University of Baghdad
College of Engineering
Soil Mechanics Laboratory (2016-2017)
3rd year

Experiment No. 1: Water Content & Atterberg Limits Test:

Moisture in soils:

Naturally occurring soils nearly always contain water as a part of their structure. The moisture content of a soil is assumed to be the amount of water within the pore space between the soil grains which is removable by oven drying at (105-110)°C, expressed as a percentage of the mass of dry soil. By "dry" is meant the result of oven drying at that temperature to constant mass, usually for a period of about (12-14) hrs. In non-cohesive granular soils this procedure removes all water present. A drying temperature of (105-110) °C is specified as the standard procedure, and this should be used as a general rule. However, this temperature may be too high for certain types of soils. For peats and soils containing organic matter a drying temperature of 60°C is to be preferred to prevent oxidation of the organic content. For soils containing gypsum the water of crystallization may be lost at temperature above 100°C, so a temperature not exceeding 80°C should be used or others used 45°C as a drying temperature. The same can apply to some soils from tropical regions. These lower temperatures may require long drying times.

There are several ways in which water is held in cohesive soils, a simplified illustration of the zones of water surrounding a clay particle is obtained by considering the following five categories water, illustrated diagrammatically in **Figure (1.1)**.

Types of water in soil sample:

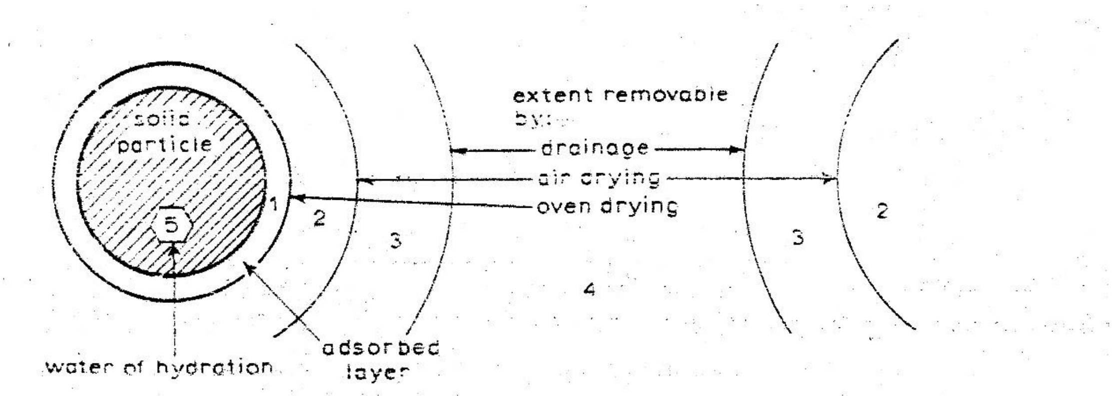


Figure (1.1): Types of water in soil sample.

1. Adsorbed water held on the surface of the particle by powerful forces of electrical attraction and virtually in a solid state. This layer is very small thickness, perhaps of the order of 0.005μm. This water cannot removed by oven drying at 110°C, and May therefore, be considered to be part of the solid soil grain.
2. Water which is not tightly held and can be removed by oven drying, but not by air drying.
3. Capillary water, held by surface tension, generally removed by air drying.
4. Gravitational water, which can move in the voids between soil grains and is removable by drainage.

5. Chemically combined water, in form of water of hydration within the crystal structure. Expect for gypsum and some tropical clay, this water is not generally removable by oven drying.

Purpose of the test:

- To determine the moisture content of the soil in situ.
- To determine the plasticity and shrinkage limits of fine grained soil.
- To measure the moisture content of samples used for laboratory testing before and after the test.

So, *Water content*: is defined as the ratio of the weight of water to the weight of dry soil grains in the mass.

$$\omega, \% = \frac{weight\ of\ water}{weight\ of\ dry\ solid} \times 100$$

Apparatus:

1. Controlled drying oven.
2. Desiccator container.
3. Balance (0.01 gm sensitivity).
4. Containers.

Testing procedure:

1. Weighing containers.
2. Selection of samples(the approximate mass of specimen required for different soil types is as follows:
   - Homogeneous clays and silts ≈ 30 gm
   - Medium-grained soils ≈300 gm
   - Coarse-grained soils ≈3 kg
3. Wet weighing (depending on sample type).
4. Oven drying (depending on sample type also).
5. Cooling in the desiccators.
6. Dry weighing.
7. Calculation:

$$\omega, \% = \frac{weight\ of\ water}{weight\ of\ dry\ soil\ mass} = \frac{w_2 - w_3}{w_3 - w_1} \times 100$$

Where:

w_1: Weight of empty container, gm,

w_2: Weight of container+ wet soil, gm and

w_3: Weight of container+ dry soil, gm.

Results and discussion:

- Engineering significance of water content.
- Drying temperature.
- Quantity of sample.
- Presence of salts and organic matters.
- Drying period.
- Values of the Test results.

The Atterberg Limits:

The condition of a clay soil can be altered by changing the moisture content; the softening of clay by the addition of water is a well known example. For clayey soil there is a range of moisture contents within which the clay is of a plastic consistency, and the Atterberg limits provide a means of measuring and describing the plasticity range in numerical terms as shown in **Figure (1.2)**.

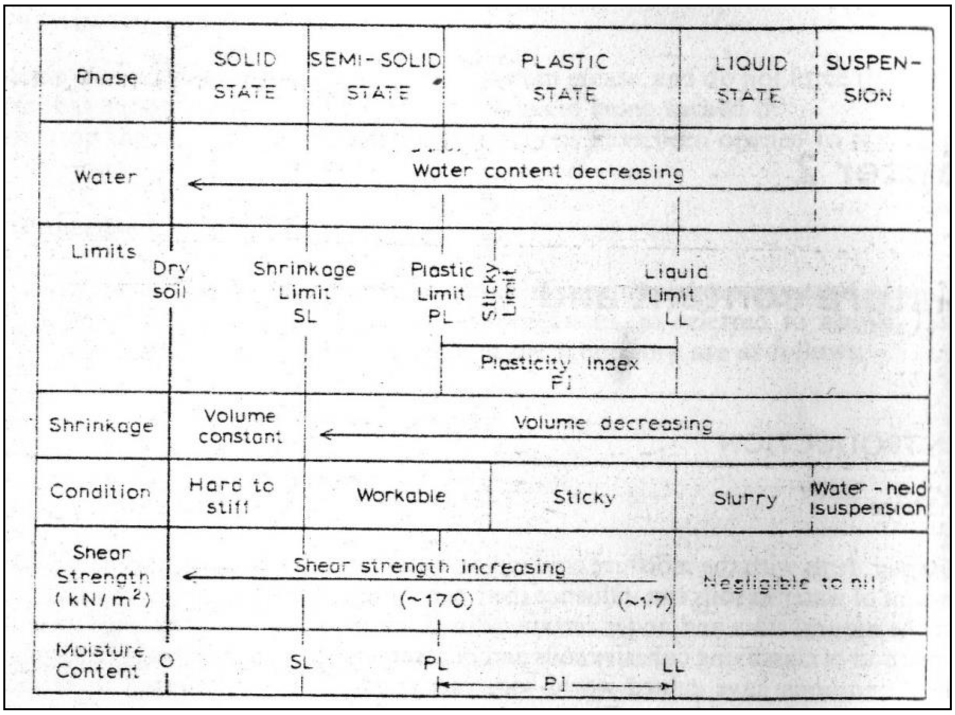


Figure (1.2): Phases of soil and the Atterberg Limits.

Definitions:

- **Liquid limit:** the water content in a percent of a soil at the boundary between liquid state and plastic state. At this water content the soil has a small shear strength that it flows to close a groove of standard width when tested in a Casagrande device.
- **Plastic limit:** the water content in percent of the soil at the boundary between plastic state and semi solid state. At this water content the soil begins to crumble when rolled into threads of specified size.

- **Shrinkage limit:** is the moisture content at which further loss of water in the soil will not cause further in the volume of the soil.

Significance and use:

- These limits are used to classify the fined grained soils in groups (see USCS).
- Liquid limit and plastic limit, individually or together used to make correlation with other soil behavior parameters such as compressibility, permeability, compatibility, shrinkage and swell and shear strength.
- Finding the following indices:
  - ✓ Plasticity Index, P.I. %
  - ✓ Liquidity Index, L.I. or B,
  - ✓ Activity, A, %.

Liquid Limit:

Apparatus:

1. A mechanical device (Casagrande apparatus) as shown in **Figure (1.3)**. The cup must fall freely, which is lifted 10 mm above the base. This can be checked with the spacer gauge (a steel block 10mm thick or a block on the handle of the grooving tool) which should just pass between cup and base when the cup is at its maximum height. The adjusting screw provides a simple means of this adjustment.

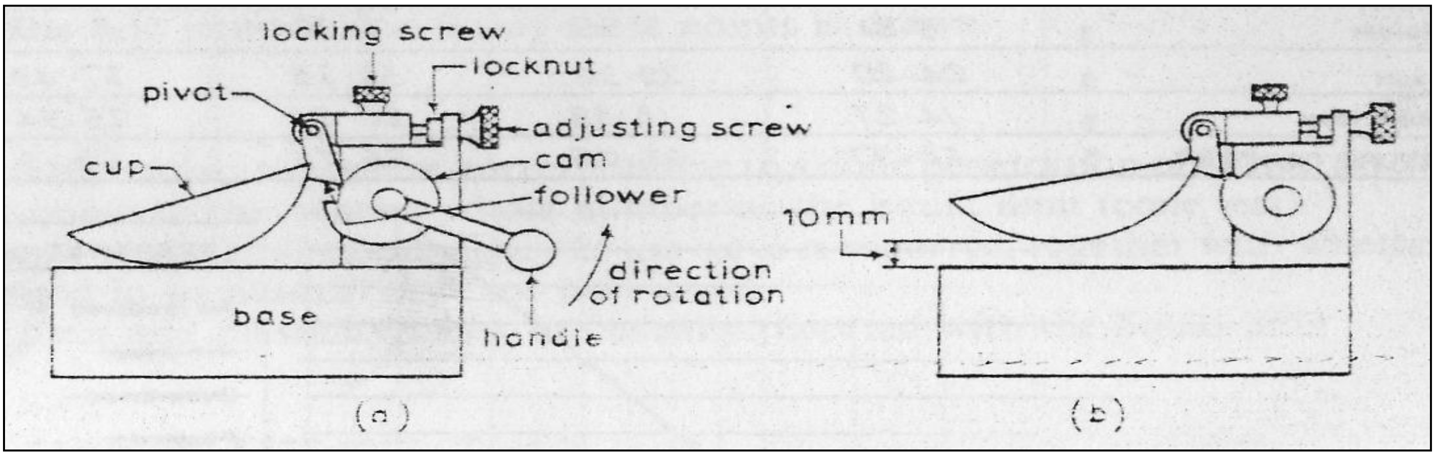


Figure (1.3): Principle of Casagrande device.

2. Grooving tool.
3. Flat glass plate (10 mm-thicknesses).
4. Wash bottle containing distilled water.
5. Spatula.
6. Standard moisture content test apparatus.

Procedure:

1. A sample of 200-250 gm of soil passing the 0.425mm (425µm) sieve is required.

2. Place the soil on the glass plate and mix thoroughly with a little distilled water. Set aside a small portion in a sealed bag before adding too much water for plastic limit test.
3. Place the paste into an air tight container or bag and seal it. Leave standing for a curing period of 24 hours to allow water to permeate through the soil mass.
4. Adjust the apparatus.
5. Remove the soil from the container after maturing and remix with the spatula for at least 10 min.
6. Place the paste in cup: A) press the soil paste against the side of the cup to avoid trapping air. B)press more soil paste well into the bottom of the cup to maximum depth of 10 mm or 1/2") level the surface of the soil paste with a small spatula as shown in **Figure(1.4-a)**.

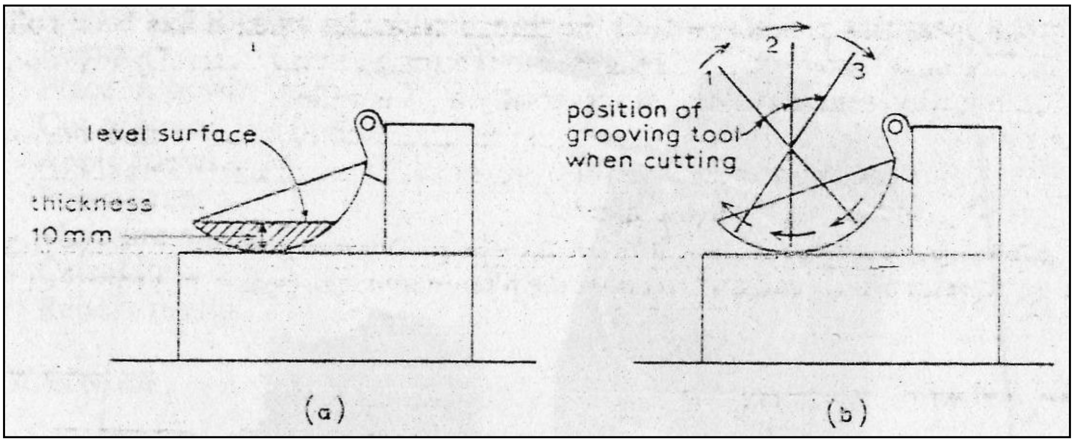


Figure (1.4): Soil placed in Casagrande cup and use of grooving tool.

7. A groove is cut through the sample from back to front along the symmetrical axis of the cup.
8. Turn the crank at a rate of about 2revolutions per second and count the blows necessary to close the groove in the soil for a distance of 13 mm. The groove should be closed by the plastic flow of soil not by slippage between the soil and the cup as shown in Figure (1.4-b).
9. When a consistent value in the range of (15-35) blows has been obtained, take a small quantity (about 10 gm) from the zone of the groove and place it in a moisture content container for water content determination.
10. By altering the water content of the soil and repeating steps obtain four water content determinations in the range of (15-35) blows.
11. Make a plot of water content against log of blows such a plot know as a flow curve is usually approximately a straight line.

Plastic limit

Procedure:

1. Mix thoroughly about 15 gm of the moist soil.
2. Roll the soil on a glass plate with the hand until it reaches (1/8)" diameter.
3. Repeat step 2 until a (1/8)" diameter thread shows signs of crumbling.

4. Take some of the crumbling material obtained in step 3 for a water content determination

Calculations:

The limits, expressed as water contents, are obtained as follows:

1. Liquid limit, L.L., is read from the flow curve as the water content at twenty five (25) blows.
2. Plastic limit, P.L., is the water content of the soil which crumbled when threads in 1/8" diameter were rolled.
3. Water plasticity ratio or liquidity index, B, is found from the equation:

$$L.I.\ or\ B = \frac{w_n - P.L.}{L.L. - P.L.}$$

In which w_n = Natural water content.

