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Laboratory testing of soils

Part I. The solid components, physical properties and permeability of soils

Методические указания к выполнению лабораторных работ
по курсу «Грунтоведение»
для студентов обучающихся по направлению 130100
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INTRODUCTION

Soil mechanics has been developed in the beginning of the 20th century. The need for the analysis of the behavior of soils arose in many countries, often as a result of spectacular accidents, such as landslides and failures of foundations.

Soil mechanics is the science of understanding and predicting how soil will respond to externally applied forces (or pressures).

Soil mechanics has become a distinct and separate branch of engineering mechanics because soils have a number of special properties, which distinguish the material from others. Its development has also been stimulated, of course, by the wide range of applications of soil engineering in civil engineering, as all structures require a sound foundation and should transfer its loads to the soil.

Laboratory testing of soils is an important element of soil mechanics. The laboratory testing must be planned in advance but flexible to be modified based on subsurface findings and test results. The complexity of testing required for a particular project may range from simple moisture content determinations to specialized strength testing.

The word "soil", in its traditional meaning, is a natural body comprised of solids (minerals and organic matter), liquid, and gases that occurs on the land surface, occupies space, and is characterized by simplicity the following: horizons, or layers, that are distinguishable from the initial material as a result of additions, losses, transfers, and transformations of energy and matter. Soil covers the earth's surface as a continuum, except on bare rock, in areas of perpetual frost or deep water, or on the bare ice of glaciers.

Soil tests are performed to determine specific soil properties and how the soil responds to imposed conditions. Types of behavior depend on the strength, compressibility, permeability and index properties. There are a number of tests that can be used to determine the desired properties, depending on the soil type and application. The Engineer determines the number, types, and requirements (such as site-specific confining stress levels for triaxial tests) of needed tests. The Engineer should be familiar with each test procedure and should verify that the tests are being performed according to his directions. Familiarity with testing procedures and the soil samples helps the Engineer to appropriately apply the test results in his subsequent analyses.

In tests are used disturbed and undisturbed sample. *Undisturbed sample* (as close to undisturbed as possible) keeps the same form or condition it had when in the ground. Undisturbed samples are used to determine the in place

strength, compressibility (settlement), natural moisture content, unit weight, permeability, discontinuities, fractures and fissures of subsurface formations. *Disturbed sample* has been "disturbed" and no longer has the same form (i.e. density). The grain size, liquid limit, plastic limits, specific gravity, and some compaction tests can be performed on this sample. Disturbed samples are generally obtained to determine the soil type, gradation, classification, consistency, density, presence of contaminants, stratification, etc. Figure 1.0 depicts the general process of traditional drilling, sampling, and laboratory testing of collected samples.
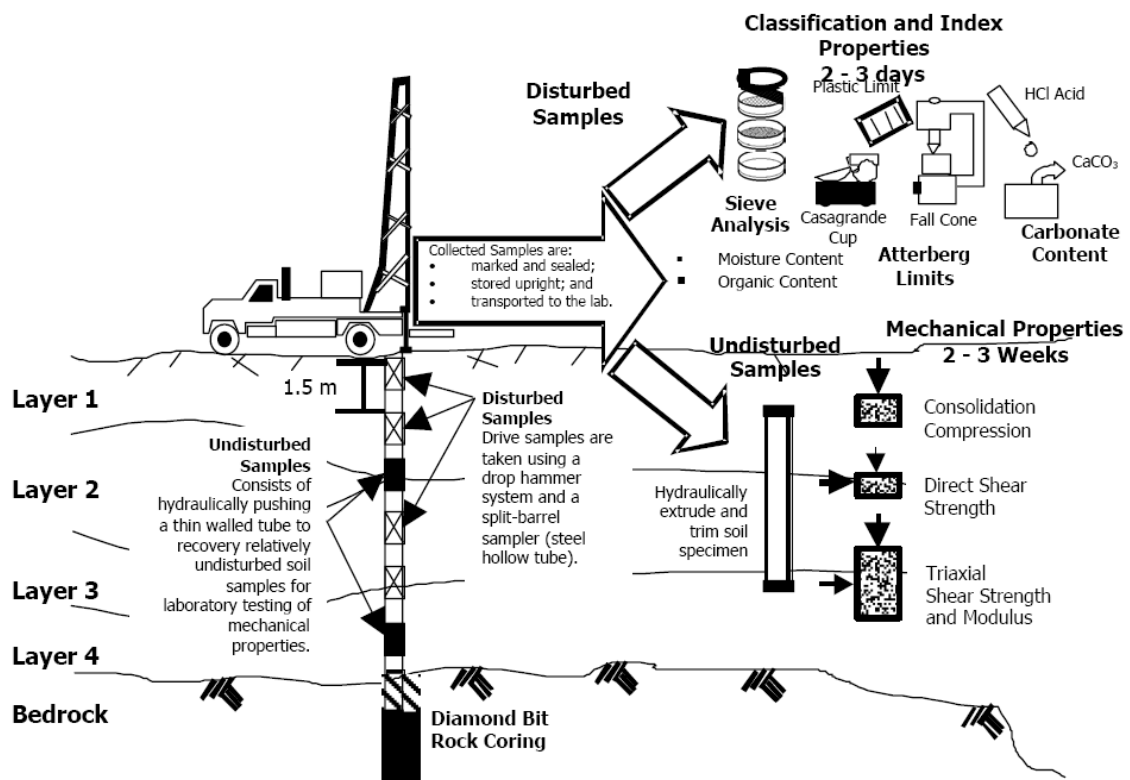

Figure 1.0. Traditional drilling, sampling, and laboratory testing of collected samples

The most important tests and special properties of soils will be described briefly in this work.

1. THE SOLID COMPONENTS OF SOILS

1.1. The mineralogical composition of soil

The engineering characteristics of a soil depend on three major components of the soil structure mineral's particles, water and air. *Mineral particles* provide the bulk and therefore the strength of the soil. *Water* in small quantities provides a lubricating effect to the soil particles thus

assisting in compaction and adhesion. *Air* (voids) in the soil structure will lead to a deterioration of the material and should therefore be removed e.g. by compaction.

For a civil engineer the mineralogical composition of soil may be useful as a warning of its characteristics, and as an indication of its difference from other materials, especially in combination with data from earlier projects. The mineralogical and organic composition of soil can be helpful in distinguishing between various types of soils.

The mineral composition of site materials varies greatly from place to place, depending upon the genesis of the materials and the geologic processes involved. The mineral composition may vary also with particle size at a particular site. The proportion of platy minerals usually increases over equidimensional minerals as the particle size decreases.

Sand and gravel usually consist of the same minerals as the original rock from which they were created by the erosion process. *The coarse-grained materials* are normally dominated by those rock-forming minerals, which are more resistant to chemical weathering, such as quartz and the heavy minerals. Rock fragments and unaltered rock-forming minerals, such as feldspar, calcite, and mica also may be present. The less complex minerals in the coarse-grained fractions can be identified readily by megascopic methods. Wherever this is possible, the predominant rock or mineral constituents and those rocks and minerals having a deleterious effect on engineering properties should be noted, using standard geologic terms. *The fine-grained materials* represent the products of chemical and mechanical weathering. The mineral composition, together with weathering processes, controls the ultimate size and shape of the fine-grained particles. Quartz, feldspar, and many other minerals may, under mechanical weathering, be reduced to finegrained equidimensional particles, such as in rock flour. Some types of minerals are broken down mechanically into platy particles. Micas are of this type. Alteration products of other types of minerals may result in the formation of platy particles.

Clay soils may contain the same minerals, but they also contain the so-called clay minerals, which have been created by chemical erosion. There are three principal groups of clay minerals: kaolinites, montmorillonites, and illites. Because of variable influence of each type on the engineering property of soils, it is important that the predominant clay mineral be properly identified whenever possible. These minerals consist of compounds of aluminum with hydrogen, oxygen and silicates.

They differ from each other in chemical composition, but also in geometrical structure, at the microscopic level. The microstructure of clay usually resembles thin plates. On the microscope there are forces between

these very small elements, and ions of water may be bonded. Because of the small magnitude of the elements and their distances, these forces include *electrical forces and the Van der Waals forces.* Clay minerals are composed of layers of two types: (1) silicon and oxygen (silica layer) and (2) aluminum and oxygen or aluminum and hydroxyl ions (alumina or aluminum hydroxide layer).

The kaolinite clays consist of two layer molecular sheets, one of silica and one of alumina. The sheets are firmly bonded together with no variation in distance between them. Consequently the sheets do not take up water. The kaolinite particle sizes are larger than those of either montmorillonite or illite and are more stable.

The montmorillonite clays consist of three layer molecular sheets consisting of two layers of silica to one of alumina. The molecular sheets are weakly bonded, permitting water and associated chemicals to enter between the sheets. As a result, they are subject to considerable expansion upon saturation and shrinkage upon drying. Particles of montmorillonite clay are extremely fine, appearing as fog under the high magnification of the electron microscope. Montmorillonite clays are very sticky and plastic when wet, and are of considerable concern in respect to problems of shear and consolidation.

Illite has the same molecular structure as montmorillonite but has better molecular bonding, resulting in less expansion and shrinkage properties. Illite particles are larger than montmorillonite and adhere to each other in aggregates.

1.2. The organic composition of soil

`Organic content of soils` help classify the soil and identify its engineering characteristics. Organic soils are those formed throughout the ages at low-lying sediment-starved areas by the accumulation of dead vegetation and sediment. Organic material accumulates in wet places where it is deposited more rapidly than it decomposes. A sample composed primarily of vegetable tissue in various stages of decomposition and has a fibrous to amorphous texture, a dark-brown to black color, and an organic odor should be designated as a highly organic soil and shall be classified as peat. In some countries, such as the Russia, Canada, Netherlands, soil may also contain layers of peat.

In describing organic soils, the material is called *peat* (fibric) if virtually all of the organic remains are sufficiently fresh and intact to permit identification of plant forms. It is called *muck* (sapric) if virtually all of the material has undergone sufficient decomposition to limit recognition of the plant parts. It is called *mucky* peat (hemic) if a significant part of the material can be recognized and a significant part cannot.

It is not sufficient to simply label a soil as "organic" without showing the organic content. Descriptions of organic material should include the origin and the botanical composition of the material to the extent that these can be reasonably inferred. The principal general kinds of peat, according to origin are:

‒ *sedimentary peat* consists the remains mostly of floating aquatic plants, such as algae, and the remains and fecal material of aquatic animals, including coprogenous earth;

‒ *moss peat* includes the remains of mosses, including sphagnum (figure 1.1), magellanicum, angustifolium;

‒ *herbaceous peat* contains the remains of sedges, reeds, cattails, and other herbaceous plants;

‒ *woody peat* involves the remains of trees, shrubs, and other woody plants.

This peat in turn may become parent material for soils. Many deposits of organic material are mixtures of peat. Some organic soils formed in alternating layers of different kinds of peat. In places peat is mixed with deposits of mineral alluvium or volcanic ash. Some organic soils contain layers that are largely or entirely mineral material.

Chemically peat consists partly of carbon compounds. The organic content is then calculated from the weight of the ash generated. Oven-dried (at 110±5 ^{o}C) samples after determination of moisture content are further gradually heated to 440 ^{o}C which is maintained until the specimen is completely ached (no change in mass occurs after a further period of heating).
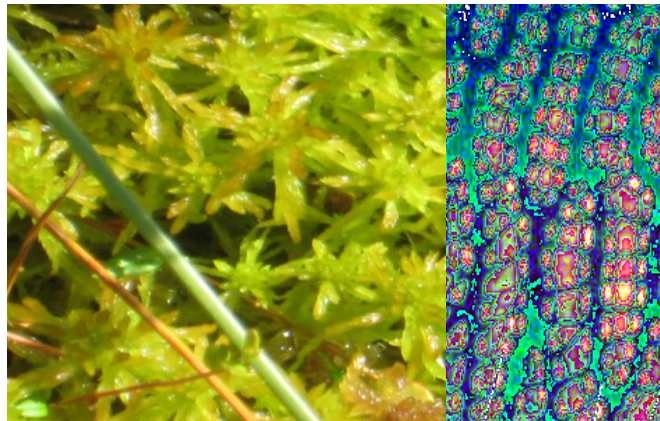
Figure 1.1. Sfagnum moss and cells of leaf

1.3. Specialty of the basic behavior of soils

Soils have a number of properties that distinguish it from other materials. *Firstly,* a special property is that soils can only transfer compressive normal stresses, and no tensile stresses. *Secondly,* shear stresses can only be transmitted if they are relatively small, compared to the normal stresses. *Furthermore* it is characteristic of soils that part of the stresses is transferred by the water in the pores.

Soils found in nature are usually a combination of soil types. A *well-graded* soil consists of a wide range of particle sizes with the smaller

particles filling voids between larger particles. The result is a dense structure that lends itself well to compaction.

The physical and mechanical behavior of the main types of soil, sand, clay and peat, is rather different. There are three basic soil groups: *cohesionless, cohesive, or organic.*

Cohesionless (granular) soils have particles that do not tend to stick together. Mostly of granular soil composed of sand, maybe some silt. Coarse grains can be seen. Feels gritty when rubbed between fingers. When water and soil are shaken in palm of hand, they mix. When shaking is stopped they separate. Very little or no plasticity. Little or no cohesive strength when dry. Soil sample will crumble easily. Granular soils are known for their water-draining properties. Sand and gravel obtain maximum density in either a fully dry or saturated state. Sand usually is rather permeable, and rather stiff, especially under a certain preloading. It is also very characteristic of granular soils such as sand and gravel, that they can not transfer tensile stresses. The particles can only transfer compressive forces, no tensile forces. Only when the particles are very small and the soil contains some water, can a tensile stress be transmitted, by capillary forces in the contact points.

Cohesive soils are characterized by very small particle sizes where surface chemical effects predominate. They are both "sticky" and "plastic". Cohesive soils have the smallest particles. Grains cannot be seen by naked eye. Feels smooth and greasy when rubbed between fingers. When water and soil are shaken in palm of hand, they will not mix. They are plastic when wet and can be molded, but become very hard when dry. Can be rolled. Has high strength when dry. Crumbles with difficulty. Slow saturation in water. Clay is used in embankment fills and retaining pond beds. Cohesive soils are dense and tightly bound together by molecular attraction. Proper water content, evenly distributed, is critical for proper compaction. Cohesive soils usually require a force such as impact or pressure. Silt has a noticeably lower cohesion than clay. However, silt is still heavily reliant on water content. Clay usually is much less permeable for water than sand, but it usually is also much softer.

Although the interaction of clay particles is of a different nature than the interaction between the much larger grains of sand or gravel, there are many similarities in the global behavior of these soils. There are some essential differences, however. The deformations of clay are time dependent, for instance. When a sandy soil is loaded it will deform immediately, and then remain at rest if the load remains constant. Under such conditions a clay soil will continue to deform, however. This is called creep. It is very much dependent upon the actual chemical and mineralogical constitution of the clay. Also, some clay, especially clays containing large amounts of

montmorillonite, may show a considerable swelling when they are getting wetter.

Thus it is important to determine the organic content of soils. The consolidation characteristics, permeability, strength and stabilization of these soils are largely governed by the properties of organic materials. It may be mentioned that some clays may also contain considerable amounts of organic material. Organic materials affect the behavior of soils in varying degrees. The behavior of soils with low organic contents (<20% by weight) generally are controlled by the mineral components of the soil. When the organic content of soils approaches 20%, the behavior changes to that of organic, or peaty soils.

Peat is usually is very light (some times hardly heavier than water), and strongly anisotropic because of the presence of fibres of organic material. As a foundation material it is not very suitable, also because it is often typically spongy, crumbly, very compressible. They are undesirable for supporting structures. It may even be combustible, or it may be produce gas.

