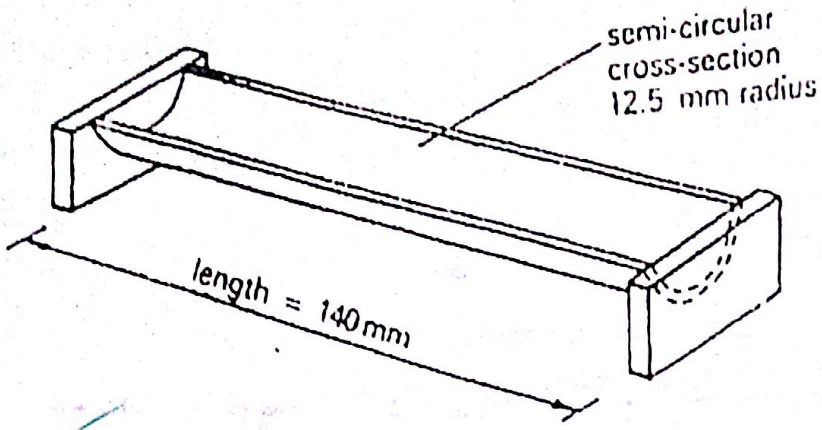

Fig. 3.26 Linear shrinkage apparatus (after Whitlow, 1996)

Table 3.5 Swelling against shrinkage limit and linear shrinkage

Shrinkage limit, w_S	Linear shrinkage, LS	% Colloids (<0.001 μm)	Liquid limit, w_L	Plasticity index, I_P	Swelling potential
> 15	0-5	< 15	< 39	< 18	Low
10 - 16	5-8	13 - 23	39 - 50	15 - 28	Medium
7 - 12		20 - 31	50 - 63	25 - 41	High
< 11	> 8	> 28	>63	> 35	Very High

→ coming in contact with H_2O, how much vol^n changes.

3.5.2.4 Use and significance of consistency limits and indices

(পড়ে নিতে হবে।)

The consistency tests are usually done on remoulded soils. However the shrinkage limit can also be obtained for the undisturbed sample. Since the actual behavior of a soil depends upon its natural structure, the consistency limits do not give complete information about the in-situ soils. They give at best a rough estimate about the behavior or in-situ soils.

Although it is not possible to interpret the consistency limits and other plasticity characteristics in fundamental terms, yet these parameters are of great practical use as index properties of fine-grained soils. The engineering properties of such soils can be empirically related to these index properties as under.

(i) It has been found that both the liquid and plastic limit

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1. Unified Soil Classification System (USCS)
2. AASHTO Soil Classification System.

Unified soil classification system is used to classify the soils that are encountered in foundations design. AASHTO classification is used for the soils in road or airport construction.

3.6.1 Unified soil classification system → Expressed by Two letters

⇓
First noun
Last Adjective
⇓
e.g. Well-graded Gravel (GW) and Highly Plastic Clay (CH)
⇓
Gravel — G
Sand — S
Silt — M
Clay — C

In Unified soil classification system, soil is basically classified into (1) two groups: i. Coarse grained and ii. fine grained. However, two other additional groups are also included though not precise enough in classification criteria; they are organic and peat soils.

The basis of Unified soil classification is that coarse grained soils are classified according to their grain size characteristics, whereas, the fine grained are to their plasticity characteristics.

If a soil mass contains more than 50 percent of the materials coarser than 0.75 µm (that is retained on No. 200 sieve), then the whole of the soil mass is to be known as coarse grained soil. Otherwise, it is fine grained. The coarse grained soils are subdivided into gravel and sand. If 50 percent or more of the coarser portion (retained on No. 200 sieve) retains on No. 4 sieve (opening = 4.75 mm), then the whole of the soil mass has the basic name Gravel. Otherwise, it is sand. To be more precise, either the grain size characteristics or the plasticity properties of the soils passing No. 40 sieve (opening = 0.425 µm) are considered and used as a qualifier.

The fine grained soils are subdivided into silt and clay depending on the plasticity characteristics of the portion passing No. 40 sieve. They are further qualified by their degree of compressibility (plasticity).

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If ≤ 5% passing through # 200 sieve

Clean gravel / elean sand ⇒ then এর আগে Grain size distrib^n curve দেখতে হবে

* If sand ⇒ then & ≥ 12% to determine 2nd letter we should see Plasticity chart i.e. we use only consistency test data only for coarse grained soil.

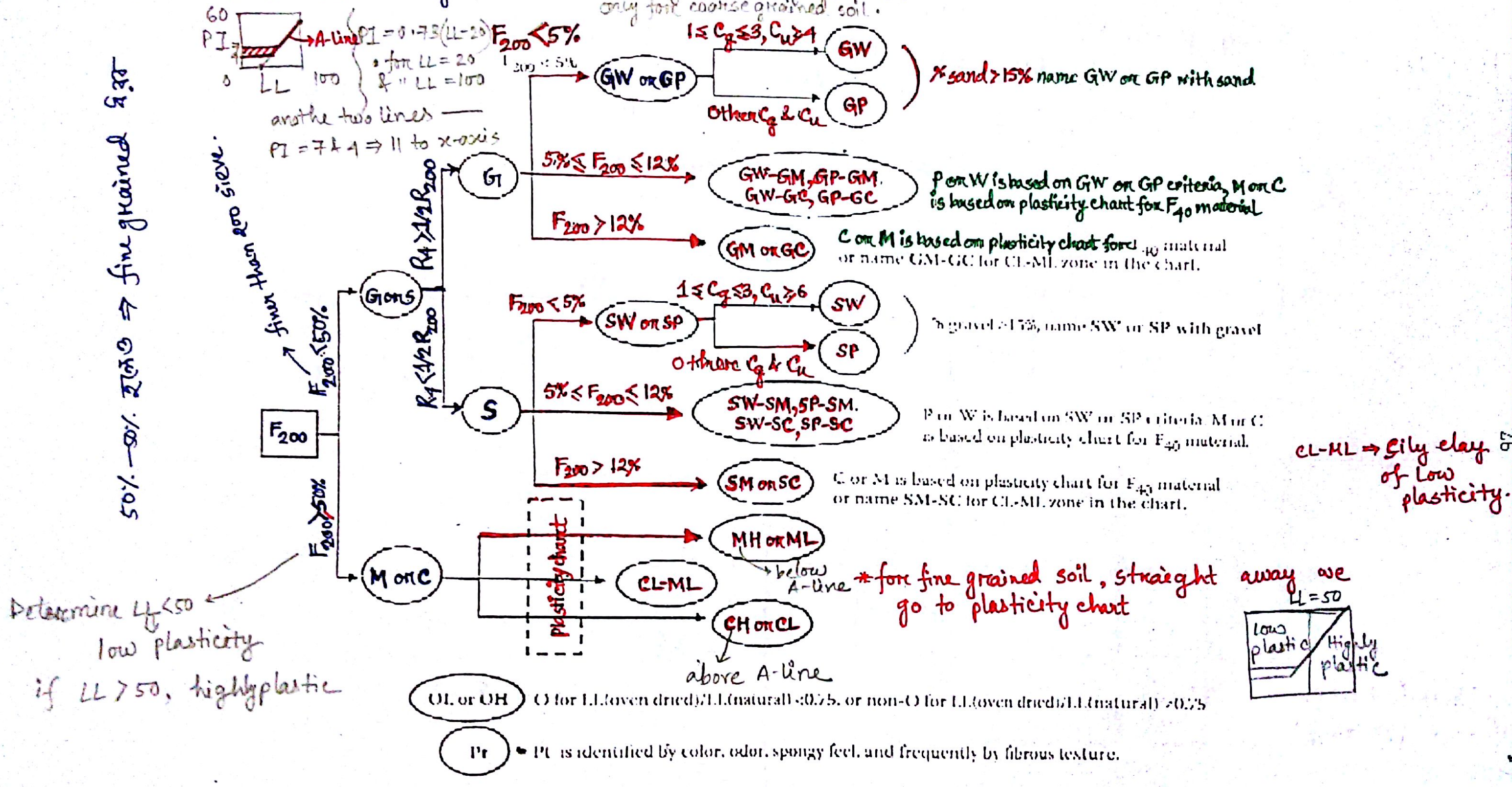

Fig. 3.31 Continued: Unified soil classification system Flowchart (after Ishibashi & Hazarika, 2011).

for $5\% < F_{200} \leq 12\%$ ⇒ we'll go for duel classification ⇒ Priority হবে gradation i.e. GM-GW লিখতে পারব না।

যদি মনে হয় black color soil হয় then দুই ভাবে LL determine করব।

⟹ if $\frac{LL_{ovendry}}{LL_{airdry}} \leq 0.75$ ⟹ It is organic soil. ⟹ Then first letter will be O ⟹ $LL_{airdry} \geq 50 \Rightarrow$ OH

$LL_{airdry} \leq 50 \Rightarrow$ OL

* Loss on ignition : High temp. এ soil কে burn করব at 400-500°C.

LOI ≥ 0.75 ⟹ Pit soil (Pt)

if LOI < 0.75 ⟹ Organic soil

⊛ Pit soil always highly plastic

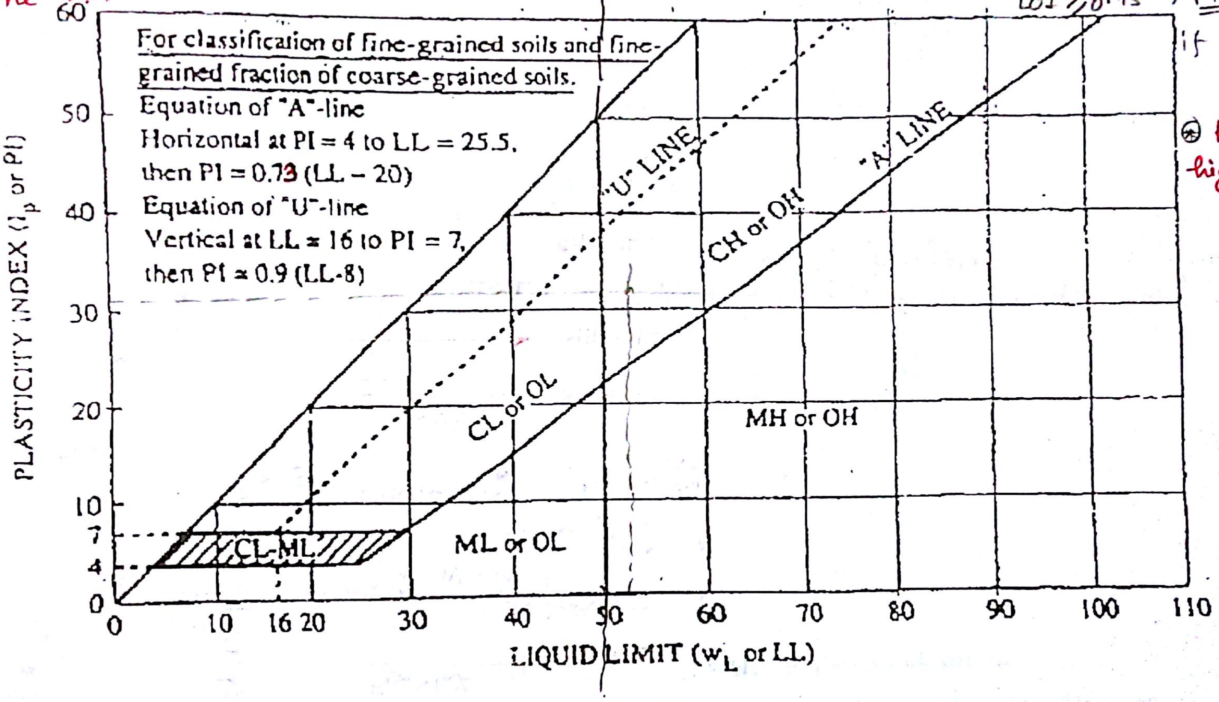

Fig. 3.32 Plasticity chart for Unified soil classification system (Atkins, 1997).

Say LL = 61, PL = 31 ∴ PI = 61 − 31 = 30 ∴ Point ≈ (61,30)

∴ From eqn ⟹ PI = 0.73(61 − 20) = 29.9

∴ It is less than i.e. below the A-line. then SM/GM

above " " " SC/GC.

(This classification is only for design of subgrades.)

㊉ Total 13 types of soil

㊉ $A_1 - A_7 \Rightarrow$ Inorganic soil & $A_8 \Rightarrow$ Organic soil

㊉ Soil-এর number যত বেশি তত waste material for the construction of subgrades.

for pavement, road construction এর জন্য basic or soil [illegible]

General Classification	Granular Material (35% or less passing No. 200 sieve)							Silt Clay Materials (More than 35% passing No. 200 Sieve)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analysis: Percent Passing											
No. 10	50 max	---	---	---	---	---	---	---	---	---	---
No. 40	30 max	50 max	51 min	---	---	---	---	---	---	---	---
No. 200 max 35% passing	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No. 40											
Liquid Limit (LL)	---		---	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min*
Plasticity Index (PI)	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min*
Usual types of significant constituent materials	Stone Fragments: gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General Rating as Subgrade	Excellent to good							Fair to poor			

এর fine grained portion soil এর property A_4 এর মত

Similar meaning

(এইগুলো data থাকলে AASTHO classification করা যাবে।)

- *Plasticity index of A-7-5 subgroup is equal to or less than L.L. minus 30.
- *Plasticity index of A-7-6 subgroup is greater than L.L. minus 30.

Fig. 3.34 AASHTO soil classification system (after Atkins, 1997)

㊉ Suppose Given $\boxed{\#200 - 30}$ ⇒ This means granular soil

#10 – 68

#40 – 50

LL = 55

PI = 35

A-7 হলে, whether it is A-7-5/A-7-6.

প্রথমে #200 দিয়ে granular / silt or clay বের হবে। Then other subgroup-এ যাবো

- Smaller GI ⇒ Better soil
  Higher GI ⇒ lower quality

The group index is used to further evaluate soils within a group. The group index is based on the service performance of many soils, especially while used as pavement subgrades. Group index is a function of liquid limit and plasticity index, and given by

Group Index, ⇒ also whole number, can't be −ve; if −ve, then it is considered as ≈ 0.

[illegible] $$GI = (F - 35)[0.2 + 0.005(w_L - 40)] + \underbrace{0.01(F - 15)(I_p - 10)}_{PGI \Rightarrow \text{ for A-2-6 \& A-2-7 soil}} \qquad (3.16)$$

Where, Normally GI = 0 − 20

F = percent passing No. 200 sieve (i.e. finer than 0.075 μm)
w_L = liquid limit (%)
I_P = plasticity index (%)

If #200 > 35 ⇒ then the Plasticity chart is used directly.

