ФЕДЕРАЛЬНОЕ АГЕНТСТВО ПО ОБРАЗОВАНИЮ

Государственное образовательное учреждение высшего профессионального образования

«НАЦИОНАЛЬНЫЙ ИССЛЕДОВАТЕЛЬСКИЙ ТОМСКИЙ ПОЛИТЕХНИЧЕСКИЙ УНИВЕРСИТЕТ»

УТВЕРЖДАЮ
Проректор-директор ИПР

___________________А.К. Мазуров
« __ » ____________ 2011 г.

Laboratory testing of soils

Part II. Mechanical properties of soils

Методические указания к выполнению лабораторных работ
по курсу «Грунтоведение»
для студентов обучающихся по направлению 130100
«Геология и разведка полезных ископаемых».

Составитель **Крамаренко В.В.**

Издательство
Томского политехнического университета
2010

УДК 624.131.37(076.5)

ББК 26.3я73

Л-125

Крамаренко В.В.

Laboratory testing of soils. Part II. Mechanical properties of soils: Методические указания к выполнению лабораторных работ по курсу «Грунтоведение» для студентов, обучающихся по направлению 130100 «Геология и разведка полезных ископаемых». / сост. В.В. Крамаренко; Национальный исследовательский Томский политехнический университет. – Томск: Изд-во Томского политехнического университета, 2011.– 52 с.

УДК 624.131.37(076.5)

ББК 26.3я73

Методические указания рассмотрены и рекомендованы
к изданию методическим семинаром кафедры
гидрогеологии, инженерной геологии и гидрогеоэкологии ИГНД
«26» января 2009

Зав. кафедрой ГИГЭ
Доктор геолого-минералогических наук __________*С.Л. Шварцев*

Председатель учебно-методической
комиссии __________*Н. Г. Наливайко*

Рецензент

канд. геол.-минер. наук доцент ТПУ Т.Я. Емельянова

5. MECHANICAL PROPERTIES OF SOILS

5.1. Stress in soils

5.1.1. Load-deformation process in soils

The design and analysis of shallow and deep foundations, excavations, earth retention structures, and fills and slopes require a thorough understanding of soil strength parameters. The selection of strength parameters needed and the corresponding types of tests to be performed vary depending on the type of construction, the foundation design, the intensity, type and duration of loads to be imposed, and soil materials existing at the site.

For simple projects, sufficient strength information can be obtained from strength correlations using field and laboratory index tests. However, strength testing may be needed when there is uncertainty in the knowledge of material properties, or if design loads/stresses are significant. The type of tests needed depends on whether the analysis is performed using total or effective stresses.

The purpose of laboratory testing is to simulate in-situ soil loading under controlled boundary conditions. There are three general ways to induce deformations in solids or semi-solids: *tension, compression, and shear*.

When a load is applied to a soil sample, the deformation which occurs will depend on the grain-to-grain contact (intergranular) forces and the amount of water in the voids. If no porewater exists, the sample deformation will be due to sliding between soil grains and deformation of the individual soil grains. The rearrangement of soil grains due to sliding accounts for most of the deformation. Adequate deformation is required to increase the grain contact areas to take the applied load. As the amount of pore water in the void increases, the pressure it exerts on soil grains will increase and reduce the intergranular contact forces.

In fact, tiny clay particles may be forced completely apart by water in the pore space.

Deformation of a saturated soil is more complicated than that of dry soil as water molecules, which fill the voids, must be squeezed out of the sample before readjustment of soil grains can occur. The more permeable a soil is, the faster the deformation under load will occur. However, when the load on a saturated soil is quickly increased, the increase is carried entirely by the pore water until drainage begins. Then more and more load is gradually transferred to the soil grains until the excess pore pressure has dissipated and the soil grains readjust to a denser configuration. This process is called *consolidation* and results in a higher unit weight and a decreased void ratio.

5.1.2. Principle of effective stress

The consolidation process demonstrates the very important principle of effective stress. Under an applied load, the total stress in a saturated soil sample is composed of the intergranular stress and porewater pressure (neutral stress). As the porewater has zero shear strength and is considered incompressible, only the intergranular stress is effective in resisting shear or limiting compression of the soil sample. Therefore, the intergranular contact stress is called the effective stress. Simply stated, this fundamental principle states that the effective stress (σ') on any plane within a soil mass is the net difference between the total stress (σ) and porewater pressure (u).

When porewater drains from soil during consolidation, the area of contact between soil grains increases, which increases the level of effective stress and therefore the soils shear strength. In practice, staged construction of embankments is used to permit increase of effective stress in the foundation soil before subsequent fill load is added. In such operations the effective stress increase is frequently monitored with piezometers to ensure the next stage of embankment can be safely placed.

Soil deposits below the water table will be considered saturated and the ambient pore pressure at any depth may be computed by multiplying the unit weight of water (γ_w) by the height of water above that depth. For partially saturated soil, the effective stress will be influenced by the soil structure and degree of saturation. In many cases involving silts & clays, the continuous void spaces that exist in the soil behave as capillary tubes of variable cross-section. Due to capillarity, water may rise above the static groundwater table (phreatic surface) as a negative porewater pressure and the soils may be nearly or fully saturated.

5.1.3. Overburden stress

Soils existing at a depth below the ground surface are affected by the weight of the soil above that depth.

The influence of this weight, known generally as the *overburden stress*, causes a state of stress to exist which is unique at that depth for that soil. When a soil sample is removed from the ground, that state of stress is relieved as all confinement of the sample has been removed. In testing, it is important to reestablish the in-situ stress conditions and to study changes in soil properties when additional stresses representing the expected design loading are applied. In this regard, the effective stress (grain-to-grain contact) is the controlling factor in shear, state of stress, consolidation, stiffness, and flow. Therefore, the designer should try to re-establish the effective stress condition during most testing. The test confining stresses are estimated from the total, hydrostatic, and effective overburden stresses. The engineer's first task is determining these stress and pressure variations with depth. This involves determining the total unit weights (density) for each soil layer in the subsurface profile, and determining the depth of the water table. Unit weight may be accurately determined from density tests on undisturbed samples or estimated from in-situ test measurements.

The total vertical (overburden) stress (σ_{vo}) at any depth (z) may be found as the accumulation of total unit weights (γ_t) of the soil strata above that depth:

$$\sigma_{vo} = \int \gamma_t\, dz = \Sigma \gamma_t \Delta z. \qquad (5.1)$$

For soils above the phreatic surface, the applicable value of total unit weight may be dry, moist, or saturated depending upon the soil type and degree of capillarity. For soil elements situated below the groundwater table, the saturated unit weight is normally adopted.

The hydrostatic pressure depends upon the degree of saturation and level of the phreatic surface and is determined as follow:

Soil elements above water table:

$$u_o = 0 \text{ (completely dry);} \qquad (5.2)$$
$$u_o = \gamma_w (z - z_w) \text{ (full capillarity).} \qquad (5.3)$$

Soil elements below water table:

$$u_o = \gamma_w (z - z_w), \qquad (5.4)$$

where z = depth of soil element, z_w = depth to groundwater table.

Another case involves partial saturation with intermediate values between (2 and 3) which literally vary daily with the weather and can be obtained via tensiometer measurements in the field. Usual practical calculations adopt (3) for many soils, yet the negative capillary values from (2) often apply to saturated clay & silt deposits. The effective vertical stress is obtained as the difference between (1) and (2):

$$\sigma_{vo} = \sigma_{vo} - u_o \qquad (5.5).$$

A plot of effective overburden profile with depth is called a σ'_v diagram and is extensively used in all aspects of foundation testing and analysis.