2. PHYSICAL PROPERTIES OF SOILS

2.1. Weight-volume concepts

Soils usually consist of soil grains, water and gas (air), figure 2.1. The soil grains are irregularly shaped solids which are in contact with other adjacent soil grains. In order to describe a soil various parameters are used to describe the distribution of these three components, and their relative contribution to the volume of a soil. These are also useful to determine other parameters, such as the weight of the soil.

The weight and volume of a soil sample depends on the specific gravity of the soil grains (solids), the size of the space between soil grains (voids and pores) and the amount of void space filled with water.

Figure 2.1. The constituent of soil

An important basic parameter is the *porosity, n,* defined as the ratio of the volume of the pore space and the total volume of the soil (usually expressed as a percentage),
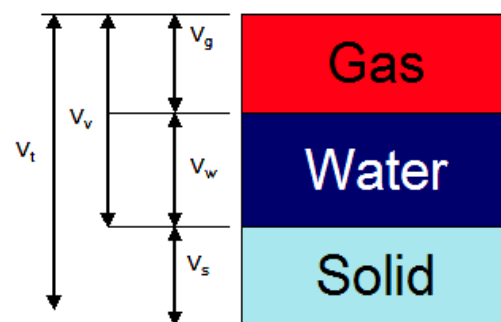
$$n = \frac{Volume\ of\ voids(V_v)}{Total\ volume\ of\ soil\ sample(V_t)}.$$

For most soils the porosity is a number between 0,30 and 0,45 (or, as it is usually expressed as a percentage, between 30 % and 45 %). When the porosity is small the soil is called densely packed, when the porosity is large it is loosely packed.

The amount of pores can also be expressed in the **void ratio e,** defined as the ratio of the volume of the pores to the volume of the solids,

$$e = \frac{Volume\ of\ voids\,(V_v)}{Volume\ of\ solids\,(V_s)}.$$

In many countries this quantity is preferred to the porosity, because it expresses the pore volume with respect to a fixed volume (the volume of the solids). Because the total volume of the soil is the sum of the volume of the pores and the volume of the solids,

$$V_t = V_v + V_s.$$

This value can be determined directly by weighing a volume of dry soil. In order to dry the soil a sample may be placed in an oven. The temperature in such an oven is usually close to 100 degrees, so that the water will evaporate quickly. At a much higher temperature there would be a risk that organic parts of the soil would be burned.

The porosity and the void ratio can easily be related,

$$e = n/(1-n);$$
$$n = e/(1+e\ ).$$

This is a common method to determine the porosity in a laboratory.

Unfortunately, this procedure is not very accurate for soils that are almost completely saturated, because a small error in the measurements may cause that one obtains, for example, $S = 0,97$ rather than the true value $S = 0,99$. In itself this is rather accurate, but the error in the air volume is then 300 %. In some cases, this may lead to large errors, for instance when the compressibility of the water-air-mixture in the pores must be determined.

2.2. Water content

Water content, w, is defined as the ratio, expressed as a percentage, of the weight of water in a given soil mass to the weight of solid particles. The water content is especially useful parameter, particularly for clays.

Determination of the moisture content of soils is the most commonly used laboratory procedure. Purpose is to determine the amount of water present in a quantity of soil in terms of its dry weight and to provide general correlations with strength, settlement, workability and other properties. The moisture content of soils, when combined with data obtained from other tests, produces significant information about the characteristics of the soil. For example, when the in situ moisture content of a sample retrieved from below the phreatic surface approaches its liquid limit, it is an indication that the soil in its natural state is susceptible to larger consolidation settlement.

By definition the water content w is the ratio of the weight (or mass) of the water and the solids:

$$w = \frac{Mass\ of\ water\ (M_w)}{Mass\ of\ soil\ solids\ (M_s)} \cdot 100\%.$$

The method of moisture content determination in laboratory conditions specifies the amount of water contained in soil. It is done by drying the soil sample at a temperature of about 105±5 ^{o} C to a constant weight (evaporate free water); this is usually achieved in 12 to 24 hours.

Apparatus: thermostatically controlled oven maintained at a temperature of 105± 5°C; weighing balance, with accuracy of 0.04% of the mass of the soil taken; airtight container made of non-corrodible material with lid; tongs.

Procedure

1. Clean the container, dry it and weight it with lid (M_1).

2. Take the required quantity of the wet specimen in the container and clean it with lid. Take the mass (M_2). The soil specimen should be representative of the soil mass. The quantity of the specimen taken would depend upon the gradation and the maximum size particles.

3. Place the container, with its lid removed, in the oven till its mass becomes constant (Normally for 24 hours).

4. When the soil has dried, remove the container from the oven, using tongs.

5. Find the mass (M_3) of the container with lid and dry soil sample.

The water content (w) of a soil sample is equal to the mass of water divided by the mass of solids:

$$w = [(M_2 - M_3) / (M_3 - M_1)]\ x100;$$

where M_1 – Mass of empty container, with lid; M_2 – Mass of the container with wet soil & lid; M_3 – Mass of the container with dry soil & lid.

For organic soils, a reduced drying temperature of approximately 40-60 ^{o}C is recommended. The wet sample is weighed, and then oven-dried to a

constant weight at. The weight after drying is the weight of solids. The change in weight, which has occurred during drying, is equivalent to the weight of water.

Serious errors may be introduced if the soil contains other components, such as petroleum products or easily ignitable solids. When the soils contain fibrous organic matter, absorbed water may be present in the organic fibers as well as in the soil voids. The test procedure does not differentiate between pore water and absorbed water in organic fibers (although the procedure does suggest evaluating organic soils at a lower temperature of 60 $^{\circ}$C to reduce decomposition of highly organic soils). Thus the moisture content measured will be the total moisture lost rather than free moisture lost (from void spaces).

Degree of saturation. Degree of saturation, *S*, is the ratio (expressed as a percentage) of the volume of water in a given soil mass to the total volume of voids. The pores of a soil may contain water and air. To describe the ratio of these two the degree of saturation *S* is introduced as

$$S = \frac{Total\ volume\ of\ voids\ contains\ water\,(V_w)}{Total\ volume\ of\ voids\,(V_v)} \times 100\%.$$

Here V_w is the volume of the water, and V_v is the total volume of the pore space. The volume of air (or any other gas) per unit pore space then is 1. For a completely saturated soil (*S* = 1) and assuming that $\rho_p/\rho_w = 2.65$, it follows that void ratio *e* is about 2.65 times the water content.

2.3. Density

The term density refers to mass per unit volume. The density of a mass of soil is of interest to the engineer for a variety of reasons including the design of earthworks and foundations and in slope stability analysis.

Dry unit weight, or dry density, is the weight of ovendried soil solids per unit of total volume of soil mass. Dry density is – the density of the soil in dry state:

$$\rho_d = \frac{Mass\ of\ soil\ solids\,(M_s)}{Total\ volume\ of\ soil\ sample\,(V_t)}.$$

Wet density, is the weight (solids plus water) per unit of total volume of soil mass, irrespective of the degree of saturation. Volume density of soil with natural moisture is expressed as a ratio of the weight and total volume of a sample with natural moisture

$$\rho = \frac{Mass\ of\ soil\ sample\,(M_s + M_w)}{Total\ volume\ of\ soil\ sample\,(V_t)}.$$

M_s+M_w – mass of soil solids and water (0%<S<100%, unsaturated), V_t – volume of a sample with natural moisture, units: kg/m^3

Saturated density (S=100%)

$$\rho_{sat} = \frac{Mass\ of\ soil\ solids + water(M_s + M_w)}{Total\ volume\ of\ soil\ sample(V_t)}.$$

The test consists in determining the moisture content of a volume unit of soil. It is determined as a quotient of soil weight and its volume. In laboratory conditions, volume density is determined using a sampling ring, by direct calculation of volume (for samples of regular geometric shape) or by means of underwater weighing. This sets the limitations concerning the applicability of laboratory methods, which include the following soil types:

fine-grained cohesive soils of soft to stiff consistency without sand admixture – sampling ring or cylinder method, underwater weighing method;

fine-grained cohesive soils of solid consistency – underwater weighing method;

fine-grained cohesive soils with sand admixture up to 2 mm, single grains or chips up to 4 mm – sampling cylinder method.

For the description of the density and the volumetric weight of a soil, the densities of the various components are needed. For water the density is denoted by ρ_w, and its value is about 1000 kg/m^3. Small deviations from this value may occur due to temperature differences or variations in salt content. In soil mechanics these are often of minor importance, and it is often considered accurate enough to assume that ρ_w = 1000 kg/m^3. For the analysis of soil mechanics problems the density of air can usually be disregarded.

Particle density or specific gravity of the soil grains is a measure of the actual particles which make up the soil mass and is defined as the ratio of the mass of the particles to the mass of the water they displace. A knowledge of the particle density is essential in relation to other soil tests. A value of specific gravity is necessary to compute the void ratio of a soil and porosity, it is used in the hydrometer analysis, and it is useful to predict the unit weight of a soil and is particularly important when compaction and consolidation properties are being investigated.

The density of the solid particles depends upon the actual composition of the solid material. Specific gravity of the soil grains typically varies between 2600 and 2800 kg/m^3. In many cases, especially for quartz sands, its value is about ρ_s = 2650 kg/m^3, for clay – ρ_s = 2750 kg/m^3. Occasionally, the specific gravity may be useful in soil mineral classifications; e.g., iron minerals have

a larger value of specific gravity than silica.

The specific gravity is determined as the ratio of the weight of a given volume of soil solids at a given temperature to the weight of an equal volume of distilled water at that temperature, both weights being taken in air:

$$G = \rho_s / \rho_w.$$

Apparatus: 50 ml density bottle with stopper (figure 2.2); oven; constant temperature water bath (27°C); vacuum desiccator; weighing balance of accuracy 0,001 g; spatula.

Figure 2.2. Density Bottles

Procedure

1. Clean the bottle with distilled water, dry it in oven, cool in desiccator and weigh it with stopper.

2. Keep about 10-15 g of this soil in the bottle. Disturbed soil sample is enough for this test. Take pulverized soil passed through 2 mm sieve.

3. Cover the soil with air free distilled water from the glass wash bottle and leave for a period of 2 to 3 hours for soaking. Add water to fill the bottle to about half.

4. Entrapped air can be removed by heating the density bottle on a water bath or a sand bath.

5. Keep the bottle without stopper in vacuum desiccator for about 1 to 2 hours until there is no further loss of air.

6. Gently stir the soil in the density bottle by a clean glass rod, wash off carefully adhering particles from the rod with some drops of distilled water and see that no more soil particles are lost.

7. Repeat the process till no more air bubbles are observed in the soil water mixture.

8. Observe the temperature of the constant in the bottle and record.

9. Insert the stopper in the density bottle, wipe and weigh.

10. Now make the bottle empty, clean thoroughly till the density bottle with distilled water at the same temperature. Insert the stopper in the bottle, wipe dry from the out side and weigh.

11. Take at least two such observations for the same soil.

Some qualifying words like *true, absolute, apparent, bulk or mass,* etc. are sometimes added to "specific gravity". These qualifying words modify the sense of specific gravity as to whether it refers to soil grains or to soil mass. The soil grains have permeable and impermeable voids inside them. If all the internal voids of soil grains are excluded for determining the true volume of grains, the specific gravity obtained is called *absolute* or *true*

specific gravity. Complete de-airing of the soil-water mix during the test is imperative while determining the *true* or *absolute* value of specific gravity.

The unit weight is frequently used than the density is (e.g. in calculating the overburden pressure).

$$Unit\ weight,\ \gamma = \frac{Weight}{Volume} = \frac{Mass \cdot g}{Volume},$$

$$\gamma = \rho \cdot g = \rho \cdot 9.8 \frac{m}{sec^2},$$

$$Water,\ \gamma = 9.8 \frac{kN}{m^3}.$$

The measurement of unit weight for undisturbed soil samples in the laboratory is simply determined by weighing a portion of a soil sample and dividing by its volume. This is convenient with thin-walled tube (Shelby) samples, as well as piston, Sherbrooke, Laval, and others samplers, as well. The water content should be obtained at the same time to allow conversion from total to dry unit weights, as needed.

Where undisturbed samples are not available, the unit weight is evaluated from weight-volume relations between the water content and/or void ratio, as well as the assumed or measured degree of saturation.

2.4. Standard compaction test

Purpose of standard compaction test is to determine the maximum dry density attainable under specified nominal compaction energy for a given soil and the (optimum) moisture content corresponding to this density. In the construction of highway embankments, earth dams, retaining walls, structure foundations and many other facilities, loose soils must be compacted to increase their densities. Compaction increases the strength and stiffness characteristics of soils. Compaction also decreases the amount of undesirable settlement of structures and increases the stability of slopes and embankments. Where a variety of soils are to be used for construction, a moisture-density relationship for each major type of soil present at the site should be established.

The void ratio is also used in combination with the relative density. This quantity is defined as

$$D_R = 100\ (e_{max} - e)/(e_{max} - e_{min}) .$$

Here e_{max} is the maximum possible void ratio, and e_{min} the minimum possible value.

These values may be determined in the laboratory. The densest packing of the soil can be obtained by strong vibration of a sample, which then gives e_{min}. The loosest packing can be achieved by carefully pouring the soil into a container, or by letting the material subside under water, avoiding all disturbances, which gives e_{max}.

The accuracy of the determination of these two values is not very large. After some more vibration the sample may become even denser, and the slightest disturbance may influence a loose packing. It follows from equation that the relative density varies between 0 and 1. A small value, say $D_R < 0.5$, means that the soil can easily be densified. Such a densification can occur in the field rather unexpectedly, for instance in case of a sudden shock (an earthquake), with dire consequences.

Compaction is the process of densification of soil by reducing air voids. The degree of compaction of a given soil is measured in terms of its dry density. The dry density is maximum at the optimum water content. A curve is drawn between the water content and dry density to obtain the maximum dry density and optimum water content.

Dry density = $(M / V)/(1+w)$,

where M = total mass of soil, V = volume of soil; w = water content

Apparatus: cylindrical metal compaction mould (capacity : 1000 cc with dia 100 mm + 0.1; 2250 cc with dia 150 mm + 0.1; internal diameter : 100 mm + 0.1; 150 mm + 0.1; internal effective height of mould : 127.3 + 0.1 mm; collar : 60 mm high; detachable base plate; hammer mass: for light compaction = 2.5 kg; heavy compaction = 4.6 kg; dia : 50 mm; sieve; oven : thermostatically controlled to maintain a temperature of 105°C; weighing balance : sensitivity – 1 g for capacity 10 kg; 0.01g for capacity 200 g; steel straight edge of about 300 mm in length with one edge leveled; gradation jar; large mixing pan; spatula.

Procedure

1. Preparation of sample. A representative portion of air dried soil sample (in case of oven drying temp. < 60 °C) break the clods, remove the organic matter like free roots, piece of bark etc.

2. Take about 6 kg – (for soil is not susceptible to crushing during compaction) 15 kg – (for soil is susceptible to crushing during compaction). Sieve above material through 19 mm sieve and 4.75 mm sieve and % passing 4.75 mm sieve. Do not use the soil retained on 20 mm sieve. Determine the ratio of fraction retained and that passing 4.75 mm sieve. If % passing retained on 4.75 mm sieve is greater than 20 mm sieve, use the larger mould

of 150 mm diameter. Mix the soil sample retained on 4.75 mm sieve and that passing 4.75 mm sieve in the proportion determined.

3. Thoroughly mix water in sandy and gravely soil : 3 to 5 %; cohesive soil : 12 to 16 % approx.

4. Store the soil sample in a sealed container for minimum period of 16 hours.

5. Clean and dry the mould and base plate. And apply a thin layer of grease on inside the mould.

6. Weigh the mould to the nearest 1 gram. Attach the collar to the mould and place on a solid base.

7. Compact the moist soil in to the mould in five layers of approximately equal mass, layer being given 25 blows from 4.54 kg hammer dropped from the height of 457 mm above the soil (figure 2.3). The blows should be distributed uniformly over the surface of each layer.