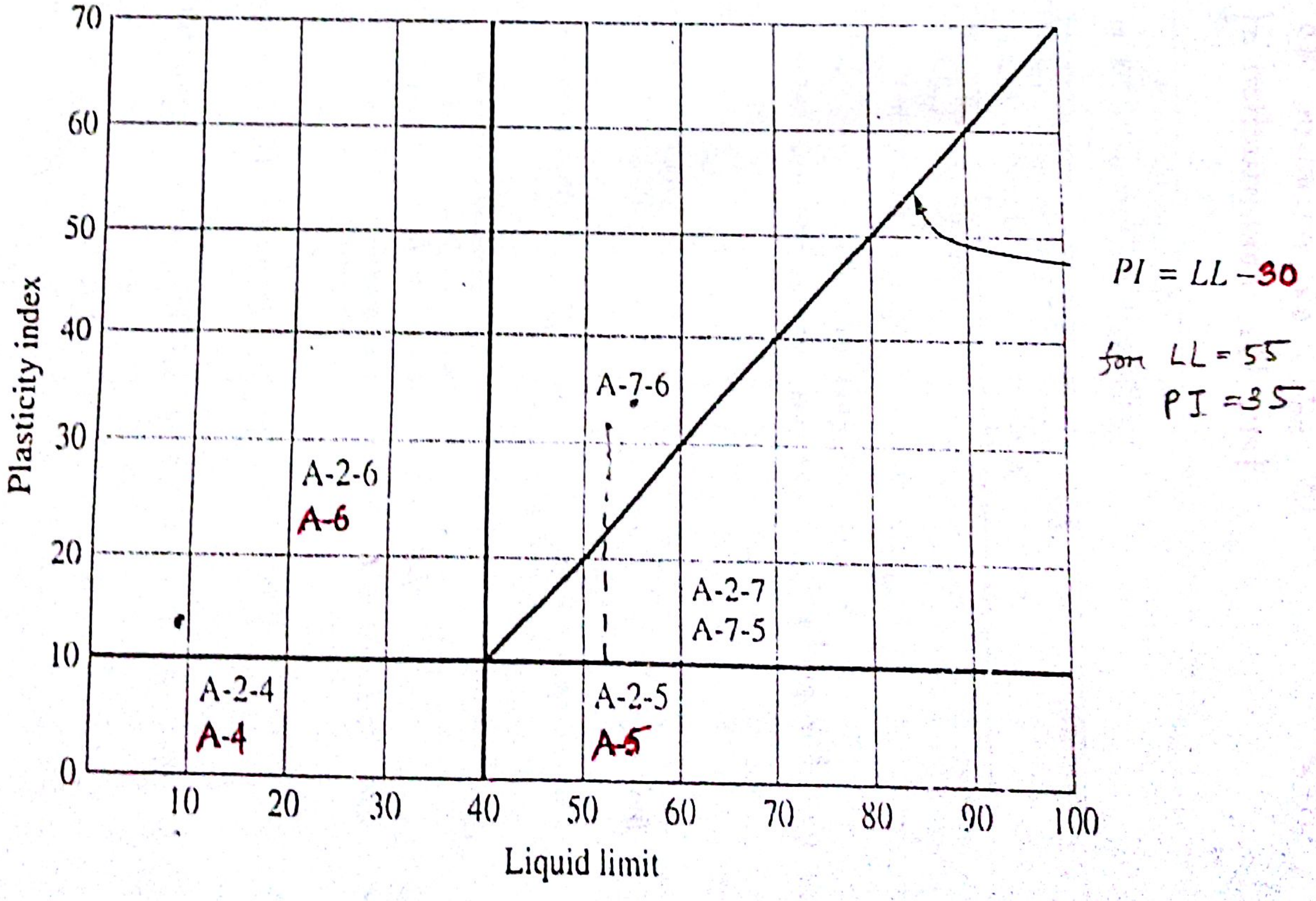

Fig. 3.35 Plasticity chart for AASHTO soil classification (after Das, 2004)

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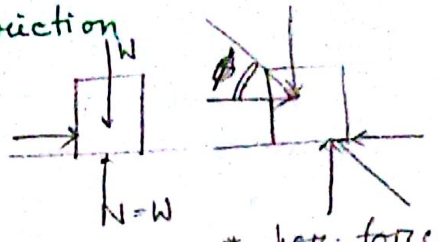
Chapter - 4

* Friction



CT → Chapter 3

1. Class note
2. স্কেল
3. Calculator

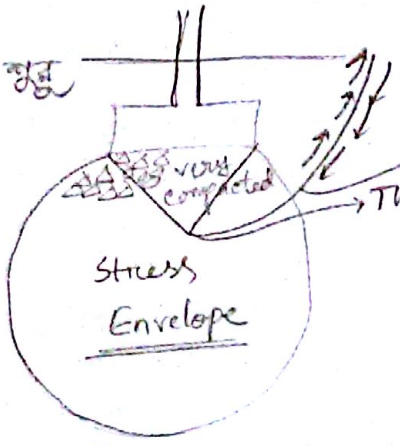
* hor. force বাড়ালে θ will change
⇓
It can be increased upto the movement
⇓
At the instant moves ⇒ Impending Motion. (IM)
⇓
friction angle depends

V. Imp.

* Angle of obliquity ⇒ the angles formed until the movement is caused or IM.

* Angle of friction
⇓
(যখনই movement শুরু হবে।)

Soil ⇒ semi infinite mass.



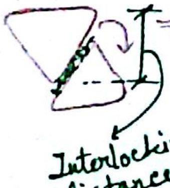
→ But this path is not like this uniform in practical.

→ This wedge will try to penetrate & it will displace the soil.
⇓
So the movement tendency is ↑ cz left / right - a semiinfinite type of soil.

⇓

practically failure surface will be a wave cz soil particles can not be broken.

(*)



⇒ It will try to overtake. Then the factors will be —

3 factors affecting shear strength

1. Very small particles carry charges. যদি H_2O/ধাতু cation থাকে, there will be bonding ⇒ cohesion. cohesion overcome না করলে it can't slip over.
2. Friction force ⇒ $F = N\mu = N\tan\phi$
   ↳ friction angle.
3. Magnitude of Interlocking — large par; Interlocking tough
   small " , " easy.
   ⇓
   or distance বেশি হবে to slip over.

(*) 3 things are needed —

1. Cohesion
2. Friction
3. Mohr's Circle Diagram

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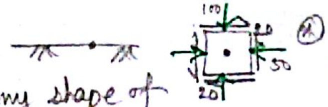

⊛ Vertical stress দিলেও হবে normal force induce হবে। ⇒ Then Mohr Circle.

⊛ for Mohr circle any shape of element can be considered ⇒ for simplicity square is there

7.3.3 Stresses on planes passing through a point and Mohr's circle

According to basic principles of mechanics, in general, both normal and shear stresses act on planes passing through a point that has been subjected to external loading. These stresses may be represented graphically by an extremely useful device known as Mohr's circle of stress. The device is based on a unique point on a circle called the pole or origin of planes. This point has such a useful property that " any straight line drawn through this point (pole) will intersect the circle at a point which represents the state of stress on a plane inclined at the same the orientation in the space of the line." According to this Pole method, atleast two lines from known points of stresses on Mohr's circle parallel to the corresponding planes on which they act are drawn to intersect a point on the circle which is the pole. Once the pole is known the stresses on any plane can be found simply by drawing a line from the pole parallel to that plane; the coordinates of the point of intersection with the Mohr's circle determine the stresses on that plane. Mohr's circle in soil mechanics follows certain sign convention for simplicity. ⊛ σ & τ scale should be same

v.v. imp.
- The compressive stress is considered as positive
- The counter-clockwise shear stress is positive

imp.

as for soil most of the stresses are comp. & shear is counter clockwise

The horizontal and vertical coordinates represents normal and shear stresses respectively. For example, in Fig. 7.3a the normal and shear stresses are acting on the planes of a recangular element. Let us assume that $\sigma_A > \sigma_B$. In Fig. 7.3b the method to determine pole is presented. A line is drawn from the known point $P_A(\sigma_A, \tau)$ of Mohr's circle parallel to the plane A (Fig. 7.3a) on which these stresses act. Similarly another line from the other known point $P_B(\sigma_B, \tau)$ is drawn parallel to their plane B on which the stresses of coordinate point P_B act. It is a geometrical truth that these two lines will intersect at a point P on the circumference of the circleas these two lines are perpendicular to each other and line P_AP_B is the diameter of the Mohr's circle. Now, let us consider a plane on the element (Fig. 7.3a) making an angle α with the horizontal. If a line is drawn parallel to this plane from the Pole point P to intersect the circumference of the Mohr's circle a point P_α , the coordinates of P_α (say $\sigma_\alpha, \tau_\alpha$) will represent the stresses acing on the plane in question.

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The stress τ_f at failure (rupture) plane is called the shear strength of the material. σ_f is the normal stress on the failure plane. Fig. 7.4 shows the relationship as expressed by Eq. (7.2) and the element at failure with principal stresses causing the failure.

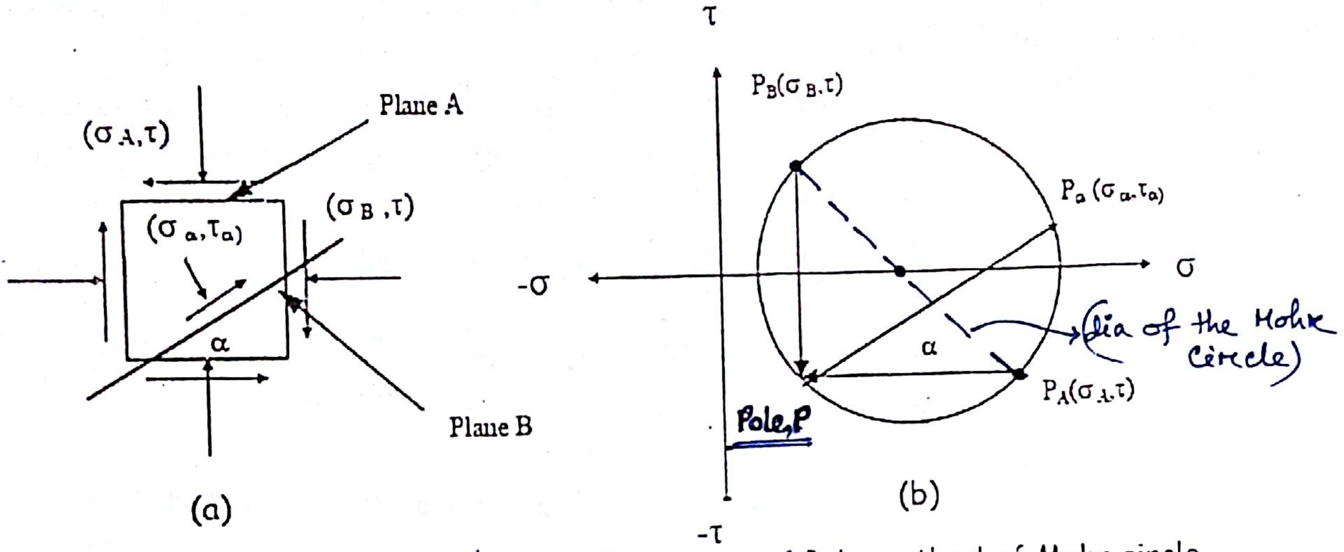
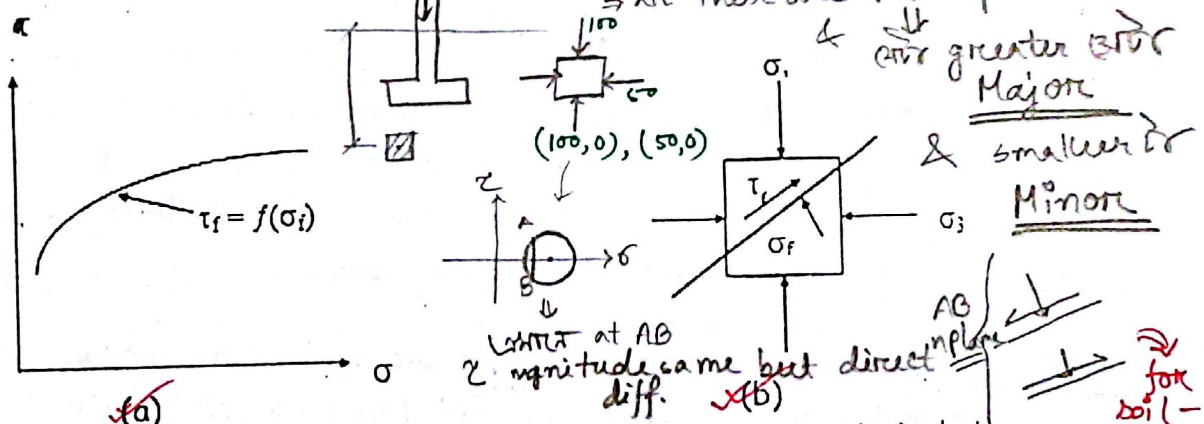


Fig. 7.3 (a) Stresses on an element; (b) Illustration of Pole method of Mohr circle.



→ All these are Principle stresses & এদের greater টা Major & smaller টা Minor

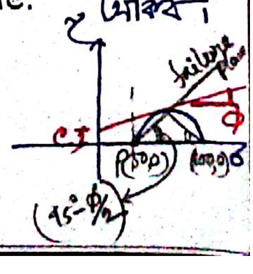
ওখানে at AB τ magnitude same but direct diff.

Fig. 7.4 (a) Mohr's failure criteria; (b) Element at failure showing principal stresses

for soil-n এই direct change doesn't matter much.

Thus, if the principal stresses at failure can be estimated, a Mohr's circle can be constructed to represent this state of stress. Similarly, several tests to failure at various combination of principal strsses would lead to construct several Mohr's circle. Such a series is plotted in Fig. 7.5. Since the Mohr's circles are determined at failure, it is possible to construct limiting or failure envelope of the shear stress. This envelope is called the Mohr's failure envelope which expresses the relationship between τ_f and σ_f as represented by Eq. (7.2) and Fig. 7.4(a). It is to be taken in to note that the failure envelope defined by Mohr (Eq. 7.3a) is a curved line.

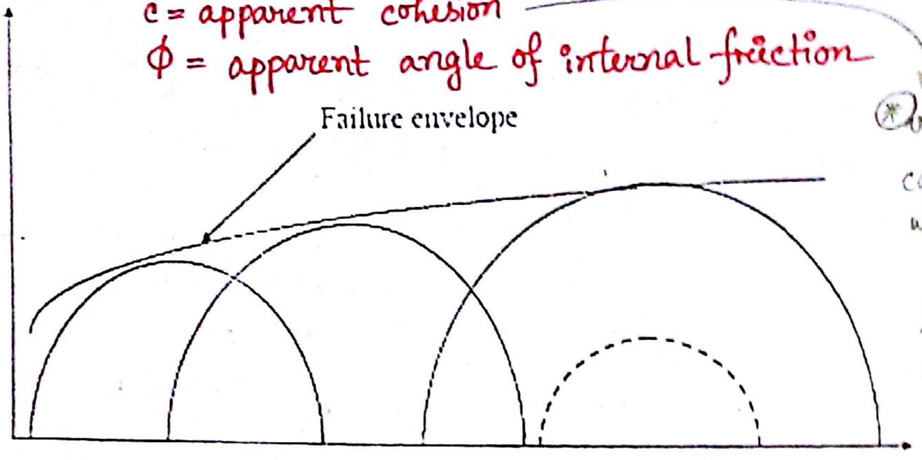
So total Mohr circle আঁকার দরকার নেই। only half আঁকব।



- Shear strength parameters —

c = apparent cohesion

φ = apparent angle of internal friction



(*) Dry soil (clay) এতে cohesion কম। but water add করলে cohesion বাড়ে & so it can be molded into any shape.
⇓
So apparent cohesion বলার হয় ct H₂O হলে আপাত cohesion বাড়ে, water বেশি আপাত cohesion বেশি

It is co

Fig. 7.5 Mohr circle at failure and Mohr's failure envelope

Mohr circle lying below the envelope, shown as a dotted circle in Fig. 7.5, represents a stable condition and the circle going above the failure envelope cannot exists. Failure occures only when the combination of normal and shear stresses is such that the Mohr's circle is tangent to the Mohr's failure envelope. If this envelope is unique for a given material, then the point of tangency of the Mohr failure envelope defines the condition on the failure plane. Using the Pole method, the angle of friction plane from the point of tangency of the Mohr corcle and the Mohr failure envelope. That is, Mohr's failure hypothesis states that the point of tangency of the Mohr failure envelope with the Mohr circle at failure determines the inclination of the failure plane.

Coulomb (1776) was interested in the sliding friction charactersistics of different materials and observed that there exists both normal stress independent and dependent components of shear strength. The normal stress dependent component is similar to sliding friction in solid and that he termed as angle of internal friction, φ. The other component is related to intrinsic cohesion of the material, c. Coulomb gives the following linear equation for shear strength of soil involving parameters φ and c.

$$\tau_f = c + \sigma \tan\varphi \qquad (7.3)$$

Eq. (7.3) is presented in terms of total stress, σ on the failure plane. The equation can be rewritten for effective stresses as follows.

$$\tau_f = c' + (\sigma - u)\tan\varphi = c' + \sigma' \tan\varphi' \qquad (7.4)$$

where, c′ and φ′ are called effective stress parameters as against total stress parameters c and φ respectively. u is the pore water pressure.