5.1.4. Total and effective stress analysis

Soils are controlled by the effective stress strength envelope (c and φ')

and therefore the proper determination of these parameters is paramount.

The strength envelope is best determined by either a series of

(1) consolidated undrained triaxial shear tests with porewater pressure measurements (CU);

(2) consolidated drained triaxial tests at slow strain rates (CD);

or (3) drained direct shear tests (DDS).

For long-term analyses, the drained parameters are equal to effective cohesion intercept c and effective friction angle φ from the effective stress Mohr-Coulomb envelope (see Figure 5.0). The shear strength (τ_{max}) is given by:

$$\tau_{max} = c' + \sigma' \tan(\varphi) \qquad (5.6).$$

Usually, $c' \approx 0$ is adopted because lab tests are affected by rate & duration effects and c' is a bond that weathers with time. Effective strength parameters apply to all soil types, including gravels, sands, silts, and clays.

The stress dependency of soil can be characterized by the stress path method. A stress path gives a numerical and graphical representation of the past, present and future state of stress on a representative soil element. It captures the geologic stress history of the element, the current stresses acting on the element, and the anticipated future changes in stress on the element. The stress path method determines what these stresses are, subjects representative elements of soil to these stress paths, and measures the resulting mechanical behavior of the soil. The measurements are used to determine strength, compressibility and permeability for specific stress paths. These stress path dependent mechanical properties are then used in analysis and design to predict the future performance of a constructed facility.

The CU triaxial test results can be used to develop the stress path of the soil under the test conditions by plotting the effective strength for each load increment from the start to finish of the test. Using the stress path method, the test results can then be analyzed with respect to the approximate field stress and strain conditions before, during,

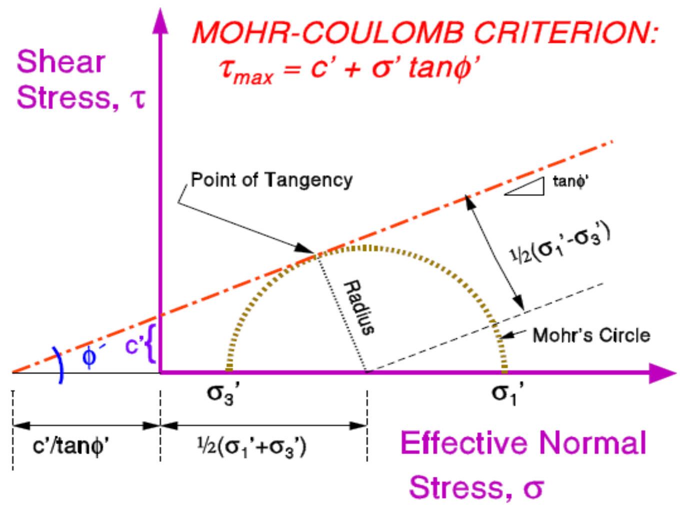


Figure 5.0. Definitions of effective stress parameters for Mohr-Coulomb failure criterion

and after construction.

For short-term loading of clays & silts, total stress analysis uses the undrained shear strength (designated s_u or c_u, the old archaic term ”cohesion” designated “*c*” has been replaced with undrained shear strength) that is a soil behavioral response that reflects the combination of the effective stress frictional envelope (c' and φ') plus excess porewater pressures that depend on stress history. From this regard, perhaps the simple shear is the most appropriate test for stability & bearing capacity analyses, however, the device is not in widespread use. Other modes of s_u include triaxial compression & extension, plane strain active & passive, true triaxial, hollow cylinder, and directional shear, all of which provide different values of s_u depending upon the boundary conditions, direction of loading, strain rate, and initial stress state. As this is a complex issue, the best value is calculated from the normalized value

$$s_u / \sigma_{vo}' = 0.5 \sin\varphi' \, OCR^{0.8}.$$

For extensively fissured clays and tills, the macrofabric of discontinuities reduces the overall strength should be reduced by a factor of 2. In the case of fissured geomaterials, it is also common that these exhibit past problems with landsliding and slope instability, therefore the drained strength parameters may be more appropriately assigned to the residual values (c_r' and φ_r'). Residual strengths can be determined by ring shear tests or series of repeated drained direct shear box tests.

Subsurface Investigations provides guidance on the selection of test types and the determination of applied loads/pressures. The general design cases and applicable stress categories are summarized below:

- foundations total stress – undrained strengths;
- excavation effective stress – drained strengths;
- natural slope effective stress – drained strengths.

Typically, at least three strength tests are performed to obtain a strength envelope and identify any obviously erroneous test results. The confining pressures for each test should be estimated by the engineer, which should be based on the range of stress levels to be experienced during various stages of construction (including the initial overburden pressures). Typically, the first test has a confining stress similar to the existing overburden pressure, and the second and third tests would have higher confining stresses to determine φ.

Testing pressures are commonly about 10, 20, and 40 psi (effective stresses). In cases where the Mohr envelope is not linear (because of past preconsolidation) or the project is complex, additional tests should be conducted in the lower stress range to better model the Mohr envelope and to better define the cohesion intercept.

Figure 5.1 provides an overview of the various laboratory tests used to measure soil strength showing the imposed stresses and loading conditions. Laboratory testing for most projects may include unconfined compression, triaxial shear, and direct shear. Other tests including direct simple shear, plane strain, torsional shear, hollow cylinder, cubical triaxial, and ring shear are also available, but are most often used on large projects. The common methods of ascertaining shear strength parameters in the laboratory are discussed below.