4. Plasticity index, P.I.:

$$P.I. = L.L. - P.L.$$

5. Flow index, I_f, =slope of flow curve
6. Toughness index, I_t,

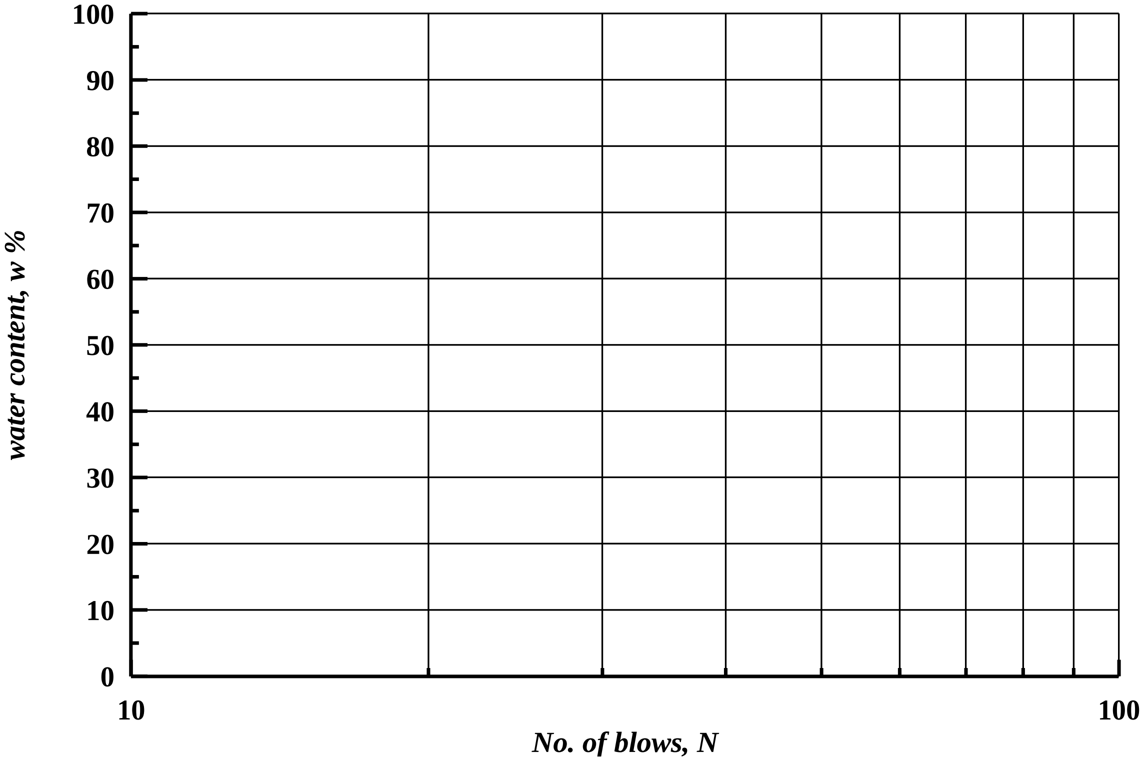
$$I_t = \frac{Plasticity\ Index}{Flow\ Index}$$

7. Activity, A ,

$$A = \frac{Plasticity\ Index}{\%\ clay\ fraction}$$

Liquid limit Determination:

Trial No.					
Container No.					
wt of container (gm)					
Wt of wet soil +container (gm)					
Wt of dry soil + container (gm)					
Water content, ω %					
No. of blows, N					



Plastic Limit Determination:

Determination No.			
Container No.			
wt of container (gm)			
Wt of wet soil +container (gm)			
Wt of dry soil + container (gm)			
Water content, ω %			

L.L., %=--------- **P.L., %=---------** **P.I., %=---------**

L.I., %=---------- **I_f=---------** **I_t=------**

Experiment No. 2: Specific Gravity Test:

Specific Gravity:

The specific gravity of the soil is the ratio of the weight in air of a given volume of soil particles to the weight in air of an equal volume of distilled water at a temperature of 4°C.

$$G_s = \frac{weight\ in\ air\ of\ a\ given\ volume\ of\ soil\ particle}{weight\ in\ air\ of\ an\ equal\ volume\ of\ distilled\ water\ at\ 4\ ^{\circ}C}$$

$$G_s = \frac{\gamma_s}{\gamma_w at\ 4\ ^{\circ}C}$$

Where :

G_s: specific gravity of soil grains,

γ_s: unit weight of the soil grains(kN/m^3).

γ_w: unit weight of water at 4°C(kN/m^3).

Purpose of the test:

- From specific gravity, void ratio and degree of saturation we can compute the unit weight of the soil.
- The specific gravity is used in calculation of some important properties such as porosity, void ratio, ….etc.
- The specific gravity is used in the computation of most laboratory tests.

Tests methods:

1- **By vacuum:**

Apparatus:

1. Density bottle (50 ml) with stopper.
2. Constant temperature water bath (usually 25°C).
3. Vacuum desiccators.
4. Drying oven.
5. Balance(0.01 gm sensitivity)
6. Source of vacuum and vacuum tubing.
7. Distilled water, wash bottle, rubber, gloves,…..etc.

Test procedure:

- Prepare the density bottle (w1) wash, clean, dry in oven (105-110) ºC.
- Preparation of test specimen between (10-30) gm, oven (105-110) ºC, cooling.
- Placing in density bottles, weigh each density bottle (w2).
- Adding liquid and applying vacuum used de-aired distilled water half full for 24 min.
- Removal of air.
- Transfer to constant temperature bath half an hour.
- Weighing w3.
- Weighing bottle with liquid (w4).
- Calculation:

$$G_s = \frac{(w2 - w1)}{(w4 - w1) - (w3 - w2)} \times G_t$$

Where:

G_s: specific gravity of the liquid.
w_1: weight of density bottle.
w_2: weight of density bottle + dry soil.
w_3: weight of density bottle + dry soil + liquid.
w_4: weight of density bottle + liquid only.

2- By heating:

Apparatus:

1. Pycnometer (volumetric bottle).
2. Constant temperature water bath (usually 25°C).
3. Heat source (such as a burner or hot plate).
4. Desiccators.
5. Drying oven.
6. Balance (0.01 gm sensitivity)
7. Thermometer (graduated to 0.1 °C).
8. Distilled water, wash bottle, pipette, gloves,…..etc.

Test procedure:

- Prepare the pycnometer (w1) wash, clean, dry in oven (105-110) ºC.
- Put approximately 50 gm of oven-dry soil into a calibrated pycnometer which is already half full of de-aired distilled water
- Remove all of the air which is entrapped in the soil by 10 minutes of boiling using a burner or hot plate accompany with agitation.
- Cool the pycnometer to some temperature within the range of the calibration curve for the pycnometer
- Add water to bring the bottom of meniscus to the calibration mark
- Dry the outside of the pycnometer and the inside of the neck above the meniscus.
- Weigh the pycnometer with water and soil
- Weighing bottle with liquid (w4).
- Calculation:

$$G_s = \frac{(w2 - w1)}{(w4 - w1) - (w3 - w2)} \times G_t$$

Where:

G_s: specific gravity of the liquid.

w_1: weight of density bottle.

w_2: weight of density bottle + dry soil.

w_3: weight of density bottle + dry soil + liquid.

w_4: weight of density bottle + liquid only.

Typical values between (2.65-2.85).

Type of soil	Specific gravity
Ottawa sand 2.67[1]	2.67
Diatomaceous earth 2.0[1]	2.0
Boston blue clay 2.78[1]	2.78
Mexico city clay 2.35[1]	2.35
Leda clay[1]	2.77
Sand[2]	2.65-2.67
Silty sand[2]	2.67-2.70
Inorganic clay[2]	2.70-2.80
Soil with mica and iron[2]	2.75-3.0
Organic soils[2]	Variable but may be under 2.00

[1]: typical values from soil testing for engineers, T. William Lambe (1951)

[2]: typical values from ASTM D854-02.

Notes:

The specific gravity of soil solid containing extraneous matter such as cement, lime, water soluble matter such as sodium, chloride, gypsum and soil containing matter with a specific gravity less than one, require special treatment.

Calibration Curve of Density Bottle or Pycnometer:

In the computation of the specific gravity of a soil from laboratory data, the weight of the Pycnometer filled with distilled water at the test temperature will be needed. This value is usually taken from a plot of temperature versus weight of Pycnometer plus water. The plot or calibration curve can be determined either by experimental or by theoretical means:

1- Experimental procedure:

This procedure consists of obtaining at least three sets of concurrent temperature and weight measurements about 4°C apart and within the temperature range of (20-30) °C. Each set, representing the coordinates for a point on the calibration curve, is obtained as follows:

a- To a clean Pycnometer add de-aired, distilled water at room temperature until the bottom of the meniscus is at calibration mark.
b- Dry the outside of the bottle and the inside of the neck above the water surface.
c- Weigh the bottle plus water to 0.01 gm.
d- Measure the water temperature to 0.1°C.
e- Heat the bottle of water slightly by placing it in a warm water bath and repeat steps (b-d), each time removing enough water to bring the meniscus down to the calibration mark. Repeat this procedure until enough points are obtained to plot the calibration curve.

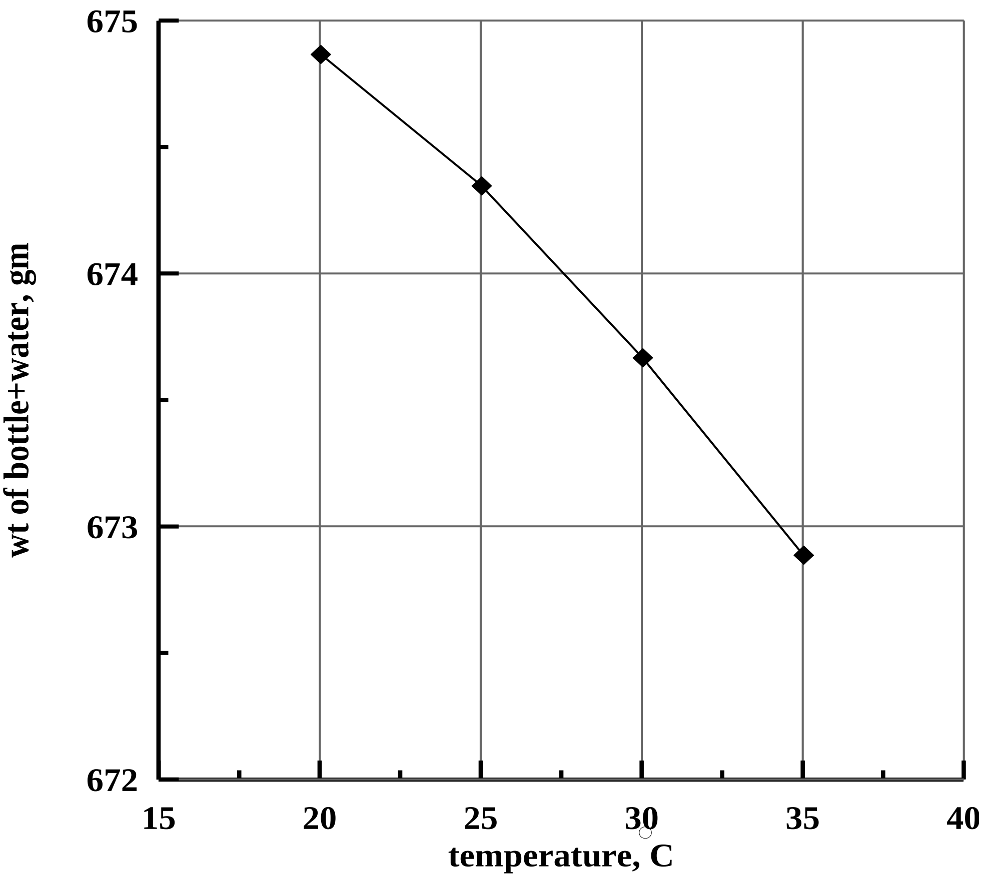


Figure (2.2): example of calibration curve.

2- Theoretical Procedure:

Points for the calibration curve can be obtained by successively substituting different temperatures in the following equation:

$$w_4 = w_B + V_B(1 + \Delta T . \epsilon)(\gamma_T - \gamma_a)$$

In which:

$w_4 = w_2 = weight\ of\ pycnometer\ plus\ water\ at\ temperature\ T,$
$w_B = weight\ of\ clean\ dry\ bottle,$
$V_B = calibrated\ volume\ of\ bottle\ at\ T_c\ ,$
$V_B = volume\ of\ bottle\ at\ temperature, T$

$$\Delta T = T - T_c$$

$T = temperature\ in\ degrees\ centigrade\ at\ which\ w_2\ is\ desired.$
$T_c = calibration\ temperatre\ of\ pycnometer(usually\ 20\ °C).$
$\epsilon = thermal\ coefficient\ of\ cubical\ expansion\ for\ pyrex\ glass$
$= 0.1 \times 10^{-4}\ per\ °C.$

γ_T = unit weight of water at T (See the Textbook "Soil Testing for Engineers").

γ_a = unit weight of air at T and atmospheric pressure
(an average value of γ_a accurate enough for use in this test is 0.0012 gper cc.)

The only bit of laboratory data needed is the weight of the Pycnometer, which must be absolutely clean and dry. A good practice is to obtain one check point for the calibration curve by the previously described experimental method.

From the foregoing definition:

$$w_4 = w_B + V_{BT}\gamma_T - V_{BT}\gamma_a$$
$$w_4 = w_B + V_{BT}(\gamma_T - \gamma_a)$$
$$w_4 = w_B + [(V_B + V_B(\Delta T.\epsilon)](\gamma_T - \gamma_a)$$
$$w_4 = w_B + V_B(1 + \Delta T.\epsilon)(\gamma_T - \gamma_a)$$

Specific Gravity Data Sheet

1. Pycnometer Calibration:

A. Experimental Procedure:

Determination No.	**1**	**2**	**3**	**4**
Wt of bottle+water, w_4 in gm				
Temperature T in $^{\circ}C$				

B. Theoretical Procedure:

Wt. bottle, W_B in gm				
Temperature of calibration T_C				
Volume of bottle at T_C V_B in cc				
Determination No.	**1**	**2**	**3**	**4**
Temperature T in $^{\circ}C$				
Unit weight of water at T_C, gm/cm^3				
Wt of bottle+water, w_4 in gm				

1. Specific gravity Data Sheet:

Determination No.	**1**	**2**	**3**	**4**
Bottle No.				
Wt. bottle+water+soil, w_1, in gm				
Temperature, T , in gm				
Wt. of soil, w_s, gm				
Specific gravity of water at T, G_T				
Specific gravity of soil, G_s				

Remarks:

Experiment No. 3: Grain Size Analysis:

a) Grain Size Analysis.
b) Hydrometer Analysis.

In general, the soil may be divided on the base of their size into seven categories which are:

Table (1): Grain size ranges according to several engineering soil classification systems.

Classification System	***Grains Size (mm)***						
	Boulders	***Cobbles***	***Gravel***	***Sand***	***Silt***	***Clay***	***colloids***
ASTM	**>300**	**300-75**	**75-4.75**	**4.75-0.075**	**0.075-0.005**	**0.005-0.001**	**<0.001**
AASHTO	**>75**	**---------**	**75-2**	**2-0.075**	**0.075-0.005**	**0.005-0.001**	**<0.001**
USCS	**>300**	**300-75**	**75-4.75**	**4.75-0.075**	**<0.075**		
MIT	**>200**	**200-60**	**60-2.0**	**2.0-0.06**	**0.06-0.002**	**<0.002**	

Purposes:

1- Grain size curves have been used in identification and classification of soils.
2- Soils used for the construction of embankments and earth dams are required to be within specified limit as defined by particle size distribution curve.
3- The method of designing filters for dams, levees uses particle size distribution of the soil involved.

a) Grain Size Analysis:

Apparatus:

1- Test sieve
2- Mechanical sieve shaker
3- Balance (0.01 gm sensitivity).
4- Drying oven (105-110) ºC.
5- Sieve brushes and metal trays.
6- Mortar and rubber tipped pestle.
7- Scoop and miscellaneous small tools.

Procedure:

1- Weigh each sieve.
2- Select a soil sample (500 gm).
3- Select a suitable sieve set.
4- Sieve the soil through a nest of sieve by mechanical shaker or hand.
5- Weigh each sieve and the pan with the soil retained on them.
6- Calculation:

$$\%\, retained = \frac{wt\ of\ soil\ retained}{total\ wt} \times 100$$

$$\%passing = 100 - \sum \%\, retained$$

$$Coefficient\ of\ Uniformity, C_u = \frac{D_{60}}{D_{10}}$$

$$Coeficient\ of\ curvature, C_c = \frac{(D_{30})^2}{D_{10}\,D_{60}}$$

Where:
C_u: $Coefficient\ of\ Uniformity.$
C_c: $Coeficient\ of\ curvatur.$
D_{60}: Diameter of particle
D_{30}: Diameter of particle
D_{10} : Diameter of particle.

The most useful sieves:

Sieve No.	Sieve opening (mm)
No. 4	4.75
No. 8	2.38
No. 10	2.00
No. 18	1.00
No. 20	0.840
No. 40	0.425
No. 100	0.149
No. 200	0.075

Example:

Sieve No.	Opening (mm).	Mass retained	% retained	Cumulative retained	% Passing
4	4.75	0	0	0	100
10	2.00	40	5.49	5.49	94.51
20	0.850	60	8.23	13.72	86.28
40	0.425	89	12.21	25.93	74.07
60	0.250	140	19.20	45.13	54.87
80	0.180	122	16.74	61.87	38.13
100	0.150	210	28.81	90.68	9.32
200	0.075	56	7.68	98.36	1.64
Pan	-	12	1.65	100.0	0
		$\sum$W= 729			

Notes:

Comparison between AASHTO and USCS

1- They both based on the texture and plasticity.
2- Both of them divided into main categories coarse and fine grained soil.
3- In an AASHTO system, the soil is considered fine grained when (35%) passing sieve no. 200.
4- In USCS, the soil is considered fine grained when more than (50%) passing sieve 200.
5- In an AASHTO sieve No. 10 is used to separate gravel from sand.

6- In an USCS sieve No. 4 is used to separate gravel from sand;
7- The classification of organic soil and peat is provided in USCS but in AASHTO there is no place for organic and peat soil (peat soil: high moisture content, low specific gravity and low unit weight).

b)Hydrometer Analysis:

Apparatus:

1- Hydrometer.
2- Mixer.
3- Dispersion agent (Sodium HexametaPhosphate ($NaPO3)_n$)
4- Constant temperature bath.
5- Two graduated cylinders (1 liter, 1000 ml capacity).
6- Thermometer (graduated 0.1 ºC).
7- Timer.
8- Drying oven.
9- Others (spatula, desiccators…etc.).