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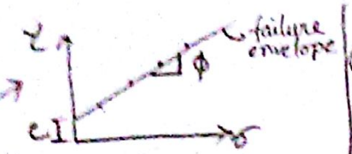
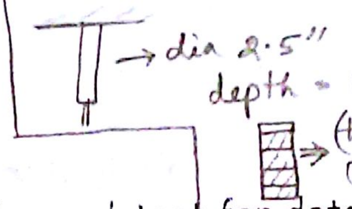


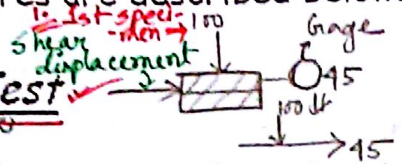
⊛ We will collect soil sample from bore hole.

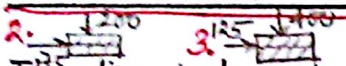
widely been used through out the world for their simplicity and inexpensiveness. However, engineering judgement and experience play the most important role to correlate the results which depends on the type of the soil and also on the other factors. The common laboratory tests and their salient features are described below.



(top, middle & bottom থেকে identical soil নিয়ে sample বানাবো।)

Direct Shear Test





The direct shear test is a simple and widely used test for determing the shear strength of soil. The direct shear apparatus is essentially a rectangular or circular box with separated lower and upper halves. A schematic diagram of direct shear apparatus is shown in Fig. 7.6. The lower half of the box is fixed to a frame whereas, the upper section is capable of moving horizontally relative to the lower one. The soil sample ois placed in the box, with approximately half of the sample within either section. The sample size is usually **60 mm square or 75 mm circular** having a **thickness in between 20 mm to 25 mm.** In case of cohesive soil prism of soil is eithet prepared or taken from the undisturbed sample using a cutter of similar dimension of shear box. Whereas, for cohesionless soil, the specimen has to be prepared in the box itself at the required void ratio. Porous disc may be placed on top and bottom of the specimen to facilitate the desired drainage condion, though seems very difficult. A normal load is applied to the plane of shear via a loading plate placed over the sample (Fig. 7.6). The upper half of the shear box is then moved laterally forcing the sample to shear across the plane between the two halves of the box by controlling rates of strain or stress.

In strain controlled apparatus, the shearing deformation is continuously applied at a constant speed and shear force is measured by means of a proving ring or a load transducer. Such a test can be continued even after the failure of the specimen. **In stress controlled apparatus, the magnitude of the shear stress is increased uniformly or in increments.** In case of load increment, each increment is applied and held constant until the shearing deformation ceases. A stress contrilled test can not be continued as soon as the specimen fails and as such the strain controlled preferred. Not only that a mechanically operated strain contrilled apparatus is simplest to devise.

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In direct shear test the test procedure is repeated using **atleast three identical specimen** with variable normal stresses. It is sometimes convenient to interpret the results, if the normal stresses are so choosen that one is closer to the existing effective overburden and the one of each of the other two on the either side of the overburden. Usually, a record of magnitude of the shearing force and the shear displacement is maintained to obtain the peak shear stress (failure stress) from the shear stress-strain plot for a particular normal load. Changes in sample thickness that occurs during shearing are also recorded so that the volume change versus shearing stress or shearing strain can be studied.

⊛ [illegible] soil may be sometimes normal & sometimes over consolidated.

⊛ shear box dia = 2·5"

Soil sample ⇒ thickness 1" & dia 2·5"

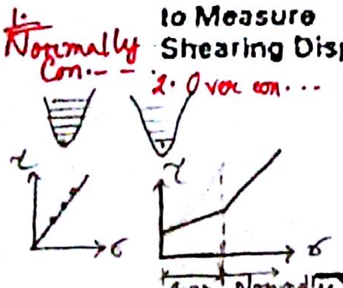
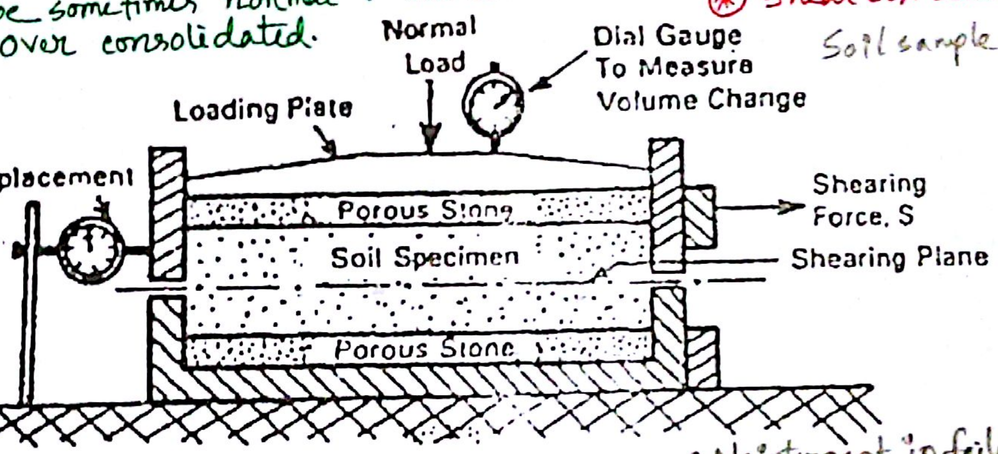


Fig: failure envelops.



• No intercept in failure envelope i.e. starts from origin.

Fig. 7.6 A schematic diagram of a direct shear apparatus

⊛ 2 types of soil — 1. Normally Consolidated Soil → present load is max^m.
2. Over consolidated "
• failure envelope-এ intercept আসবে।

Test results

Typical shear stress displacement and volume change displacement curves for a single specimen (one normal load) are shown in Fig. 7.7. The slope of the stress strain curve is known as modulus of elasticity or tangent modulus of elasticity of soil, E_S. This modulus varies directly with the stiffness of a soil; the more its value, the more is the stiffness and strength. The peak value of shear stress or the maximum value at a relatively higher shear deformation (normally 20 per cent) gives shear strength of the soil at a particular normal load. Normal stress and shear stress are thus estimated and plotted in a normal stress shear stress curve as shown in Fig. 7.7b. A best possible straight line is fitted to the observed point to represent the failure envelope. The shear strength parameters c' and φ' is thus obtained from the plot. It is to be noted that usually we get the drained strength parameters from direct shear test.

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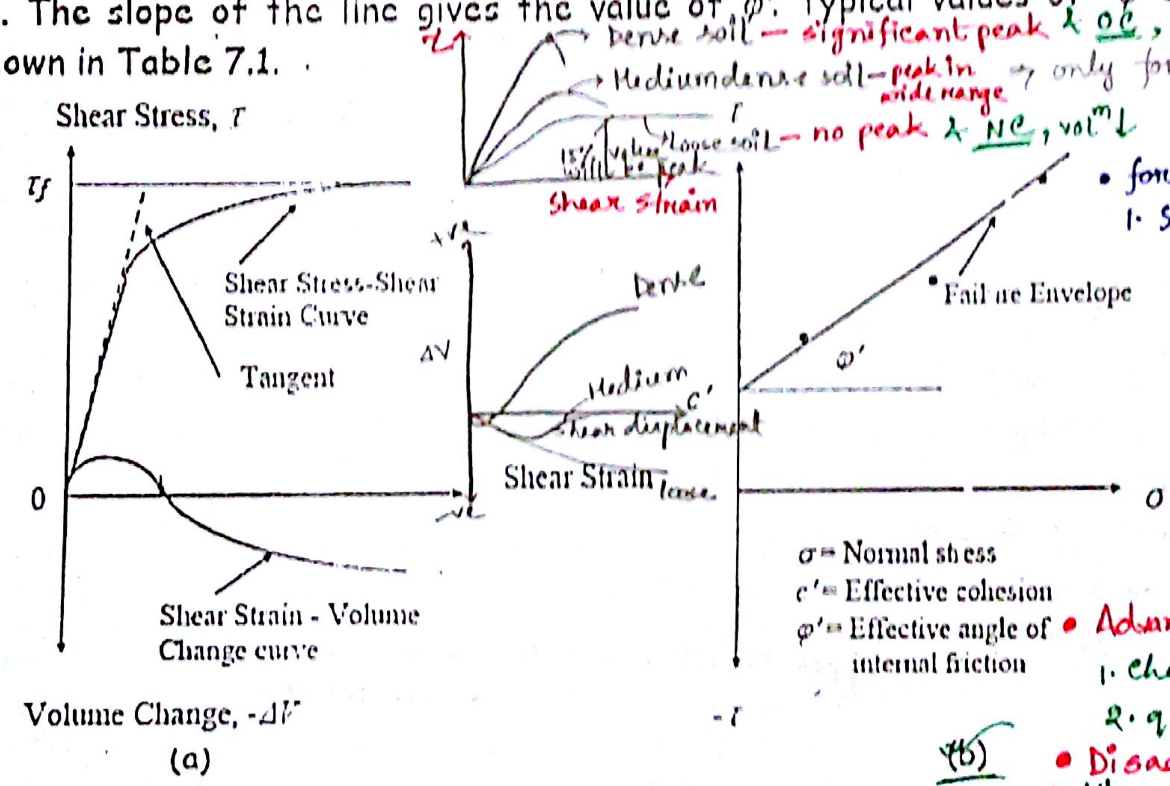
For granular soil as the cohesion is zero the available strength parameter would be only φ'.

The effective angle of friction for a particular density is obtained by plotting the maximum value of shear stress τ_f against the effective normal stress' σ' (Fig. 7.7b). Atleast three tests are carried out at different normal stresses and a straight line passing through the origin and the respective points defines the failure envelope for sand soil (as c= 0). The slope of the line gives the value of φ'. Typical values of φ' are shown in Table 7.1.



- for clay—
  1. Stiff & 2. soft.

- Advantage—
  1. cheap
  2. quick
- Disadvantages—
  1. We are not giving the soil to fail in the weakest plane.

Fig. 7.7 Direct shear test results: (a) Shear stress -shear strain-volume change diagram; (b) Failure envelope and strength parameters

Table 7.1 Typical values of effective angle of internal friction for granular soil (after Terzaghi and Peck, 1967)

Soil type	Effective angle of internal friction, φ' (degrees)	
	Loose	Dense
Nonplastic silt	27-30	30-34
Silty sand	27-33	30-35
Uniform sand	28	34
Well gradred sand	33	45
Sandy gravel	35	50

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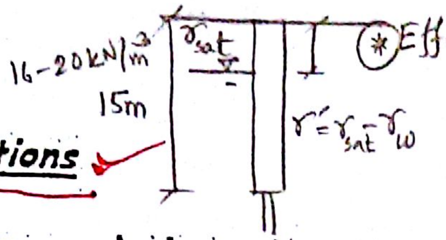

(*) Effective overburden pressure, P

$= 5\times 20 + 10(20-9.81) = 100+100 = 200\ kN/m^2$

⇓

Test-এ ৩টার load দেয়া হয় P-এর সমান, একটার half, আরেকটার দ্বিগুণ

⇓

তাই P must be known

Other observations

The design of normal shear box does not allow control of drainage of the sample. As sands and gravels are free draining materials and as such they will shear under fully drained conditions. For clay deposits, however, depending on the rate at which the soil mass is stressed, a soil element in the field may fail either completely undrained (no dissipation of excess pore water pressure), partially drained (some dissipation of excess pore water pressure) or fully drained (complete dissipation of excess pore water pressure). Although an attempt could be made to measure the undrained strength by shearing a sample rapidly in a few minutes, beacause there is no control over the drainage there would obviously be a degree of uncertainty as to whether this represented the true value of the undrained strength. For this reason the undrained shear strength of a clay soil is usually measured by the more sophisticated triaxial test. The direct shear test may, however, be used to measure the drained strength of clay soils by first consolidating the sample fully under the applied normal load and then shearing the sample at a sufficiently slow rate to ensure the fuul dissipation of pore water pressure generated by the applied shear stress. Hence, for drained clays and sands the effective stress normal to the shear plane is given by [illegible] $= N/A$ and the shear stress $\tau = S/A$, where N and S are applied normal and shear force repectively on the plan area of shear box, A.

A typical stress strain volume change relationships obtained for drained tests on loose and dense sand is shown in Fig. [illegible] The shearing in a loose sand helps to rearrange the particles to fill in the voids thus causing a denser packing and hence a decrease in volume. At larger strains of approximately 20 per cent the sample shears at constant volume under a constant value of shear stress. At this large strains the tendency for a volume increase as some particles move up and over each other is foiled by adjacent particles moving down into the voids [illegible] created, reslting in zero net volume change. This condition of s[illegible] known as critical void ratio state. For a dense sand, the interlocking particles have to move apart in order to allow relative movement or shear between the particles to occur. Thus the sample expands during shear and this phenomenon is known as dilatancy. The consequent result is [illegible] against the confing

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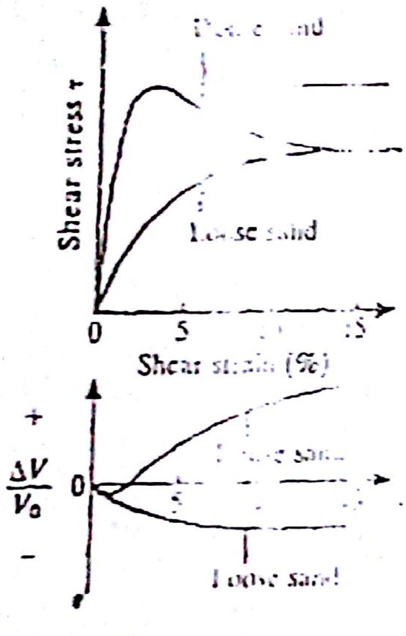
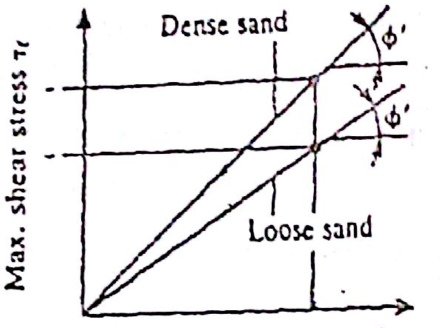
pressure. The peak shear stress therefore occurs at that particular value of shear strain at which dilatancy rate is maximum. With increasing shear strain, the dilatancy rate decreases as the sample approaches to a constant void ratio and shear stress decreases to a residual value. For the same confining pressure, the residual shear stress of a dense sample is equal to the maximum shear stress of a loose sample. Typical failure strains for loose sands are around 12-16 percent and that for dense sand are 2-4 percent.

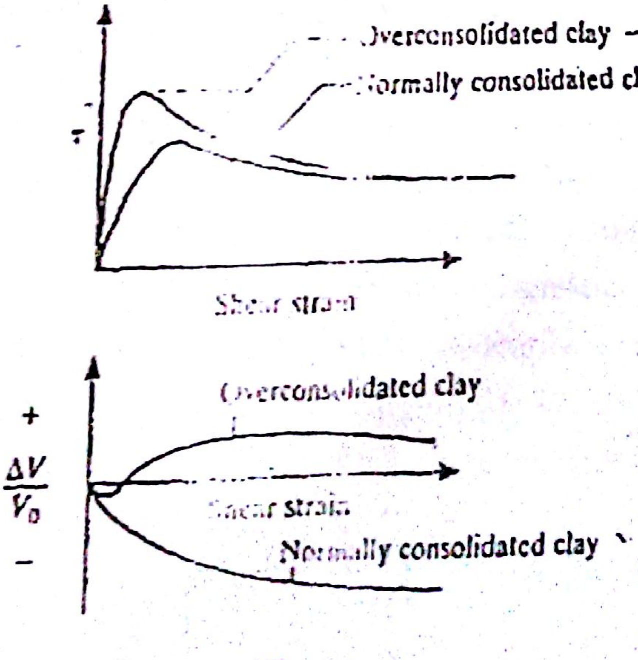


(b)

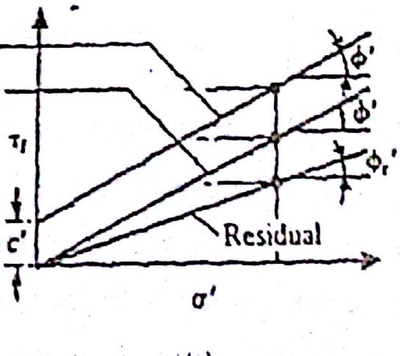


(c)

● In direct shear test disadvatages—
1. Predertermined failure plane
2. We couldn't know the dgree of saturat
3. but we are not considering inclined load.