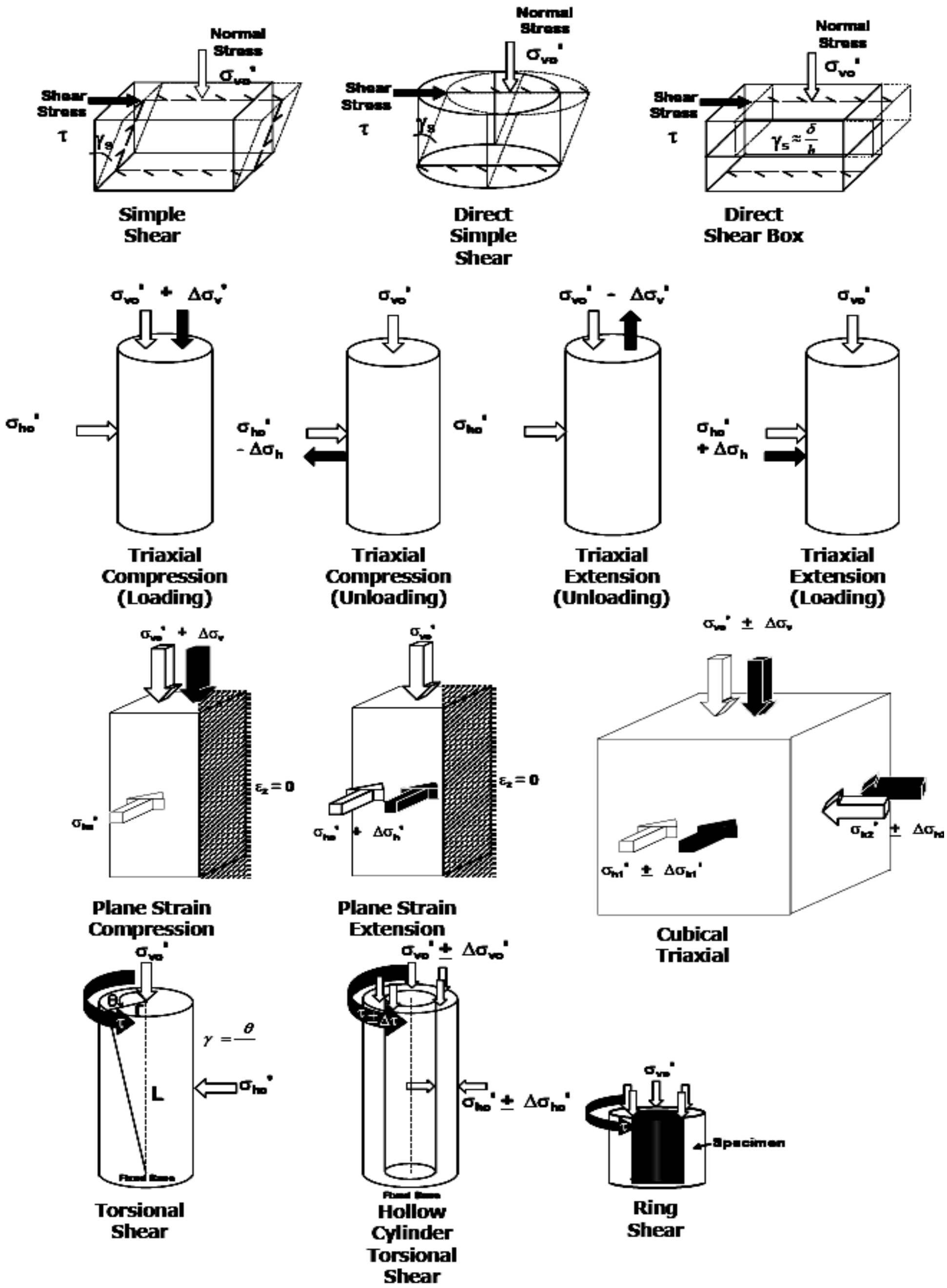


Figure 5.1. Various laboratory tests used to measure soil strength showing the imposed stresses and loading conditions.

5.2 Laboratory testing of mechanical properties

5.2.5. Consolidation test

Consolidation is an important fundamental phenomenon which must be understood by everyone who attempts to gain a knowledge of soil behavior in engineering applications. Consolidation is a time-dependent process, in some soils it may take long time (100 years ?) to achieve complete settlement. The amount of soil volume change that will occur is often one of the governing design criteria of a project. Every building or structure which is founded in or on the earth imposes loads on the soil that supports the foundations. The stresses set up in the soil cause deformation of the soil. The quantitative data that an engineer needs depend upon the mechanical properties such as stiffness and strength, and these must be determined from mechanical tests.

The main purpose of consolidation tests is to obtain soil data which is used in predicting the rate and amount of settlement of structures founded on clay. Although some of the settlement of a structure on clay may be caused by shear strain, most of it is normally due to volumetric changes. This is particularly true if the clay stratum is thin compared to the width of the loaded area or the stratum is located at a significant depth below the structure.

If the settlement is not kept to tolerable limit, the desire use of the structure may be impaired and the design life of the structure may be reduced. It is therefore important to have a mean of predicting the amount of soil compression or consolidation. It is also important to know the rate of consolidation as well as the total consolidation to be expected.

Consolidation is the process of time-dependent settlement of saturated clayey soil when subjected to an increased loading. Clay soils undergo consolidation settlement not only under the action of “external” loads (surcharge loads) but also under its own weight or weight of soils that exist above the clay (geostatic loads). Clay soils also undergo settlement when dewatered (e.g., ground water pumping) – because the effective stress on the clay increases

Coarse-grained soils do not undergo consolidation settlement due to relatively high hydraulic conductivity compared to clayey soils. Instead, coarse-grained soils undergo immediate settlement.

Consolidation refers to the compression or settlement that soils undergo as a response of placing loads onto the ground. These loads produce corresponding increases in the vertical effective stress.

The compression is caused by:

- deformation of soil particles;
- relocations of soil particles;
- expulsion of water or air from void spaces;
- most of the settlement of a structure on clay is mainly due to volumetric

changes and rarely due to shear strain.

We can understand the consolidation process by looking at the spring analogy (the Terzaghi one-dimensional theory of consolidation) shown in Figure 5.2. The saturated soil element is represented by Figure 5.2 a, in which the spring corresponds to the soil structure and the water to the soil pore water. If a weight *W* is placed on the water and the spring with the valve *y* closed (Figure 5.2 a), the weight is almost entirely carried by the water, since it is incompressible as compared to the spring. If the valve is opened and water is allowed to escape, the load will eventually be carried entirely by the spring (Figure 5.2 c). The elapsed time required to transfer the load from the water to the spring depends upon how rapidly water is allowed to escape from the valve.

The rate at which the volume change, or consolidation, occurs in a soil is directly related to the permeability of the soil because the permeability controls the speed at which the pore water can escape. The permeability of most sands is so high that the time required for consolidation after a load application can be considered negligible except for cases where a large mass of sand is subjected to a rapid shear or shock loading. Conversely, the low permeability of a clay makes the rate of volume change after application of a load a factor which must be considered. Laboratory consolidation studies, therefore, are almost entirely limited to soils of low permeability.

When a saturated soil mass is subjected to a load increment, the load is usually carried initially by the water in the pores because the water is incompressible when compared with the soil "skeleton". The pressure which results in the water because of the loading is called *hydrostatic excess pressure* because it is in excess of that pressure due to the weight of water.

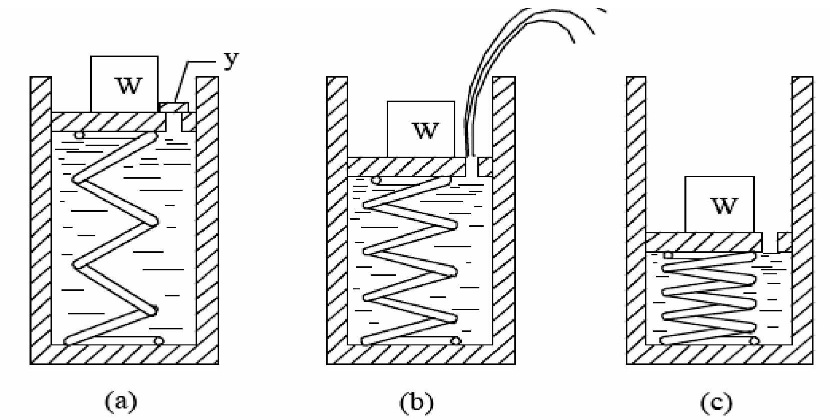


Figure 5.2: The spring analogy to consolidation.