Figure 2.3. Compaction - Lab Equipment

8. Remove the collar and trim off the excess soil projecting above the mould by using straight edge. Take the weight of mould with compacted soil in it.

9. Remove the 100 g compacted soil specimen for the water content determination.

10. Add water in increment of 1 to 2 % for sandy and gravely soils and 2 to 4 % for cohesive soils.

11. Above procedure will be repeated for each increment of water added.

The total number of determination shall be at least four. Hamming should be done continuously taking of height of 457 mm free fall accurately. The amount of soil taken for compaction should be in such a way that after compacting the last layer, the soil surface is not more than 5 mm above the top rim of the mould. Weighing should be done accurately.

Procedure of Compaction tests are performed using disturbed, prepared soils with or without additives. Normally, soil passing the sieve is mixed with water to form samples at various moisture contents ranging from the dry state to wet state. The order of soil preparation is showed on figure 2.4. Dry
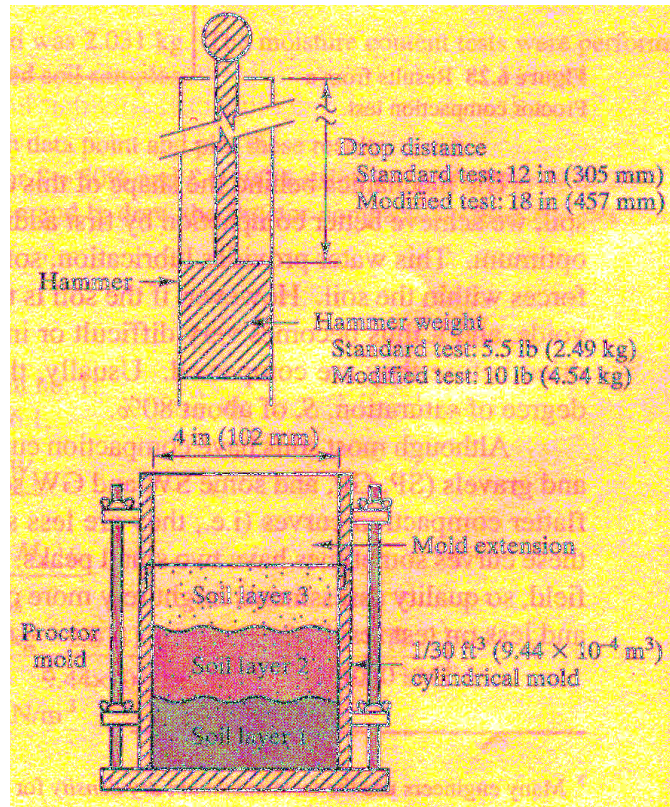
density is determined based on the moisture content and the unit weight of compacted soil.

Figure 2.4. Procedure of soil preparation for compaction tests

The density of soils is measured as the unit dry weight, ρ_d, (weight of dry soil divided by the bulk volume of the soil). It is a measure of the amount of solid materials present in a unit volume. The higher the amount of solid materials, the stronger and more stable the soil will be.

A curve of dry density versus moisture content is plotted in figure 2.5 and the maximum ordinate on this curve is referred to as the maximum dry density. The water content at which this dry density occurs is termed as the optimum moisture content. To provide a "relative" measure of compaction, the concept of relative compaction is used.

Relative compaction is the ratio (expressed as a percentage) of the density of compacted or natural in-situ soils to the maximum density obtainable in a compaction test. Often it is necessary to specify the achieving of a certain level of
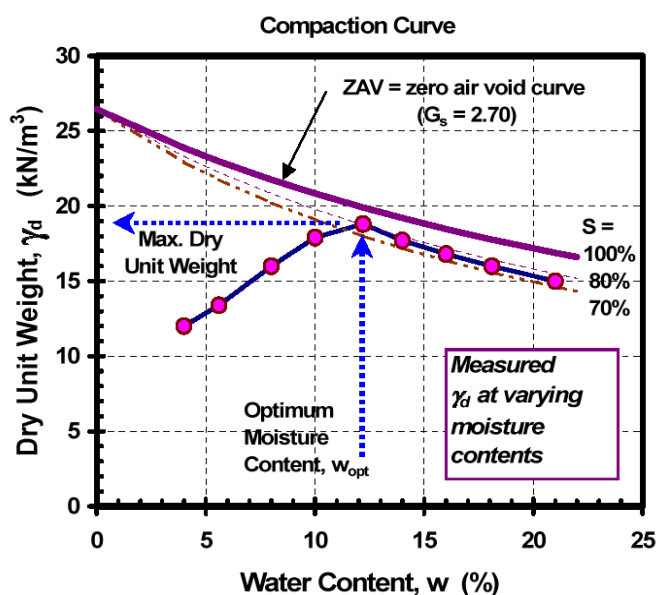

Figure 2.5. Representative moisture-density relationship from a standard compaction test.

relative compaction (e.g. 95%) in the construction or preparation of foundations, embankments, pavement sub-bases and bases, and for deep-seated deposits such as loose sands. The design and selection of a placement method to improve the strength, dynamic resistance and consolidation characteristics of deposits depend heavily on relative compaction measurements.

It is also convenient to plot the zero air voids (ZAV) curve on the moisture-density graph, corresponding to 100 percent saturation (see figure 2.5). The measured compaction curve response should not fall on or above this ZAV line. The maximum dry unit weight ("density") found as the peak value often corresponds to saturation levels of between 70 to 85 percent.

3. CLASSIFICATION TESTS OF SOILS
3.1. Grain-Size Analysis

Soil index properties are used extensively by engineers to discriminate between the different kinds of soil within a broad category, e.g. clay will exhibit a wide range of engineering properties depending upon its composition. Classification tests to determine index properties will provide the engineer with valuable information when the results are compared against empirical data relative to the index properties determined. The gradation tests are performed to determine the particle size distribution of the soil which could be used for soil classification.

Grain-size analysis is a process in which the proportion of material of each grain size present in a given soil (grain-size distribution) is determined. The tests consist of two types: sieve analysis for coarse-grained soils (sands, gravels), hydrometer analysis for fine-grained soils (clays, silts). Materials containing both types of soils are tested by both methods and the results are merged to create one particle size distribution result. These tests shall be performed on samples that were obtained for verification of the field classification of the major soil types encountered during the investigation. The number of tests shall be limited to reasonably establish the stratification without duplication, unless approved otherwise. A minor soil type, if not critical, may be given a visual classification, instead of performing classification tests for reference.

Sieve analyze. The usual procedure is to use a system of sieves having different mesh sizes, stacked on top of each other, with the coarsest mesh on top and the finest mesh at the bottom.

Apparatus: set of fine sieves; set of coarse sieves; weighing balance, with accuracy of 0.1% of the mass of sample; oven; mechanical shaker;

mortar, with rubber pestle; brushes; trays.

Figure 3.1 shows a selection of sieves and soil particle sizes. Sieves larger than the #4 sieve are designated by the size of the openings in the sieve. Smaller sieves are numbered according to the number of openings per inch.

Special standardized sets of sieves are available, as well as convenient shaking machines (Figure 3.2). Hand sieving of a large number of samples can often be tedious and sometimes lead to inaccuracy of results. Shaker machines are ideal for laboratory or on site use. The machines provide a wide choice of options for the busy laboratory. Shakers are robust, compact and sufficiently lightweight to be portable. The digital display incorporated in the unit makes the setting of the microprocessor controlled functions very straightforward. The unique action of the sieve shaker imparts a circular motion to the material being sieved so that it makes a slow progression over the surface of the sieve. At the same time, a rapid vertical movement agitates the sample which assists in clearing the apertures. A variable time switch, with a range of 10 to 60 minutes, is incorporated to set the duration of test. The shaker stops automatically at the end of set duration. The Octagon (figure 3.2) is powered by an electromagnetic drive that has no rotating parts to wear making it maintenance free and extremely quiet in operation.
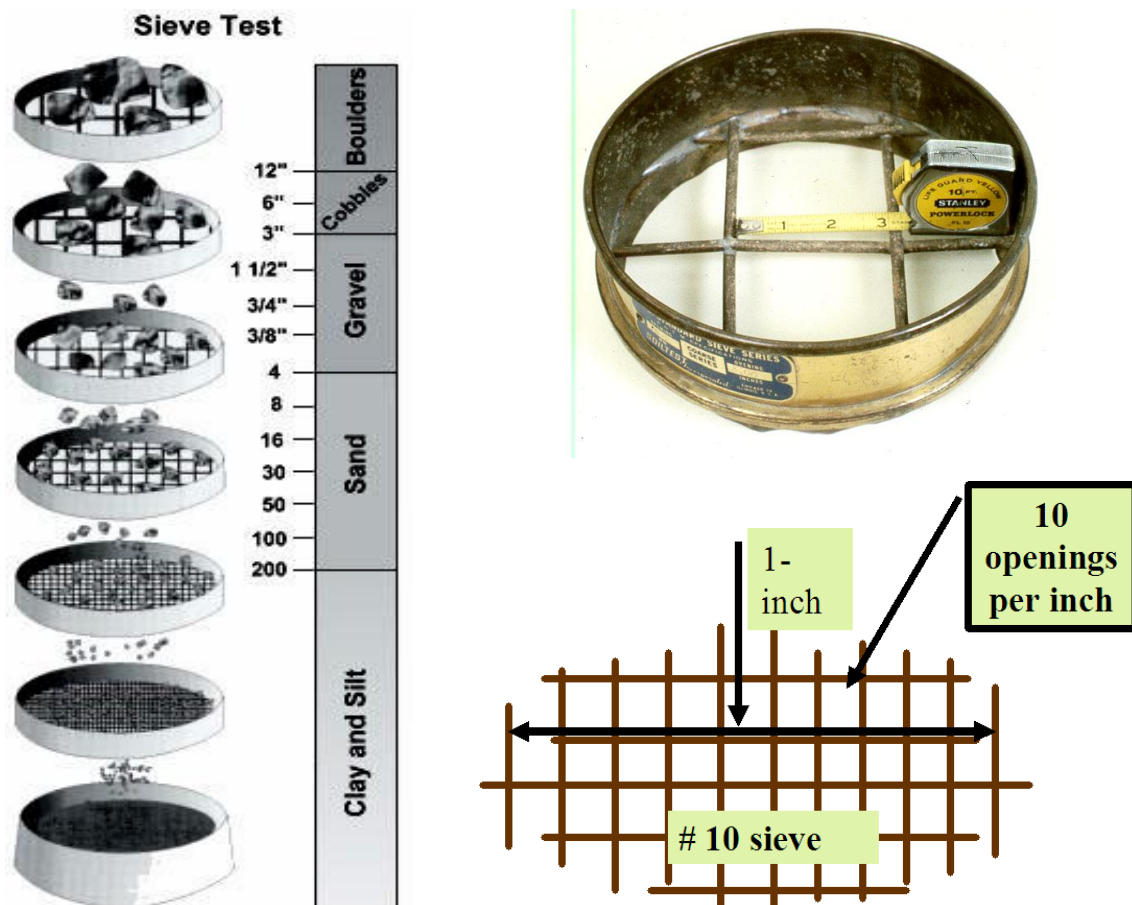
Figure 3.1 Soil particle sizes and selection of sieves.

The vibratory action produced by the power unit moves the sample over the sieve in a unique way producing faster more efficient sieving, while the rapid vertical movements also help keep the apertures from blinding.

Procedures

1. Soil sample, as received from the field shall be dried in air or in sun. In wet weather the drying apparatus may be used in which case the temperature of the sample should not exceed 60 ^{o}C.

2. The clod may be broken with wooden mallet to hasten drying the organic matter, like tree root and pieces of bark should be removed from the sample.

3. The big clods may be broken with the help of wooden mallet.

4. Care should be taken not to break up the individual soil particles.

5. A representative soil sample of required quantity is taken and dried in oven at 105 -120 ^{o}C.

6. The dried sample is taken in tray and soaked with water and mixed 2 g of sodium hexametaphosphate of 2 g or sodium hydroxide of 1 g and sodium carbonate of 1 g per liter of water added as dispersive agent. The soaking of soil continued for 10 -12 hours.

7. Sample is washed through 4.75 mm sieve with water till substantially clean water comes out. Retained sample on sieve shall be oven dried for 24 hours. This dried sample is sieved through 20 mm, 10 mm set of sieves.

8. The portion of the passing sieve shall be oven dried for 24 hours. This oven dried material is riffled and is taken of about 200 g.

9. This sample of about 200 g is washed on 75 micron sieve with half litre distilled water till substantially clear water comes out.

10. The material retained on 75 μ IS sieve is collected and dried in oven at 105 - 120 ^{o}C for 24 hours. The dried soil sample is sieved through 2 mm, 600 μ, 425 μ, 212 μ sieves. Soil retained on each sieve is weighed.

11. If the soil passing 75 μ is 10% or more, hydrometer method is used to analysis soil particle size.

After shaking the assembly of sieves, by hand or by a shaking machine, each sieve will contain the particles larger than its mesh size, and smaller than the mesh size of all the sieves above it. In this way the grain size diagram can be determined. The amount retained on each sieve is collected dried and weighed to determine the percentage of material

Figure 3.2. Sieve shaker

passing that sieve size.

Obtaining a representative specimen is an important aspect of this test. When samples are dried for testing or "washing," it may be necessary to break up the soil clods. Care should be made to avoid crushing of soft carbonate or sand particles. If the soil contains a substantial amount of fibrous organic materials, these may tend to plug the sieve openings during washing. The material settling over the sieve during washing should be constantly stirred to avoid plugging.

Openings of fine mesh or fabric are easily distorted as a result of normal handling and use. They should be replaced often. A simple way to determine whether sieves should be replaced is the periodic examination of the stretch of the sieve fabric on its frame. The fabric should remain taut; if it sags, it has been distorted and should be replaced. A common cause of serious errors is the use of "dirty" sieves. Some soil particles, because of their shape, size or adhesion characteristics, have a tendency to be lodged in the sieve openings.

Hydrometer Analysis. This method determines particle size distribution (percentage) of particle sizes, and identify the silt, clay, and colloids percentages in the soil. This is because the soil behavior for a cohesive soil depends principally on the type and percent of clay minerals, the geologic history of the deposit, and its water content rather than on the distribution of particle sizes. For particles smaller than about 0.05 mm according to USCS (0.075 mm according to British standards) the grain size can not be determined by sieving, because the size of the holes in the mesh would become unrealistically small, and also because during shaking the small particles might by up in the air, as dust.

Soil passing the sieve is mixed with dispersant and distilled water and placed in a special graduated cylinder in a state of liquid suspension (Fig. 3.3).

Mechanical End-over-End Shaker (figure 3.3. b) fitted with friction safety device, capable of rotating two Jars at approximately 50 rpm. Mechanical analysis stirrer (figure 3.3. c) is used for dispersing soil samples in water for hydrometer analysis. The stirrer is supplied complete with mixing paddle and dispersion cup. The specific gravity of the mixture is periodically measured using a calibrated hydrometer (Figure 3.3. d) to determine the rate of settlement of soil particles.