Procedure:

1- Take 50 gm of oven dry soil sample passing sieve No.200, then mixed with 125 cm^3 of solution (4 %) in the cup of mixing machine.
2- After mixing, wash the specimen into a graduated cylinder and add enough distilled water to bring the level to the 1000 cm^3 mark.
3- Mix the soil and water in the graduate cylinder by placing the palm of the hand over the open end and turning the cylinder upside down and back.
4- Place the cylinder on the table, insert the hydrometer in the suspension and start the timer.
5- Take the hydrometer readings at total elapsed times of 1/4, 1/2, 1 and 2 minutes without removing the hydrometer. The suspension should be mixed and this set of four readings repeated until a consistent pair of sets has been obtained.
6- After the 2 minutes reading, remove hydrometer, remix and restart the test but take no reading until the 2 minutes gone.
7- Take hydrometer readings at total elapsed time intervals : 2,
Also take temperature of the suspension every 20 minutes.

8- Take temperature observations and hydrometer readings in the jar of distilled water every (20, 30) minutes.
9- Obtain the height of meniscus rise of pure distilled water on the stem of the hydrometer.
10- Continue taking observation until the hydrometer reads approximately one.

Calculations:

a) **Equivalent particle diameter D:**
The equivalent particle diameter at a known depth and after a certain time interval from the start of sedimentation can be calculated from the following equation:

$$D = \sqrt{\frac{18\mu}{\gamma_s - \gamma_w}} . \sqrt{\frac{Z_r}{t}}$$

$$D(mm) = \sqrt{\frac{18\mu}{\gamma_s - \gamma_w}} . \sqrt{\frac{Z_r}{t}} = k\sqrt{\frac{Z_r(cm)}{t(min)}}$$

Where:
D= Effective diameter (mm).
μ= Dynamic viscosity of water at the temperature of the test. (millipascal.sec=centipoises). *This value can be gotten from Soil Testing for Engineers of T. William Lambe (page 36).*
γ_s= Unit weight of the soil grains.
γ_w= Unit weight of water at the temperature of the test.
Z_r= (take from calibration curve) Distance from the surface of suspension to the center of volume of the hydrometer (Z_r must be corrected for meniscus correction).
t=Total elapsed time (min.).

b) Percentage finer (percentage smaller than D):

The percentage finer is denoted by N and can be calculated by equation:

$$N = \frac{G_s}{G_s - 1} . \frac{V}{w_s} . \gamma_C (r - r_w) \times 100\%$$

N= Percentage finer, %.
G_s= Specific gravity of solids.
V= Volume of the suspension (1000 cm^3).
w_s= weight of dry soil (50 gm).
γ_c= unit weight of water at temperature of hydrometer calibration (usually 20ºC) see Table A-2, Page 147 of *Soil Testing for Engineers of T. William Lambe*.
r= hydrometer reading in suspension.
r_w= hydrometer reading in water at the same temperature of suspension.

$$N' = N \times \frac{w_1}{w_2}$$

N'= Corrected percentage finer.
w_1= weight of dry soil passes sieve No. 200.
w_2= weight of dry soil used for combined sieve analysis.

Calibration of Hydrometer:

$$First\ point: \quad Z_{r1} = L + N + \frac{h}{2} \quad \rightarrow \quad r1(0.995)$$

$$Second\ point: \quad Z_{r2} = N + \frac{h}{2} \quad \rightarrow \quad r2(1.03)$$

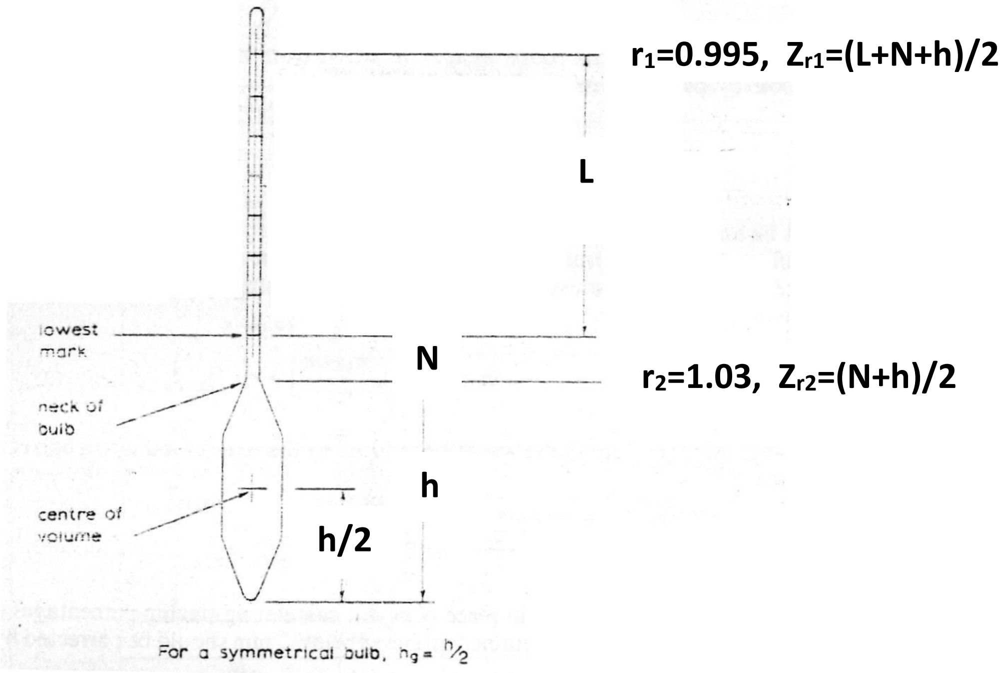


Figure (4.1): Hydrometer.

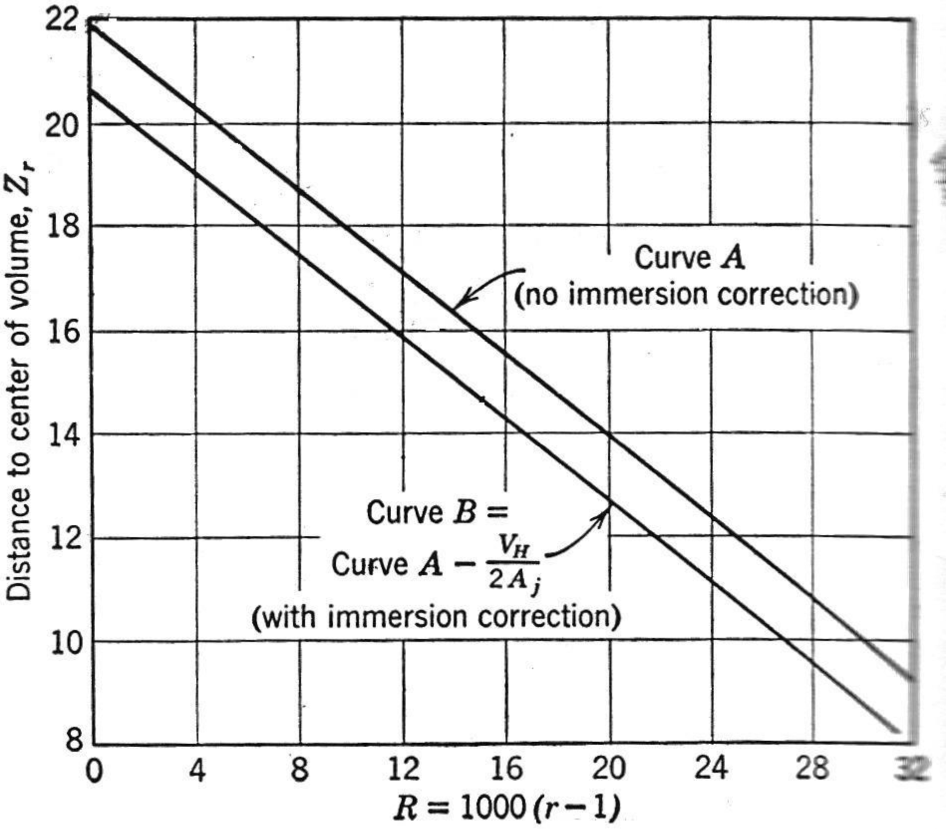


Figure (4.2): Calibration curve of Hydrometer.

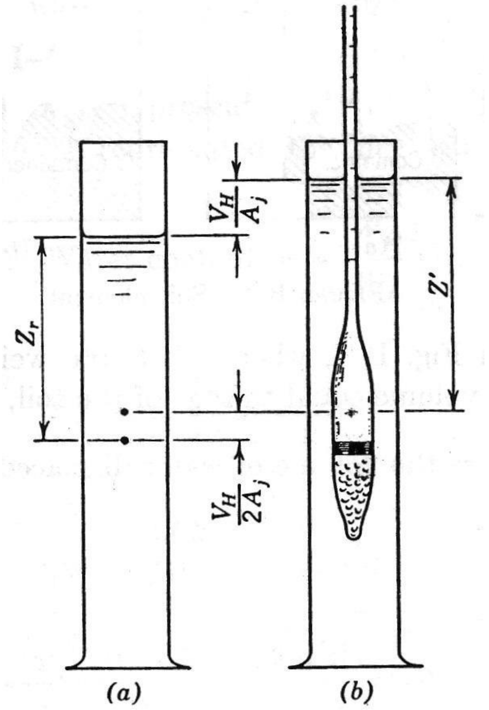


Figure (4.3): Effect of immersion of Hydrometer.

The corrected reading for immersion is:

$$Z_r' = Z_r - \frac{V_H}{2A_j}$$

Where:

V_H= volume of hydrometer, cm3
A_j= area of the graduated cylinder, cm2

Correction for Hydrometer reading:

For each reading Z_r or hydrometer reading must be corrected for four corrections. These are:

a) Meniscus correction, C_M
b) Temperature correction, M_t
c) Dispersion agent correction, χ
d) Water density correction, C_W

a) Meniscus correction, C_M:
The meniscus correction has to be added to hydrometer reading because density reading on the stem increase downwards.

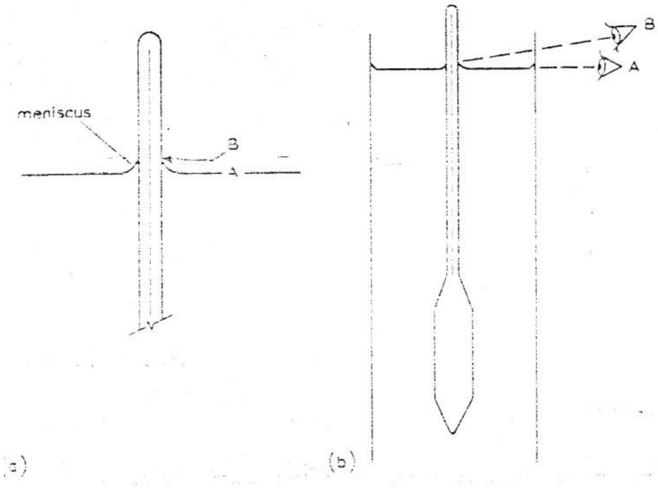
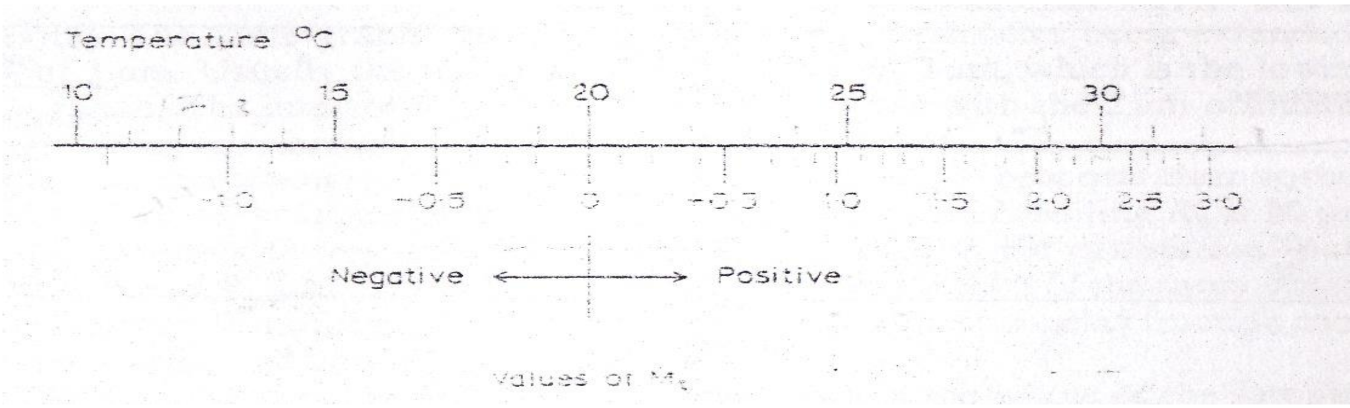


Figure (4.4): Meniscus correction of Hydrometer.

$$C_M = (A - B) \times 1000$$

If reading A=0.998
B=0.995
C_M= +0.5

b) Temperature correction, M_t:
Hydrometer is usually calibrated at 20ºC. If a test is carried out at a different temperature, both the density of water and the density of the hydrometer will be different. The value of M_t is added to the true hydrometer reading algebraically according to the sign (plus or minus).



The addition of the dispersion agent results in the density of liquid in which sedimentation take place being greater than that of water.
$\chi=2m_d$ where m_d is the dry weight of 50 ml of the prepared Dispersion agent. The correction is always subtracted from the Z_r value. Typical value of x (3.5-4.0) for the standard dispersant solution.
$\chi=3.5$

d) Water density correction, C_W:
The British standard specifies that the scale of the hydrometer shall be calibrated in g/ml to read 1000 at 20ºC. The density of pure water is 1.0 g/cm^3 at 4ºC. At 20ºC the density

0.9982 g/cm^3 which would correspond to a Z_r value on the hydrometer of (-1.8). Therefore, all hydrometer reading must be increased by (1.8).

C_W =1.8 (this value is added in any temperature).

The fully corrected hydrometer reading Z_r' is given by:

$$Z_r' = Z_r + C_M + M_t + x + 1.8$$

Hydrometer reading correction:

Temp.,ºC	C_M	M_t	C_W	χ	Total correction
15	+0.5	-0.75	+1.8	-3.5	-1.95
20	+0.5	0	+1.8	-3.5	-1.2
25	+0.5	+1	+1.8	-3.5	-0.2
30	+0.5	+2.3	+1.8	-3.5	+1.1

Note: It is interesting to observe that @25ºC the total correction is -0.2 which for most practical purposes may be taken as zero.

Note: Fully corrected reading used to determine percentage finer only while reading corrected for meniscus only was used for computing the particle diameter D.

the specific gravity scale multiplied by the unit weight of water.

After determining the immersion correction, $V_H/2A_j$, plot a curve, such as curve B, Fig. B–7, by subtracting the correction from curve A. When using Eq. IV–1, read curve A for the observations in the first two minutes and read curve B for the other observations. Allowance should be made for the rise of meniscus on the hydrometer stem before entering the hydrometer calibration curve. This allowance is made by numerically adding the height of meniscus rise, as determined in pure water at the temperature of the test, to the hydrometer reading.

Equation IV–1.

The equation comes from Stokes' law for the terminal velocity, v, of a freely falling sphere, which is,

$$v = \frac{\gamma_s - \gamma_w}{18\mu} D^2$$

where D = the sphere diameter

or

$$D = \sqrt{\frac{18\mu}{\gamma_s - \gamma_w}} \sqrt{v}$$

The velocity of particle fall is also equal to the distance of fall, Z_r, divided by the elapsed time, t, or

$$v = \frac{Z_r}{t}$$

Therefore,

$$D = \sqrt{\frac{18\mu}{\gamma_s - \gamma_w}} \sqrt{\frac{Z_r}{t}} \qquad \text{(IV–1)}$$

Equation IV–2.

If a soil-water suspension is completely mixed, the unit weight, γ_i, of the suspension is

$$\gamma_i = \frac{W_s}{V} + \left(\gamma_w - \frac{W_s}{GV}\right)$$

where W_s = the weight of soil,
V = the volume of container (i.e., graduate),

or

$$\gamma_i = \gamma_w + \frac{G-1}{G}\frac{W_s}{V}$$

If the suspension is allowed to settle, at any time, t, and down a distance, Z, the distribution of all grains finer than D, which is the diameter of the smallest grain which could have fallen Z in t, remains unchanged. Let N_1 be the ratio of the weight of soil grains smaller than D to the total weight of the soil. The weight of solids per unit of volume at depth Z and time t is N_1W_s/V, and the suspension unit weight, γ, is

$$\gamma = \gamma_w + \frac{G-1}{G}\frac{N_1W_s}{V}$$

or

$$N_1 = \frac{G}{G-1}\frac{V}{W_s}(\gamma - \gamma_w)$$

or, in percentage,[4]

$$N = \frac{G}{G-1}\frac{V}{W_s}\gamma_c(r - r_w) \times 100\% \qquad \text{(IV–2)}$$

It should be noted that in Eq. IV–2 the difference of hydrometer readings in the suspension and distilled water, or $r - r_w$, cancels any effect of temperature or meniscus rise on the stem.

Equation IV–3 (see Fig. B–8).