As the water drains from the soil pores, the load increment is gradually shifted to the soil structure. This transfer of load is accompanied by a change in the total volume of soil equal to the volume of water drained. This process is known as ***consolidation.***

Therefore are three types of consolidation:

–*immediate consolidation*; caused by elastic deformation of dry soil or moist and saturated soil without change in moisture content;

–*primary consolidation*; caused as a result of volume change in saturated cohesive soils due to exclusion of water occupied the void spaces;

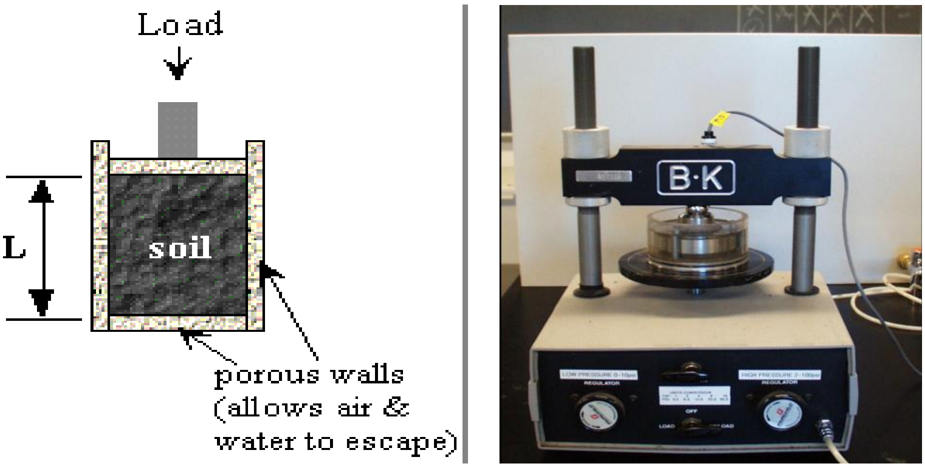
–*secondary consolidation*; occurs in saturated cohesive soils as a result of the plastic adjustment of soil fabrics.

The one-dimensional consolidation test (or *oedometer* test) provides one of the most useful and reliable laboratory measurements for soil behavior. The test determines the compressibility parameters (C_c, C_s, C_r), stiffness in terms of constrained modulus ($D_r = 1/m_v$), preconsolidation stress (σ_p), rate of consolidation (c_v), creep rate (C_{α}), and approximate value of permeability (k).

Data obtained from these tests together with classification data and knowledge of the soils loading history, enables estimates to be made of the behavior of foundations under load. Tests are carried out on specimens prepared from undisturbed samples.

This test method assumes that dimensional change due to consolidation will take place in the vertical direction**.** This assumption is generally acceptable for stiff or medium, confined cohesive soils, but it is not true for soft soils or for soils that are not confined (i.e., bridge approaches). The data and the analysis produced from this test have proved to be reasonably reliable.

All loads and recorded deformations are in the vertical direction (figure 5.3 a). Prepared samples are placed in a rigid-walled loading device called a consolidometer or oedometer (figure 5.3 b).



a) b)

Figure 5.3. Schematic diagram of a) consolidation test; b) consolidometer

Apparatus: consolidometer with loading device; specimen ring, made of a non-corroding material; water reservoir to saturate the sample; porous stones; soil trimming tool, like knife, spatula; dial gauge, accuracy 0.002 mm; pressure pad; weighing balance, accuracy 0.01g; oven; ball.

Procedure

1. Preparation of sample:

   undisturbed soil specimen: Clean dry and then lubricate the consolidation ring from inside with silicon grease. Push the sample directly into the consolidation ring and hold the ring firmly about 5 mm above the sample tube keeping the cutting face down ward and eject the sample gently and steadily out of the tube by means of hydraulic jack so that it intrudes into the ring. During this process, continue trimming the specimen care fully from outside the consolidation ring to reduce friction. Trim and flush the soil sample with the ends of the consolidation ring.

   remoulded soil specimen: Prepare the soil sample by compaction method. Place the consolidation ring on glass plate with cutting edge upward. Press the remoulded soil into the ring by suitable means. Flush the soil specimen with the top end of the ring and weigh. Alternatively, the soil specimen may be intruded into the consolidation ring.

1. Clean and dry the metal ring. Measure its diameter and height. Take the mass of the empty ring.

2. Press the ring into the soil sample at the desired density and the water content. The ring is to be pressed with hands. Any voids in the specimen due to the removal of large particles should be filled back by pressing the soil lightly.

3. Trim the specimen flush with the top and bottom of the ring.

4. Saturate the porous stones by boiling them in distilled water for about 15 minutes.

5. Assemble the consolidometer. Place the bottom porous stone, bottom filter paper, specimen, top filter paper and the top porous stone, one by one.

6. Position the loading block centrally on the top porous stone. Mount the mould assembly on the loading frame. Centre it such that the load applied is axial in the lever-loading system.

7. Set the dial gauge in the position. Allow sufficient margin for the swelling of the soil.

8. Connect the mould assembly to the water reservoir having the water level at about the same level as the soil specimen. Allow the water flow into the specimen till it is fully saturated.

9. Take the initial reading of the dial gauge.

10. Apply an initial setting load to give a pressure of 0.05 kg/cm^2 to the assembly so that there is no swelling and allow the setting load to stand till

there is no change in the dial gauge reading or for 24 hours. Take the final gauge reading under the initial setting load.

11. Normal sequence of pressure to be applied is 0.25, 0.50, 1.0, 2.0, 4.0, 8.0 and 16.0 kg /cm^2 and take the dial gauge reading after application of each load at a time sequence of 0.25, 1.0, 2.25, 4.0, 6.25, 12.25, 16, 20, 25, 36, 49, 64, 81, 100, 121, 144, 169, 196, 225, 289, 324, 361, 400, and finally 1440 minutes.

12. After the last load increment had been applied and the reading taken, decrease the load to . of the last load and allow it stand for 24 hours. Take the dial gauge reading after 24 hours. Further reduce the load and repeat the above procedure, likewise further reduce the load and repeat the procedure. Finally reduce the load to the initial setting load and keep out for 24 hours and take the final dial gauge reading.

13. Dismantle the assembly. Take out the ring with the specimen. Wipe out the excess surface water using bloating paper and remove the filter paper both side the specimen.

14. Take weight of the ring with specimen.

15. Dry the specimen in oven for 24 hours and determine the dry weight of the specimen.

16. Determine the specific gravity of soil from the dried specimen.

The consolidometer can be either a floating ring consolidometer (figure 5.4, a) or a fixed ring consolidometer (figure 5.4, b). During the consolidation test, when load is applied to the soil specimen, the nature of variation of side friction between the surrounding brass ring and the specimen are different for the fixed ring and the floating ring consolidometer, and this is shown in figure 5.4. In most cases, a side friction of 10% of the applied load is a reasonable estimate.

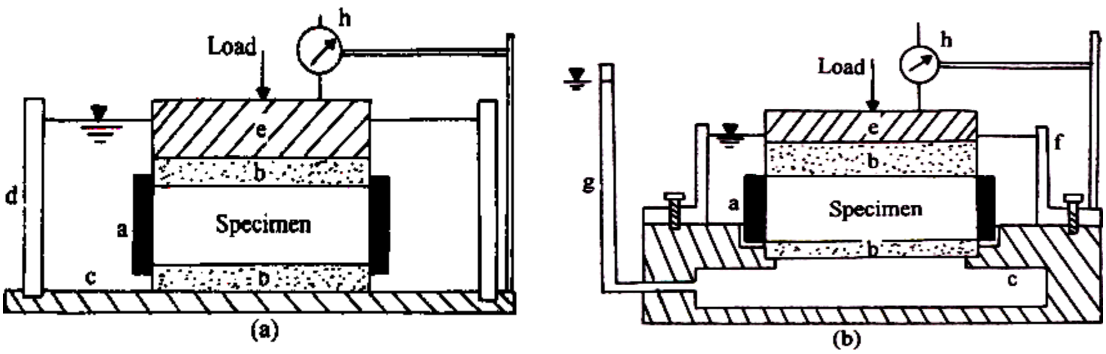


Figure 5.4. Schematic sketch of Consolidometer: a) floating ring consolidometer; b) fixed ring consolidometer

The floating ring consolidometer usually consists of a brass ring (a) in which the soil specimen is placed. One porous stone (b) is placed at the top of the specimen and another porous tone at the bottom. The soil specimen in the ring with the two porous stones is placed on a base plate. A plastic ring surrounding the specimen fits into a groove on the base plate. Load (e) is applied through a loading head that is placed on the top porous stone. In the floating ring consolidometer compression of the soil specimen occurs from the top and bottom towards the center. The order of loading consolidometer is showed on figure 5.5.