Procedures

1. Particles passed through 75 μ sieve along with water is collected and put into a 1000 ml jar for hydrometer analysis. More water if required is added to make the soil water suspension just 1000 ml. The suspension in the jar is vigorously shaken horizontally by keeping the jar in between the palms of two hands. The jar is put on the table.
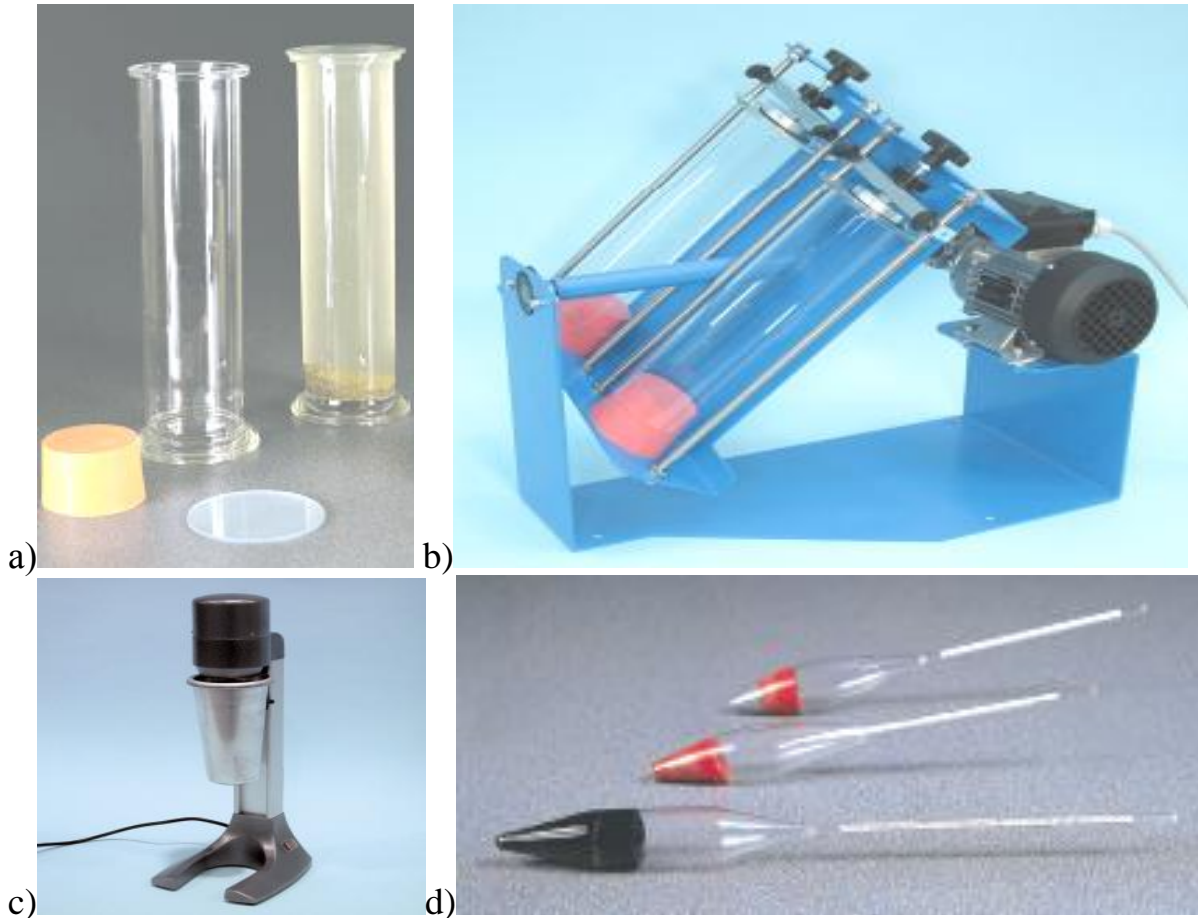
Figure 3.3. Apparatus: a) hydrometer sedimentation cylinders b) end-over-end Shaker, c) stirrer, d) soil hydrometers

2. A graduated hydrometer is carefully inserted in to the suspension with minimum disturbance.

3. At different time intervals, the density of the suspension at the c.g. of the hydrometer is noted by seeing the depth of sinking of the stem. The temperature of suspension is noted for each recording of hydrometer reading.

4. Hydrometer reading is taken at a time of 0.5, 1.0, 2.0, 4.0, 15.0, 45.0, 90.0, 180.0 minutes, 6 hrs, 24 / 48 hours.

5. By using the nomogram the diameter of the particles at different hydrometer reading is found out.

The amount of particles of a particular size can then be determined much better by measuring the velocity of deposition in a glass of water. This method is based upon a formula derived by Stokes for settlement of idealized spherical particles. This formula expresses that the force on a small sphere, sinking in a viscous liquid, depends upon the viscosity of the liquid, the size of the sphere and the velocity. It should be noted that Stokes Law does not apply to particles < 0.0002mm.

Because the force acting upon the particle is determined by the weight of

the particle under water, the velocity of sinking of a particle in a liquid can be derived. The formula is

$$v = \frac{(\gamma_p - \gamma_f) \cdot D^2}{18 \cdot \mu} \qquad (3.1)$$

where γ_p - is the volumetric weight of the particle, γ_f - is the volumetric weight of the liquid, D - is the grain size, μ - is the dynamic viscosity of the liquid.

Because for very small particles the velocity may be very small, the test may take rather long. The test does not require the weighing accuracy necessary for pipette sedimentation and is suitable for use in site laboratories. Replicable results can be obtained when soils are largely composed of common mineral ingredients. Results can be distorted and erroneous when the composition of the soil is not taken into account to make corrections for the specific gravity of the specimen. Particle size of highly organic soils cannot be determined by the use of this method.

The Pipette Method produces similar results to the hydrometer method, but is considered to be more precise. It requires delicate apparatus and greater accuracy in weighing. In the pipette test samples of a suspension are taken from a fixed elevation in a measuring cylinder at times, *t*, and the percentage of various grain sizes determined. Andreasen Pipette Stand (figure 3.4.) with moving carriage assembly which can be operated with no vibration transmitted to the pipette whilst the pipette is being inserted and withdrawn from the liquid suspension. The stand is fitted with a holder for securing the pipette and is supplied with a stand for the sedimentation cylinder.

Specially designed for the sedimentation testing of soils and other fine grained material, the bath is supplied with a false bottom to assist in circulation of the bath liquid. Adequate clearance is provided when the tank is used with a Pipette sedimentation stand.

Figure 3.4. Pipette Stand

Results of grain tests are sometimes needed to complete classification of soils. Figure 3.5 shows several grain size distributions obtained from sieving and hydrometer methods. Shown (right to left) are sieve and example soil particle sizes including (right to left): medium gravel, fine gravel, medium-coarse sand, silt, and dry clay (kaolin).

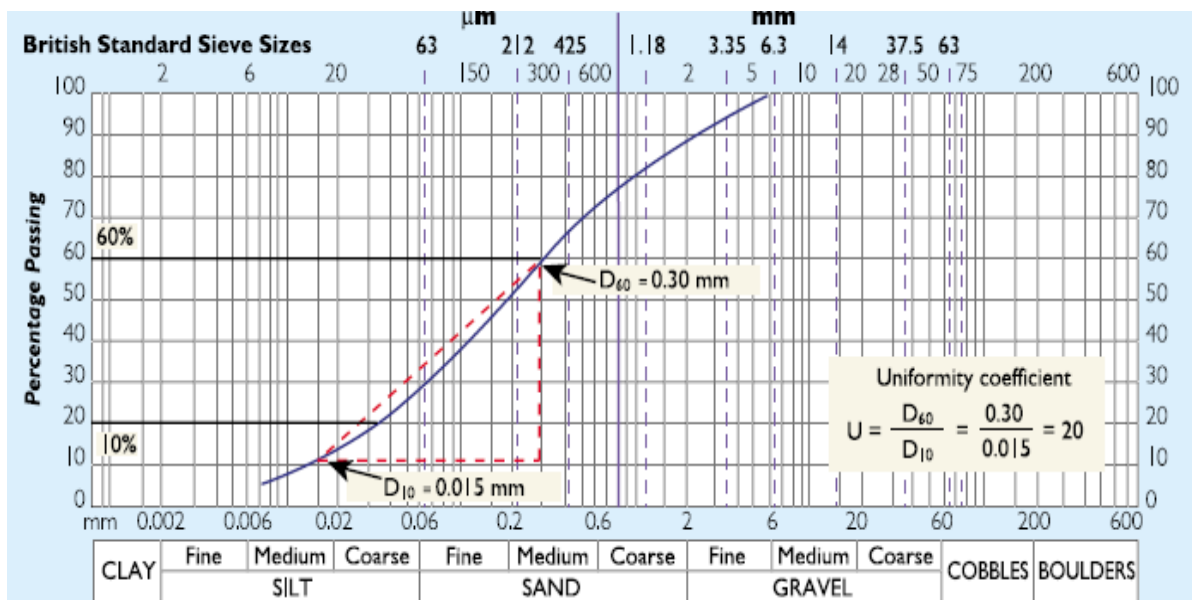*Figure 3.5. Sieves for mechanical analysis for grain size distributions.*

Figure 3.6. Grain-size diagram

The size of the particles in a certain soil can be represented graphically in a grain size diagram (figure 3.6). Such a diagram indicates the percentage of the amount of particles smaller than a certain diameter, measured as a percentage of the weight. The results are plotted on a semi log graph with particle size as abscissa (log scale) and the percentage smaller than the specified diameter as ordinate. A steep slope of the curve in the diagram indicates a uniform soil, a shallow slope of the diagram indicates that the soil contains particles of strongly different grain sizes.

The resulting gradations can provide data for determining several parameters, such as effective diameter (D_{10}) and coefficient of uniformity

(C_u). The grain size distribution can be characterized by the quantities D_{60} and D_{10}. These indicate that 60 %, respectively 10 % of the particles (expressed as weights) is smaller than that diameter. The ratio of these two numbers is denoted as the ***uniformity coefficient*** C_u,

$$C_u = D_{60}/D_{10} \ .$$

In a poorly graded soil the particles all have about the same size. The uniformity coefficient is than only slightly larger than 1, say $C_u = 2$.

3.2. Consistency (Atterberg) Limits

The Atterberg limits provide general indices of moisture content relative to the consistency and behavior of cohesive soils. With high moisture content, silty clay becomes slurry to liquid. Liquid consistency corresponds to the case when soil shows practically no resistance to shear strain. After significant reduction of its moisture content, soil obtains characteristics of a brittle material, and when exposed to strain it fails. This state is called stiff consistency.

Purpose the Atterberg limits tests is to describe the consistency and plasticity of fine-grained soils with varying degrees of moisture. For the portion of the soil passing the № 40 sieve, the moisture content is varied to identify three stages of soil behavior in terms of consistency. These stages are known as the liquid limit (*LL*), plastic limit (*PL*) and shrinkage limit (*SL*) of soils. In total, individual consistencies and limits may be graphically displayed as follows phases on figure 3.7.
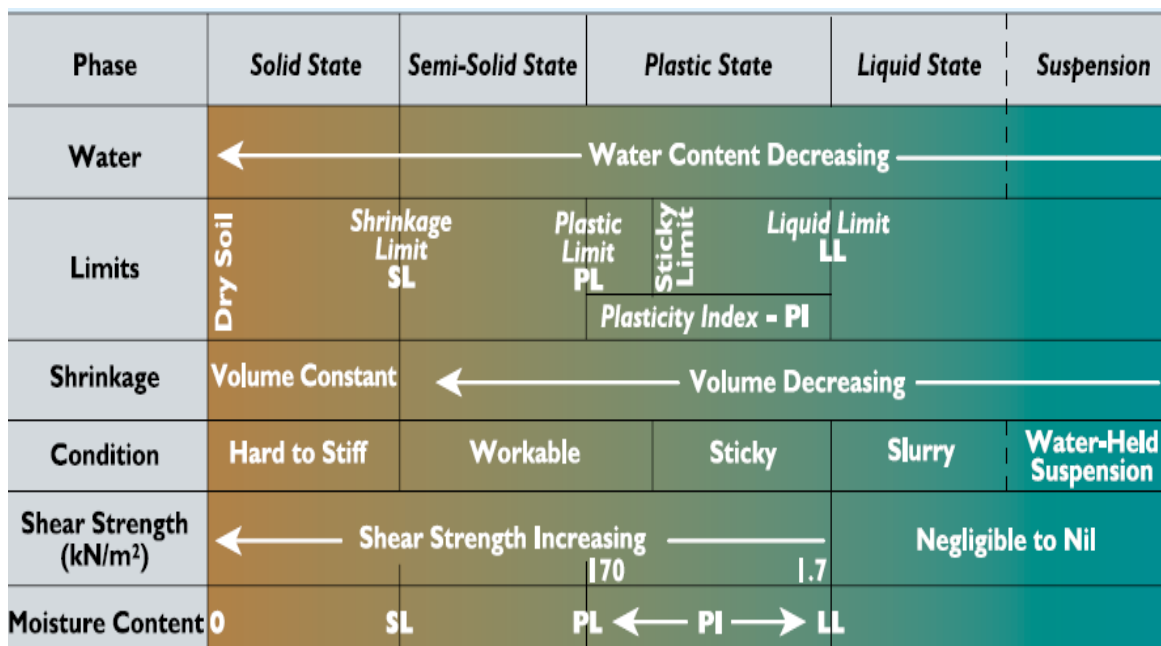

Figure 3.7. The Atterberg limits and phases of soils

The liquid limit. The moisture content at which soil already shows some shear strength is considered to be the boundary between liquid and plastic consistency. It is called the liquid limit (*LL*), units: %.

Procedure of the definition of the liquid limit test by manual Casagrande cup device. As this limit is not absolute, it has been defined as the value determined in a certain test, due to Casagrande. The test procedure has remained, in principle, the same since 1932, when Casagrande proposed to define the various limits by relating the moisture content characteristics of soil under certain conditions.

The liquid limit is determined in a standardized Casagrande dish. The apparatus required is simple yet effective. The soil in the dish is divided by a cutting knife into 2 parts, and the whole dish falls on a pad from a height of 1 cm. In the test a hollow container with a soil sample may be raised and dropped by rotating an axis (figure 3.8). The liquid limit is the value of the water content for which a standard V-shaped groove cut in the soil, will just close after 25 drops. If both parts along the cut are reconnected at a length of 12.5 mm after 25 blows, the soil's moisture content is at its liquid limit. When the groove closes after less than 25 drops, the soil is too wet, and some water must be allowed to evaporate. By waiting for some time, and perhaps mixing the clay some more, the water content will have decreased, and the test may be repeated, until the groove is closed after precisely 25 drops. Then the water content must immediately be determined, before any more water evaporates, of course.
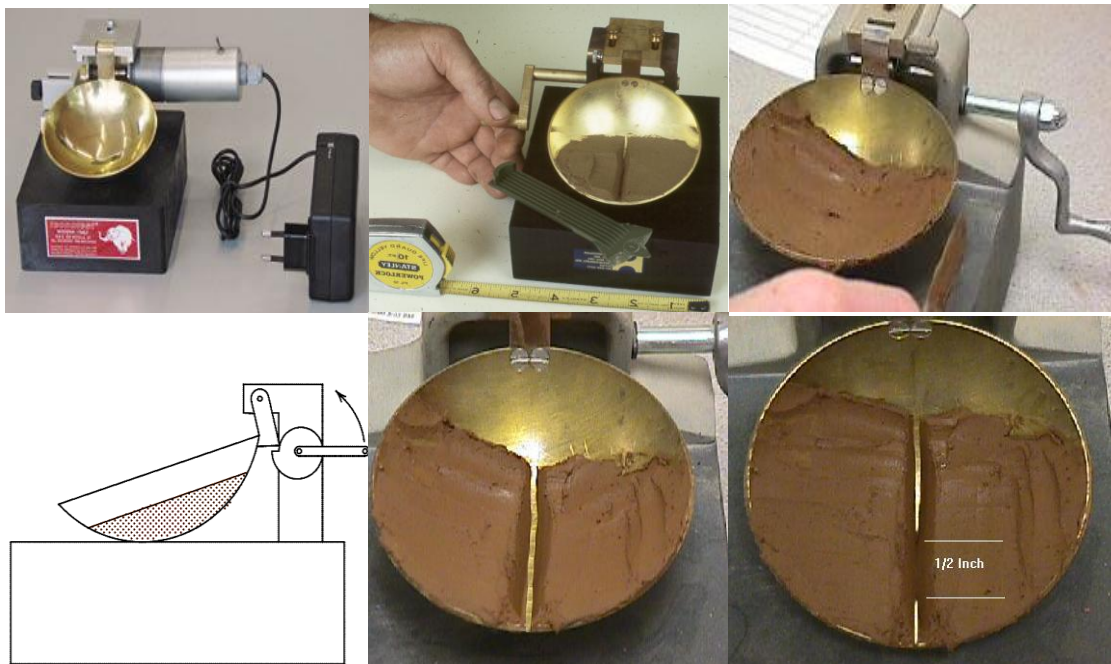

Figure 3.8. The definition of the liquid limit by manual Casagrande cup device

Procedure

1. Air dry the soil sample (in case drying) and break the clods. Remove the organic matter like tree roots, pieces of bark etc.