(d)



(e)

Fig. 2.8 (b) Stress-strain-volume change relationships for drained tests on sands; (c) Failure envelopes for drained tests on sands; (d) Stress-strain-volume change relationships for drained tests on clays; (e) Failure envelopes for drained tests on clays.

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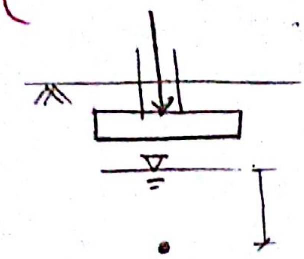
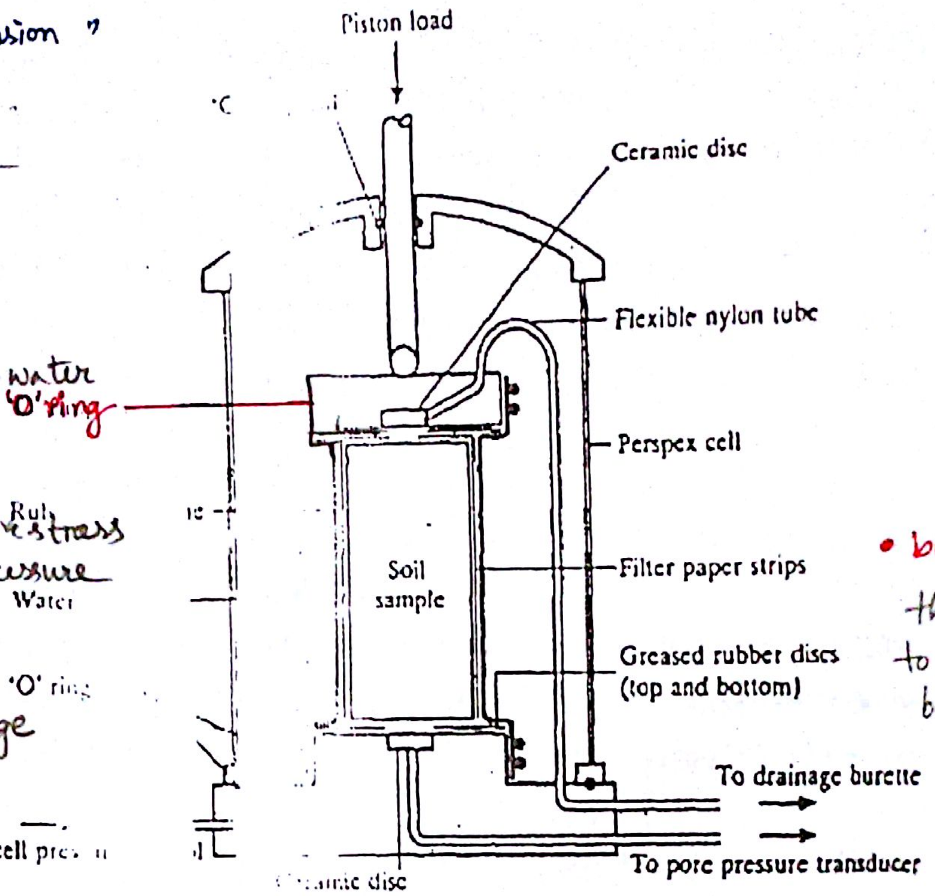
- Depending on loading mode —
  1. Triaxial Compression Test
  2. " Extension "



load পড়লে soil & water load নিবে।
⇓
Soil → Excess effective stress
Water → Pore water pressure
⇓
2 stages —
1. Construction stage
2. Loading "
⇓
ধরি



- back pressure
  ⇓
  the pressure given to soil by top & bottom drainage line
  ⇓
  for this soil will try to wash back
- So cell pressure is given

Fig. 7.9 A schematic diagram of triaxial shear test set up.

The prepared soil sample is properly placed in the cylinder, the cell is the filled with water via cell pressure control tube and subjected to a desired cell pressure. The water pressure, commonly referred to as confining pressure, acts horizontally on the cylindrical surface of the sample through the rubber membrane as well as vertically through the top cap. The pore water pressure in the sample is measured through a porous disc at the base pedestal and connected through a water filled duct to a pressure transducer. To minimise the friction at the top and bottom of tha sample and allow an unrestricted lateral deformation during shear, greased rubber discs are placed between the sample and the end caps.

An axial load commonly referred to as deviator load (stress), is the applied and steadily increased until the failure of the specimen occurs. The procedure is repeated for different confining pressures and corresponding deviator loads atleast three times using three identical soil specimen. A Mohr's circle for each combination of confining pressure (σ_3), and total axial stress (σ_3 + deviator stress = σ_1) is constructed. The tangent to the resulting circles becomes the Mohr envelope as shown in Fig. 7.10.

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Desipition of Pore H_2O pressure

Construction basis. 1. • Unconsolidated — water can't drain out 1. (undrained)

2. • Consolidated loading basis " can " " 2. • (drained)

As such a wide variety of tests are possible in triaxial shear testing to depict insitu loading and drainage conditions. The following tests are adequate for most of the engineering purposes.

based on drainage condition:

- 1. Unconsolidated Undrained test (*UU or Quick or Q test*) ⇒ loading & construction very fast
- 2. Consolidated Undrained test (*CU or Q_c or R test**) ⇒ construction slow but loading fast
- 3. Consolidated Drained Test (*CD or Slow or S test*) ⇒ loading & construction very slow

(it is simply because, R lies between Q and S in alphbatic order as is the case of CU with respect to UU and CD)*

Generally, there are three different and successive steps in a triaxial test. These are:

- Preparation of soil sample
- Application of cell pressure
- Application of excess axial stress at constant cell pressure

The first step, preparation of soil sample, for all the soil sample is essentially similar with only a minor exception that in case of *UU* test the drainage accessories like wrapping of filter paper around the soil specimen can be ommitted. The dissipation of pore water pressure or drainage during second and third steps of testing depends upon the test type. Often pore water pressure is measures in consolidated undrained test to estimate the effective stress parameters. Such a test is designated as CU test. Table. 7.2 presents the drainage and loading conditions during second and third steps.

Table 7.2 Drainage and loading conditions in different types of triaxial tests

Type of test	Second Step: **Application of cell pressure**	Third Step: Application of additional axial stress at a constant cell pressure
Unconsolidated Undrained (UU)	Drainage is not allowed. Sample is unconsolidated.	Drainage is not allowed. i.e. failure occures in undrained condition.
Consolidated Undrained (CU)	Drainage is allowed. That is, sample is consolidated.	Drainage is not allowed. i.e., failure occures in undrained condition.
Consolidated Drained (CD)	Drainage is allowed. That is, sample is consolidated.	Drainage is allowed. That is, failure occures in drained condition.

Shear strength parameters from various types of triaxial test

The triaxial test data obtained from a test on each of the specimen are separetely plotted as devator stress strain diagram to obtain the peak or maximum deviator stress and the corresponding axial strain. The excess pore water pressure reading at the failure strain is used to calculate the effective principal stresses. Thus, the principal stresses on the soil sample are estimated and Mohr circle are drawn. For details of the test procedure and computations reference can be made to Lambe(1993).

Shear strength parameters for cohesive soil

The shear strength of cohesive soils depends on many factors like **degree of saturation, stress history, loading rate and drainge conditions.** Cohesive soils are often treated in a saturated state to represent the worst condition that can prevail in the field, though the parameters estimated wil be generally on the conservative side. Even then, researchers, concentrated many of their studies on unsaturated and remoulded clays to assess the insitu behaviour by analysing the results obtained from their studies..

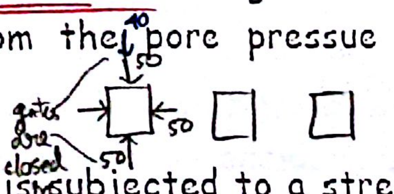
- construction test → simulated by cell pressure
- Lading " ⇒ " extra loading

Top & bottom drainage line will alwz be closed. for

Unconsolidated undrained (UU) test & (Quick test) / Q-test

In this test a minimum of three soil samples are subjected to different confining pressure σ₃ and then loaded to failure. Before applying this confining pressure, a small confining pressure and equal amount of pore water pressure may be applied to the soil and allowed to stay for some time to saturate the sample. This application of pore water pressure to the sample is known as back pressure. The degree of saturation of soil specimen can be estimated from the pore pressue parameter, B as discussed later.

* cell pressure is all around i.e. equal in every directn.

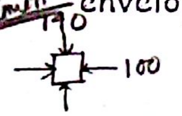


Say cell = 50

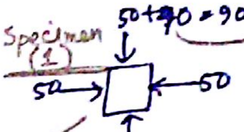
When a saturated cohesive soil is in subjected to a stress increase, it will respond simply by increasing the pore water pressure by that amount. both cases So, any change in all around pressure or axial stress will not bring any change in the structure or arrangement of soil particles within the soil mass. Hence the undrained strength remains constant and independent of cell pressure. That is, the soil will fail at an equal deviator stress for all the different cell pressures. Deviator stess which is the diameter of the

Mohr's circle being equal, the failure envelope will be a horizontal line thus depicting that only cohesion parameter will contribute to generate the shear stength of soil. The cohesion parameter at this condition is termed as undrained cohesion and designated as c_u The other parameter φ would be zero. A typical example of the Mohr,s circle, failure envelope and the estimated shear parameter is shown in Fig. 7.11. The failure envelope in this case can be defined as

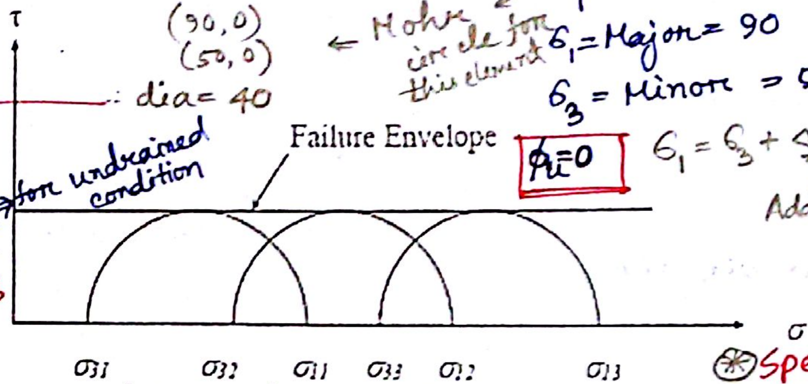
Specimen-2



$$\boxed{\tau_f = c_u} \qquad (7.5)$$



→ the external load at which it fails



- dia of the Mohr circle will be always equal to DEVIATOR STRESS

← Mohr circle for this element σ_1 = Major = 90

σ_3 = Minor = 50

$\boxed{\varphi_u = 0}$

$\sigma_1 = \sigma_3 + \Delta\sigma$

↓ Additional/Deviator Stress

Fig. 7.11 Unconsolidated undrained (UU) strength of saturated clay

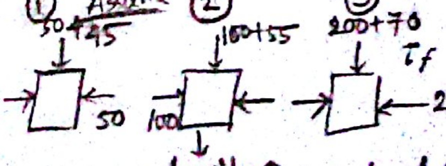
⊛ Specimen ② σ pressure বাড়ালে 3 ⇓ no vol^m change cz it is saturated. ⇓ So Deviator Stress will be $\boxed{\text{equal}}$

→ change in volume & drainage occur.

$\boxed{\text{Consolidated}}$ undrained (CU) test — (R-test)

↳ So top & bottom drainage line is open for construction & closed for loading

In this test soil sample is initially consolidated under confining pressure. This pressure is known as initial effective confining pressure. The volume change reading can also be taken at this stage to estimate the initial void ratio of the specimen. After consolidation, the confining pressure and back pressure may be increased by equal amount thus keeping the effective cell (confining) pressure constant. As usual, the back pressure is applied to saturate the soil sample and also to facilitate the measurement of negative pore water pressure during application of deviator stress in case of an overconsolidated clay. The soil sample is sheared at this elevated cell pressure (σ_3) with a deviator stress of $\Delta\sigma$, that is axial stress of $\sigma_1(=\sigma_3+\Delta\sigma)$. Morh's circles are drawn for the three specimens to determine the failure envelope and strength parameters in terms of total stress. The Mohr's circle diagram and failure envelope will be very much of similar fashion to the generalised diagram of Fig. 7.9b. The strength parameters would be designated as c_{cu} and φ_{cu} instead of c and φ as mentioned there. The failure envelope will be represented by



$$\tau_f = c_{cu} + \sigma \tan\varphi_{cu} \qquad (7.6)$$

↳ for deviator load.

density ↑ ⇒ so load for failure/deviator stress ↑ than specimen①

⊛ vol^m change → density higher & ↑ load for failure.

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As mentioned earlier, consolidated undrained test can also be carried out with pore water pressure measurement while shearing the soil sample. The effective stress parameters in such a case are determined drawing Mohr circle and hence the failure envelope taking in to consideration the effective cell pressure σ_3' as $(\sigma_3 - u_f)$ and effective axial stress σ_1' as $(\sigma_1 - u_f)$. Here u_f is the pore water pressure at failure. Thus the effective minor and major principal stresses σ_1' and σ_3' respectively are to be computed for all the three specimens. It is understandable that the Mohr circles for this effective stress condition would be of similar diameter as those for total stress condition. Only thing is that they will shift towards left in the diagram thus resulting in a higher φ and lower c values interms of effective stress A typical illustration is shown in Fig 7.12.