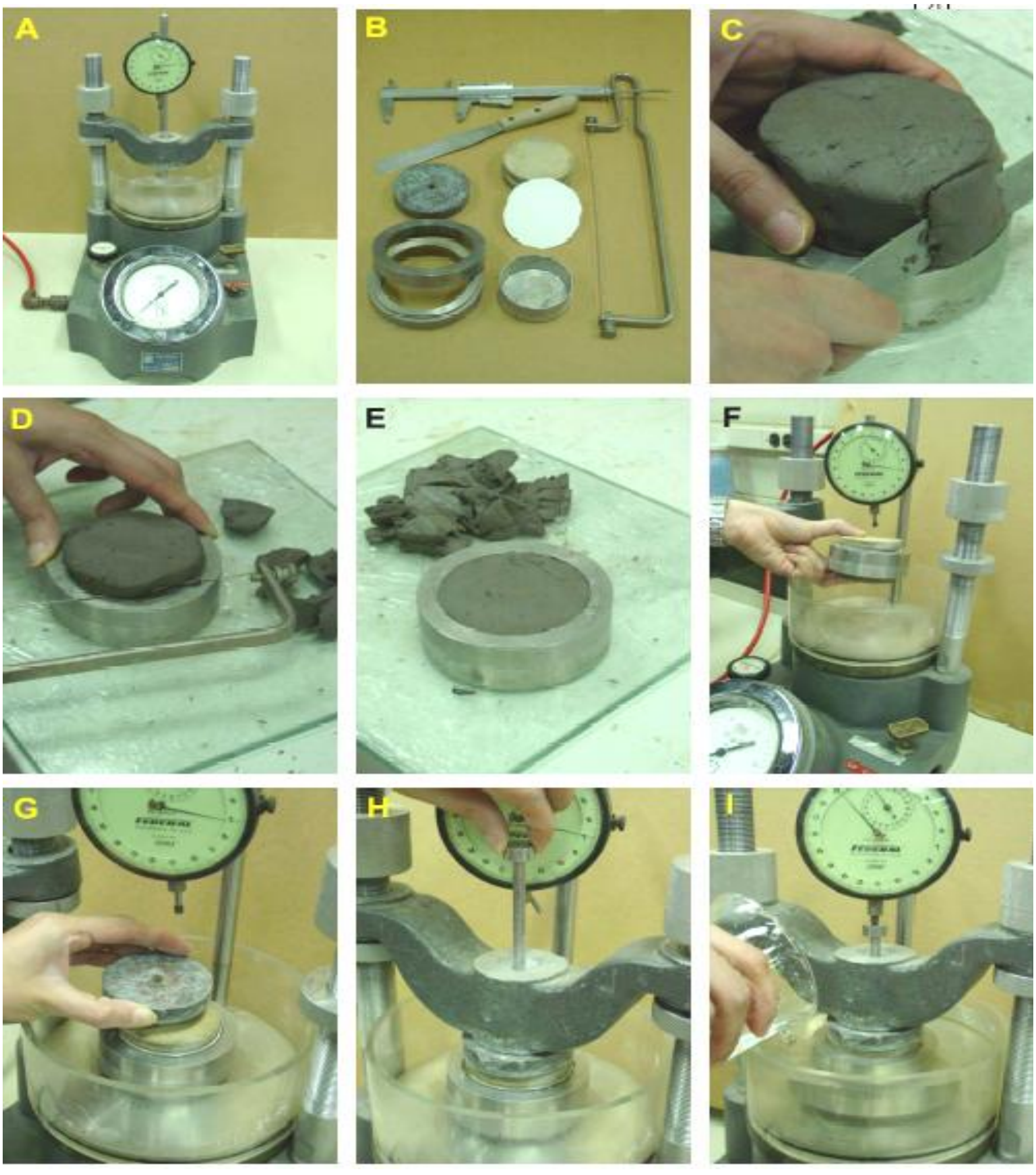


Figure. 5.5. The order of loading consolidometer

The fixed ring consolidometer essentially consists of the same components, i.e., a hollow base plate, two porous stones, a brass ring to hold the soil specimen, and a metal ring that can be fixed tightly to the top of the base plate. In the fixed ring consolidometer, the compression of the specimen occurs from the top towards the bottom.

The specifications for the loading devices of the consolidation test unit vary depending upon the manufacturer. Consolidation testing system (figure 5.6.) is a state-of-the-art, fully-automated consolidation testing system designed for soil. The system is based on either the Rowe and Barden or the Rowe type consolidation cells and the pressure/volume controllers. An overview of the