In Fig. B–8,

W_s = total soil weight used in combined analysis
W_1 = weight of soil finer than No. 200 sieve
W_2 = weight of soil used in hydrometer analysis

When W_2 is used for W_s in Eq. IV–2 to get N, then N is the percentage finer based on all the material passing the No. 200 sieve. This is true if W_2 is a representative sample of W_1. Therefore, to reduce N to a basis of the entire soil, W_s,

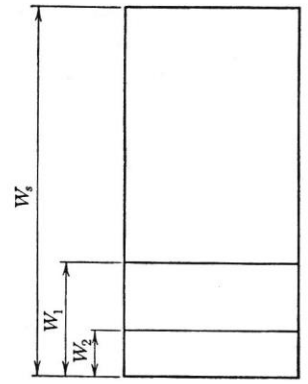


FIGURE B–8. Weights in grain size analysis.

$$N' = N \cdot \frac{W_1}{W_s} = N \cdot \% \text{ finer than No. 200 sieve} \qquad \text{(IV–3)}$$

where N' = percentage finer based on entire soil sample, W_s.

---

[4] This derivation is abbreviated from that in Taylor's *Fundamentals of Soil Mechanics.*

Table(4-1):Variation of k with Gs.

	Temperature (°C)													
G_s	17	18	19	20	21	22	23	24	25	26	27	28	29	30
2.5	0.0149	0.0147	0.0145	0.0143	0.0141	0.0140	0.0138	0.0137	0.0135	0.0133	0.0132	0.0130	0.0129	0.0128
2.55	0.0146	0.0144	0.0143	0.0141	0.0139	0.0137	0.0136	0.0134	0.0133	0.0131	0.0130	0.0128	0.0127	0.0126
2.6	0.0144	0.0142	0.0140	0.0139	0.0137	0.0135	0.0134	0.0132	0.0131	0.0129	0.0128	0.0126	0.0125	0.0124
2.65	0.0142	0.0140	0.0138	0.0137	0.0135	0.0133	0.0132	0.0130	0.0129	0.0127	0.0126	0.0124	0.0123	0.0122
2.7	0.0140	0.0138	0.0136	0.0134	0.0133	0.0131	0.0130	0.0128	0.0127	0.0125	0.0124	0.0123	0.0121	0.0120
2.75	0.0138	0.0136	0.0136	0.0133	0.0131	0.0129	0.0128	0.0126	0.0125	0.0124	0.0122	0.0121	0.0120	0.0118
2.8	0.0136	0.0134	0.0134	0.0131	0.0129	0.0128	0.0126	0.0125	0.0123	0.0122	0.0120	0.0119	0.0118	0.0117

Hydrometer Test analysis (sample of calculations)

The calculations related to the hydrometer tests are summarized as given below.

1. The required input data

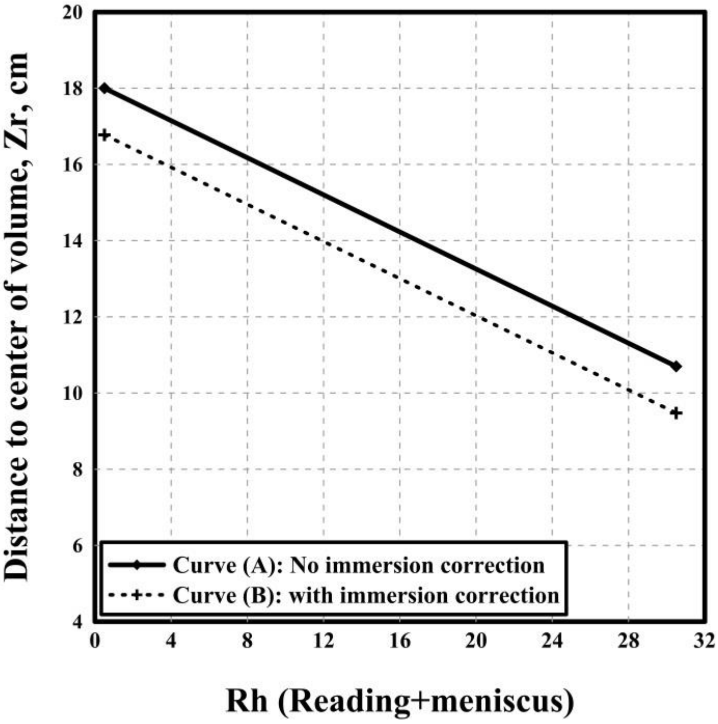
The following data should be given and recorded during the test

G_s	2.76
Temperature, °C	15
V_H, cm^3	70
Distance between two volumetric reading on the cylinder(900-1000) cm^3 , cm	3.5
m_s, gm	50.0
m_s, gm after completing the test	48.5
Aj cm2	28.57
(Vh/2Aj), cm	1.225
Zr, cm at r=1.000 (from calibration data)	18
Zr, cm at r=1.030 (from calibration data)	10.7
Meniscus correction, C_m	+0.5
Dispersion correction, x	-3.5
Water density correction, C_w	+1.8
Temperature correction, mt	-0.75
Full correction	-1.95

Note: the given values are for illustration only

2. Calibration of Hydrometer

The calibration of hydrometer is done as explained in page 5 in the lecture. The results of the calibration for the given data are presented below.



3. Calculation of the equivalent diameter (D)

The diameter of particles is calculated from the following equation:

$\boldsymbol{D}\ (\boldsymbol{mm}) = \sqrt{\frac{\boldsymbol{18\,\mu}}{\gamma_s - \gamma_w}}\sqrt{\frac{\boldsymbol{Z_r}}{\boldsymbol{t}}}$, please see page 4 of the lecture.

The above equation is simplified as follows:

$$\boldsymbol{D}\ (\boldsymbol{mm}) = \boldsymbol{k}\sqrt{\frac{\boldsymbol{Z_r}(\boldsymbol{cm})}{\boldsymbol{t}\ (\boldsymbol{min})}}$$

The value of k is taken from Table (4.1) presented in the lecture (Page 10). For the given data, the value of k is about 0.0137.

4. Calculation of the percentage finer (N)

The percentage finer cab ne obtained from the following equation as given in page 4 of the lecture.

$$\boldsymbol{N} = \frac{\boldsymbol{G_s}}{\boldsymbol{G_s - 1}}\frac{\boldsymbol{V}}{\boldsymbol{m_s}}\,\gamma_c\,(\boldsymbol{r} - \boldsymbol{r_w}) \times \boldsymbol{100}\%$$

The above equation is simplified as follows:

$$\boldsymbol{N} = \frac{\boldsymbol{G_s}}{\boldsymbol{G_s - 1}}\frac{\boldsymbol{1}}{\boldsymbol{m_s}}\,\boldsymbol{R'_h} \times \boldsymbol{100}\%$$

The value of $\mathrm{R'_h}$ represents the full correction of the readings. This value can be taken for some specific temperature as given in Page 8 of the lecture or calculated for each correction as explained in the lecture on pages 7 and 8. For the given data, the value of $\mathrm{R'_h}$ is -1.95. This means that full correction value (-1.95) must be added to all readings of the hydrometer.

5. Presentation of the results:

Elapsed time, min.	r, in suspension	t, °C	Rh=1000 (r-1)	Cm	Rh+Cm	Rh +full correction	Zr, cm	D, mm	N, %
0.25	1.03000	15	30.00	0.5	30.50	28.05	10.698	0.0896	90.70
0.5	1.03000	15	30.00	0.5	30.50	28.05	10.698	0.0634	90.70
1	1.02750	15	27.50	0.5	28.00	25.55	11.219	0.0459	82.61
2	1.02350	15	23.50	0.5	24.00	21.55	12.054	0.0336	69.68
4	1.02200	15	22.00	0.5	22.50	20.05	11.143	0.0229	64.83
8	1.02000	15	20.00	0.5	20.50	18.05	11.560	0.0165	58.36
16	1.01850	15	18.50	0.5	19.00	16.55	11.873	0.0118	53.51
30	1.01750	15	17.50	0.5	18.00	15.55	12.081	0.0087	50.28
60	1.01525	15	15.25	0.5	15.75	13.30	12.551	0.0063	43.00
120	1.01250	15	12.50	0.5	13.00	10.55	13.124	0.0045	34.11
180	1.01150	15	11.50	0.5	12.00	9.55	13.333	0.0037	30.88
300	1.01025	15	10.25	0.5	10.75	8.30	13.594	0.0029	26.84
1440	1.00900	15	9.00	0.5	9.50	7.05	13.854	0.0013	22.80
2880	1.00900	15	9.00	0.5	9.50	7.05	13.854	0.0010	22.80

Experiment No. 5: Compaction Test:

Soil placed as engineering fill (embankments, foundation pads, road bases) is compacted to a dense state to obtain satisfactory engineering properties such as, shear strength, compressibility, or permeability. In addition, foundation soils are often compacted to improve their engineering properties. Laboratory compaction tests provide the basis for determining the percent compaction and water content needed to achieve the required engineering properties, and for controlling construction to ensure that the required compaction and water contents are achieved.

Benefits of Compaction:

1. Compaction increases the dry density of soils for a wide range of construction applications such as highway embankments, backfilled trenches, earth dams and sub−foundation soils.
2. Compaction generally leads to the following desirable effects on soils:

1) An increase in shear strength;

This means that great loads can be applied to compacted soils since they are typically stronger.

2) A decrease in compressibility;

This also means that larger loads can be applied to compacted soils since they will produce smaller settlements.

3) A decrease in permeability;

This inhibits soils' ability to absorb water, and therefore reduces the tendency to expand/shrink and potentially liquefy.

Explanation of the Process of Compaction:

The soil mass involved in the laboratory compaction process starts as a three-phase system (soil, air, and water). During the initial trials, one has a cylinder of soil with voids; some of the voids are partially filled with water, and the remainder is filled with air. At this state the structure of soil mass is described as flocculated structure as shown in Figure (1). Because of lubrication, the addition of a small amount of water to a dry soil aids the compaction process. Up to a certain point additional water replaces air from the soil voids, but after a relatively high degree of saturation is reached, the water occupies space, which could be filled with soil particles, and the amount of entrapped air remains essentially constant. There is, therefore, an optimum amount of mixing water for a given soil and compaction process, which will give a maximum weight of soil per volume.

As more water is added, the efficiency of the compaction process increases due to the lubrication effect of the water and the breaking of inter-particle bonds. Then, the void spaces become smaller, and the amount of air in the voids reduces (the degree of saturation increases) as water replaces the air in the voids and the structure of soil mass becomes oriented as shown in Figure (1).

If the process were completely efficient, it might be possible to expel all the air from the voids, in which case one could compute a dry density corresponding to a state of zero-air voids. Since it is never possible to get all the air out of the voids (a condition of S = 100 percent), any compaction curve will always fall below the "ideal" or zero air-voids curve.

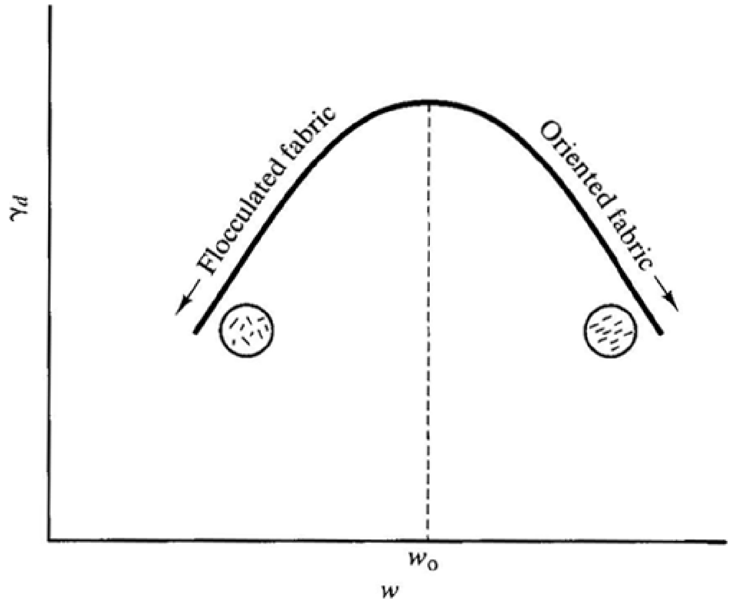


Figure (1): The structure of soil during compaction.

Test Specifications and Standards for Compaction Test

1. ASTM D698 (Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft^3(600 kN-m/m^3).

2. **ASTM D1557 (**Standard Test Methods for Laboratory Compaction Characteristics of Soil Using modified Effort (56,000 ft-lbf/ft^3 (2,700 kN-m/m^3).

Compaction Energy (E):

Compaction Energy (E) applied to soil per unit volume is computed from the following equation:

$$Compaction\ Effort = \frac{No.of\ Blows\ per\ Layers * No.of\ Layers * Hammer\ Height * Height\ of\ Drop}{Volume\ of\ the\ Mould}$$

Table below gives the comparison between the Standard and Modified Compaction Test Properties:

Type of Compaction Test	Mold Diameter (cm)	Mold Height (cm)	Hammer mass (kg)	Hammer Height of Drop (cm)	No. of Blows per Layer	No. of Layers	standard	Compaction Energy
Standard	10.16	11.64	2.5	30.5	25	3	See ASTM D 698	594 kN.m/m3 (≈600)
Modified	10.16	11.64	4.5	45.7	25	5	See ASTM D 1557	2670 kN.m/m3 (≈2700)

Apparatus and Materials

1. Mold Assembly. Mold has a volume of (1/30) ft^3 or ≈944cm^3, and a diameter of 4in or 10.16cm. The height of the mold is about 4.6 in (11.76 cm). The mold is accompanied with mold collar of about 4 in diameter and 2 in height,
2. Hammer (For the Standard Effort, The hammer shall fall freely through a distance of 12 in. (304.8 mm) from the surface of the specimen. The mass of the rammer shall be 5.5 lbm (≈2.5 kg). For the modified Effort, the rammer shall fall freely through a distance of 18in. (457.2 mm) from the surface of the specimen. The mass of the rammer shall be 10 lbm (4.54 kg),
3. Sample Extruder (optional)—a jack, frame or other device adapted for extruding compacted specimens from the mold,
4. Balance (Electrical balance of 0.01 gm sensitivity as well as mechanical or electrical balance of 1.0gm readability),
5. Straightedge or Spatula,

6. Mixing Tools—Miscellaneous tools such as mixing pan, spoon, trowel, spatula, spray bottle, etc., or a suitable mechanical device for thoroughly mixing the sample of soil with increments of water,
7. Sieve No. 4 (4.75mm),
8. Drying oven,
9. Drying cans and
10. Rubber hammer.

Testing Procedures

The amount of soil required for the standard compaction test varies with the kind and gradation of the soil to be tested. For soils passing the No. 4 sieve that are to be tested in the 4.0 in mold, 20 kg soil is normally sufficient for the test. The soil to be tested shall be air-dried.

Aggregations present in the sample shall be thoroughly broken, but care should be taken that the natural size of the individual particles is not reduced. The material shall then be screened through a No. 4 sieve. It is worth to mention here in that in the past years, the mass of the soil was about 2 to 3 kg for the test.

The soil is mixed with varying amounts of water to achieve different water contents. For each different water content, w:

a) The soil is placed into the mold in three layers

b) For each lift, the soil is compacted by dropping a hammer of certain mass 25 times onto the confined soil from a specific height.

c) The compacted soil is removed from the mold and its dry density (or dry unit weight) is measured.

The testing procedures for each trial are summarized below:

1. Obtain the mass of the compaction mold and its base without collar.
2. The soil of about 3 kg must be placed in large pan. The required water content is spread on the soil and left for a certain period. When the soil absorbs the added water, mixing

procedure is starting by hand. The mixing must continue until homogenous mixture is attained. Then the soil within the pan is divided in to three or five parts. Each part should be placed carefully in the mold.

3. Use either the standard or the modified method, as specified by the instructor, and compact the soil. Be sure to scarify the sample between layers. When compacting, try not to let the hammer hit the specimen more than once per drop at the same location.
4. Carefully strike the top of the compacted soil with a steel straight edge. Use a slow sawing motion. Fill in any holes in the compacted specimen with soil if the smoothing process removes any small pebbles. If the mold is not filled above the collar joint from the last compacted layer, do not add soil to make up the deficiency. Care should be followed to ensure that the soil above the mold is not large. The addition soil should be removed carefully. If the collar is hard to remove, do not risk twisting off the third (or last) layer of soil; take a spatula and trim along the sides of the collar until it comes off easily.
5. Weigh the mold filled with the soil and the base.
6. Extrude the soil from the mold, split it, and take three water content samples from the center of the specimen: one near the top and the other near the bottom. Obtain as much soil as the moisture cups will hold (about 100 g).
7. Repeat the above procedure at 2 to 6 with different water contents. The instructor will provide guidelines as to which water contents to use, but the actual decision rests with the group. The goal is to produce an accurate compaction curve; optimally, at least four points should be obtained for compaction curve. Two of them at the dry side and the others are within the wet side. After your first 2 tests, it should roughly decide the next water added to the soil.
8. Return to the laboratory the following day and weigh the oven-dry water content samples to find the actual average water content of each test.