2. About 100 g of the specimen passing 425 micron IS sieve is mixed thoroughly with distilled water in the evaporating dish and left for 24 hours for soaking.

3. A portion of the paste is placed in the cup of the Liquid limit device.

4. Level the mix so as to have a maximum depth of 1 cm.

5. Draw the grooving tool through the sample along the symmetrical axis of the cup, holding the tool perpendicular to the cup.

6. For normal fine grained soil: the Casagrande tool is used which cuts a groove of width 2 mm at the bottom, 11 mm at the top and 8 mm deep.

7. For sandy soil : The tool is used which cuts a groove of width 2 mm at bottom, 13.6 mm at top and 10 mm deep.

8. After the soil pat has been cut by proper grooving tool, the handle is rotated at the rate of about 2 revolutions per second and the nos. of blows counted till the two parts of the soil sample come into contact for about 10 mm length.

9. Take about 10 g of soil near the closed groove & find water content.

10. The soil of the cup is transferred to the dish containing the soil paste and mixed thoroughly after adding a little more water. Repeat the test.

11. By altering the water content of the soil and repeating the foregoing operations, obtain at least 5 readings in the range of 15 - 35 blows. Don't mix dry soil to change its consistency.

12. Liquid limit is determined by plotting a 'flow curve' on semi-log graph between nos. of blows on logarithmic scale and water content on arithmetical scale.

13. Generally these points lie in a straight line.

14. Water content corresponding to 25 blows is the value of Liquid limit.

An alternate procedure for Casagrande's test in Europe and Canada uses a fall cone device to obtain better repeatability (figure 3.9). In this test a steel cone, of 60 grams weight, and having a point angle of 60°, is placed upon a clay sample, with the point just at the surface of the clay. The cone is then dropped and its penetration depth is measured. The liquid limit has been defined as the water content corresponding to a penetration of exactly 10 mm.
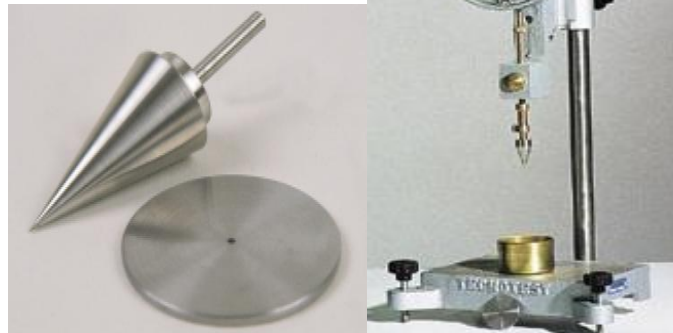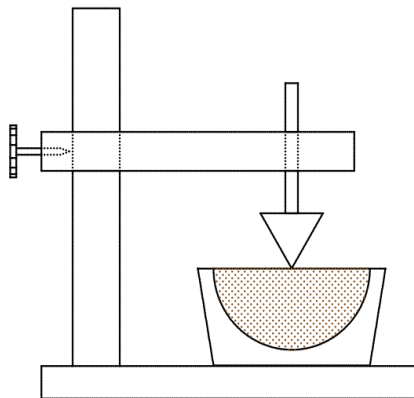
Figure 3.9. Fall Cone

Procedure

1. The 4 numbered cans were weighed.

2. The container of the cone penetrometer device was filled with soil specimen on three times, and were stroke on the bench to prevent air voids from taking place in the container, and it was stuck flush.

3. The container containing the specimen was placed under the cone of the penetrometer, and the cone is then released for 5 seconds, and the reading of the penetration was taken.

4. Small portion of the tested soil was taken into one of the identified cans and its weight was recorded.

5. The four cans were then removed to the oven for 24 hours, and then their weights were recorded.

Again the liquid limit can be determined by doing the test at various water contents. It has also been observed, however, that the penetration depth, when plotted on a scale, is an approximately linear function of the water content (figure 3.10). This means that the liquid limit may be determined from a single test, which is much faster, although less accurate.

Procedure of the definition of the plastic limit. The transitory moisture content between plastic and stiff consistency is called the *plasticity limit (PL)*. The
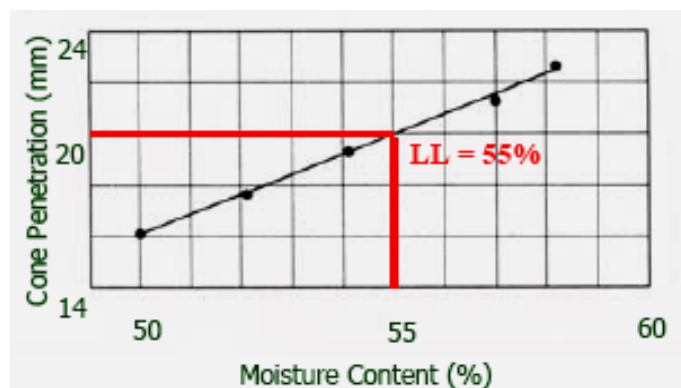

Figure 3.10. Liquid Limit determined by doing the test at various water contents

plasticity limit is determined by saturating soil with water and by making rolls of it with a diameter of 3 mm (figure 3.11) on an absorbent pad. If the rolls start to crumble into 1 cm lengths, the soil's moisture content is at its plasticity limit.

Apparatus: porcelain evaporating dish about 120 mm diameter; spatula; container to determine moisture content; balance with 0.01 g accuracy; oven; ground glass plate 20 x 15 cm for rolling.

Procedure

1. Take out 30 g of air dried soil from a thoroughly mixed sample of the soil passing sieve, mix the soil with distilled water in a evaporating dish and leave the soil mass for naturing. This period may be up to 24 hours.

2. Take about 8 g of the soil and roll it with fingers on a glass plate. The rate of rolling shall be between 80 to 90 strokes per minutes to form a 3 mm diameter.

3. If the diameter of the threads becomes less than 3 mm without cracks, it shows that water content is more than its plastic limit. Kneed the soil to reduce the water content and roll it again to thread.

4. Repeat the process of alternate rolling and kneading until the thread crumbles.

5. Collect the pieces of crumbled soil thread in a moisture content container.

6. Repeat the process at least twice more with fresh samples of plastic soil each time.

The definition of the plastic limit very wet clay can be rolled into very thin threads, but dry clay will break when rolling thick threads. The (arbitrary) limit of 3 mm is supposed to indicate the plastic limit. In the laboratory the
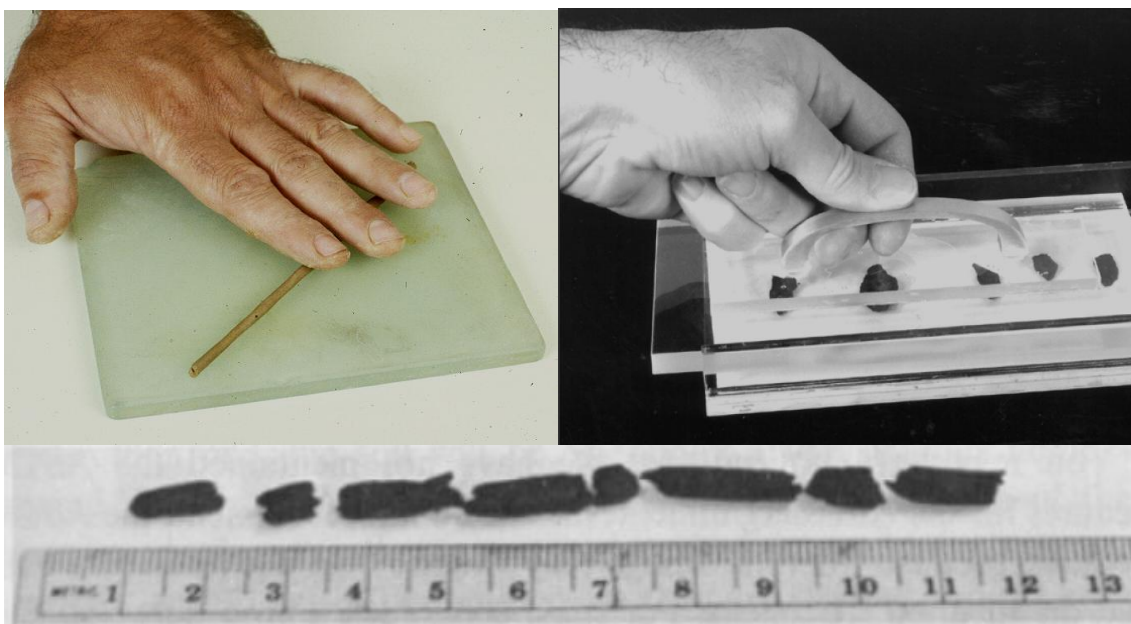

Figure 3.11. The definition of the plastic limit

test is performed by starting with a rather wet clay sample, from which it is simple to roll threads of 3 mm. By continuous rolling the clay will gradually become drier, until the threads start to break by evaporation of the water or with help devise (figure 3.12).

Depending on their liquid limit, soils may be further specified as soils showing the following plasticity (figure 3.13): L – low LL below 35 %, I – medium LL = 35 – 50 %, H – high LL = 50 – 70 %, V – very high LL = 70 – 90 %, E – extremely high LL exceeding 90 %.

Figure 3.12. Warm-air dryer and Wille Geotechnik equipment for the plastic limit definition.
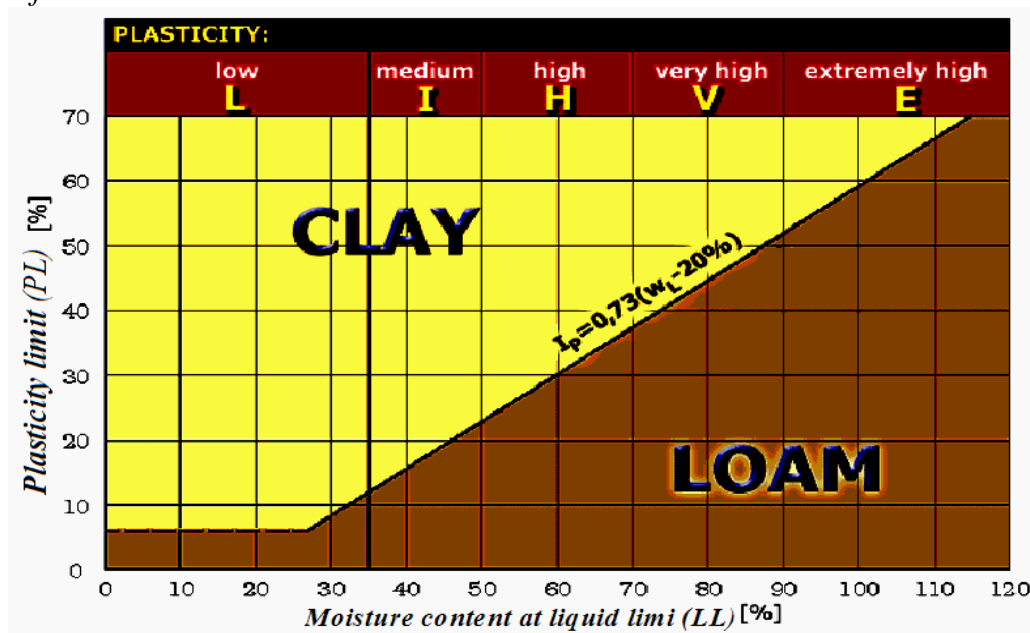

Figure 3.13. Specification of soil by consistency limit

In countries with very thick clay deposits (England, Japan, Scandinavia, Russia) it is often useful to determine a profile of the plastic limit and the liquid limit as a function of depth, see figure 3.14. In this diagram the natural water content, as determined by taking samples and immediately determining

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the water content, can also be indicated.

On the basis of these consistency limits, the characteristics of cohesive soils are determined – their plasticity index *PI*, consistency index I_C, liquidity index I_L and colloidal clay activity index *A*.

Plasticity is expressed by means of the plasticity index, which is the range of moisture content at which soils show plastic behavior. For many applications it is especially important that the range of the plastic state is large. This is described by the *plasticity index PI*. The plasticity index is a useful measure for the possibility to process the clay. It is important for potteries, for the construction of the clay core in a high dam, and for the construction of a layer of low permeability covering a deposit of polluted material. In all these cases a high plasticity index indicates that the clay can easily be used without too much fear of it turning into a liquid or a solid.

Figure 3.14. Profile of the plastic and the liquid limit as a function of depth

Plasticity index is defined as the difference of the liquid limit and the plastic limit,

$$PI = LL - PL .$$

where *LL* - moisture content at liquid limit, *PL* - moisture content at plasticity limit, units: *non-dimensional.*

The plasticity index *PI* grows with the percentage of clay fraction.

The consistency index is the most common way of expressing the consistency of cohesive soils. ***Consistency index - I_c***

$$I_c =(LL-w)/(LL-PL)$$

where *LL* - moisture content at liquid limit, *PL* - moisture content at plasticity limit, w - specific moisture content of soil at which its consistency is determined, units: *non-dimensional.*Depending on its value, the following states (consistencies) are distinguished:

consistency index I_C	consistency
greater than or equals 1	solid to hard
1.0 - 0.5	Stiff
0.5 - 0.05	Soft
smaller than 0.05	slurry - liquid

In foreign academic literature, the consistency of cohesive soils is also expressed by means of the *index of liquidity I_L*.

$$I_L=(w-PL)/(LL-PL),$$

where *LL* - moisture content at liquid limit, *PL* - moisture content at plasticity limit, *w* - specific moisture content of soil at which its consistency is determined, units: *non-dimensional.*

The liquidity index is *LI = (w-PL)/PI* is an indicator of stress history; *LI #1* for normally consolidated (*NC*) soils and *LI #0* for over-consolidated (*OC*) soils.

The colloidal clay **activity index A** was defined by Skempton in 1953.

$$A=PI / \% \ clay \ fraction \ (weight)$$

PI - plasticity index , units: *non-dimensional.*

A	Classification	Potential for swelling
< 0.75	Inactive Clays	Low
0.75 – 1.25	Normal Clays	Med
>1.25	Active Clays (e.g. montmorillonite)	High

By and large, these are approximate and empirical values. They were originally developed for agronomic purposes. Their widespread use by engineers has resulted in the development of a large number of rough empirical relationships for characterizing soils.

Considering the abstract and manual nature of the test procedure, Atterberg limits should only be performed by experienced technicians. Lack of experience, and lack of care will introduce serious errors in the test results.

The shrinkage limit (**SL**). The transitory moisture content between its stiff and solid consistency of the soil, in percent, at which the decrease in soil volume ceases, is defined as the *shrinkage limit*, units: %. Volume changes in soil can be a very dangerous problem in engineering structures; for example if a soil used in a highway expands or contracts this will produce a bumpy road. Volume changes occur over a period of time and depend on both soil type and the change in water content, but most of the damage will occur if differential water contents produce different amount of volume change.