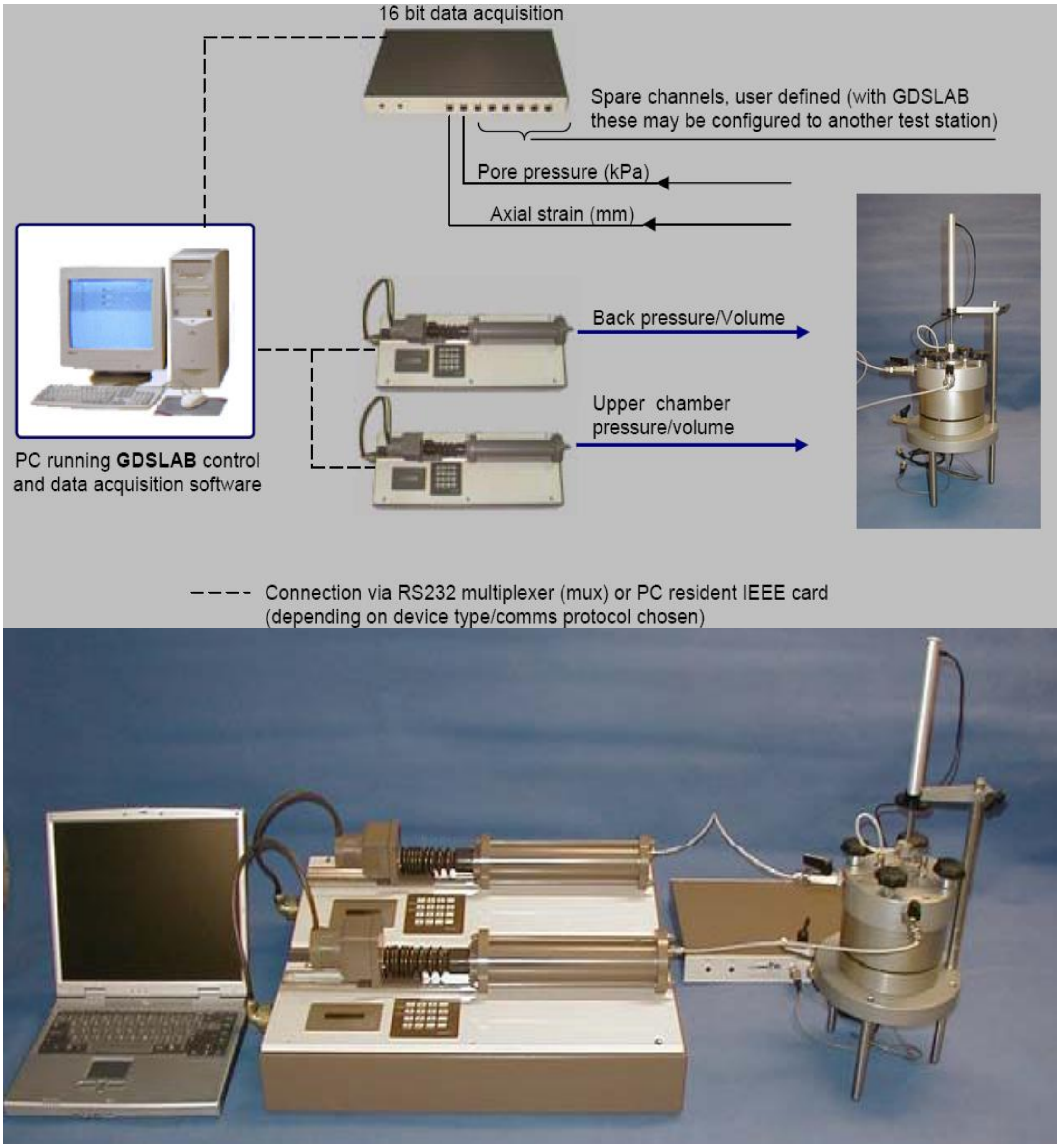


Figure 5.6. Diagram of elements consolidation testing system and standard consolidation testing system

two types of consolidation cell is described later. Two of these pressure controllers link the computer to the test cell as follows: one for axial stress and axial displacement control, one for setting back pressure and measuring volume change. System can run classic tests such as step loading to more advanced tests such as automated testing rate by controlled hydraulic gradient or cyclic loading, all under PC control. Using the flexibility of software, almost any user-defined test may be performed. The hardware used may be chosen to suit your testing and budgetary requirements. The test is performed using a small 50-mm to 75-mm diameter thin specimen (25 mm thick) taken from an undisturbed sample. Selection of representative samples for testing is critical.

The range of Rowe-type consolidation cells is a development of the original cell designed by Professor P.W. Rowe at Manchester University, England. These cells, in which load is applied to the sample hydraulically, offer many advantages and considerably widen the scope of laboratory testing. In addition, the hydraulic loading system gives accurate control of applied loads over a wide range, including high pressures on large diameter samples (figure 5.7).

The use of hydraulic consolidation allows tests to be performed on samples of large diameter. The most important feature of this system is the ability to control drainage and to measure pore water pressure during the consolidation test. Several drainage conditions are possible and back pressure can be applied to the sample.

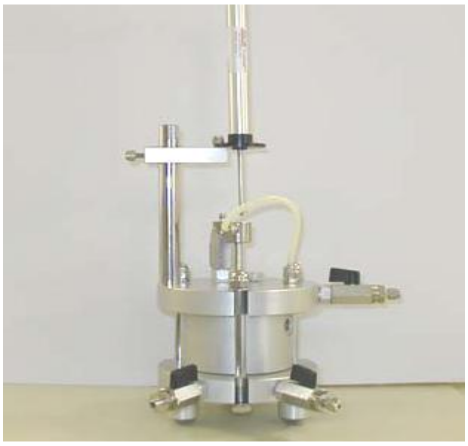
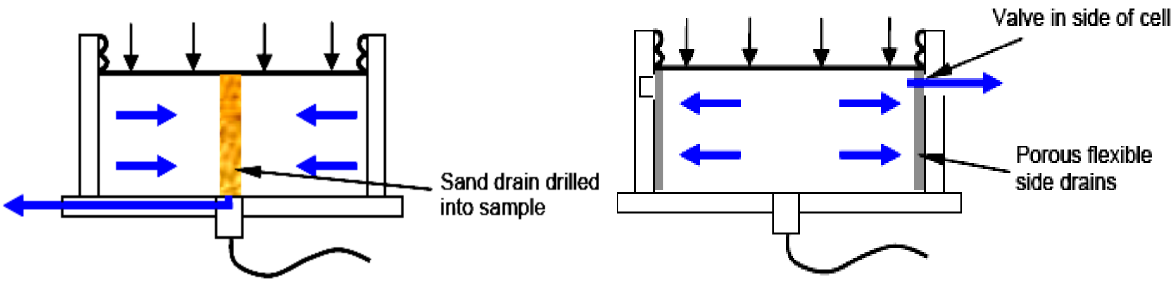


Figure 5.7. Rowe type cell

A unique feature of the classic Rowe cell is the ability to perform consolidation tests with horizontal drainage using side drains or a centre drain. Figures 5.8 a, b



a) *b)*

Figure 5.8. Rowe cell: a) Rowe cell showing horizontal drainage with inward radial flow; b) Rowe cell showing horizontal drainage with outward radial flow

below show schematics of the horizontal drainage.

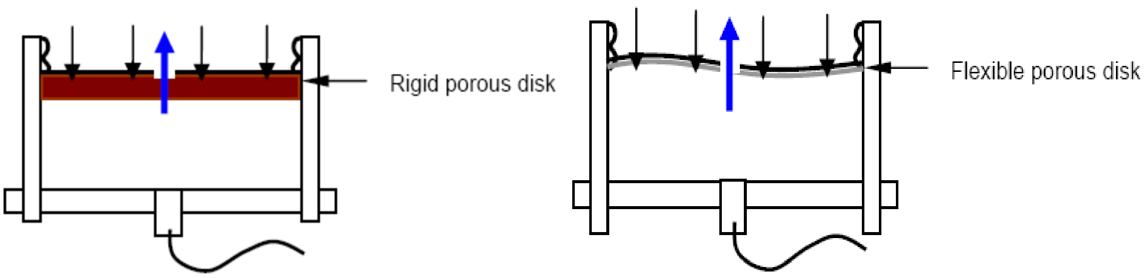
Figure 5.9. Rowe and Barden consolidation cell

Rowe and Barden consolidation cell (figure 5.9) is available in a range of sizes for test specimens of 50- 100mm diameter. Back pressure is applied to the top drain of the cell so that field hydraulic gradients can be modeled. The bottom drain is provided with a tapping for a pressure transducer. The Rowe and Barden cell incorporates the novel Bishop and Skinner floating ring which allows the top bag to move with the specimen vertically.

The main advantage of this method is that it allows measurement of the upper chamber volume change to be used as a calculation of axial strain. This is the fundamental difference between the Rowe and Barden cell and the Rowe cell (figures 5.9).

The hydraulic Rowe cell system is used because of its multiple drainage (up to eight conditions) options as well as the capability of testing large diameter samples through the use of water pressure on a flexible diaphragm. Both the hydraulic Rowe & Barden consolidation cell and the Rowe consolidation cell can be used with either a rigid porous disk for constant strain (see figure 5.10, a) or flexible porous disk for constant stress (figure 5.10, b).

Results of one-dimensional consolidation tests can be presented in a variety of ways. Consolidation data is taken at the usual time increments. The data will either be plotted on a square root of time graph or on a logarithm of time graph. The most common include e-log σ_v' graphs where by the



a) *b)*

Figure 5.10. a) Rowe and Barden cell showing rigid porous disk for constant axial strain; b) Rowe and Barden cell showing the flexible porous disk for constant stress tests

compression indices are determined as the slopes of Δe versus *Δlog* σ_v' for the recompression, virgin compression, and swelling lines, respectively (figure 5.11).