The above mentioned procedures are summarized as follows:

1. Soil Sample (Type and Quantity).
2. Sample preparation.
3. Compaction of the Sample.
4. Data Recording and Observation.
5. Data Presentation.

Col. No.	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
			Water Content Determination								
Trial No.	Wt. of Mold +Wet soil (gm)	Wet Soil in the Mold (gm)	Tin No.	Tin Wt. (gm)	Tin Wt. + Wet soil (gm)	Tin Wt .+dry soil (gm)	Water Content, %	Average Water Content, %	Wet density (gm/cm^3)	Dry density (gm/cm^3)	ZAVD (gm/cm^3)
1			**A1**								
			A2								
			A3								
2			**B1**								
			B2								
			B3								
3											
4											
5											

Calculation

1- Calculation of the dry Density

$$\gamma_{\mathrm{d}} = \frac{\gamma_{\mathrm{wet}}}{1+\omega} = \frac{\dfrac{\mathrm{Col1} - \mathrm{M}_{\mathrm{mold}}}{\mathrm{V}}}{1+\omega}$$

2- Calculation of the water content

$$\omega = \frac{\mathrm{col.}\,5 - \mathrm{col.}\,6}{\mathrm{col.}\,6 - \mathrm{col.}\,4}$$

3- Calculation of the S Lines

$$\gamma_d = \frac{Gs\,(1-A)}{1+\omega\, Gs}\gamma_w$$

4- Calculation of the ZAVD Line (Saturation Line)

$$\gamma_d = \frac{Gs}{1+\omega\, Gs}\gamma_w$$

Presentation of the Test Results

The Results of the laboratory compaction test would be presented as a graph of the water content versus unit weight or density as shown in the figure below. From this graph, the value of the maximum dry density and optimum water content can be found. In addition, the ZAVD can be drawn as shown in Figure (2).

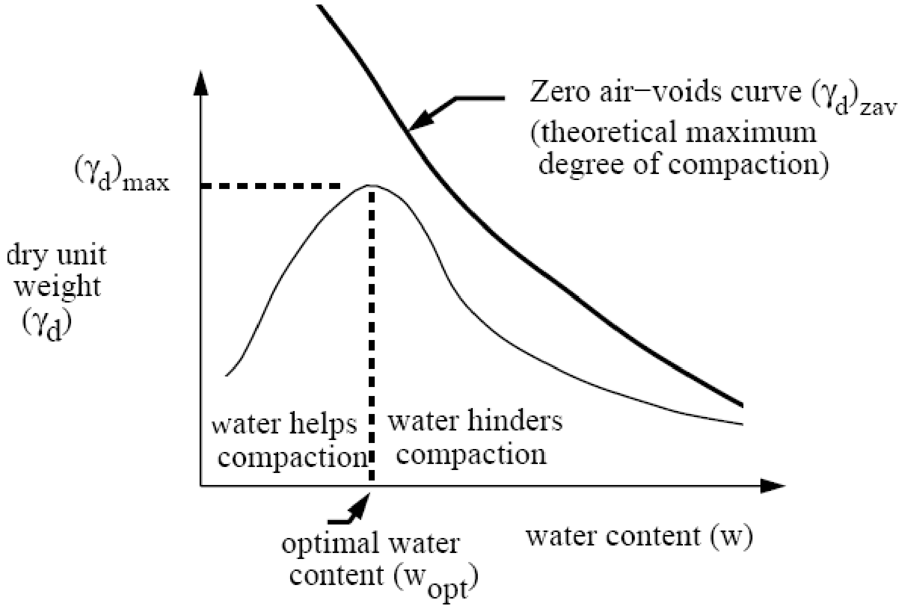


Figure (2): Compaction curve.

Experiment No. 4: Permeability Test:

1. Definition of Permeability

It is the measure of soil capacity to allow the flow of a fluid through it (the fluid may be either water or gas) in unit of velocity (i.e. L/T such as cm/sec, m/min…etc.).

2. The purpose of the test

The purpose of the permeability test is to find the coefficient of permeability, k.

3. The engineering applications of the coefficient of permeability

- Finding the seepage under dams.
- The squeezing out of water from a soil by the application of a load, and drainage of subgrades, dams, and back fills. In addition, the effective strength of a soil is often indirectly controlled by its permeability.
- Dewatering process.
- Design of sewerage system, and
- others

4. Factors affecting permeability

- The size of soil grains: and it is influenced by its particle size distribution, especially by the finer particles. (The smaller the particles, the smaller the voids between them, and therefore the resistance to flow of water increase, this means that the permeability decrease with decreasing particle size).
- Particle shape and texture.
- The mineralogical composition: the mineral type influence the permeability of clay largely than does the size of particle for cohesive soil, but it generally has little effect on the permeability of sand and gravels.
- Void ratio and porosity.
- Soil fabric.
- Nature of fluid (depending on density and dynamic viscosity).
- Type of flow (which is "laminar" or "streamline").

- Temperature: an increase in temperature causes a decrease in the viscosity of water, i.e. the water becomes more fluid which affects the value of the measured permeability.
- Degree of saturation: an increase in the degree of saturation of a soil causes an increase in permeability.

5. Laboratory Methods to measure Permeability (k):

There are different methods to find the permeability of soil. These methods are mainly depends on the type of the soil and direction of flow.

1- The constant head method (for cohesionless soil).
2- The variable head method (for fine grained soil).
3- Permeability from the consolidation test (Indirect method).

The basic of the constant and variable head is illustrated in Figure (1)

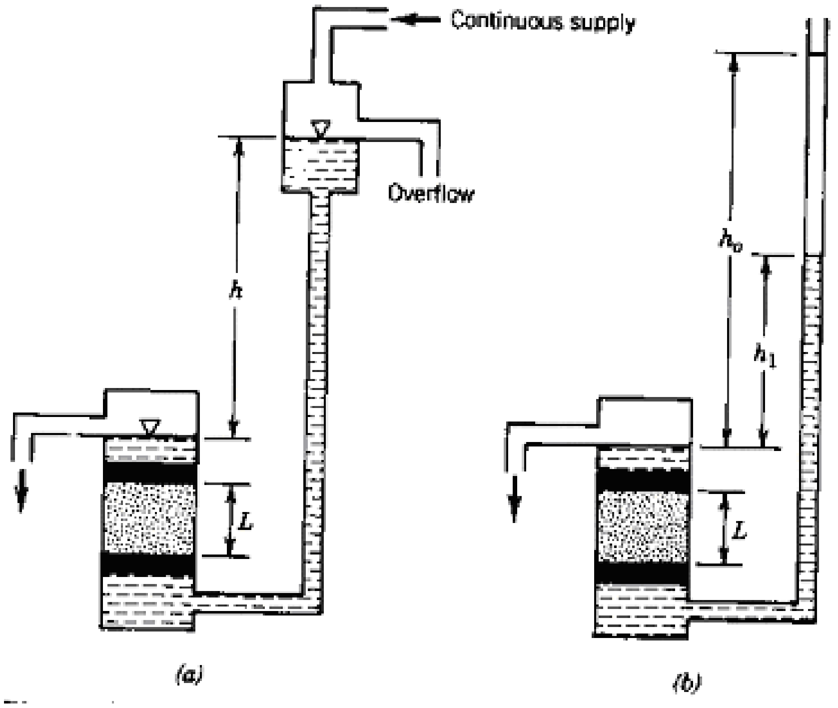


Figure (1): The basic of the permeability tests in the laboratory

<u>(a): The Constant Head Method</u>

This test is suitable for coarse grain soil (Sand and fine gravel)

Apparatus:

1- Permeameter tube.
    a. Two screens.
    b. Two rubber stoppers.
    c. Springs.
2- Stand pipe.
3- Dearing and saturating device.
4- Support frame and clamps.

General tools and equipment:

1- Hammer.
2- Distilled water.
3- Vacuum supply.
4- Balance.
5- Drying oven.
6- Scale.
7- Desiccators.
8- Thermometer.
9- Stop clock.
10- Rubber tubing, evaporating dish.
11- Funnel.

Testing Procedure:

1- Place the soil in a measured permeameter, weight, and saturate the sample.
2- Measure the value of the head (h), and specimen length (L).
3- Start flow by opening value at the base of the sample.
4- After allowing a few minutes for equilibrium conditions to be reached obtain graduate and time readings.
5- After a sufficient amount of water has collected in the graduate for a satisfactory measure of its volume, take graduate and time observations.

Subtract the graduate and time readings obtained in step 4 from the respective values obtained in this step to give (Q) and (t).

6- Record the temperature of the water every few minutes.

7- Change the void ratio of the soil as was done in variable head test, and take another series of graduate and time readings. Measure the specimen length at each void ratio.

Calculation

$$\mathbf{k} = \frac{\mathbf{Q\ L}}{h\ \mathbf{A\ t}}$$

Where:

k=Coefficient of permeability.

Q=Total quantity of water which flowed through in elapsed time.

h=Total head lost.

A=cross-sectional area of the permeameter.

t= Elapsed time.

Note: In the case, that the temperature in lab is not 20 C^o then it must be corrected by using:

$$\mathbf{K_{20}} = \mathbf{K_T}\ \frac{\mu_T}{\mu_{20}}$$

Where:

K_{20}= Permeability at temp. t.

K_T= Permeability at temp. 20C^o.

μ_T= viscosity at temp. t.

μ_{20}= viscosity at temp. 20 C^o.

(Please see Soil testing for engineers by (Lambe) p.148)

(b):Variable Head Test

This is suitable for fine-grained soils like clays:

$$k = \frac{2.3\,aL}{At} \log \frac{h_1}{h_2}$$

Where:

a= cross-sectional area of the stand pipe.

L=Length of soil sample in permeameter.

A=cross-sectional area of the permeameter.

t= elapsed time (t_2-t_1).

h= The head between which the permeability is determined.

From Consolidation Test

$$C_v = \frac{K}{\gamma_w m_v} \qquad m_v = \frac{\Delta e}{(1+e_\circ)\Delta\sigma}$$

6. Classification of soils According to their Permeability:

The soil can be classified according to the value of k as given below

Table (1) typical values of Hydraulic conductivity of saturated soils

Soil type	Permeability (k) cm/sec	Permeability (k) ft/min
Clean gravel	100-1.0	200-2.0
Coarse sand	1.0-0.01	2.0-0.02
Fine sand	0.01-0.001	0.02-0.002
Silty clay	0.001-0.00001	0.002-0.00002
Clay	< 0.000001	< 0.000002

(Taken from Principles of Geotechnical Engineering (Braja, M. Das) p.143)

University of Baghdad
College of Engineering
Soil Mechanics Laboratory (2016-2017)
3rd year

Consolidation Test (One dimensional Consolidation Test)

Introduction:

Consolidation is the gradual reduction in volume of a fully saturated soil of low permeability due to drainage of some of the pore water, the process continue until the excess pore water pressure set up by an increase in total stress has completely dissipated.

Settlement is the increase in stress within soil layer due to the application of surface load will be accompanied by some strain and concurrent surface settlement.

Type of Settlement:

1- Immediate settlement (in Sands only).
2- Consolidation settlement (in clay only).
3- Secondary consolidation (creep).

The equation of settlement:

$$\rho = \frac{\Delta e}{1+e_\circ} H_\circ$$

Where:

ρ = Settlement of soil.

$H_\circ$ = Thickness of clay layer

Consolidation may occur due to following reasons:

1- Lowering of the ground water table.
2- External static loads from structures.
3- Self weight of recently placed soils.
4- Desiccation.

Consolidation settlement:

1- It occurs due to the process of consolidation.
2- Clay and Organic soil are most prone to consolidation.
3- Consolidation is the process of reduction in volume due to expulsion of water under an increased load.
4- It is time related process occurring in saturated soil by draining water from void.

5- It is often confused with compaction.
6- Consolidation theory is required to predict both rate and magnitude of settlement.
7- Since water flows out in any direction, the process is three dimensional.
8- But, soil confined laterally. Hence, vertical one dimensional consolidation theory is acceptable.
9- Spring analogy explains consolidation settlement as shown in figure (1).
10- Permeability of soil influences consolidation.

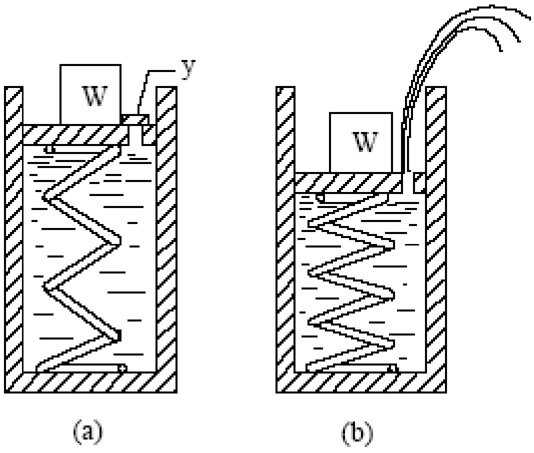


FIGURE 1: The spring analogy to consolidation.

Primary consolidation

This method assumes consolidation occurs in only one-dimension. Laboratory data is used to construct a plot of strain or void ratio versus effective stress where the effective stress axis is on a logarithmic scale. The plot's slope is the compression index or recompression index. The equation for consolidation settlement of a normally consolidated soil can then be determined to be:

$$\delta_c = \frac{C_c}{1+e_0} H \log\left(\frac{\sigma'_{zf}}{\sigma'_{z0}}\right)$$

where

δ_c is the settlement due to consolidation.
C_c is the compression index.
e_0 is the initial void ratio.
H is the height of the soil.
σ_{zf} is the final vertical stress.
σ_{z0} is the initial vertical stress.

C_c can be replaced by C_r (the recompression index) for use in verconsolidated soils where the final effective stress is less than the preconsolidation stress. When the final effective stress is greater than the preconsolidation stress, the two equations must be used in combination to model both the recompression portion and the virgin compression portion of the consolidation processes, as follows,

$$\delta_c = \frac{C_r}{1+e_0} H \log\left(\frac{\sigma'_{zc}}{\sigma'_{z0}}\right) + \frac{C_c}{1+e_0} H \log\left(\frac{\sigma'_{zf}}{\sigma'_{zc}}\right)$$

Where σ_{zc} is the preconsolidation stress of the soil.

Secondary compression

Secondary compression is the compression of soil that takes place after primary consolidation. Even after the reduction of hydrostatic pressure some compression of soil takes place at slow rate. This is known as secondary compression. Secondary compression is caused by creep, viscous behavior of the clay-water system, compression of organic matter, and other processes. In sand, settlement caused by secondary compression is negligible, but in peat, it is very significant. Due to secondary compression some of the highly viscous water between the points of contact is forced out.

Secondary compression is given by the formula

$$S_s = \frac{H_0}{1+e_0} C_a \log\left(\frac{t}{t_{90}}\right)$$

Where

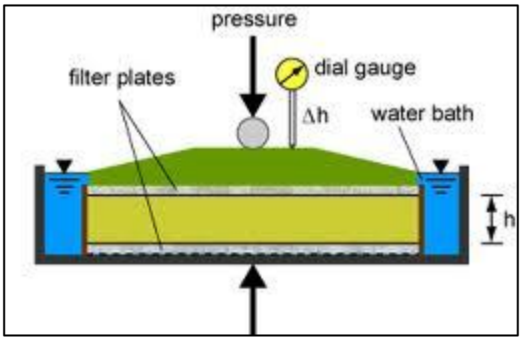
H_0 is the height of the consolidating medium
e_0 is the initial void ratio
C_a is the secondary compression index
t is the length of time after consolidation considered
t_{90} is the length of time for achieving 90% consolidation

The purpose of the test

This test is performed to determine the magnitude and rate of volume decrease that a laterally confined soil specimen undergoes when subjected to different vertical pressures. From the measured data, the consolidation curve (pressure-void ratio relationship) can be plotted. This data is useful in determining the compression index, the recompression index and the preconsolidation pressure (or maximum past pressure) of the soil. In addition, the data obtained can also be used to determine the coefficient of consolidation and the coefficient of secondary compression of the soil.

Apparatus and Supplies

1- Consolidation unit as shown in figure below.
2- Loading frame.
3- Timing device.
In general:
1- Balance (0.01 gm sensitivity).
2- Drying oven.
3- Desiccator.
4- Timer.
5- Watch glasses or moisture content cans.
6- Scale.
7- Evaporating dish.



Specimen preparation and consolido-meter set-up:

1. First, a sample of the soil to be tested must be prepared. This is usually taken from a thin-walled Shelby tube, but for practice in laboratory exercises, a Proctor compaction specimen will suffice.
2. The consolido-meter ring should be measured and massed at this point.
3. The soil sample is next extruded from its container (either the Shelby tube or the Proctor mold) a certain length. It should be extruded a little more than the height of the ring into which it will eventually be placed.
4. Using a thin wire saw, this disk of soil is cut off. Remember which side of the soil which is (up and down). We'll need to properly orient the specimen in the consolidation test later.
5. At this point, one should have a disk of soil with a diameter greater than the inside diameter of the ring, and a height greater than that of the ring. Now, the soil must be placed into the ring.