Mohr circlepoints — I

① → (50,0),(95,0) ⇒ dia = 45

② → (100,0),(155,0) ⇒ " = 55

③ → (200,0),(275,0) ⇒ " = 75

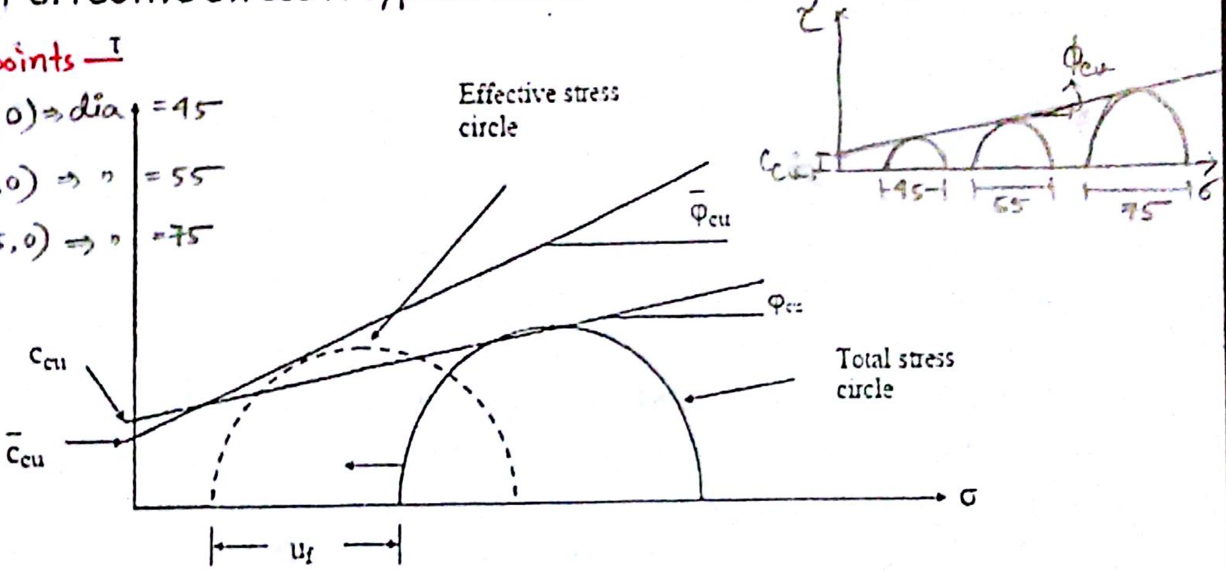


Fig. 7.12 Total and effective stress parameters from a CU test (Typical one circle of each are shown, others are hidden)

The effective strength parameters are $\bar{c}_{cu}$ and $\bar{\varphi}_{cu}$ respectively for cohesion and friction and the equation for failure envelope is

⇒ additional load is larger than CU test

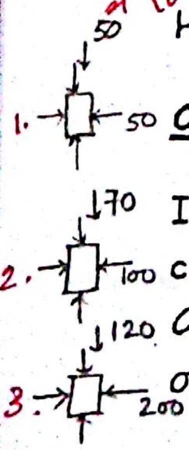
$$\tau_f = \bar{c}_{cu} + \sigma' \tan \bar{\varphi}_{cu} \qquad (7.7)$$

However, for normally consolidated soil $\bar{c}_{cu}$ will be zero as explained later.

additional loading is given very slowly

Consolidated drained (CD) test (Slow test) / S-test ⇒ both the drainage lines are opened and so additional load will change ⇓ so loading is very high

In this type of test, the sample is first consolidated under confining or cell pressure. Back pressure may be applied in a similar manner to that of CU test to saturate the sample, if necessary. Thus the confining pressure σ_3 here is also the effective confining pressure σ_3' as that of CU test. The



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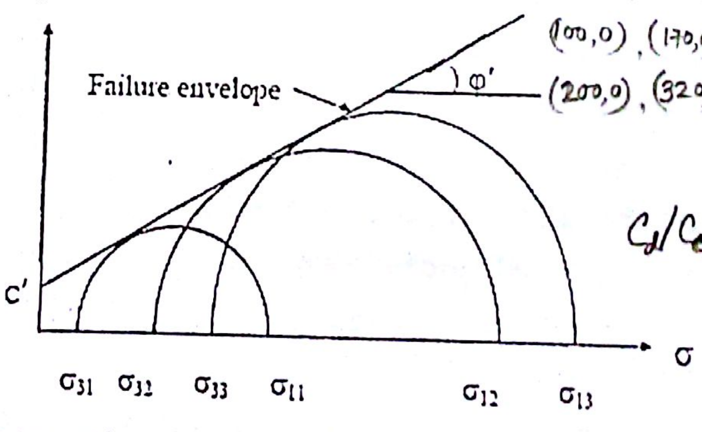
specimen is then sheared to failure with a very small strain rate so that no excess pore water pressure can develop on the failure surface. That is, the pore water pressures remains zero at all the stages of the test thus prevailing effective stress conditions althrough. The volume change reading can also be taken to examine the void ratio of the soil at failure condition. Mohr's circle interms of effective minor and major principal stresses at failure for all the three specimen are are drawn to obtain the effective shear strength parameters c' and φ'. The failure envelope is of typical in nature as represented by Fig. 7.10b, and c and φ would be replaced by the parameters c' and φ'. Fig. 7.13 shows the related diagrams. The values of these parameter are different. The failure envelope can be given by:

⊛ Drained test ⇒ this means consolidated drained test

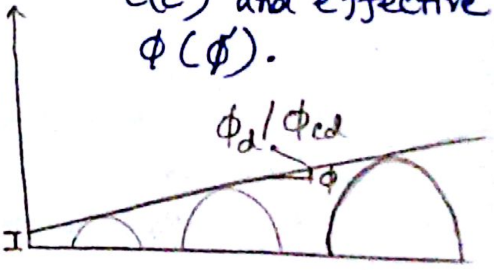
$$\tau_f = c' + \sigma' \tan\varphi' \qquad (7.8)$$

⊛ Here is no chance of developing pore water pressure → So total pressure or stress is only effective stress ⇓ So C_d & ϕ_d is also called effect $c(c')$ and effective $\phi(\phi')$.

The parameters c' and φ' are often replaced by c_{CD} and φ_{CD} to remind that these parameters are for drained condition. The parameters obtained from a drained test always refers to effective stress conditions though these can also be obtained from a CU test and that should be specifically spelled out. However, in laboratory testing there may be slight differences in effective stress parameters determined from CU test and CD tests. However, the magnitude strength obtained from a CD test is always higher than that form a CU test for a normally consolidated clay.



(50,0), (100,0)
(100,0), (170,0)
(200,0), (320,0)



⊛ load magnitude is higher so ϕ will be greated that CU test

Fig. 7.13 Shear strength parameters from a CD test.

Strength parameter for cohesionless soil

When load is applied to a soil, the soil is stressed and causes the soil-water matrix to compress and some of the water is squeezed out. Because cohesionless soils have a high hydraulic conductivity (coefficient of permeability), this water is able to move quickly and easily. The potential drainage rate is atleast as high as the loading rate. This condition is known as drained condition and the stresses acting on a cohesionless soil are always the effective stresses. Hence, in case of cohesionless soils the measured strength parameters from shear tests are always the effective one and it is almost independent of type of triaxiat tests. As cohesionless soil has smaller surface forces, the cohesion parameter can be considered as zero the the strength envelope can be explained by the equation

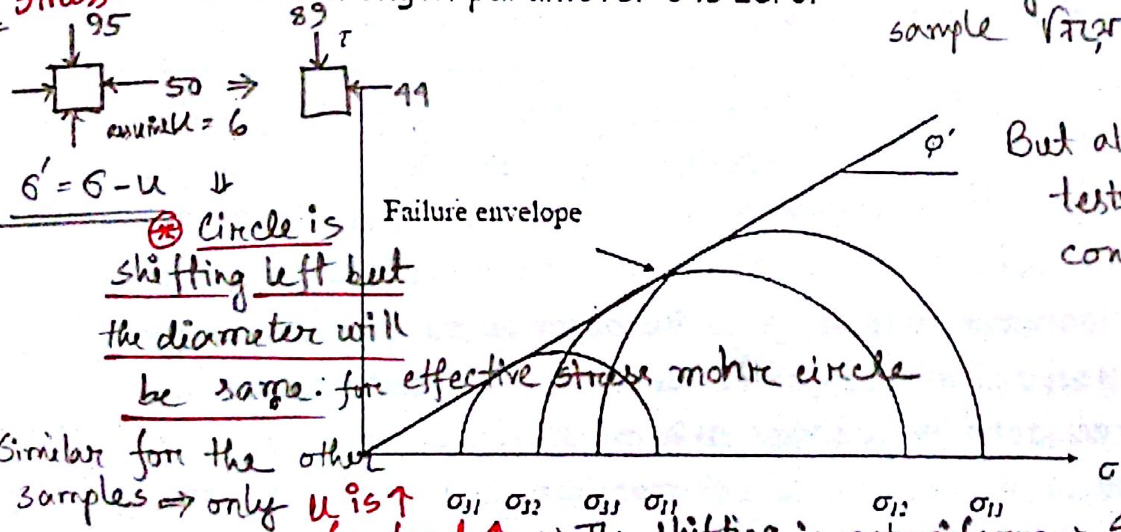
* for effective (σ) stress measurument → CU-Test
" pore water pressure (u) → $\overline{CU}$ /CU bar Test
⇓
अब त्यहाँ u and σ' दुवै निकाल्न सकिन्छ।

$$\tau_f = \sigma' \tan\varphi' \quad (7.9)$$

As such, only one Mohr's circle is required to define the straight line failure envelope of strength as another point is known to be the origin of σ-τ coordinate system. However, confining pressure has a little effect on strength the parameter and as such, conventionally three specimen are tested at different confining pressures, Mohr's circles are drawn to obtain the failure envelope of the form represented by Eq. (7.9) and thus the angle of internal friction φ' is determined. A typical Mohr circle diagram for triaxial test on dry cohesionless soil is shown in Fig. 7.14, where the strength parameter c is zero.

Effective Stress
1.

Normally bore hole बाट soil sample लिएर test गर्नुपर्छ
⇓
But all the specimen tests are very time consuming.



σ' = σ − u ⇓

* Circle is shifting left but the diameter will be same: for effective stress mohr circle

Similar for the other samples → only u is ↑ for load ↑. → The shifting is not uniform → So φ will be changed & ↑
⇓
This ↑ φ is called $\boxed{\phi'}$

Fig. 7.14 Shear strength results of cohesionless soil from a triaxial test

However, the moist cohesionless soil might show a very small value of apparent cohesion which is due to surface tension. In all the shear strength test of sand volume change is a very important factor. Hence, it is preferable to record the volume change readings for better explantion of the test results.

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Undrained shear strength of normally consolidated clays varies with effective overburden pressure (i.e. with depth). Several empirical relations are available in terms of index properties of soil. Of them, the one given by Skempton is mentioned below.

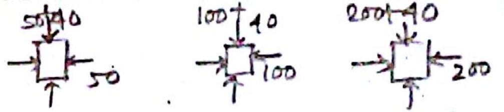
$$c_u/\sigma'_o = 0.11 + 0.0037\, I_p \qquad (7.10)$$

Where, c_u is the undrained cohesion, σ'_o is the effective over burden pressure and I_p is the plasticity index in percent. Eq. 7.10 afford a quick but approximate estimation of the increase in undrained shear strength of a normally consolidated clay with depth.

Unconfined Compression Test

→ the additional load will be always same → Main assumption.

Unconfined compression test is essentially a special case of triaxial test where the minor pricipal stress or cell pressure is kept as zero. No rubber membrane is required to encase the soil sample As such, the apparatus becomes much simpler. This is the easiest, quickest and simplest test for determining the undrained strength of saturated cohesive soil. This test can also be done on unsaturated sample, but it should be borne in mind that undrained shear strength of cohesive soil varies significantly with the moisture content and density and in the design due consideration should be given. As in triaxial tests, three cylindrical specimens are tested for axial compression to failure in an unconfined compression machine and their average values are considered as the strength.



→ If zero stress is given, it is also 40.

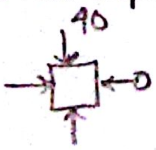
The unconfined compression testing macine essentially consists of a loading frame where the cylindrical soil sample can be placed in between base platform and a top cap. The top cap is attached to either with a proving ring or load transducer. The bottom platform can be raised either by gear system or by a hydraulic pump system. A view of the machine is shown in Fig. 2.15.

⊛ 1. Without cell pressure, sample stand করানোর tough, so vacuum is applied.

2. If the soil is 100% saturated i.e. $\overset{\nabla}{\square}$ → then UC test

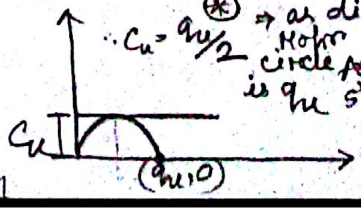
& Prove in UC Test, the parameter $C_u = q_u/2$

A prepared soil specimen is placed properly and compressed to failure very quickly within a couple of minutes to maintain the undrained condition. The soil appears to fail in compression, and the test results are often expressed in terms of the compressive strength; it actually fails in

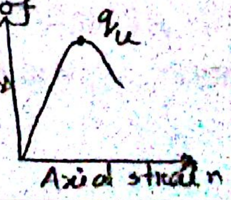


(40,0), (0,0)

⇓ the circle is same for all the circles.



∴ $C_u = q_u/2$ ⊛ → as dia of Mohr circle is q_u



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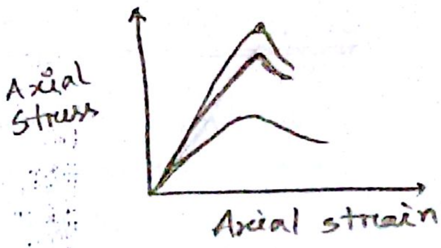
shear in diagonal planes. The unconfined compressive strength of the soil, usually designated as q_u, is given by

$$q_u = (P_f / A_f) \tag{7.11}$$

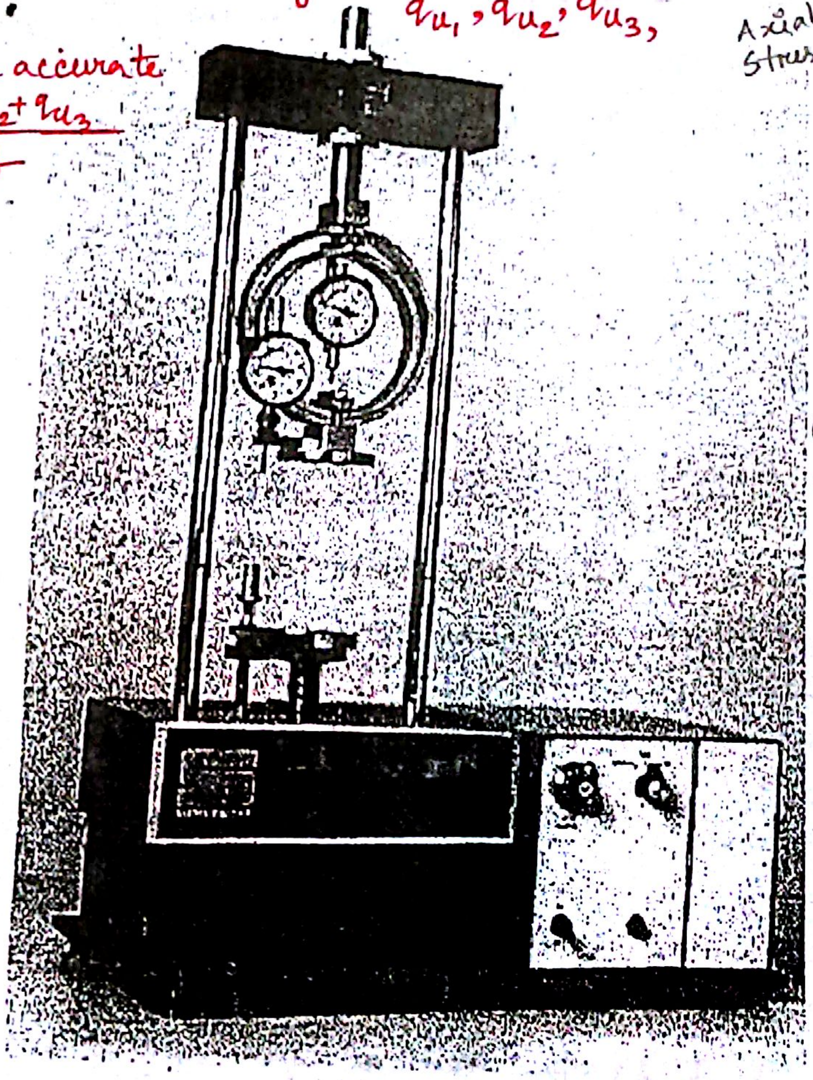
Where, q_u is unconfined compressive strength and A_f is the area of the sample at failure, often termed as corrected area. Details of these can be found in Lambe(1993). If the Mohr's circle of stresses on the sample at failure is drawn, Fig. 7.16, the undrained shear strength can be found as

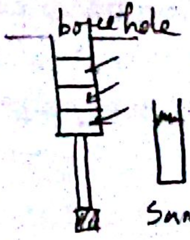
$$c_u = (q_u / 2) \tag{7.12}$$

⊛ যদি 3 results are very close such that $q_{u_1}, q_{u_2}, q_{u_3}$, then the accurate $q_u = \frac{q_{u_1} + q_{u_2} + q_{u_3}}{3}$



⇓
3 টে curve for top, middle & bottom sample.
⇓
একেবারে যেটা 100% saturation-এ থাকবে সেটার জন্য যে curve সেটাই perfect.





Sample is sealed so that there is no change in moisture
⇓

Fig. 7.15 A view of unconfined compression testing machine

Theoretically, an unconfined compression test should yield the same results of undrained shear strength as obtained from triaxial UU test. However, this supposition is only valid if several asumptions are satisfied.