Shrinkage is soil contraction and is mainly a cause of soil suction, which is the phenomenon that produces capillary rise of water in soil pores above the water table. Two main sources of soil shrinkage are:

1. *Capillary rise:* At the top of the capillary column the pressure will be

negative pressure i.e. tension, which will cause the soil grains to be in tension and gets closer to each other.

2. *Drying:* As the soil dries, the pores start to empty from water, during this empting process surface tension develop and the grains gets closer to each other.

A saturated clayey soil, when gradually dried, will lose moisture and, subsequently, there will be a reduction in the volume of the soil mass. During the drying process, a condition will be reached when any further drying will result in a reduction of moisture content without any decrease in volume (figure 3.15).

The shrinkage limit is determined by a test during which a soil sample, mostly prism-shaped, is exposed to slow drying, and its moisture content is measured in relation to shrinkage. *SL* test covers the determination of shrinkage of soils and indicates the plastic properties of soils with a low clay content. This method of test covers the determination of the *shrinkage limit, shrinkage ratio, volumetric shrinkage and linear shrinkage.*
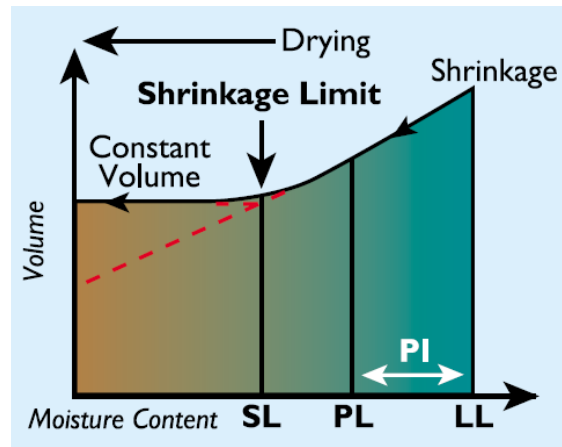

Figure 3.15. Shrinkage curve for clay soil

In this experiment we are considered in the drying shrinkage, where the contraction continues till the shrinkage limit. The shrinkage limit is defined *as the water content below which no further change in volume of soil occurs* (figure 3.15). From this definition it can be seen that the higher the shrinkage limit he soil has the more preferred this soil is.

The shrinkage limit can be measured by using the following formula:

$$Sl = \left(\frac{V_{dry}}{M_s} - \frac{1}{\rho_s} \right) \cdot \rho_w \cdot 100$$

or

$$Sl = w_i - \left(\frac{(V_i - V_{dry}) \cdot \rho_w}{M_s} \right) \cdot 100,$$

where: w_i -initial water content, V_{dry} - volume of final volume (dry volume), M_s - mass of the dried sample.

The test is simply held by placing the soil sample in the shrinkage dish and stuck flush, and then dried gradually, so that no cracking for the soil sample will occur, for 48 hours where the first 24 hours will be air drying. Dry volumes in equation can be approached by *mercury method.*

Apparatus: shrinkage dish, having a flat bottom, 45 mm diameter and 15 mm height; two evaporating dishes about 120 mm diameters, with a pour out and flat bottom; one small mercury dish, 60 mm diameter; two glass plates, one plane and one with prongs, 75 x 75 x 3 mm size; glass cup, 50 mm diameter and 25 mm height; sieve 425 micron; oven; desiccator; weighing balance, accuracy 0.01 g; spatula; straight edge; mercury. Equipment for definition of volumetric shrinkage limit are showed on figure 3.16.
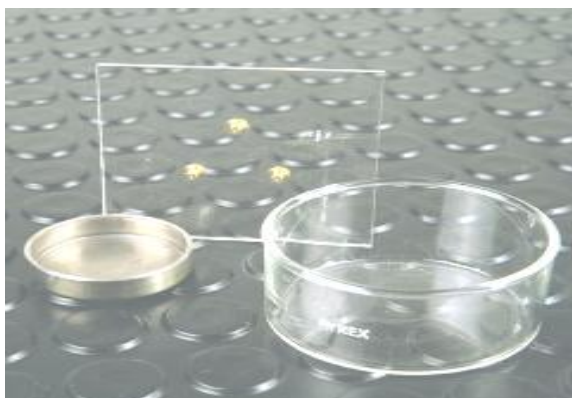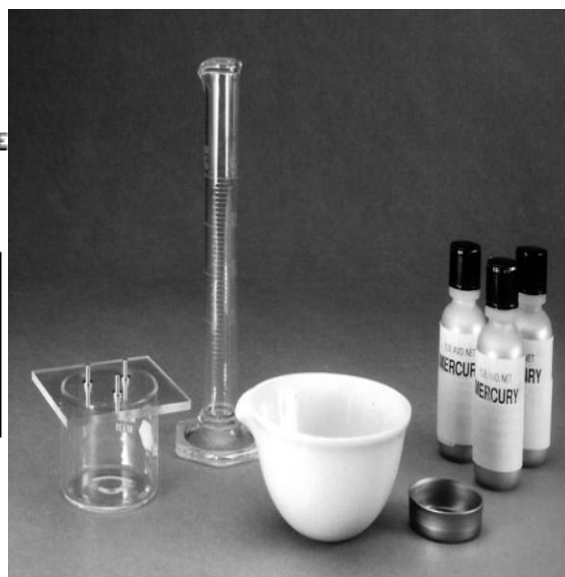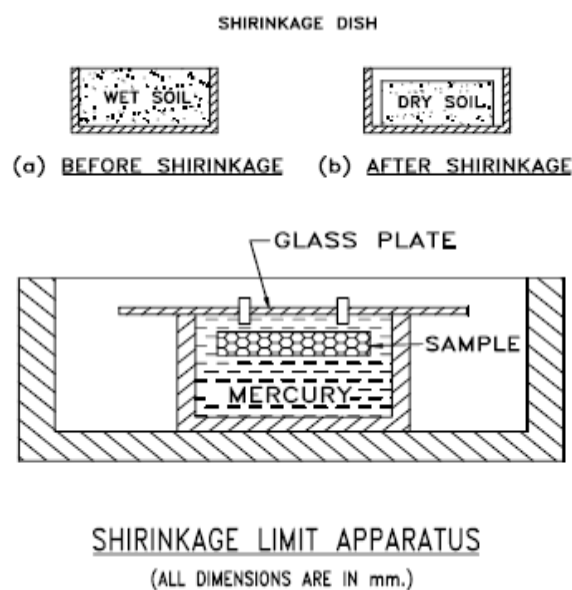
Figure 3.16. Apparatus for determining the volume of dry soil

Procedure

1. Take a sample of mass about 100 g from a thoroughly mixed soil passing 425 micron sieve.

2. Take about 30 g of soil sample in a large evaporating dish. Mix it with distilled water to make a creamy paste, which can be readily worked without entrapping the air bubbles.

3. Take the shrinkage dish, clean it and determine its mass. The dish was greased before starting the lab so that the soil won't stick on its walls.

4. Fill mercury in the shrinkage dish. Remove the excess mercury by pressing the plain glass plate over the top of the shrinkage dish. The plate should be flush with the top of the dish, and no air should be entrapped.

5. Transfer the mercury of the shrinkage dish to a mercury weighing dish and determine the mass of the mercury to an accuracy of 0.01 g. The volume of the shrinkage dish is equal to the mass of the mercury in grams divided by the specific gravity of the mercury.

6. Coat the inside of the shrinkage dish with a thin layer of silicon grease or vaseline. Place the soil specimen in the center of the shrinkage dish, equal to about one third volume of shrinkage dish. Tap the shrinkage dish on a firm, cushioned surface and allow the paste to flow to the edges.

7. Add more soil paste, approximately equal to the first portion and tap the shrinkage dish as before, until the soil is thoroughly compacted. Add more soil and continue the tapping till the shrinkage dish is completely filled, and excess soil paste projects out about it's edge. Strike out the top surface of the paste with the straight edge. Wipe off all soil adhering to the out side of the shrinkage dish. Determine the mass of the wet soil.

8. Dry the soil in the shrinkage dish in an air until the colour of pat turns from dark to light. Then dry the pat in the oven at 105 to 110 ^{o}C to constant mass.

9. Cool the dry pat in a desiccator. Remove the dry pat from the desiccator after cooling and weigh the shrinkage dish with the dry pat to determine the dry mass of the soil.

10. Place a glass cup in a large evaporating dish and fill it with mercury. Remove the excess mercury by pressing the glass plate with prong firmly over the top of the cup. Wipe of any mercury adhering to the outside of the cup. Remove the glass cup full of mercury and place it in another evaporating dish, taking care not to spill any mercury from the glass cup.

11. Take out the dry pat of soil from the shrinkage dish and immerse it in the glass cup full of mercury. Take care not to entrap air under the pat. Press the plate with the prongs on the top of cup firmly.

12. Collect the mercury displaced by the dry pat in the evaporating dish, and transfer it to the mercury weighing dish. Determine the mass of mercury to an accuracy of 0.01 g. The volume of the dry pat is equal to the mass of mercury divided by the specific gravity of mercury.

13. Repeat the test at least 3 times.

The linear shrinkage is defined as the decrease in one dimension of a soil mass, expressed as a percentage of the original dimension, when the water content is reduced from a given value to the shrinkage limit. Standards specifies a method for measuring the linear shrinkage, where standardized mold takes the shape of half cylinder of 12.5 mm diameter and 140 mm length (figure 3.17).

Using the following formula the Linear shrinkage is measured:

LS(%)=100%·L$_s$/L

where *L*- initial length (the length of the mold), L_s- final length of the specimen.

Figure 3.17. Linear Shrinkage apparatus

The mold is fill with soil sample and then dried in the same manner the shrinkage sample was dried, and then the final length of the specimen was measured. For purposes of accuracy several readings are taken for each length and then the average for each reading is calculated and used in the formula. The length of dried sample is measured by using a string and a ruler as the specimen will be buckled and its length can't be measured by using a ruler only.

Procedure

1. The standardized mold was filled with soil sample to the top and the surface was leveled.

2. The mold containing the sample was then placed into the oven and dried gradually to avoid cracking of the soil sample.

3. After 48 hours, the mould was removed from the oven and the length of the mold was measured two times and the length of the specimen was measured 3 times.

In the case of heavy clay soils, it is desirable to allow the wet soil to stand for about 24 hours in an airtight container before performing the test to allow the water to permeate throughout the soil mass. After curing, it is necessary to re-mix before testing. Highly aggregated soil may require as much as 40 minutes continuous mixing immediately before testing.

As operators gain experience, it will not be necessary to test the mixture in the liquid limit machine as moisture content is not critical within a few per cent. The curling of a specimen can generally be prevented by extremely slow drying.

Alternatively, when excessive curling is expected, the specimens may be air-dried for 24 hours and then weighted in three places in such a manner to prevent undue curling but to allow moisture to evaporate.

It is possible to prepare a special scale calibrated for the length of the mould to read off linear shrinkage direct. For a 250 mm mould, scale

subdivisions of 1.25 mm correspond to 0.5% increments of linear shrinkage. For a 135 mm mould, scale subdivisions of 1.35 correspond to 1% increments of linear shrinkage.

3.3. Classification of Soils

Classification helps in grouping soils according to properties, provides a way of referring to soils universally, and makes selection of soils easy for a certain application. The engineering classification of soils can be used as a preliminary estimate of the potential engineering use of soils. Will the soils be freely draining or impermeable, will they be highly compressible etc. These factors are obviously important in choosing a suitable material for the core material of an earth dam or for the foundation of a building. Some soils are liable to collapse, others to liquefaction. Some clays containing high montmorillonite contents are particularly susceptible to shrinkage and swelling with seasonal changes in water content.

Various engineering classifications of soils have been adopted throughout the world. There are US systems, French systems, Russian systems, Australian systems, Brazilian systems, British systems, Canadian systems, Dutch systems, German systems, Polar systems and others.

The global character of a classification according to grain size is well illustrated by the characterization sometimes used in Germany, saying that gravel particles are smaller than a chicken's egg and larger than the head of a match, and that sand particles are smaller than a match head, but should be visible to the naked eye.

Soils may be crudely classified also according to grain size, table 3.1, in to fine grained (predominantly clay and silt size) or coarse grained (sand and gravel sizes predominant); or in shear strength terms: cohesive (eg. clays, clay silt mixtures, organics) – (c-soils or $\varphi = 0$ soils); cohesionless (eg. sands and gravels) ($c=0$ or frictional soils); mixtures eg. sandy clays etc (c-φ soils).

Table 3.1

Size range	Grain size term	Grain Size British System /European mm	ASTM mm (in)	Confer
Boulders		>200	350 (12) and above	Larger than basketball
Cobbles		>60<200	75 (3)-350 (12)	Grapefruit
Gravel	Coarse	>20<60	19 (0.75) –75 (3)	Orange or Lemon
	Medium	>6<20		
	Fine	>2 <6	4.75 (3/16) - 19 (0.75)	Grape
Sand	Coarse	> 0.6 <2	2.0 (3/32) -4.75 (3/16)	Rock Salt

	Medium	> 0.2 < 0.6	0.42 (0.016) - 2.0(3/32)	Sugar
	Fine	> 0.06 < 0.2	0.074 - 0.42 (#200-#40)	Powdered Sugar
Silt	Coarse	>0.02 <0.06	0.005-0.074	Cannot be seen with naked eye at distance > 200mm
	Medium	>0.006 <0.02		
	Fine	>0.002 <0.006		
Clay*		<0.002	0.002 and below	

It is essential that a common language exist between engineers for the classification and description of soils. Several different classification systems have been developed by different agencies to describe soils. Most soil classification systems include a comprehensive description of the soil material characteristics. The principal material characteristics are grain size distribution and plasticity. Secondary material characteristics include the colour of the soil and the shape, texture and composition of the particles.

The three most common systems in North America are the American Association of State Highway and Transportation Officials (AASHTO), USDA Soil Taxonomy and the Unified Soil Classification System (USCS).

American Association of State Highway and Transportation Officials system (AASHTO) Origin of AASHTO: (commonly used by pavement engineers for road construction). This system was originally developed by Hogentoglerand Terzaghiin 1929 as the Public Roads Classification System. Afterwards, there are several revisions. The present AASHTO (1978) system is primarily based on the version in 1945. The final part of the AASHTO Classification is calculation of the *group index (GI).*

Group Index shall be calculated after performing the classification and Atterberg Limit and is reported along with the Classification Test. Group Index indicated the plastic nature of the portion of the material passing № 200 sieve. Reported as a positive whole number or zero. Generally, the higher the value of the group index for a given classification, the poorer the performance of the soil. The formula used to calculate the group index is as follows:

$$GI = (F - 35)[(0.2 + 0.005)(LL - 40)] + 0.01(F - 15)(PI - 10)$$

where F = Percentage passing the № 200 sieve; LL = Liquid Limit; PI = Plasticity Index.

Typical classification: A-2-6, A-3, A7-5, etc.

USDA Soil Taxonomy is a hierarchical system and used almost exclusively in the US for agricultural, biological, and geological studies. It has been applied throughout the world (Soil Survey Staff, 1975). USDA Soil Series is a nominal system of soils of the US. Easiest to use. However, not as comprehensive as USCS %. Fraction of silt, sand, and clay used for classification. Gravel fraction not considered. Probably because agricultural scientists rarely use gravels for agricultural purposes. Plasticity of Clay

fraction not considered. Plasticity of clay does not play as big of role in agricultural applications.

US Classification System (USCS) is the most common soil classification system among geotechnical engineers and geologists. Originally developed for the United States Army Corps of Engineers (USACE). The method is standardized in ASTM as "*Unified Soil Classification System (USCS)*". In the USCS, the group symbols consist of a primary and a secondary descriptive letter. The letters and their meaning are given in table 3.2. Soils exhibiting the characteristics of two groups should be given a boundary classification denoted by dual symbols connected by a hyphen. Used primarily for selection of material for highway subgrade. Soils divided into 8 groups according to *PSD, LL*, Plasticity Index (I_p), and organic material fraction. Figures 3.18-3.19 can be used as a guide for classifying different soils and providing a 2-letter designator for any soil type.