6. Set the soil on a large glass plate, or clean level surface. The *top* of the soil specimen should be *down.*
7. Orient the ring on top of the specimen such that it is approximately centered, with the cutting edge down (into the soil).
8. Place a small glass or acrylic plate on top of the ring, and then slowly and evenly start to push the ring into the soil specimen. The ring will begin to cut away excess soil around the outside of the specimen.
9. After the ring has been pushed in about ¼” to ½”, use the wire saw to trim away excess soil being cut by the ring. If this is not done, there is a tendency for soil inside the ring to be ripped out with the excess soil being cut away.
10. Keep pushing the ring, slowly and evenly into the soil, and cutting away the excess. The key here is patience, precision and carefulness. This step often takes much time and concentration, especially for beginners.
11. When the ring has been pushed far enough down that the *bottom* (which is now facing up) of the soil specimen touches the small top glass or acrylic plate, stop.
12. Using a ring with the same interior diameter, but a larger exterior diameter, finish pushing the ring downwards. This should be stopped so that the cutting edge of the ring (facing down) is *not* flush with the *top* of the soil specimen (now facing down). There needs to be, at this point, a little bit of soil (about ¼”) sticking out of either side of the ring.
13. Now, using the thin wire saw, trim the top and bottom of the specimen such that they are both flush with the top and bottom of the ring. Use a sawing motion to cut *down* into the soil until the ring is reached (can’t go any farther down). Then, using a sawing motion, cut back towards yourself keeping the saw’s cutting wire in contact with the ring. Take off small bits in this manner until the entire surface is flush with the ring. Then flip the specimen over, and repeat on the other side.
14. Take some of the *fresh* trimmings and obtain the initial water content.
15. Mass two pieces of filter paper, and place these on either side of the specimen. They should stick somewhat.
16. Mass the ring + specimen + paper.
17. Place the ring + specimen + paper into the consolidometer.
18. Mass a porous stone and place on top of the specimen.
19. Place a rigid cap and steel ball bearing of known mass on top of this top porous stone.
20. Fill the consolidometer with water.
21. Adjust the reaction beam on the pneumatic loading frame such that it barely contacts the ball bearing.
22. Set the vertical displacement dial gage so that its full range is available in compression. Zero the dial gage. Adjust units and travel direction, if digital dial gage.
23. Apply a small seating load to the specimen (~30 kPa) and take appropriate time deformation readings. Leave the seating load on for 24 hrs.

CONSOLIDATION OF THE TEST SPECIMEN:

1. Become familiar with the apparatus. Specifically,

§ How the load will be applied.

§ Units of the dial gage, and in which direction it will move when load is applied.

§ How to switch between the low and high pressure regulators.

2. Understand the loading schedule. This will be predetermined by the instructor. Each group in your section will be joined with a group from each other section (all the group A's will work together, etc.). There will be a certain number of the loading increments that each individual group will be responsible for. A load increment ratio (LIR) of 1 is normally used. This means that the load is doubled from the previous increment (1, 2, 4, 8, etc.). Other load increments may also be used, however. Sometimes, unload-reload cycles are performed.
3. Prepare a Data Sheet 2 for each loading increment that your individual group will be responsible for. You'll take vertical deformation dial readings at elapsed time increments of 0, 6, 15, 30, 45, 60 sec., 2, 4, 8, 15, 30, 60 minutes or until 90% consolidation has been reached. The time required for 90% consolidation can be determined from the square root time plots that you'll be plotting for every stage. These will be covered later.
4. Adjust the low-pressure regulator to obtain the desired initial applied stress on the specimen. Then open the loading valve to apply the load. Immediately begin to observe the vertical deformation, and take readings at the aforementioned elapsed time increments.
5. Plot vertical deformation dial readings (DR) v. the square root of time *as you go*. This will help you catch recording errors, as well as determine when primary consolidation is complete. This second part is determined by finding t90 from the plot. Your instructor will show you how to do this in lab; it is also covered in your text. When you reach t90, 90% of primary consolidation is complete; it is now satisfactory to proceed with the next loading increment.
6. When primary consolidations is complete, or as instructed, apply the next load increment, and record and plot vertical deformation readings as before.
7. Continue the incremental loading process outlined by your instructor. It is a good idea to plot vertical deformation dial readings v. applied stress (logscale) as you go. This also helps prevent mistakes, and gives you a picture of how the loading process is proceeding.
8. After the final (maximum) load increment has been applied, and primary consolidation is complete for this stage, the specimen is unloaded (rebounded) using *alternate* loading increments. When unloading, the lower load increment is applied to the specimen, and it must be allowed to rebound in this state for a certain period of time. This time is t90 for the corresponding *load* increment. For example: if we're unloading from 1200 kPa to 800 kPa, we should let the specimen rebound for a period of time equal to t90 for the loading stage from 800 kPa to 1200 kPa.
9. Rebound the specimen until only a very small seating load is applied to the specimen. Leave this load on overnight. This is necessary to be able to determine accurately the final height and void ratio of the specimen and correlate these values to the final strain dial reading.
10. Unload the specimen completely, record the final dial reading and carefully disassemble the apparatus. Quickly measure the mass of the ring plus the wet

specimen, then place the ring and specimen in the oven. This will allow calculation of the final water content of the specimen. Clean the equipment and work area.

Calculations

From consolidation test result we can find other coefficient help us to recognizing soil behavior such as:

$$a_v = \frac{\Delta e}{\Delta \sigma} \qquad \frac{\Delta H}{H} = \frac{\Delta e}{1+e_\circ}$$

$$m_v = \frac{a_v}{1+e_o} \qquad \Delta e = \frac{1+e_\circ}{H}\Delta H$$

$$c_v = \frac{T_v H^2}{t}$$

$$c_c = \frac{\Delta e}{\Delta logp}$$

Where

a_v: Coefficient of compressibility m^2/ kN,

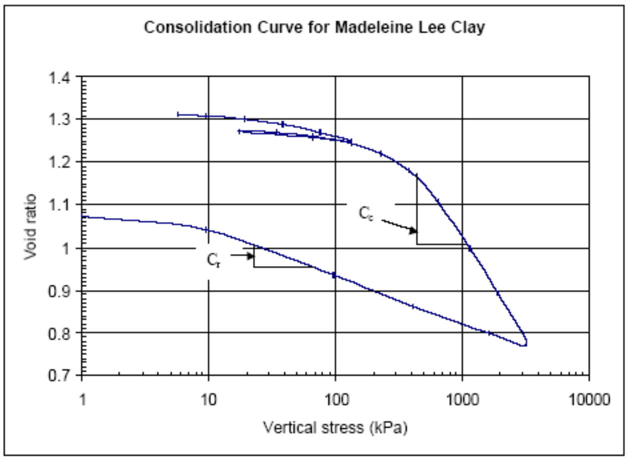
m_v: Coefficient of volume change m^2/ kN,

c_v: Coefficient of consolidation,

c_c: Compression index,

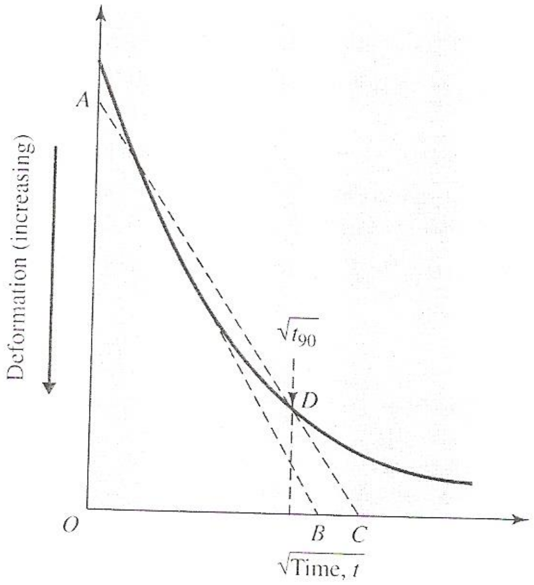
H: Path length, and

T_v: Time factor.



There are two ways to compute c_v from consolidation test data:

1. Square root time method(Taylor's method)



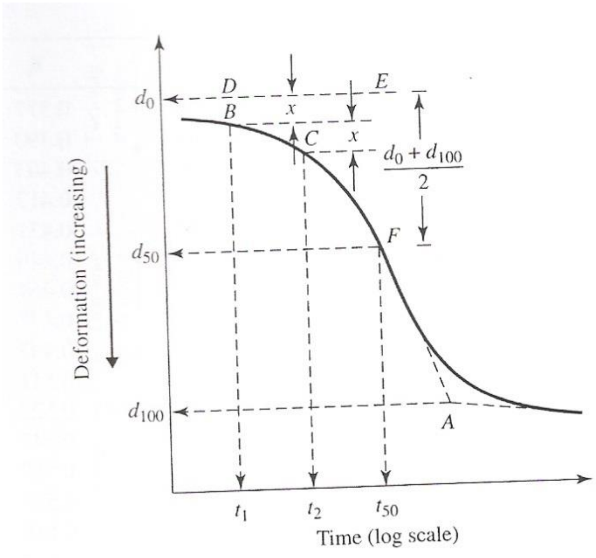
1- Draw a line AB through the early portion of the curve.
2- Draw a line AC such that (OC =1.15OB). The abscissa of point D, which is the intersection of AC and the consolidation curve, gives the square root of time for 90% consolidation($\sqrt{t_{90}}$).
3- For 90% consolidation, $T_{90=}$ 0.848, so

$$T_{90} = \frac{c_v t_{90}}{H^2} = 0.848$$

Or

$$c_v = \frac{0.848H^2}{t_{90}}$$

2. Log-time method (Casagrande method).



1- Extend the straight line portions of primary and secondary consolidation to intersect at A. The ordinate of A is represented by d_{100} that is, the deformation at the end of 100% primary consolidation.
2- The initial curved portion of the plot of deformation versus log t is approximated to be a parabola on the natural scale. Select times t_1 and t_2 on the curved portion such that ($t_2=4t_1$). Let the difference of specimen deformation during time (t_2-t_1) be equal to (x).
3- Draw a horizontal line DE such that the vertical distance BD is equal to (x). The deformation corresponding to the line DE is $d_\circ$ (that is, deformation at 0% consolidation).
4- The ordinate of point F on the consolidation curve represents the deformation at 50% primary consolidation, and its abscissa represents the corresponding time (t_{50}).
5- For 50% average degree of consolidation, T_v = 0.197, so,

$$T_v = \frac{c_v t_{50}}{H^2}$$

Or

$$c_v = \frac{0.197\,H^2}{t_{50}}$$

Calculations

1- Find the volume of sample.
2- Find the saturation unit weight of soil.
3- Find the dry unit weight of soil.
4- Find the initial void ratio.
5- Find the void ratio for each load during time.
6-

Time (min)	**Loads**						
	50	**100**	**200**	**400**	**800**	**200**	**50**
0							
0.25							
0.5							
1							
2.25							
1440(24 hr)							

To find Cv for each load.

Load	**Reading**	**Δh**	**Δh * factor**	$\boldsymbol{\Delta e = \frac{1+e_\circ}{H} * \Delta H}$	**e = $e_\circ$ - Δe**
0					
50					
100					
200					
400					
800					
200					
50					

To find Cc & Cr

Find a_v, m_v

Example:

Ws= 29%

$P_{initial}$= 600 kPa

Gs= 2.73

γ_t = 19.00 kN/m^3

e_o = 1.092

Time (min)	**Loads kg**							
	0.5	**1**	**2**	**4**	**8**	**16**	**4**	**1**
0	**0**	**432**	**728**	**1039**	**1340**	**1735**		
0.25	**86**	**495**	**810**	**1141**	**1489**	**1885**		
0.5	**91**	**499**	**818**	**1149**	**1502**	**1900**		
1	**97**	**506**	**826**	**1158**	**1517**	**1919**		
2	**106**	**516**	**838**	**1172**	**1538**	**1941**		
4	**119**	**532**	**856**	**1190**	**1566**	**1974**		
8	**140**	**555**	**883**	**1215**	**1605**	**2013**		
15	**169**	**585**	**915**	**1244**	**1640**	**2044**		
30	**214**	**626**	**953**	**1272**	**1669**	**2067**		
60	**268**	**663**	**981**	**1292**	**1687**	**2083**		
120	**332**	**690**	**1000**	**1307**	**1700**	**2095**		
240	**378**	**710**	**1014**	**1320**	**1712**	**2107**		
24 hr	**432**	**728**	**1039**	**1340**	**1735**	**2131**	**2068**	**1965**

Test No. (6): Direct Shear Test:

Types of shear strength tests:

- **Direct shear test.**
- **Vane shear test.**
- **Triaxial tests.**
- **Field tests:**
- **Cone penetration test CPT.**
- **Standard penetration test SPT.**
- **Plate load test.**

Introduction:

In all soil stability problems such as the design of foundations, retaining walls and embankments, knowledge of the strength of the soil involved is required. Depending on the source of its strength, the soil can be classified into two groups:

Cohesionless and cohesive ; **cohesionless soils** are soil which have no cohesion or attraction between individual particles and **cohesive soils** are soils the individual particles of which exhibit interact action.

The resistance to shear of a cohesion less soil is derived from friction between grains and the interlocking of grains. Friction between soil grains is similar to friction between any surfaces.

In soils, the friction may be either sliding friction or rolling friction. For example, if a large enough shear force were applied to soil grain A it could be moved to position B by either sliding or rolling or a combination of the two.

Shear strength:

It is the resistance of a soil to failure .it is very important in analyzing soil stability problems such as:

1- Bearing capacity,
2- Slope stability of dams and embankments and
3- Lateral earth pressure in retaining walls.

Principle of test:

The direct shear test is a strain-controlled test: the rate at which the soil will be strained is controlled. A specimen of soil will be placed into a shear box, and consolidated under

an applied normal load. The shear box is made of two separate halves, an upper and a lower. After the application of the normal load, these two halves of the be moved relative to one another, shearing the soil specimen on the plane that is the separation of the two halves. The direct-shear test imposes stress conditions on the soil that force the failure plane to occur at a
predetermined location (on the plane that separates the two halves of the box). On this plane

there are two forces (or stresses) acting - a normal stress, σ_n, due to an applied vertical load P_v and a shearing stress, σ_h, due to the applied horizontal load P_h. These stresses are simply
computed as:
$\sigma_n = P_v/A$

$\sigma_h = P_h/A$

Where: A is the nominal area of the specimen (or of the shear box). It is usually not corrected

for the change in sample area caused by the lateral displacement of the sample under the shear load P_h. These stresses should satisfy Coulomb's equation:

$$\tau = c + \sigma_n \tan\phi$$

As there are two unknown quantities (c and ϕ) in the above equation, two values, as a minimum, of normal stress and shear stress will be required to obtain a solution.
Since the shear stress and normal stress have the same significance as when used in a Mohr's circle construction, rather than solving a series of simultaneous equations for c and $\tan\varphi$, one may plot on a set of coordinate axes the values of τ versus σ_n from several tests (generally with on the ordinate), draw a line through the resulting locus of points, or the average locus of points, and establish the slope of the line as and the y-axis intercept as the c parameter.
This is commonly known as the Mohr-Coulomb Failure Envelope.
For cohesionless soils, the intercept is usually negligible, and Coulomb's equation becomes

$$\tau = \sigma_n \tan\phi$$

Test inaccuracies and surface-tension effects of damp cohesionless materials may give a small value of c, called the "apparent" cohesion. This should be neglected unless it is more than 1 or 2 psi. If the c value is large and the soil is a cohesionless material, the reason for the large value should be investigated.

The direct shear test was formerly quite popular, but with the development of the triaxial test
which is much more flexible, it has become less popular in recent years.

The advantages of the direct shear test are:

1- An easy and quick test.
2- Gives very good results for sandy soils.

Disadvantages of the test include:

1. Difficult or impossible to control drainage, especially for fine-grained soils.
2. Failure plane is forced--may not be the weakest or most critical plane in the field.
3. Non-uniform stress conditions exist in the specimen.
4. The principal stresses rotate during shear, and the rotation cannot be controlled. Principal stresses are not directly measured.

Because the drainage conditions during all stages of the test markedly influence the shear
strength of soils, the direct shear test is only applicable for relatively clean sands which are free draining during shear. For clay soils, some unknown amount of consolidation could occur during shear, which would give a larger shear strength than actual. Therefore the test is not generally recommended for cohesive soils.

Direct Shear test may be categorized as follows:

1- **Quick test:** shear began before the sample consolidates under the normal load. If the soil is cohesive and saturated, excess Pore water pressure may develop.
2- **Consolidated-Quick test:** the normal force is applied and the vertical dial movement is observed until settlement stops before the shearing force is applied.
3- **Consolidated-Slow tests:** the normal force is applied and the shear force is developed until all settlement stops. The shear force is then applied so slowly that no pore pressure develop in the sample.