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- *The soil specimen must be clay and 100 percent saturated.*
- *The specimen must not contain any fissures, cracks, silt seams, verves or any other defects.*
- *The specimen must be sheared rapidly to failure. If the time is too long, evaporation and surface drying may result in high strength. Typical time to failure is 5 to 15 minutes. If the sample does not show any sign of failure, the unconfined comprresive value is taken as the as 20 percent strain.*

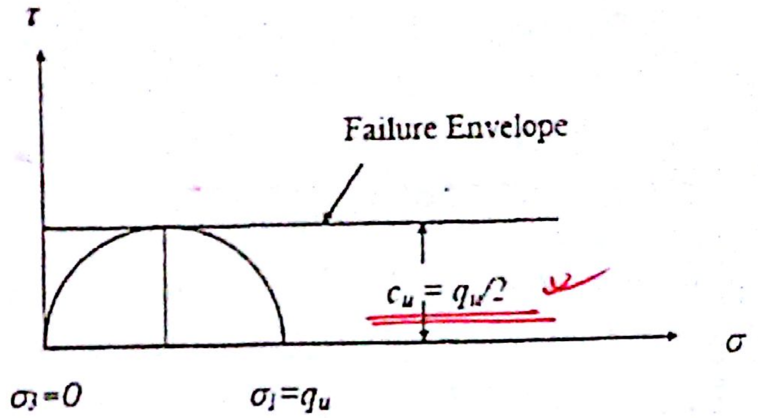

Fig. 2.16 Mohr circle diagram for unconfined compression test

Unconfined compressive strength also gives a measure of consistency of clay soil as expressed in Table 7.3. Unconfined compression test can be done both on undisturbed and remoulded samples. Their values usually gives some useful estimation about the sensitivity of clay soils. Sensitivity describes the effect of stress and environment with time on the strength of the soil. It is usually defined as the ratio of undrained shear strength of clay soil in undistubed condition to that at a remoulded state at the same water content. Though, it is referred to undrained strength it is customary to use the unconfined compressive strength to define its sensitivity. Hence,

$$Sensitivity = \frac{q_{u\ undisturbed}}{q_{u\ remoulded}}$$

→ crack/fail করার পর ঐ sample দিয়েই again same mold করব & find q_u.

The sensitivity of a soil measures the reduction of shear strength that might occur due to disturbace of the sample. Bjerrum(1954) gives a sensitivity classification os soil which is presented in Table 7.4.

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Table 7.3 Consistency and unconfined compressive strength of clay soil.

Consistency	Unconfined Compression strength, $q_u (kN/m^2)$
Very soft	< 25
Soft	25 - 50
Medium	50 - 100
Stiff	100 - 200
Very stiff	200 - 400
Hard	400 - 800

Table 7.4 Sensitivity classification of clay soil

Sensitivity value	Soil description
Less than 2	Insensitive
2 - 4	Medium sensitive
4 - 8	Sensitive
8 - 16	Very sensitive or Extra Sensitive
16 - 32	Slightly quick
32 - 64	Medium quick
Greater than 64	quick

Highly overconsolidated soils are insensitive because of low natural water content of the deposit. Glacial clays lie in the range of medium to extra sensitive classification. Some of fresh water and marine deposits are quick. Sensitivity of most of the clay deposits ranges from 2 to 4. The decrease in strength of clay soil is attributed partly due to disturbance of adsorbed water in clay layers. If the remoulded soil is left undisturbed at the same water content, partly of its strength is regained due to reorientation of the particles. This is known as thixotropy.

Vane Shear Test

Very quick test as in undrained condition ⇒ cz in field no membrane can be kept.

The vane shear test can be applied for measurement of undrained shear strength of clay soil both in the laboratory and field. In particular this test is very suitable for soft clays, which otherwise may be greatly disturbed during the extraction and testing process.

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Essentially, the vane shear apparatus consists of a series of thin rectangular blades welded to a circular shaft, so that when the shaft is rorated about its axis each blade generates the same cylinder. In its usual form, four blades are used forming a cross section. A schematic diagram of vane shear apparatus is shown in Fig. 7.17(a).

The vane dimensions may vary but a height/diameter ratio of 2.0 is commonly used. The vane is pushed into the soil to a predetermined depth and rotated rapidly (usually 0.1°/sec) until shear takes place over the surface area approximately equal to that generated by the vane. The strength is estimated from the maximum measured torque required to shear the cylindrical surface. A simple formula between shear strength and torque can be derived depending on the following assumptions.

- *The soil is purely cohesive (φ = 0), homogeneous and isotropic.*
- *The insertion of the blade causes no disturbance to the soil.*
- *No drainage takes place during shearing.*
- *Failure takes place by shearing over the surface of the cylinder generated by the rotating vane.*
- *No progressive failure takes place in the soil and the shear strength is fully mobilsed on the surface of the cylinder at failure.*

- Conditions:
  1. Soil must be clay
  2. " " " saturated

The total torque is the sum of the opposing moments on the cylinder side and on the two edges of the cylinder. This can be written as-

$\rightarrow SF = C_u \Rightarrow$ as side-interface [illegible] ISo same SF रहता।

$$T = M_{side} + (2 \times M_{edge})$$

$$M_{side} = \pi d h C_u \cdot \frac{d}{2} = \frac{\pi d^2 h C_u}{2}$$

$\Downarrow$ as cylindrical surface, it is uniform.

At center(0). $SF = 0 \Rightarrow$ as no deformation.

where,

T = total applied torque

M_{side} = resisting moment due to the shear forces on the cylindrical surface

M_{edge} = resisting moment due to shear forces on a single edge surface.

Let us consider a cylinder of soil of height, *h* and diameter, *d* (Fig. 7.17) will resist the torque until the soil fails. If the assumption of full shear mobilisation at the edges are not valid, there could be various assumptions regarding the distribution of mobilised shear strength at the edges. The possible distributions are shown in Fig. 7.17(b).

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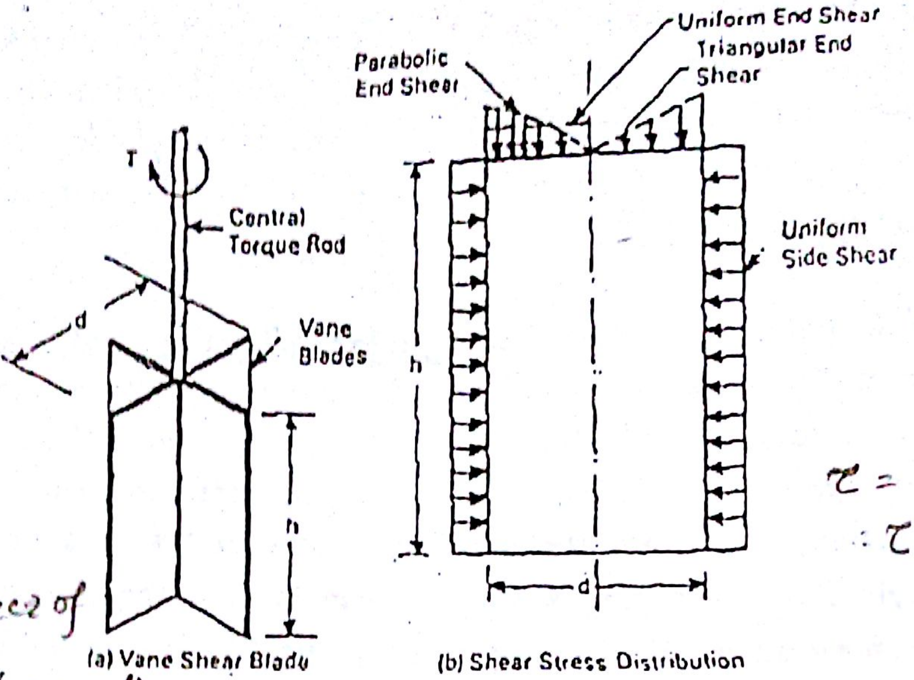
Sect^n A-A



It is failing becz of shear & area of cylindrical surface = πdh

$\tau = c + \sigma \tan\phi$ → 0 as $\phi = 0$

$\therefore \tau = c_u$ becz undrained

Fig. 7.17 Vane shear test; (a) diagram of apparatus; (b) distribution of stresses

Accordingly, the expression for resisting torque can be obtained by integrating the resulting moment for the relevant stress distribution diagram of the edges and adding the value for the moment due to surface resistance. The general expression can be obtained as follows:

$$\boxed{T = \pi c_u \left(\frac{d^2 h}{2} + \frac{a d^3}{4} \right)} \qquad (7.13)$$

⇒ a = depends on the distributio

for side/surface → for edges (top & bottom)

where,

c_u = undrained shear strength of soil

Imp. a = $\frac{2}{3}$ for uniform distribution of shear at the edges

= $\frac{1}{2}$ for triangular distribution of shear at the edges

= $\frac{3}{5}$ for parabolic distribution of shear at the edges.

Sometimes, a vane shear test is done in a borehole where to top edge of the vane concides with the bottom of the borehole. In such a case only one edge instead of two partakes in the resistance process. The value of 'a' in Eq. (7.13) will then be reduced to half of the values as given above. For a conventional event, where a uniform distribution at both the edges is considered and the height of the vane is twice the diameter of the

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failure cylinder, Eq. (7.13) takes a simplified form to estimate the undrained cohesion of soil as

Q. Prove that →

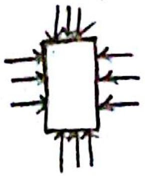
$$\boxed{c_u = \frac{6T}{7\pi d^3}} \Rightarrow \text{using } h = 2d \text{ and } a = 2/3 . \qquad (7.14)$$

7.6 Pore Water Pressure Parameters

Pore water pressure plays a significant role in determining the shear strength of soil. The change in pore water pressure due to change in applied stress is characterised by dimensionless coefficients called *'pore pressure coefficients' or 'pore pressure parameters A and B'*. These parameters have been proposed by Skempton(1954). In undrained triaxial compression, pore water pressure develops both during the application of cell pressure and application of additional axial stress.

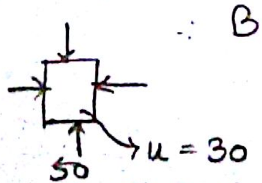
Triaxial test, in addition to determining the strength parameters of soil, also gives data for predicting the excess the excess pore water build up at a point in a soil mass by a change in total stress condition. The ratio of pore water pressure developed due to application of confining or cell pressure is termed as *B* parameter: Therefore,

$$B = \frac{\Delta u_c}{\Delta \sigma_c} = \frac{\Delta u_c}{\Delta \sigma_3} \qquad (7.15a)$$



That is, $\Delta u_c = B\Delta\sigma_3$ (7.15b)

where,
$\Delta\sigma_c$ = increase in confining pressure
Δu_c = increase in pore pressure due to $\Delta\sigma_c$
$\Delta\sigma_3$= increase in minor principal stress.



and ↑ 60 & if u↑ = 35

$$\therefore B = \frac{35-30}{60-50} = .5$$

In undrained condition, the decrease of volume of mass is equal to that in the volume of pores. Using the principle of theory of elasticity it can be shown that

$$B = \frac{1}{1 + \frac{nC_{vo}}{C_{sk}}} \qquad (7.16)$$

where,

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B → depends on degree of saturation
A → 〃 〃 overconsolidation ratio.

$$\overline{A} = \frac{\Delta u_d}{(\Delta\sigma_1 - \Delta\sigma_3)}$$

That is, $$\Delta u_d = \overline{A}(\Delta\sigma_1 - \Delta\sigma_3) \quad (7.17)$$

↳ উপরের pressure ↑

Where, Δu_d is increase in pore water pressure due to increase in deviator stress $(\Delta\sigma_1\text{-}\Delta\sigma_3)$. $\overline{A}$ is an experimentally determined dimensionless parameter and the product of two other parameters, A and B. If now Δu denotes the total excess pore water pressure within the soil mass due to increase in confining pressure and deviator stress, then using the principle of superposition, it can be related as

$$\Delta u = (\Delta u_c + \Delta u_d) = B\Delta\sigma_3 + \overline{A}(\Delta\sigma_1 - \Delta\sigma_3) = B\Delta u_3 + AB(\Delta\sigma_1 - \Delta\sigma_3)$$

That is, $$\boxed{\Delta u = B[\Delta\sigma_3 + A\,(\Delta\sigma_1\text{-}\Delta\sigma_3)]} \quad (7.18)$$

For a 100% saturated sample, where $B = 1$ the equation simplifies to the form

$$\Delta u = \Delta u_3 + A\,(\Delta\sigma_1\text{-}\Delta\sigma_3) \quad (7.19)$$

Similar to parameter B, A is not also constant. It varies with the soil type, stress level, soil anistropy, density index and stress history (*OCR*). Bishop and Hankel(1962) gave a relationship between over consolidation ratio and A value at failure, A_f. This is shown in Fig. 7.19. Typical A_f values for clay soils are also given by Skempton. They are presented in Table 7.5.

× { bottom & top drainage line → closed
Then transducer will give the reading

Theoretically,
$\boxed{B = 1}$ if the soil is fully saturated
⇓
But practically
·91 – ·95 A_f
হলেই assume
fully saturated

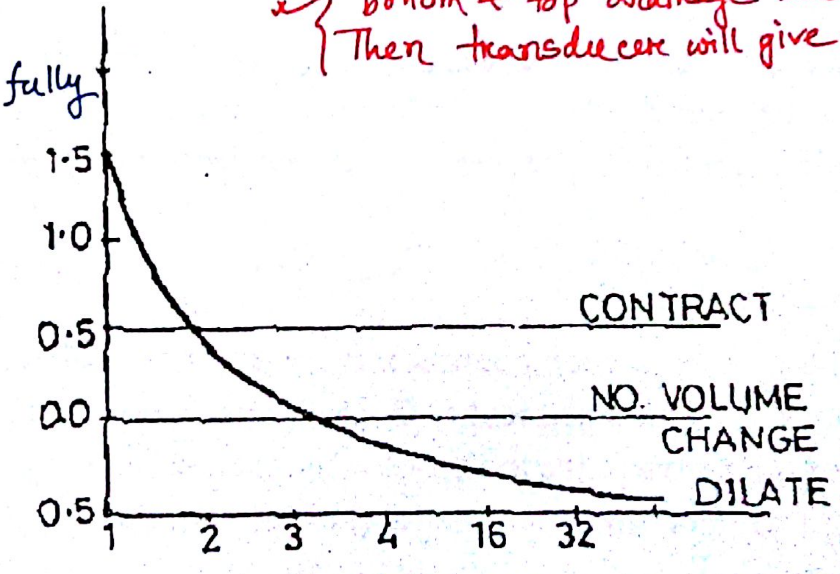

Fig. 7.19 Variation of A_f with over consolidation ratio (*OCR*)

OC clay highly dense. So এদের particle অণুগুলো উপরে চলে আসে। So volume বাড়ে & so Pore water Pressure ↓.

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Lateral Earth Pressure V.V. Imp.

- Introduction to Plastic Equilibrium in Soils
- Earth Pressure at Rest
- Active and Passive Earth Pressures by Rankine and Coulomb's theory
- Culmann and Rebhann's Graphical Construction for earth pressures

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CHAPTER 9

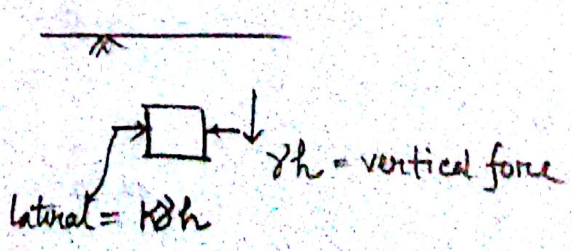
LATERAL EARTH PRESSURE ⇒ धक्का ⇒ always lateral

9.1 Introduction to Plastic Equilibrium in Soils

A mass of soil is said to be in a state of plastic equilibrium if failure is incipient or imminent at all points within the mass. This is commonly referred to as general state of plastic equilibrium and occurs in rare instances such as when tectonic forces act. Usually, however, failure may be imminent only in a small portion of the mass such as that produced by the yielding of a retaining structure in the soil mass adjacent to it. Such a situation is referred to as local state of plastic equilibrium.