The four most important soil properties furnished by a consolidation test are: the preconsolidation stress, σ_p'. This is the maximum stress that the soil has “felt” in the past, the compression index, C_c, which indicates the compressibility of a *normally-consolidated* soil, the recompression index, C_r, which indicates the compressibility of an *over-consolidated* soil, the coefficient of consolidation, c_v, which indicates the rate of compression under a load increment.

A normally-consolidated soil is defined as a soil which, at the present time, is undergoing the application of a stress that is *larger* than any stress it has undergone in its history. That is, $\sigma_{present} > \sigma_p'$. Conversely, an over-consolidated soil is defined as a soil which has experienced higher stresses in the past, $\sigma_{present} < \sigma_p'$.

The specimen is subjected to incremental loads, which are doubled after each equilibrium phases reached (after t_p corresponding to the end of primary consolidation). Tradition would use a 24-hour increment per load, although this is conservative. Alternatively, specimens can be loaded continuously with monitoring by load cells and porewater pressure transducers.

Generally, it is desirable to perform an unload-reload cycle during the test, with the unloading initiated at a loading increment along the virgin portion of the consolidation curve. The unload-reload cycle provides a more reliable estimate of the recompression characteristics of the soil.

When saturated soil masses are subjected to incremental loads, they undergo various degrees of dimensional change. Initially, the incremental

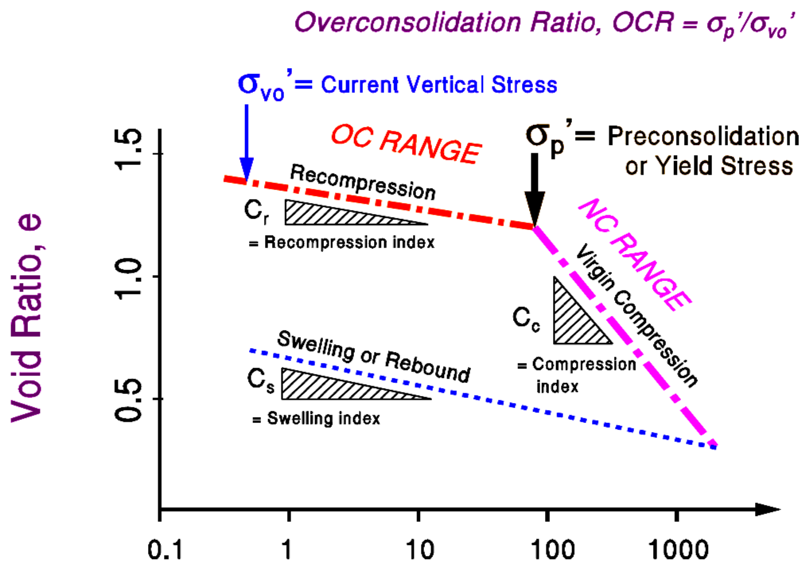


Figure 5.11. Graph of one-dimensional consolidation tests

load is resisted and carried by the liquid phase of the soil, which develops excess porewater pressures (Δu) in the soil voids. Depending on the permeability and the availability of drainage layer (*s*) in contact with the soil, the liquids in the voids begin draining and continue to do so until the Δu is dissipated.

As the hydrostatic pressure decreases, a proportional amount of the incremental load is transferred to the solid portion of the soil. When the excess hydrostatic pressure reaches zero, all of the new load is carried by the soil's solids. This process is called *primary consolidation*. In granular, high-permeability soils, this transfer take place very quickly (since water can drain fast). In clays and low-permeability soils, primary consolidation takes a longer time, which can affect the long-term performance of structures supported by these soils. Time rate is expressed by the coefficient of consolidation (c_v).

In many clays, the primary consolidation is typically followed by secondary compression or long-term creep and represented by the parameter. In thick clay deposits, the magnitude of secondary compression may be substantial. For soils known for their tendency to have significant secondary compression particularly under heavy incremental loads, it may be necessary to predict the long-term effects of secondary compression. In that case, each incremental of the test load is left in place until such time that the time-settlement curve plotted for that load becomes asymptotic to a horizontal line.

Heavy organic clays also require longer loading periods. The time-settlement curves produced by heavy organic soils may not clearly show the end of the primary consolidation. In those cases, it may be necessary to monitor the pore pressures of the soil to determine the end of the primary stage. It should be noted that the magnitude of secondary, long term, compression of highly (20% or more) organic soils may be as large or larger than the primary consolidation.

Secondary compression in these soils takes place as a result of the continuing compression of organic fibers. The substantial dissipation of the excess hydrostatic pressures during the test does not signal the end of significant compression; expulsion of absorbed water with associated compression from the body of the fiber itself may continue for a long period of time.

When performing the test, it is observed, as expected, that the increase of vertical stress caused by a loading from say 10 kPa to 20 kPa leads to a larger deformation than a loading from 20 kPa to 30 kPa. The sample becomes gradually stiffer, when the load increases. Often it is observed that an increase from 20 kPa to 40 kPa leads to the same incremental deformation as an increase from 10 kPa to 20 kPa. And increasing the load from 40 kPa to

80 kPa gives the same additional deformation. Each doubling of the load has about the same effect.

In order to predict the settlements of structures in the field, a method of extrapolating laboratory test results in the settlement analysis is needed. Total settlement is computed from knowledge of the preconsolidation pressure, and the coefficients of compression and recompression.

In engineering practice, reasonably good predictions of a structure's settlements can be made from the results of carefully run laboratory tests. Predicted settlements are larger than actual settlements more often than not. Time rate predictions are often rather poor in practice. Better predictions, naturally, can be made for those cases which have conditions more closely in agreement with the assumptions made in the theory derivation. This would be the case, for example, when the soil involved experiences most of its settlement due to primary consolidation, or when drainage conditions in the field are accurately known.

5.2.6. Swell and collapse tests

Swelling of foundation, embankment, or pavement soils result in serious and costly damage to structures above them. It is therefore important to estimate the swell potential of these soils. *Swell Potential of Clays is* used to estimate this potential of (expansive) soils. The percentage of volumetric swell of a soil depends on the amount of clay, its relative density, the compaction moisture and density, permeability, location of the water table, presence of vegetation and trees, and overburden stress.

Swelling is a characteristic reaction of some clay to saturation. The potential for swell depends on the mineralogical composition. Some soils, particularly those containing montmorillonite clay, tend to increase in volume when their moisture content increases. While montmorillonite (smectite) exhibits a high degree of swell potential, illite has none to moderate swell characteristics, and kaolinite exhibits almost none. These soils can be highly problematic resulting in costly damage to structures being supported by them.

The clay have a tendency to swell in small or more proportion when submerged in water. Free swell or differential free swell also termed as free swell index, is the increase in volume of soil with out any external constraint when subjected to submergence in water.

$$\text{Free swell index } FS\ (\%) = \frac{V_d - V_k}{V_k} \times 100;$$

where V_d = volume of soil specimen read from the graduate cylinder containing distilled water, V_k = volume of soil specimen read from the graduate cylinder containing kerosene.

Apparatus: sieve : 425 micron sieve; oven; balance : weighing accuracy of 0.01 g; graduate glass cylinder : two nos. each of 100 ml capacity.

Procedure

1. Take two specimens of 10 g each of pulverised soil passing 425 micron sieve and oven dried.
2. Pour each soil specimen in 100 ml capacity graduate glass cylinder.
3. Pour distilled water in one and kerosene oil in other cylinder upto 100 ml mark.
4. Remove entrapped air by gentle shaking or stirring with glass rod.
5. Allow attainment of equilibrium state of volume of suspension (for not less than 24 hours).
6. Final volume of soil in each of the cylinder shall be read out.