For cohesionless soils, all three of the above tests will give the same results whether the sample is saturated or unsaturated.

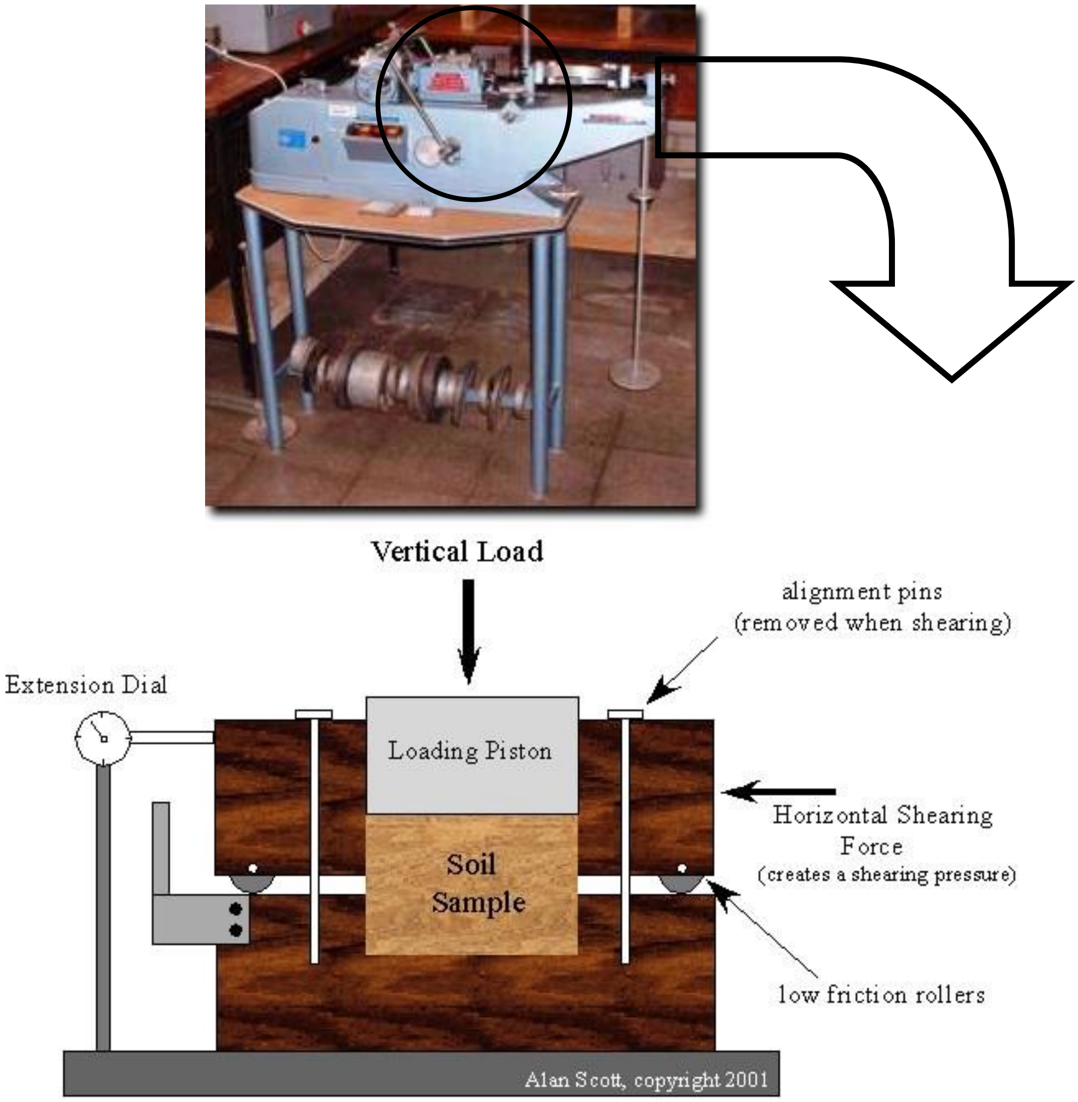


Figure No. 1: Direct Shear Test Apparatus

Apparatus:

1) Hand operated or machine driven capable of providing platen speed in the range (0.00048 - 1.2)mm/minute.
2) Load measuring ring of 3 kN ,which has a calibration factor in (kN/division).
3) Dial gauge, 25 mm travel reading to 0.002 mm sensitivity.
4) Dial gauge, 25 mm travel reading to 0.01 mm sensitivity.
5) Balance reading to 0.01 gm.
6) Drying oven.
7) Ruler.

Procedure:

1) Prepare apparatus.
2) Prepare specimen.
3) Measure specimen (length and diameter, and weight).
4) Set up specimen.
5) Record zero reading.
6) Apply.
7) Take readings: the readings shall be taken till the failure occurs in one of the following notes:
   a) Proving ring readings decreased,
   b) 4-successive readings be constant or
8) Unload .
9) Calculations:

1. Compute the shear stress as $= P_h / A$
2. Compute the normal stress as $= P_v / A$
3. Compute the horizontal displacement, εh, and the vertical displacement, εv, for each observed value.
4. Plot shearing stress against horizontal displacement and obtain the maximum value of shearing stress, τ_{max}.
5. Plot a graph of normal displacement vs. shear displacement. This should be on the same page (same horizontal scale) as the shearing stress vs. shear displacement plot.
6. Using data from all three groups, plot shearing stress against normal stress and show the angle of internal friction, ϕ, and the intercept, c.

Report:

1. Visual classification.
2. Plots of shear stress v. horizontal deflection and vertical deflection v. horizontal deflection on the same page to the same scale.
3. Mohr-Coulomb Failure Envelope.
4. State internal angle of friction and cohesion.
6. Remarks stating any errors associated with the test, specific errors witnessed, and *specifically* the manner in which these errors could have affected the results.

Direct Shear Test

Analyst Name:______________________________ Date:____________ Test
No.__________
Specific Gravity:__________ Specimen Density:__________ Normal
Load:______________
Area:______________

Specimen Dimensions: Height:_______ Area:________ Volume:________
Mass:_________
Loading Information: Prov. Ring Calibration ____________ Elapsed Time (sec)
Project Information:
Project Name:_____________________________ Project
Location:________________________
Boring No.:_______________ Sample No.:________________
Sample Description:
Source:______________________________
Condition:_________________________________
Vis. Class. & USCS
Symbol:____
Structure:_______________ Consistency:__________________

Test No. 7: Unconfined Compression Test:

Introduction:

Depending on the source of its strength, the soil can be classified into two groups:

1- Cohesion less soil: which is dependent on friction or i.e. angle of internal friction.
2- Cohesion soil which is dependent on attraction between soil particles (i.e. on C as well as φ).

Purpose of the test:

To determine the shear strength parameter for cohesive soils.

Principle of test:

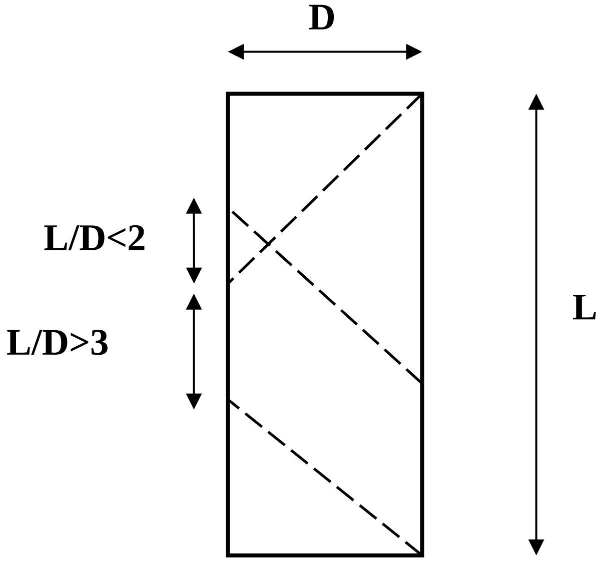
A cylindrical specimen of soil is subjected to a steadily increasing axial load until failure occurs. In the unconfined compression test, the axial load is the only force or stress which is applied. In Triaxial test, the specimen is first subjected to all round confining pressure which is then maintained constant as the axial load is increased. In either case, the rate of loading is such that failure occurs within a relatively short time usually between (5-15)minutes.

Apparatus:

1) Mechanical load frame of 10 kN capacity is suitable, either hand operated or machine driven capable of providing platen speed in the range (0.5-4)mm/minute.
2) Load measuring ring of 2 kN ,which has a calibration factor in (kN/division).
3) Dial gauge, 25 mm travel reading to 0.01 mm sensitivity.
4) Balance reading to 0.1 gm.
5) Drying oven.
6) Ruler .

Procedures:

1) Prepare apparatus:
   A rate of loading between (0.5-2%of specimen length/min), if the length of the soil specimen was 76mm, so the rate of loading must be (0.02*76/min=1.52mm/min).
2) Prepare specimen:
   The ratio of length to diameter of soil specimen should be between 2&3. This means that:
   $$3 > L/D > 2$$
   When the $(\mathrm{L/D} < 2)$ potential failure zone overlap and when$(\mathrm{L/D} < 3)$ no overlap of failure zone. If $(\mathrm{L/D} > 3)$, the specimen may act as column(buckling).



Area correction:

When a cylindrical soil specimen is compressed under an axial load, its length decreases and the ratio of the change in length(x) to the original length (L_o) at any stage is the strain,ε. In an undrained test, no water is allowed to leave the specimen and if the soil is fully saturated and its assumed that both water and the soil grains are incompressible, the volume of the specimen remains unaltered. Therefore the volume lost in the length must appear as an increase in diameter, V= constant,

<u>Before testing:</u>

$$V_O = \frac{\pi}{4} D^2 L = A_0 L_0$$

<u>After testing:</u>

$$V' = A'L' = A'(L_0 - \Delta L)$$

$$A_0 L_0 = A'(L_0 - \Delta L)$$

$$A' = \frac{A_0 L_0}{L_0 - \Delta L} \; dividing\ by\ L_O$$

$$= \frac{A_0}{1 - \frac{\Delta L}{L}} = \frac{A_0}{1 - \varepsilon_a}$$

3) Measure specimen (length and diameter, and weight).
4) Set up specimen.
5) Record zero reading.
6) Apply compression.
7) Take readings: the readings shall be taken till the failure occurs in one of the following notes:
   a) Proving ring readings decreased,
   b) 4-successive readings be constant or
   c) Take readings to 20% strain (76*0.2) =15.2 mm length decreased.
8) Unload.
9) Sketch mode of failure: there are 3 types of failure criteria:
   a) Plastic failure (barreling).
   b) Semi plastic.
   c) Brittle failure (shear).
10) Measure water content.
11) Calculate:

Table (7-1): Calculation data sheet.

Vertical dial reading, divisions	Proving ring reading, divisions	Sample deformation, ΔL , mm, (col.1*0.01)	Strain, col. (3)/L_O	Corrected area,A_O/(1 -col. (4)	Total load on sample, KN, col. (2)*CF	Stress,kPa, col. (6)/col. (5)
Col. (1)	Col. (2)	Col. (3)	Col. (4)	Col. (5)	Col. (6)	Col. (7)
0						
20						
40						
60						

12) Plot the graphs:

After the completion of the table No.(7-1), graphs of a stress-strain relationship and after that a mohr's circle graph must be plotted. The following curves show an example of plotted graphs explaining the modulus of elasticity, initial or tangent E_i or secant E_s , unconfined compressive strength, q_u and undrained shear strength, C_u which is equal to unconfined compressive strength, q_u divided by 2($C_u = \tau = \frac{q_u}{2}$).

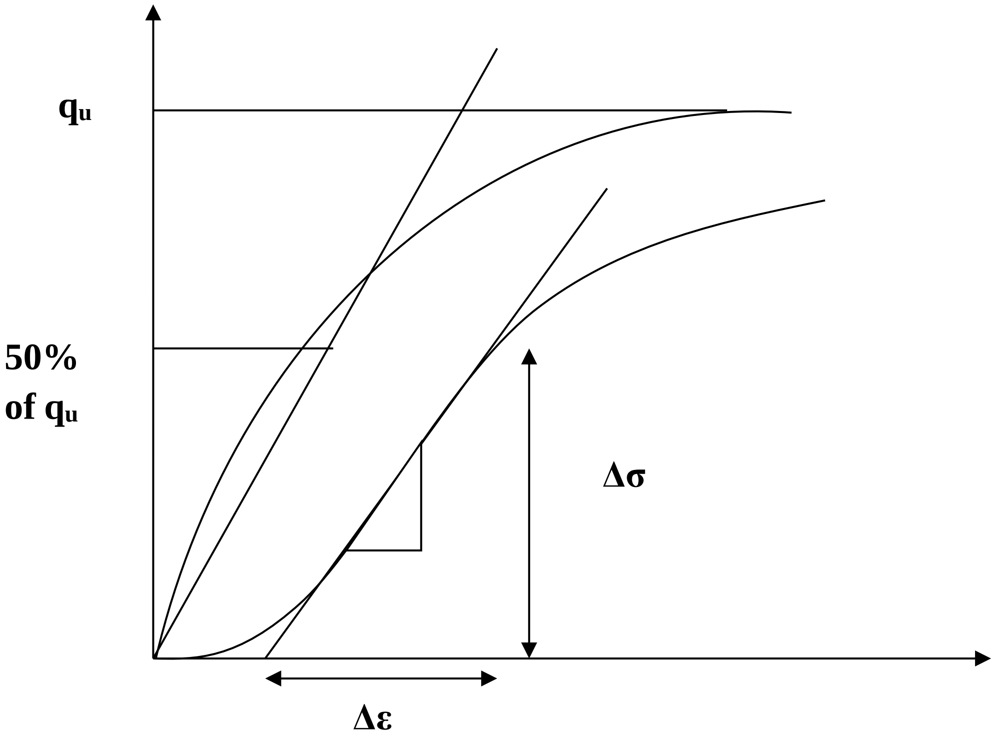


Figure (7-1): stress-strain relationship to find theq_u .

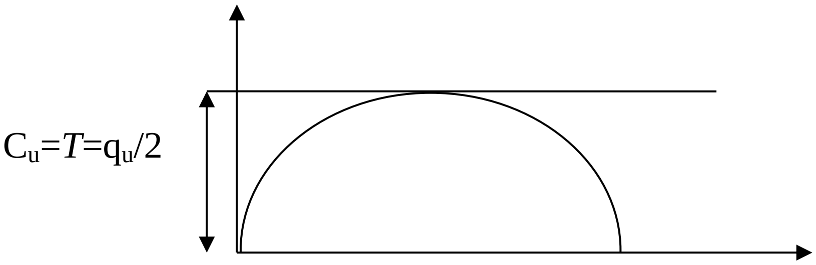


Figure (7-2): Mohr's circle graph showing the $C_u = \tau = \frac{q_u}{2}$.

Sensitivity, S_t :

An unconfined compressive strength test on a remolded specimen will give a compressive strength q_r which is lower than the compressive strength q_u of the undisturbed clay because , it becomes softer when remolded than it was when undisturbed due to breakdown of its structure. The ratio of the undisturbed to the remolded strength is known as:

$$Sensitivity = S_t = \frac{q_u}{q_r}$$

Table (7-2): some values of sensitivity.

Values	Description
<4	insensitive
4-8	sensitive
>8	Quick extra sensitive

The test has the advantage of being fast, simple, compact and, therefore, inexpensive. However, there are limitations which include:

1) The specimen must be fully saturated, otherwise compression of any air voids and expulsion of air will produce an increased strength. This should be noted when testing compacted clays practically if they are at or below optimum moisture content.
2) If the specimen contains any micro fabric such as fissures, silt partings, gravel particles or defect such as cracks or air voids, then premature failure may result because of these inherent weakness.
3) If the specimen has a low clay content then premature failure is likely since it will have poor cohesion.
4) The drainage conditions are not controlled. The test must not be carried out two slowly, otherwise undrained shear condition may not exist.