Table 3.2.

Unified Soil Classification (USC) System (from ASTM D 2487)				
Major Divisions			Group Symbol	Typical Names
Course-Grained Soils More than 50% retained on the 0.075 mm (№ 200) sieve	**Gravels** 50% or more of course fraction retained on the 4.75 mm (№ 4) sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravels with Fines	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands 50% or more of course fraction passes the 4.75 (№4) sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures

Table 3.2 (continuation)

Unified Soil Classification (USC) System (from ASTM D 2487)			
Major Divisions		Group Symbol	Typical Names
Fine-Grained Soils More than 50% passes	**Silts and Clays** Liquid Limit 50% or less	ML	Inorganic silts, very fine sands, rock four, silty or clayey fine sands
		CL	Inorganic clays of low to medium plasticity, gravelly/sandy/silty/lean clays
		OL	Organic silts and organic silty clays of low plasticity

the 0.075 mm (№ 200) sieve	**Silts and Clays** Liquid Limit greater than 50%	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
		CH	Inorganic clays or high plasticity, fat clays
		OH	Organic clays of medium to high plasticity
Highly Organic Soils		PT	Peat, muck, and other highly organic soils

Prefix: G = Gravel, S = Sand, M = Silt, C = Clay, O = Organic
Suffix: W = Well Graded, P = Poorly Graded, M = Silty, L = Clay, LL < 50%, H = Clay, LL > 50%

Table 3.3

Size range	Grain Size
Boulder	over 10 "
Cobbles	10" to 3"
Gravel	3" to #10 Sieve
Coarse Sand	#10 to #40 Sieve
Fine Sand	#40 to #200 Sieve (0.075 mm.)
Silt	0.075 to 0.002 mm.
Clay	Smaller than 0.002 mm.
Colloids	Smaller than 0.001 mm

a) Soils having 0 to 19% retained on #10 sieve (table 3.4 may be used).

Table 3.4

Classification	% Sand & Gravel	% Silt	% Clay
Sand	80-100	0-20	0-20
Sandy Loam	50-80	up to 50	up to 20
Loam	30-50	30-50	Up to 20
Silty Loam	Up to 50	50-80	0-20
Silt	0-20	80-100	0-20
Sandy Clay Loam	50-80	0-30	20-30
Clay Loam	20-50	20-50	20-30
Silty Clay Loam	Up to 30	50-80	20-30
Sandy Clay	50-70	0-20	30-50
Silty Clay	0-20	50-70	30-50
Clay	0-50	0-50	30-100
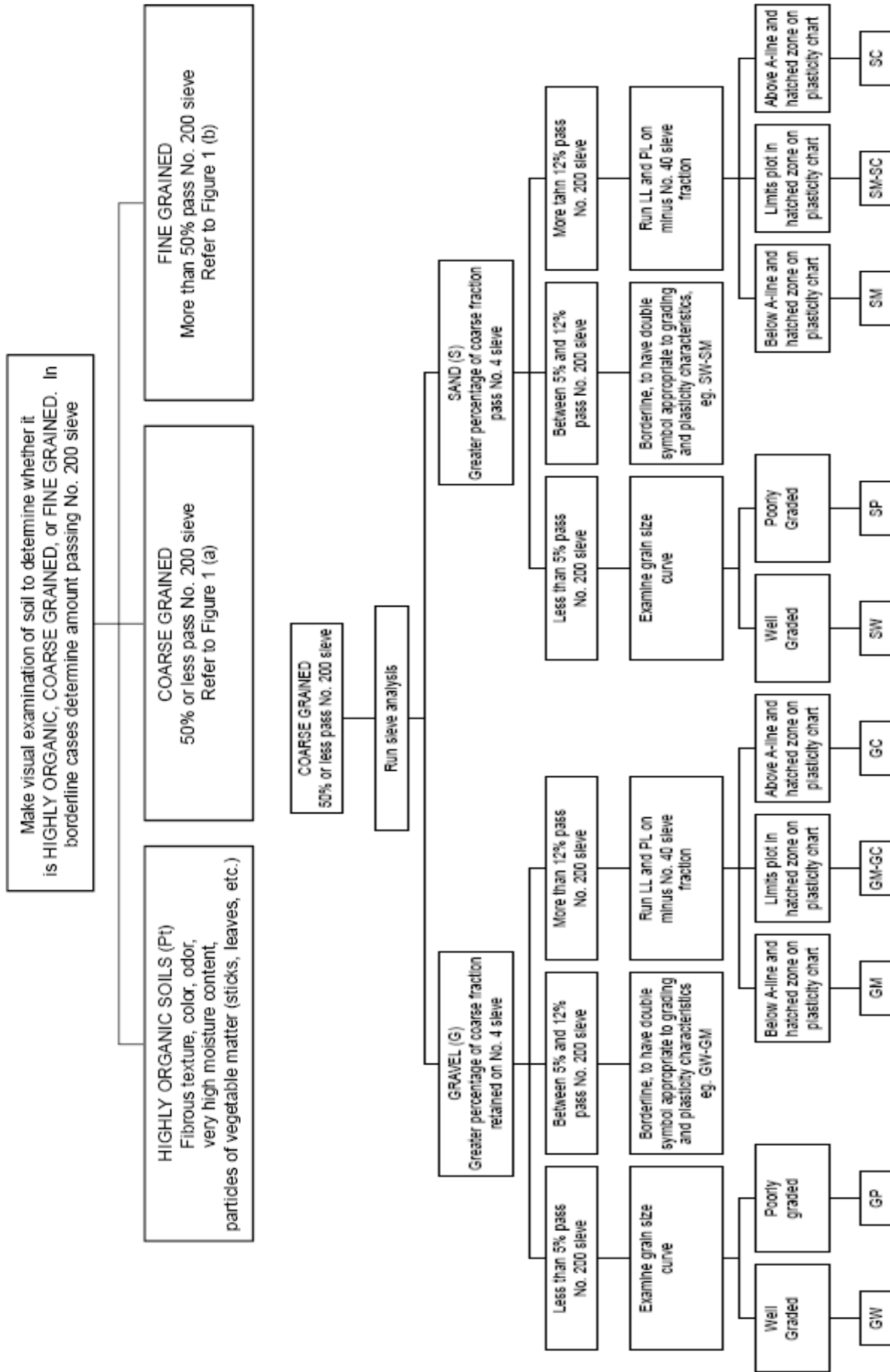
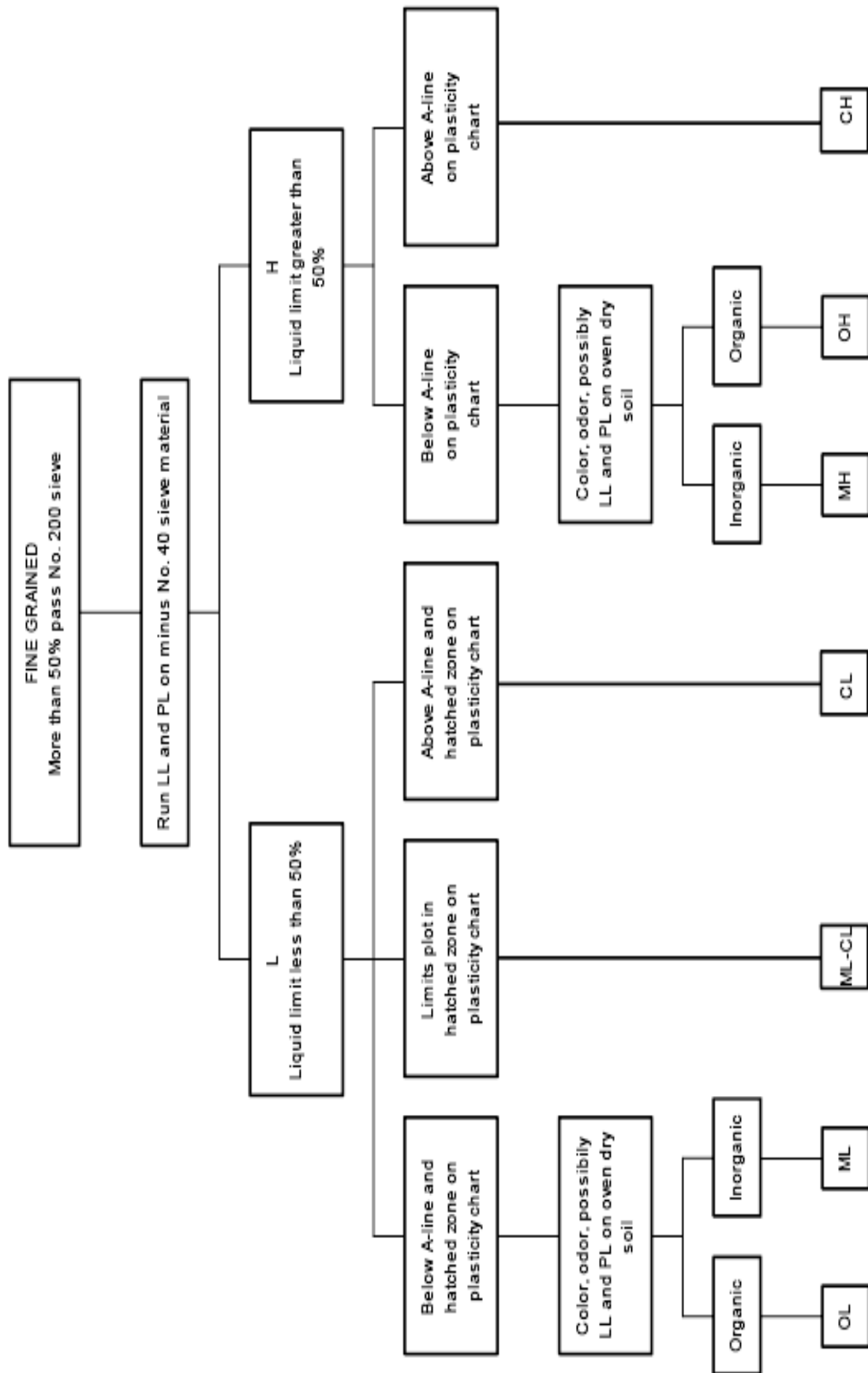Figure 3.18 Laboratory soil identification procedure for coarse grained soil

FINE GRAINED
More than 50% pass No. 200 sieve

Run LL and PL on minus No. 40 sieve material

L
Liquid limit less than 50%

H
Liquid limit greater than 50%

Below A-line and hatched zone on plasticity chart

Limits plot in hatched zone on plasticity chart

Above A-line and hatched zone on plasticity chart

Below A-line on plasticity chart

Above A-line on plasticity chart

Color, odor, possibly LL and PL on oven dry soil

Organic

Inorganic

OL

ML

ML-CL

CL

Color, odor, possibly LL and PL on oven dry soil

Inorganic

Organic

MH

OH

CH

Figure 3.19. Laboratory soil identification procedure for fine grained soil

b) Soils having 20% or more retained on #10 sieve and more than 20% passing 200 sieve (Silt and Clay).

Classify followed by term describing relative amount of gravel according to the following:

20% to 35% gravel - "With some gravel"

36% to 50% gravel - "and gravel"

Examples: Clay Loam with some gravel, Sandy Loam and gravel

c) Soils having 20% or more retained on #10 sieve less than 20% passing #200 sieve (table 3.5 may be used).

Table 3.5

Classification	% Gravel	% Sand	% Silt	% Clay
Gravel	85-100	0-15	0-15	0-15
Sandy Gravel	40-85	15-40	0-20	0-20
Gravelly Sand	20-40	40-80	0-20	0-20
Sand and Gravel	20-50	20-50	0-20	0-20

When the gradation of a given sample does not meet the requirements for any classification exactly, it shall be placed in the classification to which it come the closest.

d) **Organic Soils.** The following classification system (table 3.6) shall be used for organic soils.

Table 3.6

Classification	Percentage
With trace Organic Matter	1 to 6
With Little Organic Matter	7 - 12
With Some Organic Matter	13 to 18
Organic Soil (A-8)	19 - 30
Peat (A-8)	More than 30

e) **Marly Soils.** The following classification system (table 3.7) shall be used for marly soils with calcium and magnesium carbonate content.

Table 3.7

Classification	Percentage
With Trace Marl	1% to 9%
With Little Marl	10% to 17%
With Some Marl	18% to 25%
Marly Soil (A-8) No group index	26% to 40%
Marl (A-8) No group index	More than 40%

4. SOIL PERMEABILITY

Permeability is one of the major parameters flow property of soils used in selecting soils for various types of construction. In some cases it may be desirable to place a high-permeability material immediately under a pavement surface to facilitate the removal of water seeping into the base or sub-base courses. In other cases, such as retention pond dikes, it may be detrimental to use high-permeability materials. Permeability also significantly influences the choice of backfill materials.

Soil permeability can be measured either directly from field testing or indirectly from laboratory tests, although not as accurately. Purpose is to determine the potential of flow of fluids through soils. The Engineer should evaluate whether design for the project needs permeability data.

The ease with which a fluid passes through a porous medium is expressed in terms of coefficient of *permeability* (*k*), also known as *hydraulic conductivity*. Darcy's law for the flow of a fluid through a porous medium has been formulated, in its simplest form, as

$$q = -k\, dh\, /ds \quad (4.1).$$

This means that the hydraulic conductivity *k* can be determined if the specific discharge *q* can be measured in a test in which the gradient *dh/ds* small reservoirs of water, the height of which can be adjusted. In these reservoirs a constant water level can be maintained. Under the influence of a difference in head *h* between the two reservoirs, water will flow through the soil. The total discharge *Q* can be measured by collecting the volume of water in a certain time interval. If the area of the tube is *A*, and the length of the soil sample is *ΔL*, then Darcy's law gives

$$Q = kA\Delta h/\Delta L \qquad (4.2)$$

Because *Q* = *qA* this formula is in agreement with (4.1). Darcy performed tests as shown in Figure 4.1 to verify his formula (4.2). For this purpose he performed tests with various values of *Δh*, and indeed found a linear relation between *Q* and *Δh*. The same test is still used very often to determine the
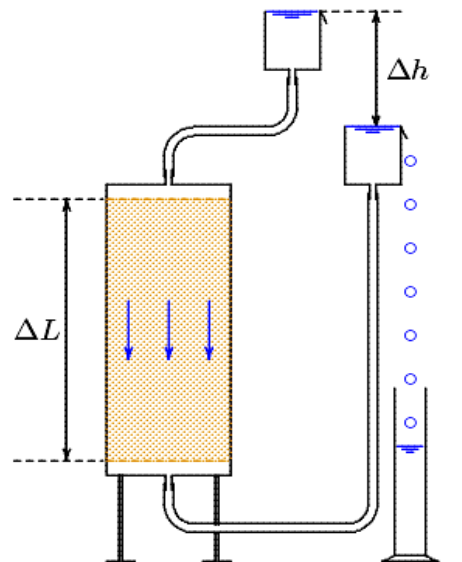

Figure 4.1. Scheme of constant-head

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hydraulic conductivity (coefficient of permeability) k.

For sand normal values of the hydraulic conductivity k range from 10^{-6} m/s to 10^{-3} m/s. For clay the hydraulic conductivity usually is several orders of magnitude smaller, for instance $k = 10^{-9}$ m/s, or even smaller. This is because the permeability is approximately proportional to the square of the grain size of the material, and the particles of clay are about 100 or 1000 times smaller than those of sand. An indication of the hydraulic conductivity of various soils is given in table 4.1.