Earth pressure → lateral
→ Verticle

The state of stress in the backfill of a retaining structure depends on the movement of structure with reference to the backfill. The backfill material is said to be in a state of elastic equilibrium when the stress involved and the corresponding strain are within elastic limit. This generally occurs for no or very less movement of the wall. Further increase in stress develops shear stresses at some point in the body, reaching the shear strength of soil. Subsequent increase in stress causes a substantial increase in strain producing a condition known as plastic flow. The soil mass prior to onset of the plastic flow condition is said to be in a state of plastic equilibrium. Plastic collapse occurs after the state of plastic equilibrium has been reached and the load or stress at this condition is referred to as collapse load. The determination of collapse load, adopting plasticity theory, is rather complex. However, plasticity theory also provides a simplified analysis.



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Rankine was the first to investigate the stress conditions associated with the states of plastic equilibrium in a semi infinite mass of homogeneous, elastic and isotropic soil mass under the influence of gravity or self weight alone.

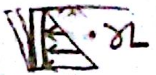
9.2 Lateral Earth Pressure

The pressure exerted by the soil against an engineering structure or acting on a surface of surrounding soil mass is called earth pressure. Strictly speaking the two terms used in geotechnical engineering related to pressure namely pressure and stress are nearly synonymous. However, pressure may be defined as the contact force per unit area between a structure and soil whereas stress is the force per unit area within the soil mass.

One of the first steps in designing any structural member is to determine the magnitude and direction of applied load. For retaining structures, the primary applied load is the resultant of the lateral earth pressure the soil imparts on the wall. Lateral earth pressure due to soil depends on:

1. Physical characteristics of soil
   3 types
   - Cohesionless soil
   - → C–ϕ → has both cohesion & friction
   - Cohesive soil
2. Deformation characteristics of structure
   - No deformation ③
   - Structure away from soil (lateral expansion) ① → pressure will be lowest
     (outward & downward)
   - Structure towards soil (lateral contraction) ② → pressure will be highest
     (inward & upward)

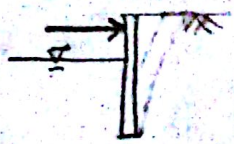
①



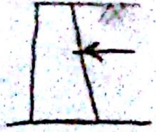
⇓
The structure is very thin & so the soil wants to move outward & downward

Depending on the deformation characteristics of structure and hence the associated soil, lateral earth pressure may have

②



③



* for the cases ① & ②
⇓
2 টা soil-এ pressure করে

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varying magnitude; as such lateral earth pressure can be classified as

- Earth pressure at rest (no deformation) → retains on its own space.
- Active earth pressure (lateral expansion) → away from the soil → lowest
- Passive earth pressure (Lateral contraction) → towards the soil → highest

When a soil maintains the same lateral dimension regardless of the vertical pressure, the lateral pressure exerted by the soil on the structure or the lateral stress induced within the soil mass is known as earth pressure at rest. The ratio of lateral effective stress, σ_3, to vertical effective stress, σ_1, at this condition is termed as *coefficient of earth pressure at rest* and *designated by K_o.*


→ lateral expansion

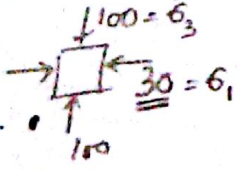
When a soil expands laterally due to stresses, the maximum lateral stress at which the soil fails due to shear is called active earth pressure. The ratio of lateral stress to vertical stress at this condition is termed as *coefficient of active earth pressure and designated by K_a.*


for lateral contraction, the soil will be coming up.

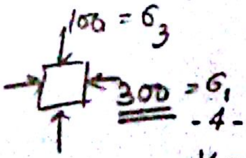
When a soil contracts laterally due to stresses, the minimum lateral stress at which it fails due to shear is called passive earth pressure. The ratio of lateral stress to vertical stress at the condition is termed as *coefficient of passive earth pressure and designated by K_p.*

In general the ratio of lateral stress to vertical stress is known as coefficient of earth pressure and designated as K. The coefficient of lateral earth pressure influences many aspects of the engineering behaviour of soil. As K increases, the following changes occur (Schmertmann, 1985).

$K_a = \frac{\sigma_1}{\sigma_3}$



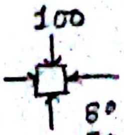
Active case



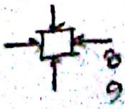
Passive case

-4-

$K_p = \frac{\sigma_1}{\sigma_3}$

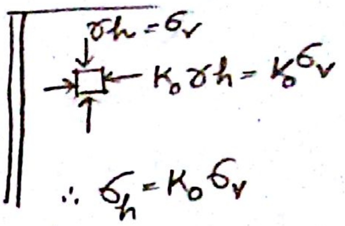


80
70
60
50
40
30 → max^m at which fails for expansion
20



80
90
100
200
300 → min^m at which fails for contraction
400
500

- Bearing capacity increases
- Slope stability decreases
- Fracture in earth dam becomes less likely
- Lateral pressure acting on retaining walls increases
- Skin friction resistance on deep foundations increases
- Foundation settlements becomes smaller
- Seismic liquefaction becomes less likely
- Soil improvement becomes more difficult



9.3 Earth Pressure at Rest

In case of rigid and unyielding retaining wall, no lateral strain occurs in the soil. The corresponding lateral stress or pressure is the earth pressure at rest. The vertical and lateral stresses, σ_v and σ_h respectively, at this condition can related as

$$\sigma_h = K_o \sigma_v = K_o \gamma h \qquad (9.1)$$

A theoretical expression of K_o for an isotropic, homogeneous and elastic material can be obtained from the condition of zero lateral strain. The lateral strain, ϵ_h, can be expressed

$$\varepsilon_h = \frac{1}{E[\sigma_h - \mu(\sigma_v + \sigma_h)]} \qquad (9.2)$$

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

If $\epsilon_h = 0$, then $\Rightarrow$ (fore this case)

$$\sigma_h = \frac{\mu}{1-\mu}\sigma_v = K_o \sigma_v \qquad (9.3)$$

Where, μ is the Poisson's ratio of soil. It is not really very easy to determine the Poisson's ratio of insitu soil, as such a

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Active Earth Pressure ⇒ Rankin's theory } Cohesionless } c-φ soil
Passive " " ⇒ Culomb's theory } Cohesive
Culmann's Graphical Method

9.4 Earth Pressure Theories

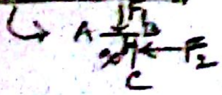
The magnitude of lateral earth pressure, especially the active and passive pressures, is evaluated by the application of one or other of the so called lateral earth pressure theories. The problem of determining the lateral earth pressure against retaining walls is the oldest in the field of engineering. Several investigators have proposed theories, but those given by Coulomb and Rankine have been considered reliable in spite of some limitations and are considered as the basic to the problem. Their theories are usually considered as the Classical Earth Pressure Theories.

9.4.1 Rankine's Theory ⇒ (Active for Earth Pressure)

Rankine developed his theory of lateral earth pressure when the backfill consists of dry, cohesionless soil. The theory was later extended by Resal (1910) and Bell (1915) to be applicable to cohesive soils. The following important assumptions are made in Rankine's theory:

Imp Q. What are the assumptions of Rankine's Theory.

- 1. The soil mass is semi infinite, homogeneous, dry and cohesionless
- 2. The ground surface is a plane which may be horizontal or inclined
- Most imp. 3. The face of the wall in contact with the backfill is vertical and smooth
- 4. Conjugate relationship exists between vertical and lateral pressures



if $F_1 \parallel BC$ and $F_2 \parallel AB$ ⇒ then it will be said conjugate stress

Rankine considered the equilibrium of a soil element in deriving the formula to estimate the lateral earth pressure.

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9.4.1.1 Active Earth Pressure in Cohesionless Soil

The Mohr's circle of stresses may be used to determine the lateral thrust on retaining walls from backfill soil for which the shear strength may be expressed by $\tau = \sigma \tan\varphi$. The value of φ here designates effective angle of internal friction.

Let us consider a vertical and smooth retaining wall, and a cohesionless backfill with a horizontal surface. At a given depth h on an element, the vertical stress σ_1 is equal to γh. The lateral stress is designated by σ_3. Both of these are principal stresses and the corresponding Mohr's circle is shown in Fig. 9.1. It is assumed that the wall has yielded sufficiently to satisfy active earth pressure condition, such that $\sigma_3 = K_a \gamma h = Ka.\sigma_1$.

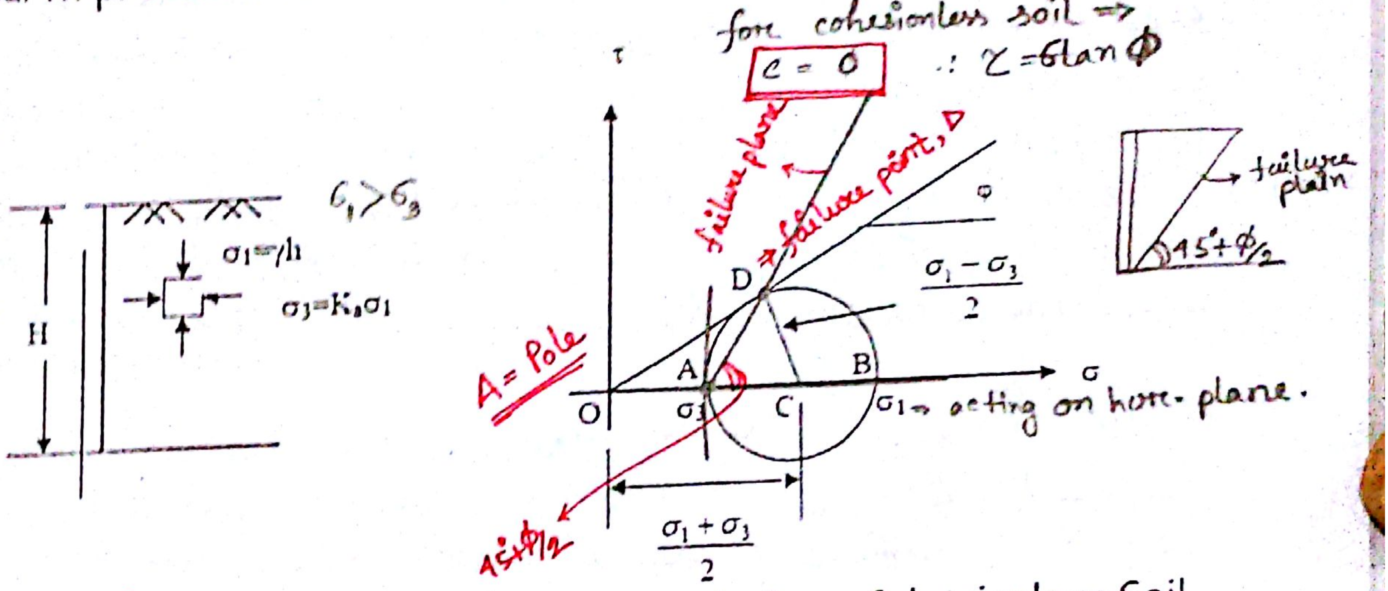

Fig. 9.1 Soil Element and Mohr's Circle for a Cohesionless Soil

$$K_a = \frac{\sigma_3}{\sigma_1} = \frac{OA}{OB} = \frac{OC - AC}{OC + CB}$$

$$= \frac{OC - CD}{OC + CD}$$

$$= \frac{\frac{OC}{OC} - \frac{CD}{OC}}{\frac{OC}{OC} + \frac{CD}{OC}}$$

$$= \frac{1 - \frac{CD}{OC}}{1 + \frac{CD}{OC}}$$

$$= \frac{1 - \sin\phi}{1 + \sin\phi}; \text{ as } \sin\phi = \frac{CD}{OC}$$

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

Now,

$$\sin\varphi = \frac{\frac{\sigma_1 - \sigma_2}{2}}{\frac{\sigma_1 + \sigma_2}{2}} = \frac{\sigma_1 - \sigma_2}{\sigma_1 + \sigma_2}$$

$$\Rightarrow \quad (\sigma_1 + \sigma_3)\sin\varphi = (\sigma_1 - \sigma_3)$$

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Now,

$$\sin\varphi = \frac{\frac{\sigma_1 - \sigma_3}{2}}{\frac{\sigma_1 + \sigma_3}{2} + c\cot\varphi} = \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3 + 2c\cot\varphi}$$

$$\Rightarrow \quad (\sigma_1 + \sigma_3)\sin\varphi + 2c\cot\varphi\,\sin\varphi = \sigma_1 - \sigma_3$$

$$\Rightarrow \quad (1 - \sin\varphi)\sigma_1 - (1 + \sin\varphi)\sigma_3 = 2c\cos\varphi$$

$$\Rightarrow \quad \sigma_3 = \left(\frac{1 - \sin\varphi}{1 + \sin\varphi}\right)\sigma_1 - \frac{2c\cos\varphi}{1 + \sin\varphi}$$

But,

$$\frac{\cos\varphi}{(1 + \sin\varphi)} = \frac{\sqrt{\cos^2\varphi}}{(1 + \sin\varphi)} = \frac{\sqrt{1 - \sin^2\varphi}}{(1 + \sin\varphi)} = \sqrt{\frac{(1 + \sin\varphi)(1 - \sin\varphi)}{(1 + \sin\varphi)^2}} = \sqrt{\frac{(1 - \sin\varphi)}{(1 + \sin\varphi)}} = \sqrt{K_a}$$

Therefore, γh $\quad \sigma_3 = K_a\gamma h - 2c\sqrt{K_a}$

If $h = 0$, $\sigma_3 = -ve$ $\Rightarrow \sqrt{K_a} = \tan(45^\circ - \phi/2)$

$\Downarrow$

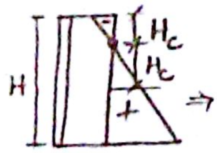
If $\phi > 90^\circ$, then $\sigma_3 = +ve$

$\Downarrow$

But $\phi \ngtr 90^\circ$ ce

That is, $\quad p_{ac} = K_a\gamma h - 2c\sqrt{K_a} \qquad (9.9)$

$\sigma_3 = -ve$ means tension



$\Rightarrow$ So when depth = H_c then $\sigma_3 = 0$

$\therefore \gamma H_c K_a - 2c\sqrt{K_a} = 0$

$\Rightarrow H_c = \frac{2c}{\gamma}\cdot\frac{1}{\sqrt{K_a}}$

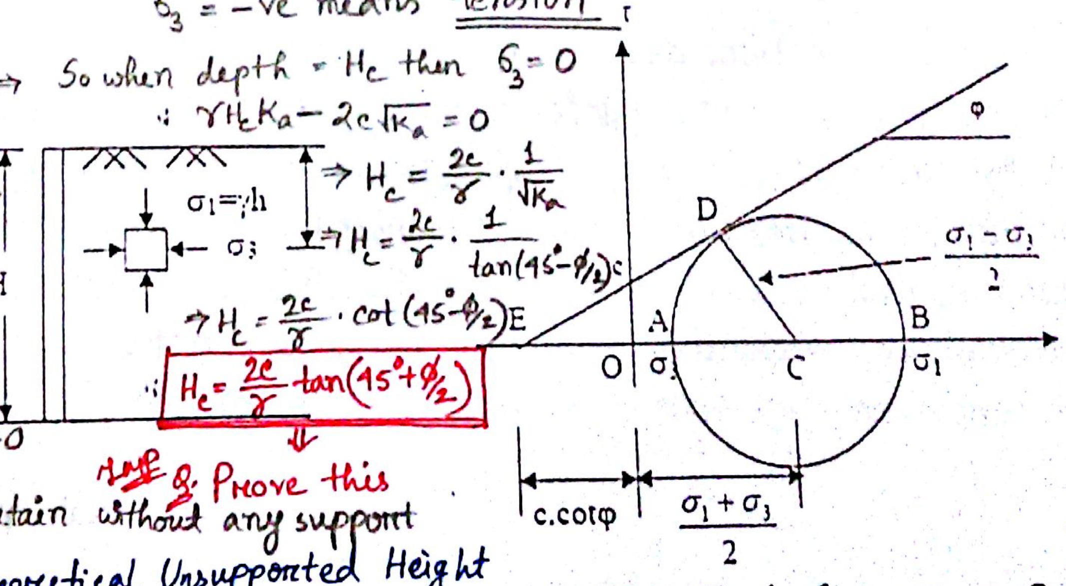
$\Rightarrow H_c = \frac{2c}{\gamma}\cdot\frac{1}{\tan(45^\circ - \phi/2)}$