Example for Unconfined Compression Test:

Type of soil	**Silty clay**
Specific gravity	**2.75**
Water content,%	**38.5%**
Bulk density	**1.78 gm/m3**
Length of specimen	**76 mm**
Width of specimen (Diameter)	**38mm**
Strain dial factor	**0.01mm/division**
Proving ring factor(2kN)	**0.00069 kN/divison**
Rate of strain	**1.2mm/min(rapid)**

Vertical dial reading, divisions	Proving ring reading, divisions	Sample deformation, ΔL , mm, (col.1*strain dial factor)	Strain, col. (3)/L_O	% strain=col. (4)*100	Corrected area,A_O/(1-col. (4)	Total load on sample, KN, col. (2)*P.R.F	Stress,kPa, col. (7)/col. (6) (kN/m^2)
Col. (1)	**Col. (2)**	**Col. (3)**	**Col. (4)**	**Col.(5)**	**Col. (6)**	**Col. (7)**	**Col. (8)**
0	0	0	0	0	11.34	0	0
20	24	0.2	0.0026	0.26	11.37	0.0166	14.59
40	30	0.4	0.0053	0.53	11.40	0.0207	18.16
60	37	0.6	0.008	0.8	11.43	0.0255	22.31
80	43	0.8	0.0105	1.05	11.46	0.0296	25.83
100	49	1	0.013	1.3	11.49	0.034	29.59
120	54	1.2	0.0148	1.48	11.51	0.037	32.15
140	57	1.4	0.0184	1.84	11.55	0.039	33.77
160	61	1.6	0.021	2.1	11.58	0.042	36.27
180	64	1.8	0.0237	2.37	11.62	0.044	37.87
200	67	2.0	0.0263	2.63	11.65	0.046	39.49
220	70	2.2	0.029	2.9	11.68	0.048	41.10
240	73	2.4	0.0316	3.16	11.71	0.05	42.70
260	76	2.6	0.0342	3.42	11.74	0.052	44.29
280	79	2.8	0.037	3.7	11.78	0.054	45.84
300	83	3.0	0.039	3.9	11.80	0.057	48.31
320	86	3.2	0.042	4.2	11.84	0.059	49.83
340	87	3.4	0.0447	4.47	11.87	0.060	50.55
360	87	3.6	0.049	4.9	11.92	0.060	50.34

Test No. (8): UU Triaxial Test for Cohesive Soil (Silt and Clay):

1. Introduction

This test will deal only with the determination of total shear strength parameters of cohesive soils using, Unconsolidated Undrained testing procedures (UU-Test).

There may be instances where effective shear strength parameters are required, and they would require other forms of tests such as Consolidated Undrained or Drained (CU and CD test).

Specimens used for the test are of cylindrical shape and should be undisturbed or pre-prepared cohesive soil samples. Specimen is subjected to a confining fluid pressure in a triaxial chamber and axial load (deviator load) is applied in a strain controlled or stress controlled manner. In all stages of the test, undrained conditions are maintained without allowing for any pore water pressure dissipation.

2. Apparatus and Tools

Figure (1) shows the assembly of the triaxial cell used in conventional triaxial tests (UU, CU and CD).

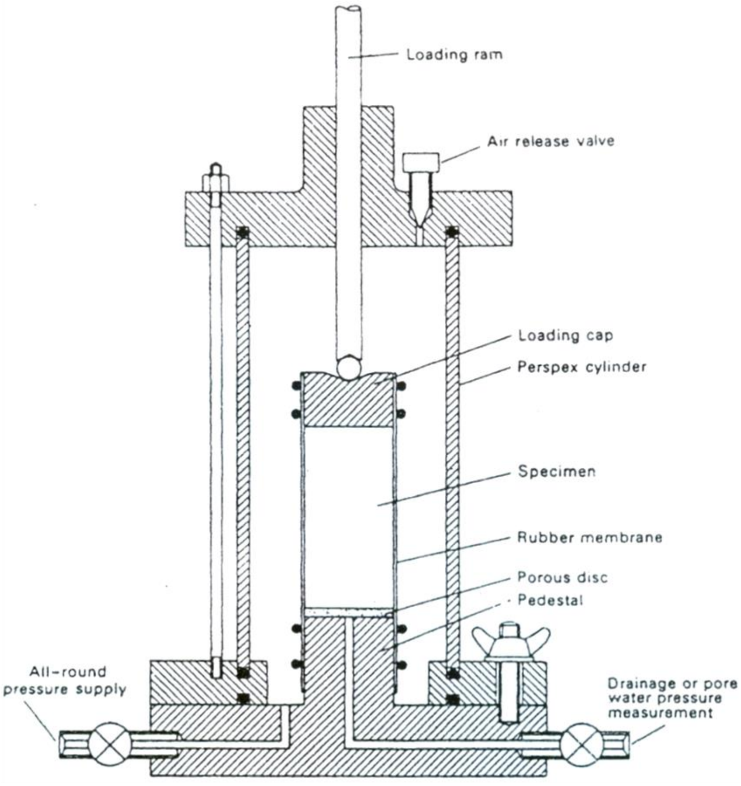


Figure 1: Assembly of Triaxial Cell

The main components of triaxial cell and other devices used in UU test are:

Triaxial Compression Chamber

An apparatus shall be provided to keep the cylindrical soil specimen, enclosed by a rubber membrane sealed to the specimen cap and base, under the applied chamber pressure. The apparatus includes a bushing and piston aligned with the axis of the specimen. Axial load is applied to the specimen through this system.

Chamber Pressure Application device

There shall be a system capable of applying and maintaining the chamber pressure constant at the desired value (within±10 kPa) throughout the test. This device is connected to the triaxial chamber through pressure control devices. Pressure may be applied through hydraulic pressure system or by compressed air. The air-water pressure system is the most widely used method for cell pressure application.

Axial Loading Device (Loading Machine)

There shall be a device to provide the axial load in a specified controlled manner at the desired rate. It should have a sufficient loading capacity and should be free from vibrations.

Axial Load Measuring Device (Proving Ring)

There shall be a device to measure the deviator load applied to the specimen. This may be a Proving ring, hydraulic load cell or an electronic load cell with sufficient accuracy.

Axial Deformation Measuring Device (Dial Gage)

There shall be a device to measure the axial deformation of the specimen. This may be a dial gauge reading to an accuracy of 0.01 mm/division.

Specimen cap and base

There should be an impermeable, rigid cap and base to prevent drainage of the specimen. Both the cap and base should have a plane surface of contact and a circular cross section of diameter equal to that of the specimen. The specimen base should be coupled to the triaxial chamber base to prevent any lateral motion or tilting.

Rubber Membranes, Membrane stretcher and o - rings

Rubber membranes should be used to encase the specimen to provide reliable protection against leakage. The membrane thickness shall not exceed 1 % of the diameter of the specimen. The membrane shall be sealed to the specimen base and cap with rubber 0 – rings.

Devices for Specimen Weighing and Measuring

There should be a device to measure the height and diameter of the specimen to the nearest 0.3 mm and a weighing device to weigh the soil to the nearest 0.01 g.

3. Testing Procedure and sample preparation

Preparation of the Sample

Specimens used for the test shall be undisturbed. They should be of a minimum diameter 33 mm and a have a length/diameter ratio between 2 and 3. Specimen should be weighed to the nearest 0.01 g prior to the testing. In traditional way, the sample size is 38mmdia by 76mm height. Specimens should be handled very carefully to minimize disturbance, change cross section or loss of moisture. Specimens shall be uniform circular cross section with ends perpendicular to the axis of the specimen.

Weight of the specimen should be determined and encased by the membrane and sealed to the specimen base and cap immediately after the preparation, with the help of O rings.

Testing Procedures

Triaxial chamber shall be assembled with the specimen encased in rubber membrane, and sealed to the specimen cap and base and kept in position. Axial load piston should be brought to contact with the specimen cap and proper seating should be provided.

When dealing with soft soils special care must be taken not to overload the specimen with the weight of the piston. Chamber shall be filled with the confining water and placed in position in the axial loading device. Special care should be taken in aligning the axial load device, the axial load-measuring device and the triaxial chamber to prevent application of lateral force to the piston during testing. Thereafter the chamber pressure maintaining and measuring device shall be attached and adjusted to provide the desired chamber pressure.

The proving ring is usually located outside the triaxial chamber and chamber pressure will produce an upward force on the piston that will react against the proving ring. In this case axial load measuring device should be adjusted to read zero prior to the application of the deviator load.

Application of the Axial Load (Using Controlled Strain)

The axial load may be applied at the desired strain rate, approximately 10-15min after the application of chamber pressure. Proving ring readings should be recorded for intervals of axial deformation. Sufficient readings should be taken to capture the stress-strain curve. Thus, more reading that is frequent is required, at the initial stages to capture the initial stiff part of the curve and as the failure approaches to capture the failure point.

The failure of the sample can be achieved by:

1. Reduction in the readings of proving ring.
2. Three to five constant readings of the proving ring.
3. If the sample has not failed showing a reduction in the deviator load, loading shall be continued to 15-20% strain. If the residual strengths are required test may be continued further.

At the end of the test specimen shall be taken out, failure patterns may be noted and moisture content of the sample should be determined. Test should be performed on at least two other identical samples at different chamber pressures to construct the failure envelope and to determine the shear strength parameters.

4. Calculations

The calculations of the UU test have the same procedures of the calculations of the unconfined compression test. However,

$$A_c = \frac{A_o}{1 - \epsilon_a}$$

$$\epsilon_a = \frac{\Delta L}{L_o}$$

Deviator stress=$\sigma_1 - \sigma_3 = \frac{proving\ ring\ factor * Reading\ of\ Proving\ Ring}{A_c}$

5. Presentation of Results

Report should include

1. The state of the sample; i. e. undisturbed / remolded
2. Rate of strain (usually 1.5mm/min.)

3. Visual description of specimen, perhaps with the soil group symbol,
4. Initial dry unit weight and moisture content for all the specimen tested at different cell Pressures,
5. Deviator stress at failure at different cell pressures, and therefore the minor and major principal stresses,
6. Axial strain at maximum deviator stress for all cell pressures,
7. Remarks about any unusual conditions observed or failure patterns observed,
8. Mohr circles of stress at failure for all the cell pressures
9. Soil shear strength parameters C_u and $ø_u$
10. Deviator stress vs. axial strain information for all the cell pressers together with the stress strain curves.

Typical test presentation and results

1. Sheet of data and readings

Dia (mm) = 38 **height (mm) = 76**
Client: **Project:**
Proving constant = 1.4*10^{-3}kN/div. **Axial strain reading=0.01mm/division**

Strain dial reading	cell pressure, kPa			Strain	Strain, %	$(\sigma_1-\sigma_3)$kPa for $\sigma_3=100kPa$	$(\sigma_1-\sigma_3)$kPa for $\sigma_3=200kPa$	$(\sigma_1-\sigma_3)$kPa for $\sigma_3=300kPa$
	100	200	300					
0	0	0	0	0	0	0	0	0
20	11	9	25	0.003	0.3	13.54	11.08	30.78
40	21	35	27	0.005	0.5	25.79	42.98	33.15
60	48	50	52	0.008	0.8	58.78	61.23	63.68
80	63	58	63	0.011	1.1	76.95	70.84	76.95
100	70	64	69	0.013	1.3	85.27	77.96	84.05
120	84	73	84	0.016	1.6	102.05	88.69	102.05
140	93	82	93	0.018	1.8	112.68	99.36	112.68
160	99	83	98	0.021	2.1	119.63	100.30	118.42
180	103	84	102	0.024	2.4	124.13	101.23	122.93
200	108	85	104	0.026	2.6	129.81	102.16	125.00
250	109	87	109	0.033	3.3	130.12	103.86	130.12
300	110	90	113	0.039	3.9	130.42	106.71	133.98
350	112	95	113	0.046	4.6	131.89	111.87	133.06
400	112	95	112	0.053	5.3	130.98	111.10	130.98
Maximum deviator stress $(\sigma_1-\sigma_3) at\ faulire$						130.42	111.87	133.98

2.Table for calculating and drawing Mohr circle

σ_3 kPa	$(\sigma_1-\sigma_3)$f, kPa	σ_1kPa	$\frac{(\sigma_1-\sigma_3)\text{f}}{2}$	$\frac{(\sigma_1+\sigma_3)\text{f}}{2}$
100	130.42	230.42	65.21	165.21
200	111.87	311.87	55.94	255.94
300	133.98	433.98	66.99	366.99

The required data to draw the stress strain relationship are:

UU Test – Data for the Stress – Strain Plot			
Strain %	**Cell Pressure 100**	**Cell Pressure 200**	**Cell Pressure 300**
0	0	0	0
0.3	13.54	11.08	30.78
0.5	25.79	42.98	33.15
0.8	58.78	61.23	63.68
1.1	76.95	70.84	76.95
1.3	85.27	77.96	84.05
1.6	102.05	88.69	102.05
1.8	112.68	99.36	112.68
2.1	119.63	100.30	118.42
2.4	124.13	101.23	122.93
2.6	129.81	102.16	125.00
3.3	130.12	103.86	130.12
3.9	130.42	106.71	133.98
4.6	131.89	111.87	133.06
5.3	130.98	111.10	130.98

From Figure 3, undrained cohesion cu=65.5 kPa, Angle of internal friction=0.0 degrees

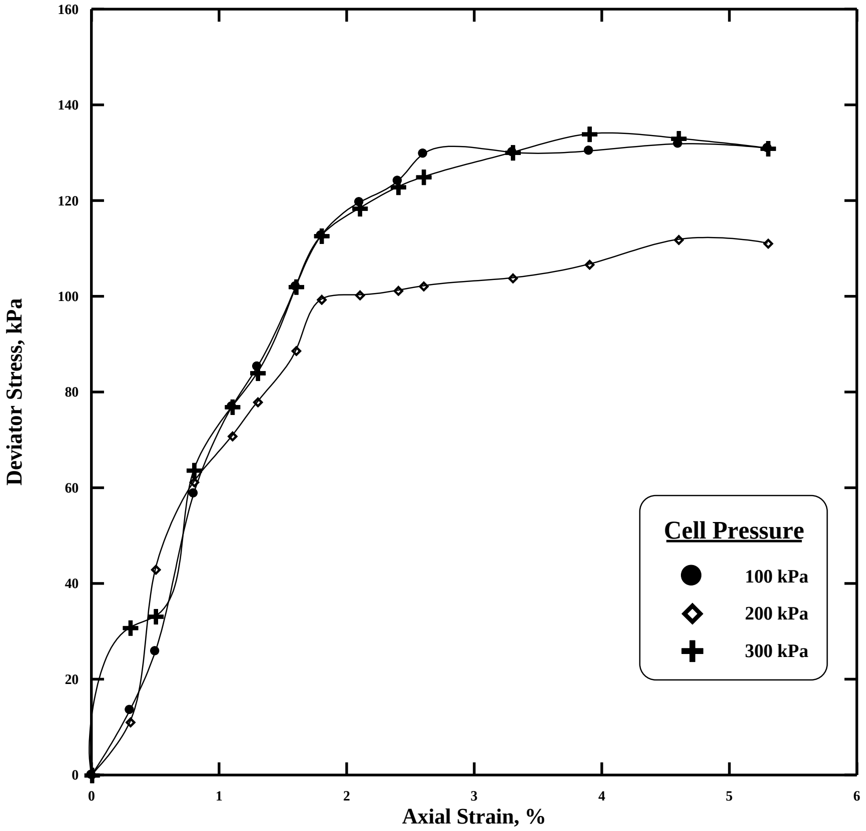


Figure 2: Stress-Strain Relationship for the UU Triaxial Test

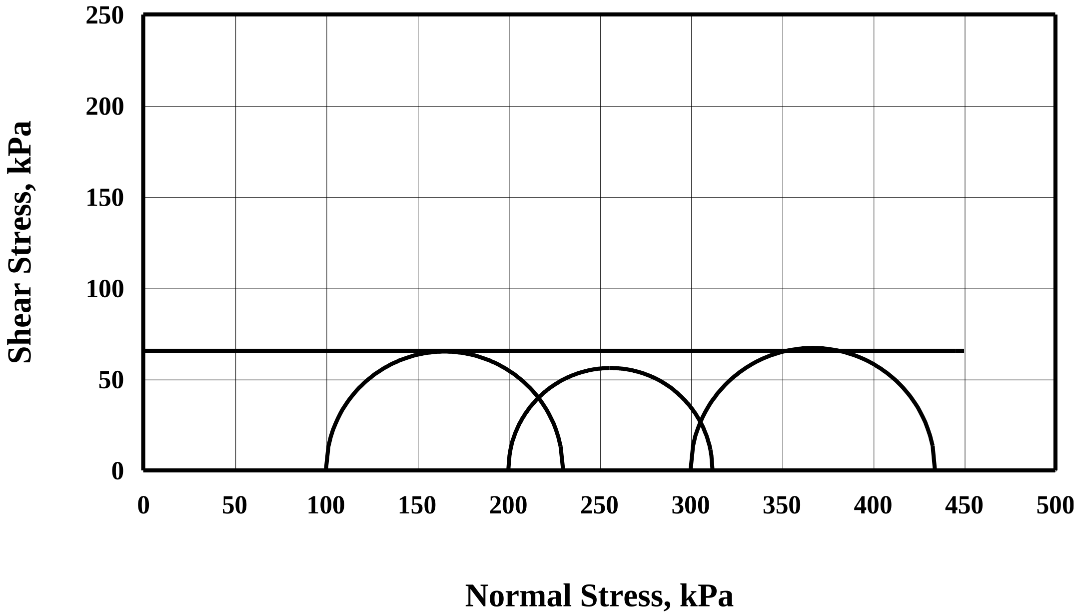


Figure 3: Mohr's circle and failure envelope

Experiment No. 8: Unconsolidated Undrained Triaxial Test