Table 4.1

Type of soil	Hydraulic conductivity, k (m/s)
gravel	$10^{-3} - 10^{-1}$
sand	$10^{-6} - 10^{-3}$
silt	$10^{-8} - 10^{-6}$
clay	$10^{-10} - 10^{-8}$

There are two basic standard types of test procedures to directly determine permeability: *constant-head and falling-head procedures* (figure 4.1 and 4.2, a). In procedures, undisturbed, remolded, or compacted samples can be used. The permeability of coarse materials is determined by constant head tests. The permeability of clays is normally determined by the use of a *falling head permeameter. The difference between the two tests is* that in the former, the hydraulic gradient of the specimen is kept constant, while in the latter, the head is allowed to decrease as the water permeates the specimen.

Both test procedures determine permeability of soils under specified conditions. The engineer must establish which test conditions are representative of the problem under consideration. As with all other laboratory tests, the engineer has to be aware of the limitations of this test. Generally, soils containing 10 percent or more particles passing a 75 μm sieve are tested using the falling head assembly. More granular soils, containing 90 percent or more particles retained on the 75 μm sieve, are tested using the constant head assembly.

The process of hydraulic conductivity is sensitive to the presence of air or gases in the voids and in the water. Prior to the test, distilled, de-aired water should be run through the specimen to remove as much of the air and gas as practical. It is a good practice to use de-aired or distilled water at temperatures slightly higher than the temperature of the specimen. As the water permeates through the voids and cools, it will have a tendency to dissolve the air and some of the gases, thus removing them during this process. The result will be a more representative, albeit idealized, permeability value.

The combination permeameter is sealed at the top so that a vacuum may

be used to saturate the specimen. This cell has a transparent plastic chamber for soil specimens of either fine-grained or coarse-grained soils. Porous stones located at the top and bottom of the cell prevent sample flaking or washout.

For constant head tests, a plastic funnel reservoir is mounted on an upright attached to the cell, providing a maximum head of 550 mm. Falling head tests are performed using the graduated pipette falling head reservoir, which gives a maximum head of 1000 mm and is graduated 0.2 ml.

Procedure of Constant Head Test. To conduct this test, a permeameter into which the sample is placed and compacted to the desired relative density is used. Water (preferably de-aired) is introduced by an inlet valve until the sample is saturated. Water is then allowed to flow through the sample while a constant head is maintained. The permeability is measured by the quantity of flow of discharge over a specified time. This method is generally used only for coarse-grained soils.

The specimen is formed in a permeability cell and water is passed through it from a constant level tank. Take-off points located along the sides of the permeability cell are connected to three manometer tubes mounted on a panel complete with a miter scale. Water passing through the specimen is collected and measured, either for a specific quantity or over a period of time. The reduction of head is noted from the variation of water level in the manometer tubes.

The coefficient of permeability is equal to the rate of flow of water through a unit cross sectional area under a unit hydraulic gradient. In the constant head parameter, the head causing flow through the specimen remain constant throughout the test. The coefficient of permeability (k) is obtained from the relation.

Apparatus: permeameter mould : internal diameter =100 mm, effective height = 127.3 mm; detachable collar : 100 mm diameter, 60 mm high; compaction equipment; drainage base, having porous disc; drainage cap , having a porous disc with a spring attached to the top; constant head water supply reservoir; constant head collecting chamber; stop watch; weighing balance; thermometer.

Procedure
1. Preparation of sample
a) *disturbed soil sample*
1. Measure the internal dimensions of the mould. Weight the mould to the nearest gram.
2. Apply a little grease on the inside to the mould. Clamp the mould between the base plate and the extension collar and place the assembly on a solid base.

3. Take about 2.5 kg of soil sample, from a thoroughly mixed wet soil in the mould. Compact the soil at the required dry density, using a suitable compacting device.

4. After the compaction, remove the collar and base plate, trim off the surplus soil mass by means of straight edge and weigh the mould with a compacted soil.

5. Saturate the stones. Place the filter paper at both the end of the soil specimen in the mould.

6. Attached this mould with the drainage base and cap having saturated porous stone.

b) undisturbed soil sample

1. For testing the undisturbed soil sample, trim off the undisturbed specimen in the shape of a cylinder of about 85 mm in dia and height equal to that of mould. Put the filter paper at the both the end of the specimen and place it centrally over the bottom saturated porous stone of the drainage base fixed to the mould.

2. Fill the annular space between mould and soil specimen with an impervious material to avoid any leakage from the sides. The impervious material may be cement slurry or a mixture of 10 % bentonite and 90 % fine sand by weight.

3. Fix the drainage cap over the top the mould.

2. Disconnect the reservoir from the bottom out let.

3. Connect the constant head reservoir to the drainage cap inlet.

4. Open the stop cock and allow the water to flow downward so that all the air is removed.

5. Close the stop cock and allow the water to flow through the soil till a steady state is attained.

6. Start the stop watch, and collect the water flowing out of the base in a measuring flask for some convenient time interval.

7. Repeat this thrice, keeping the interval the same. Check that quantity of water collected is approximately the same each time.

8. Measure the difference of head (h) in levels between the constant head reservoir and the outlet in the base.

9. Repeat the test for at least two more different interval.

10. Stop the flow of water and disconnect all the part of assembly. Record the temperature of the water used in the test.

Alternatively, a constant head test can be performed on a compacted specimen in a compaction mold.

Procedure of Falling Head Test. Clays and silts are tested using the 'falling head' technique. To conduct this test, an apparatus and a procedure similar to the constant-head test above are used. The soil is compacted into a

mold. The permeability is measured by the decrease in the head of water for a specified time.

Flow of water through the specimen is observed by monitoring the rate of fall of water in the tube. It is essential that soils of very low permeability are sealed inside the cylinder to prevent seepage along the sides of the specimen. Before testing, the specimen must be completely saturated with water as the presence of air will restrict the flow of water.

The apparatus consists of a clay sample in a circular ring, placed in a container filled with water. The lower end of the sample is in open connection with the water in the container; through a porous stone below of the sample it is connected to a thin glass tube, in which the water level is higher than the constant water level in the container. Because of this difference in water level, water will flow through the sample, in very small quantities, but sufficient to observe by the lowering of the water level in the thin tube.

In this case the head difference h is not constant, because no water is added to the system, and the level h is gradually reduced. This water level is observed as a function of time.

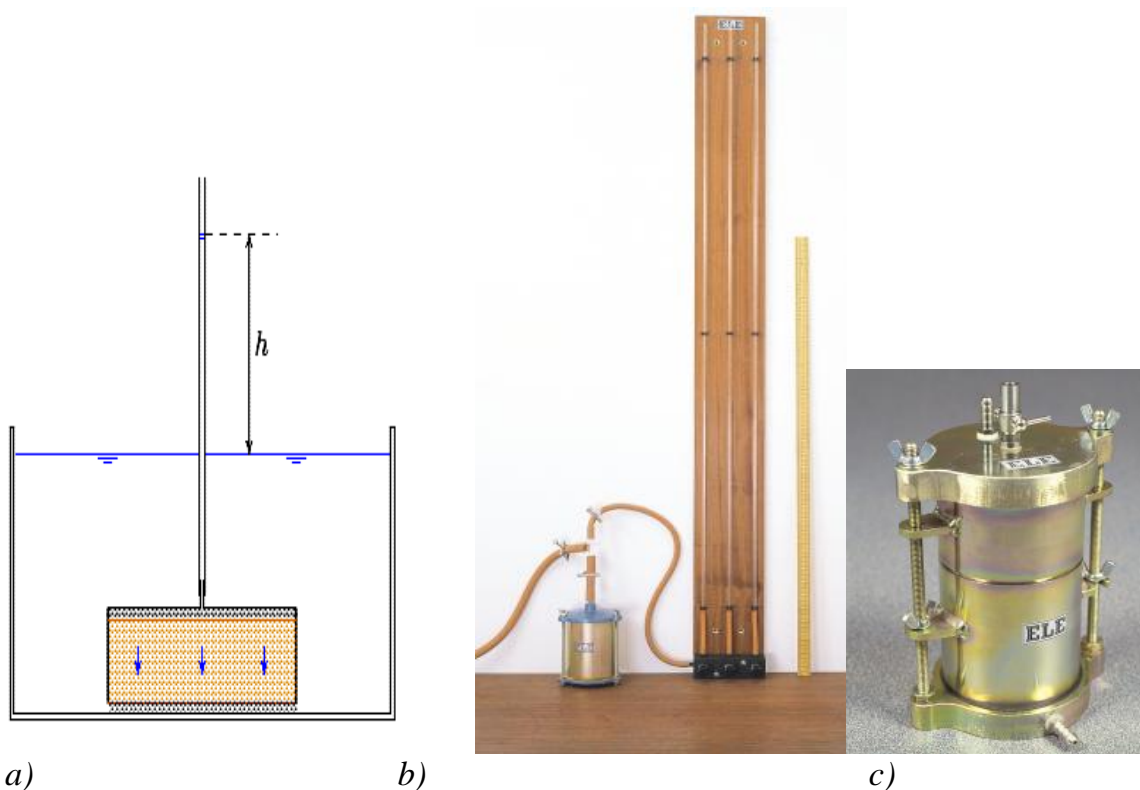a)   b)   c)

Figure 4.2. a) Scheme of falling head test; b) falling head permeability cell with manometer tubes, c) compaction permeameter

Permeability test with flexible-wall permeameters. For fine-grained soils, tests using *a triaxial cell* are generally preferred, because Constant and

Falling Head tests take too long to perform. In situ conditions can be modeled by application of an appropriate confining pressure. The sample can be saturated using back pressuring techniques. Water flows through the sample under pressure and measurements are taken until steady-state conditions occur.

For testing of fine-grained, low-permeability soils, the use of *flexible-wall permeameters* is recommended which are essentially very similar to the triaxial test apparatus (Figure 4.3). When *rigid wall units* are used, the permeant may find a route at the sample-permeameter interface, thus it may drain through that interface rather than travel through the specimen. This will produce erroneous results. *A flexi-wall permeameter system* designed for determining hydraulic conductivity of saturated porous materials, saturated leach conductivity and intrinsic permeability. The system is of modular design enabling one or three samples to be tested simultaneously when using the optional panels.

The three cell system allows the operator to have one specimen in saturation, one in consolidation and the other in shear at different confining pressures. Simple to use, the systems comprise a master control panel, permeability cell and de-aired water system. Optional 70 mm and 100 mm diameter specimen accessories are available to test nontoxic specimens. *Measurement range:* permeability 10^{-4} to 10^{-10} cm/sec, pressure 14 to 1000 kPa 14 to 1000 kPa, vacuum 0 to -100 kPa.

Figure 4.3. Three cell permeability apparatus

Pinhole test. Clay soils which contain a high proportion of sodium can be highly erodible under the action of water flowing through them. These dispersive clays can lead to 'piping' and subsequent failure of foundations and earth dams. A simple method of determining erodibility is the 'Pinhole

Test'. This pinhole dispersion test apparatus (Figure 4.4.) identifies clay soils which can be dispersed by water.

It should be emphasized that permeability is sensitive to viscosity. In computing permeability, the correction factors for viscosity and temperatures should be applied. During testing, the temperature of the permit and the laboratory should be kept constant.
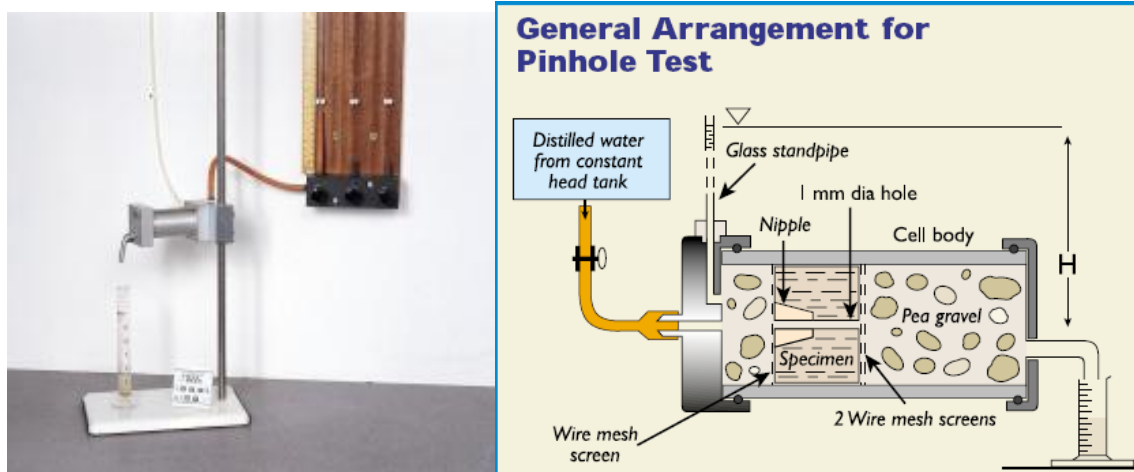

Figure 4.4. Pinhole dispersion test apparatus complete with hoses and stand.

Laboratory permeability tests produce reliable results under ideal conditions. Permeability of fine-grained soils can also be computed from one-dimensional consolidation test results, although these results are not as accurate as direct *k* measurements.

References

1. *Critical state soil mechanics. Andrew Schofield and Peter Wroth. Lecturers in Engineering at Cambridge University.*
2. American Association of State Highway and Transportation Officials (AASHTO) (1988). *Manual on Subsurface Investigations*, Developed by the Subcommittee on Materials, Washington, D.C.
3. American Association of State Highway and Transportation Officials (AASHTO). (1995). *Standard specifications for transportation materials and methods of sampling and testing: part II: tests*, Sixteenth Edition, Washington, D.C.
4. American Society for Testing & Materials. (2000). *ASTM Book of Standards*, *Vol. 4, Section 08 and 09*, Construction Materials: Soils & Rocks, Philadelphia, PA.
5. Atkinson, J.H. (2000) Non-linear soil stiffness in routine design. *Geotechnique* 50, No5, 487 - 508.
6. Bowles, J. E. 1992. Engineering properties of soils and their measurements, 4th Edition, McGraw-Hill, Enc.
7. C. Karg is with the Civil Engineering Department, Ghent University (UGent), Gent, Belgium.
8. Critical state soil mechanics. Andrew Schofield and Peter Wroth. *Lecturers in Engineering at Cambridge University.*
9. Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates. *In* 1997 Annual Book of ASTM Standards, Section 4, Volume 04.08, Soil and Rock (1): D 420 - D 4914, pg. 78 - 82. ASTM, West Conshohocken, PA, USA.
10. *Engineering and design. Laboratory soils testing. Department of the army . U. S. Army Corps of Engineers .Washington, D. C. 1986*
11. Head, K. H. 1980. Manual of soil laboratory testing, Vol. 1: Soil Classification and Compaction Tests, Pentech Press Limited, Plymouth, London.
12. *Manual on subsurface investigations. Paul W. Mayne, Barry R. Christopher, and Jason DeJong. National Highway Institute, Federal Highway Administration Washington, DC. 2001.*
13. Soil Mechanics – Laboratory Manual, B.M. DAS
14. Soil mechanics. *Arnold Verruijt.* Delft University of Technology, 2001
15. Soil Properties, Testing, Measurement, and Evaluation, C. Liu, J. Evett
16. Standards Association of Australia. *AS 1289.C4.1-1977 Methods of testing soils forengineering purposes - Soil classification tests -*

Determination of the linear shrinkage ofa soil - Standard method.

17. Wagner, A. A. 1957. The use of the unified soil classification system by the bureau of reclamation. Proceedings. 4th International Conference SMFE, Butterworths, London, Vol. 1. pp. 125 - 134.
18. www.geo-observations.com/
19. *www. geonor@geonor.com*
20. www.uic.edu/classes/cemm/cemmlab/
21. *www. Tecnotest.com*
22. *www.ele. com*
23. *www.gdsinstruments. Com*

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