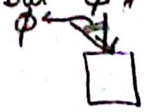
$\Rightarrow H_c = \frac{2c}{\gamma}\cdot\cot(45^\circ - \phi/2)$

$\therefore \boxed{H_c = \frac{2c}{\gamma}\tan(45^\circ + \phi/2)}$

$\Downarrow$

Imp Q. Prove this





$\Downarrow$
for $2H_c$ depth upper – ve lower +ve
$\Downarrow$
So, their resultant pressure = 0
$\Downarrow$
So it can retain without any support
$\Downarrow$
This is Theoretical Unsupported Height

Fig. 9.3 Soil Element and Mohr's Circle for a c - φ Soil

$\Downarrow$

$$\boxed{H_u = 2H_c = \frac{4c}{\gamma}\tan(45^\circ + \phi/2)}$$ → Imp Q. Prove this

$\Downarrow$
for partially cohesive soil

Suppose,
$c = 20\ kN/m^2$
$\phi = 0$
$\gamma = 18\ kN/m^3$
$\therefore H_u = \frac{4c}{\gamma} = \frac{4 \times 20}{18}$
$= 4.44\ m$
$\downarrow$

- for purely cohesive soil, $\phi = 0$

$\therefore \boxed{H_c = \frac{2c}{\gamma} \text{ and } H_u = \frac{4c}{\gamma}}$ → Imp Q. Prove this

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$$K_a = \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \tag{9.15a}$$

The active thrust is given by:

$$✓\ P_a = \frac{1}{2} K_a \gamma H^2 \cos\beta \tag{9.15b}$$

Similarly, the passive earth pressure coefficient is given by

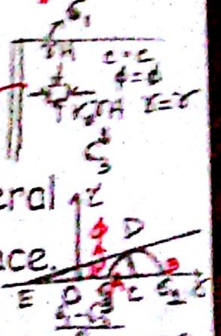
$$✓\ K_p = \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}} \tag{9.16a}$$

And passive thrust is

$$P_a = \frac{1}{2} K_a \gamma H^2 \cos\beta \tag{9.16b}$$

for 9.4.1.3 ⇑

9.4.1.6 Cohesive (c-φ) Backfill with Sloping Surface



Mazindrani and Ganjali (1997) made an analytical solution of lateral earth pressure problem of cohesive backfill with sloping surface. According to them:

$$\sin\phi = \frac{CD}{EC} = \frac{\frac{\sigma_1-\sigma_3}{2}}{EO+OC} = \frac{\frac{\sigma_1-\sigma_3}{2}}{c\cot\phi + \frac{\sigma_1+\sigma_3}{2}}$$

→ formula মনে রাখ

$$K_a = \frac{1}{\cos^2\phi}\left[2\cos^2\beta + \frac{2c}{\gamma}\cos\phi\sin\phi - \sqrt{\left\{4\cos^2\beta\left(\cos^2\beta-\cos^2\phi\right) - \left(\frac{2c}{\gamma z}\right)^2\cos^2\phi - \frac{8c}{\gamma z}\cos^2\beta\cos\phi\sin\phi\right\}}\right]$$

And,

$$\therefore \sin\phi = \frac{\sigma_1-\sigma_3}{2c\cot\phi + \sigma_1+\sigma_3}$$

$$K_p = \frac{1}{\cos^2\phi}\left[2\cos^2\beta + \frac{2c}{\gamma}\cos\phi\sin\phi - \sqrt{\left\{4\cos^2\beta\left(\cos^2\beta-\cos^2\phi\right) + \left(\frac{2c}{\gamma z}\right)^2\cos^2\phi - \frac{8c}{\gamma z}\cos^2\beta\cos\phi\sin\phi\right\}}\right]$$

The active and passive thrusts are given by

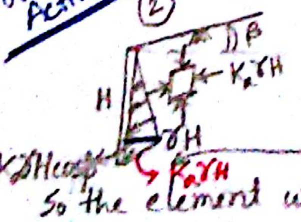
→ for cohesionless soil (Rankine's theory)

Inclined Ground for Active E. press.

$$P_a = \frac{\cos\beta}{\cos^2\phi}\left[\begin{matrix}2\gamma H\cos^2\beta + 2c\cos\phi\sin\phi \\ -\sqrt{\left\{4\cos^2\beta\left(\cos^2\beta-\cos^2\phi\right)\gamma^2H^2 + \left(2c\cos\phi\right)^2 + 8c\gamma H\cos^2\beta\cos\phi\sin\phi\right\}}\end{matrix}\right] - \gamma H\cos$$

→ Q. Draw the pressure distⁿ diagram ⇒ don't follow the text. Use this approach

②

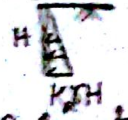


→ Mohr circle for Rhombic element → $\underline{\underline{K_a}} = \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$ and

$K_p = \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}$

So the element will be RHOMBIC

①



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

& for ② ⇒

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

for 9.4.1.5

(g) Triangular load

(h) Area load

✓(a) Point Load

Let Q be a point load acting on the backfill surface as shown in Fig. 9.9. Referring to the Figure Lateral earth pressure along the vertical section at perpendicular distance of load Q is given by:

(i) For $m > 0.4$

$$P_h = \frac{1.77Q}{H^2} \frac{m^2 n^2}{(m^2+n^2)^3}$$

(ii) For $m \leq 0.4$

$$P_h = \frac{0.28Q}{H^2} \frac{n^2}{(0.16 \div n^2)^3}$$

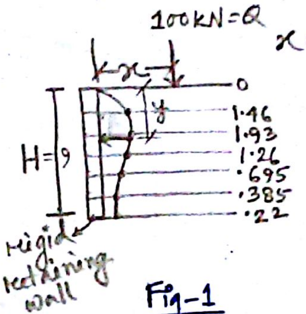


Fig-1

$x = mH$; $y = nH$

⇓

x, y, H known

So from this m, n = known

⇓

Let. $x = 3m$, $y =$

$H = 9m$

⇓

total H কে 6 division এ ভাগ করব।

⇓ min^m

Steps for solving problem :–

1. $m = \frac{x}{H} = \frac{3}{9} = 0.33 < .4$

$P_h = \frac{.28Q}{H^2} \cdot \frac{n^2}{(.16+n^2)^3} = \frac{.28 \times 100}{9^2} \cdot \frac{n^2}{(.16+n^2)^3}$

$\therefore P_h = \frac{.346 n^2}{(.16+n^2)^3}$

Lateral pressure at points along the wall on each side of a perpendicular from the concentrated load Q to the wall is

$$P_h' = P_h \cos^2(1.1\alpha)$$

Q. Imp

for two point loads Q_1 & Q_2 determine P_{hA}

for Q_1, the point is ⊥ to it, so we use direct table

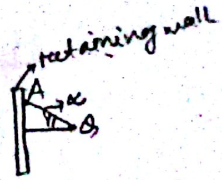
but for Q_2, P_h' will be used

Q_2 its not ⊥ to load.

2. Table –

Depth (m), y	$n = \frac{y}{H}$	$P_h = \frac{.346n^2}{(.16+n^2)^3}$
0	0	0
1·5	·167	1·46
3	·333	1·93
4·5	·5	1·26
6	·667	0·695
-20- 7·5	·833	0·385
9	1·00	0·220

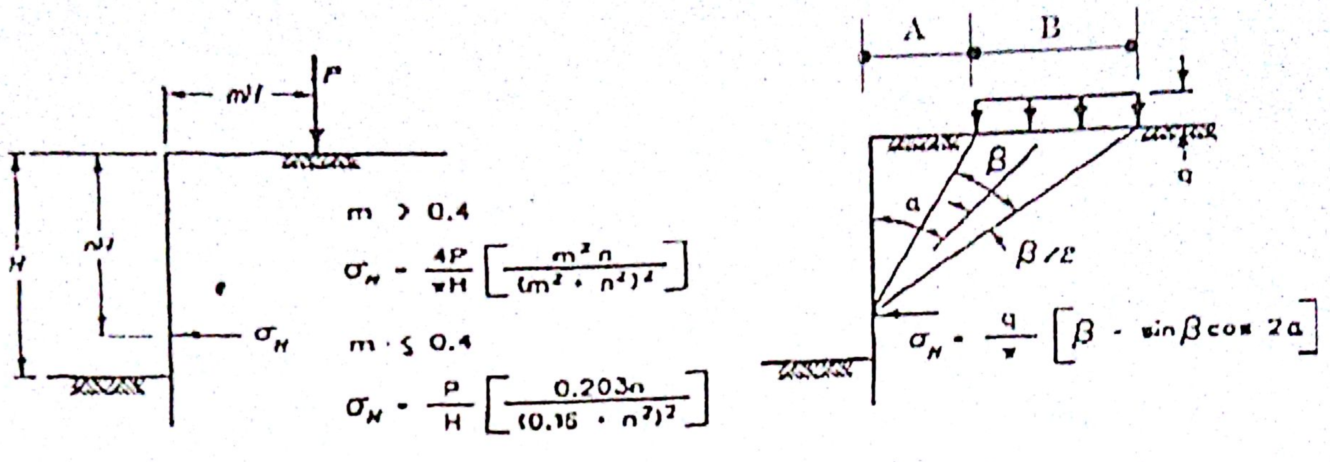
Then total pressure হবে fig-1 এর total area



$P_h' = \cos^2(1.1\alpha) \cdot P_h$

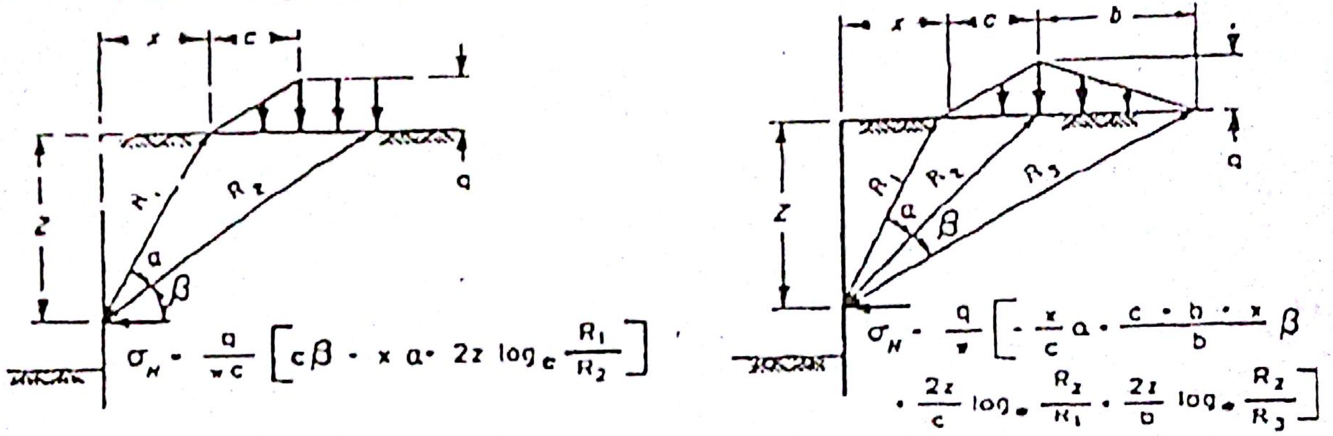
⇓

for determining stress at A, this formula is used.



a. Line load (factor of two included)
from Terzaghi (1954)

b. Strip load



~~c.~~ Ramp load

d. Triangular load

from Dawkins (1991)

NOTES:
(1) FOR FIGURES (c) AND (d) THE ANGLES α AND β ARE EXPRESSED IN UNITS OF RADIANS.
(2) NEGATIVE PRESSURES MAY BE COMPUTED AT SHALLOW DEPTHS (Z).

Fig. 9.10 Lateral Pressure due to other Surcharge Loads; (a) Line Load; (b) Strip Load; (c) Ramp Load; (d) Triangular Load

- only Point & line load-এর formula ⇒ মুখস্থ
- for other loads ⇒ formula will be given.

✱ for other rectangular area-load, some graphs are used for determining distⁿ of stress ⇒ These graphs will be provided in exam

✱ Assume, total load for one floor = 200 − 250 psf → if the the load is not given.

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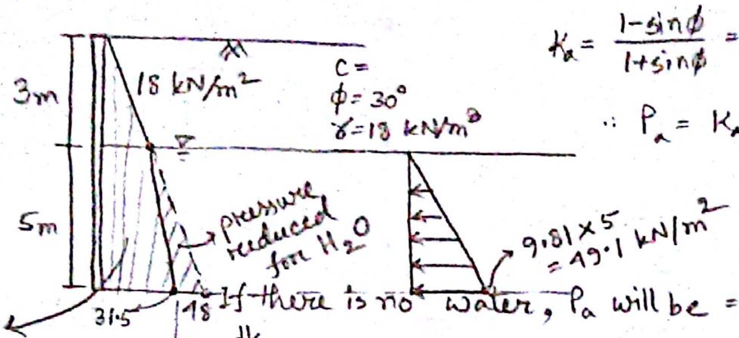
Effect of Water Table on earth pressure:



$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30°}{1+\sin 30°} = 0.333$

$\therefore P_a = K_a \gamma H = 0.333 \times 18 \times 3$

$= 18 \text{ kN/m}^2$

If there is no water, P_a will be $= 0.333 \times 18 \times 8 = 48 \text{ kN/m}^2$

⇓

But for H_2O, for 5m portion, Here γ' / submerged unit weight is there.

$\therefore P = 0.333 \times 18 \times 3 + 0.333 \times (18 - 9.81) \times 5 = 31.5 \text{ kN/m}^2$

Total pressure force

⇓

Q. [illegible] what is the language

Active Thrust ⇒ only soil pressure $= 31.5 \text{ kN/m}^2$

Total thrust $= 31.5 +$ Water pressure

$= 31.5 + 49.1$

$= 80.6$

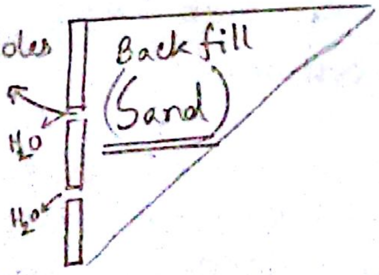
⇓

So we will not allow to retain H_2O beyond retaining wall.

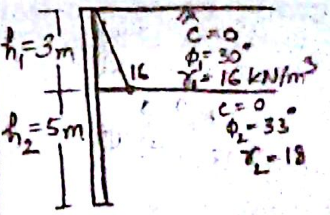
Q. What's the use of ⇒ Weep Holes

⇓

Ans ⇓ So that H_2O pressure can't act on retaining wall.



Effect of soil Streatification:



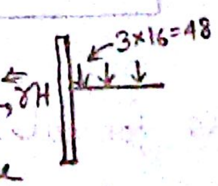
for layer - I

$K_{a_1} = \frac{1-\sin\phi_1}{1+\sin\phi_1} = \frac{1-\sin 30°}{1+\sin 30°} = \frac{1}{3}$

$P_{a_1}(h_1) = K_{a_1} \gamma_1 h_1$

$P_{a_1}(h_1 = 0) = 0$

$P_{a_1}(h_1 = 3) = 0.333 \times 16 \times 3 = 16 \text{ kN/m}^2$



Here is a pressure for type I soil on the type II soil

⇓

We will calculate how much of type II needed for not having the pressure of type I

⇓

$\boxed{\text{Equivalent Height}}, H_e = \frac{q}{\gamma_2} = \frac{48}{18}$

$= 2.67\text{m}$

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