< 20	low
20 –35	moderate
35 –50	high
>50	very high

Oedometer Swell Test. The one dimensional swell potential test is used to estimate the percent swell and swelling pressures developed by the swelling soils. The swell potential of a soil can be approximated from the consolidometer test methods. It is more convenient to perform this test than Soil Suction test since the Laboratory is equipped with the needed apparatus to conduct this test.

This test can be performed on undisturbed, remolded, or compacted specimens. If the soil structure is not confined (i.e. bridge abutment) such that swelling may occur laterally and vertically, triaxial tests can be used to determine three dimensional swell characteristic.

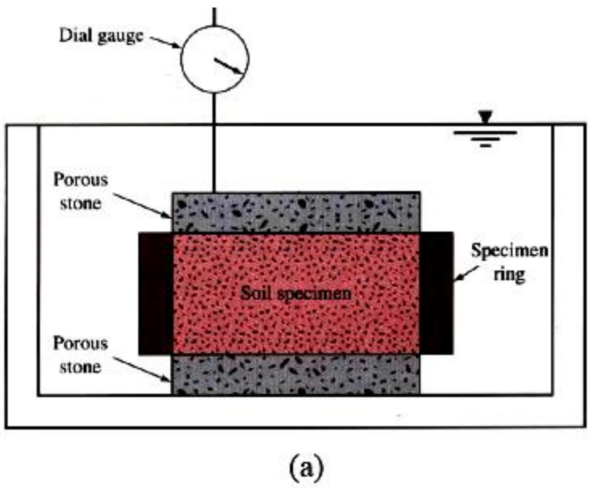
The swell test is typically performed in a consolidation apparatus. The swell potential is determined by observing the swell of a laterally-confined specimen when it is surcharged and flooded. Alternatively, after the specimen is inundated, the height of the specimen is kept constant by adding loads. The vertical stress necessary to maintain zero volume change is the swelling pressure.

A conventional oedometer steel ring of size 2.5 in. (64 mm) in diameter and 1 in. (25 mm) in height was pushed into the cores remaining after separating the specimen required for the testing. The inner face of the consolidation ring was lubricated to minimize the friction during free swell.

Two such specimens were retrieved from cores samples at regular depths. These specimens were then sealed in polyethylene bags and preserved in the 100% relative humidity room prior to testing.

On the day of testing, the free swell specimens were removed from the humidity room and weighed along with the oedometer ring prior to testing. Porous stones were placed on both the top and bottom of the specimen to facilitate movement of water into the soil. The specimens were then transferred into a container and filled with water in order to soak the specimen under a no load condition. The amount of upward vertical movement (heave or swelling) of the specimens was recorded at various time intervals by placing a dial gauge on the top porous stone. Fig. 3.4 depicts a schematic sketch of the one dimensional free swell and the test setup in present study. The recording of readings was continued until no further movement was measured for at least one day. Soaked specimens were then carefully removed from the ring, weighed, oven dried, and weighed after drying in order to calculate the moisture content of the saturated specimen. The swelling of the expansive soil, measured as strain is termed as the ***free swell index (FSI).***

The constant swell pressure tests were conducted following the procedures and are defined as the amount of load that should be applied over the expansive soil to resist any volume change in the vertical direction. The set up for this test in the present study is shown in figure. 5.12. Here, after soaking the specimen, whenever a change in height (Δh) was measured sufficient amount of load was applied to make Δh = 0. The process was continued until Δh = 0 under a constant load for at least one day. The load applied over the specimen at this point was termed as the swell pressure of the soil.



b)

Figure 5.12. Free swell test: a) schematic sketch, b) test setup

The collapse potential of suspected soil is used to estimate this potential of soils. The collapse potential of suspected soils is determined by placing an undisturbed, compacted or remolded specimen in the consolidometer ring and in a loading device at their natural moisture content. A load is applied and the soil is saturated to measure the magnitude of the vertical displacement.

Loess or loess type soils is predominantly composed of silts, and contain 3% to 5% clay. Loess deposits are wind blown formations. Loess type deposits have similar composition and they are formed as a result of the removal of organics by decomposition or the leaching of certain minerals (calcium carbonate). In both cases disturbed samples obtained from these deposits will be classified as silt. When dry or at low moisture content the in situ material gives the appearance of a stable silt deposit. At high moisture contents these soils collapse and undergo sudden changes in volume. Loess, unlike other non-cohesive soils, will stand on almost a vertical slope until saturated. It has a low relative density, a low unit weight and a high void ratio. Structures founded on such soils, upon saturation, may be seriously damaged from the collapse of the foundation soils.

The collapse during wetting occurs due to the destruction of clay binding which provide the original strength of these soils. It is conceivable that remolding and compacting may also destroy the original structure.

5.2.7. Shear strength tests

Engineers must understand the nature of shearing resistance in order to analyze soil stability problems such as bearing capacity, slope stability, lateral earth pressure on earth-retaining structures, pavement. The stresses set up in the soil cause deformation of the soil. Stress failure is caused by slippage of soil particles, which may lead to sliding of one body of soil relative to the surrounding mass. The process is known as shear failure and occurs when shear stresses set up in the soil mass exceed the maximum shear resistance which the soil can offer, i.e. its shear strength.

Soil is not capable of resisting tension; it is capable of resisting compression to some extent. In cases of excessive compression, failure usually occurs in the form of shearing along some internal surface within the soil. The stress is increased until failure. The theory of Mohr-Coulomb Failure Criteria states that a material fails because of a critical combination of normal stress and shear stress, and not from their either maximum normal or shear stress alone.

Soil strength comes from internal friction and cohesion. The shear strength of a soil is its resistance to shearing stresses. Shear failure occurs when the stresses between the particles are such that they slide or roll past each other.

The relationship between normal stress and shear is given as Coulomb's equation:

$$\tau = c + \sigma \tan(\varphi)$$

where τ = shear strength, c = cohesion, σ = effective intergranular normal (to the shear plane) pressure (figure 5.13.), and φ = angle of internal friction. The quantities τ, c, and σ have units of pressure.

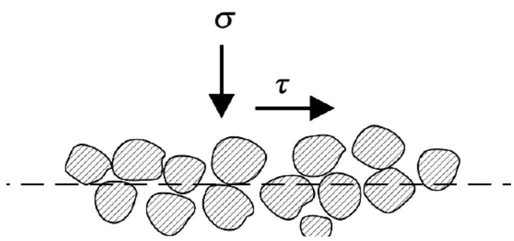


Figure 5.13. Direct of the shear strength of soils

Soil derives its shear strength from follow sources: cohesion between particles (stress independent component), cementation between sand grains, electrostatic attraction between clay particles. The shear strength of a heavy clay soil does not increase with increased load because φ = 0. The shear strength of a very sandy soil does increase with increasing load because φ does not equal 0, but c = 0 for sand.

For cohesionless soils, the intercept is usually negligible, and Coulomb's equation becomes

$$\tau = \sigma_n \tan\varphi.$$

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

Direct Shear Test *(DS)* is a low cost test that provides reasonable strength values for undisturbed or recompacted specimens of cohesionless soils. Test is also the oldest and simplest form of shear test arrangement. A purpose of test is to determine the shear strength of soils along a pre-defined (horizontal) planar surface.

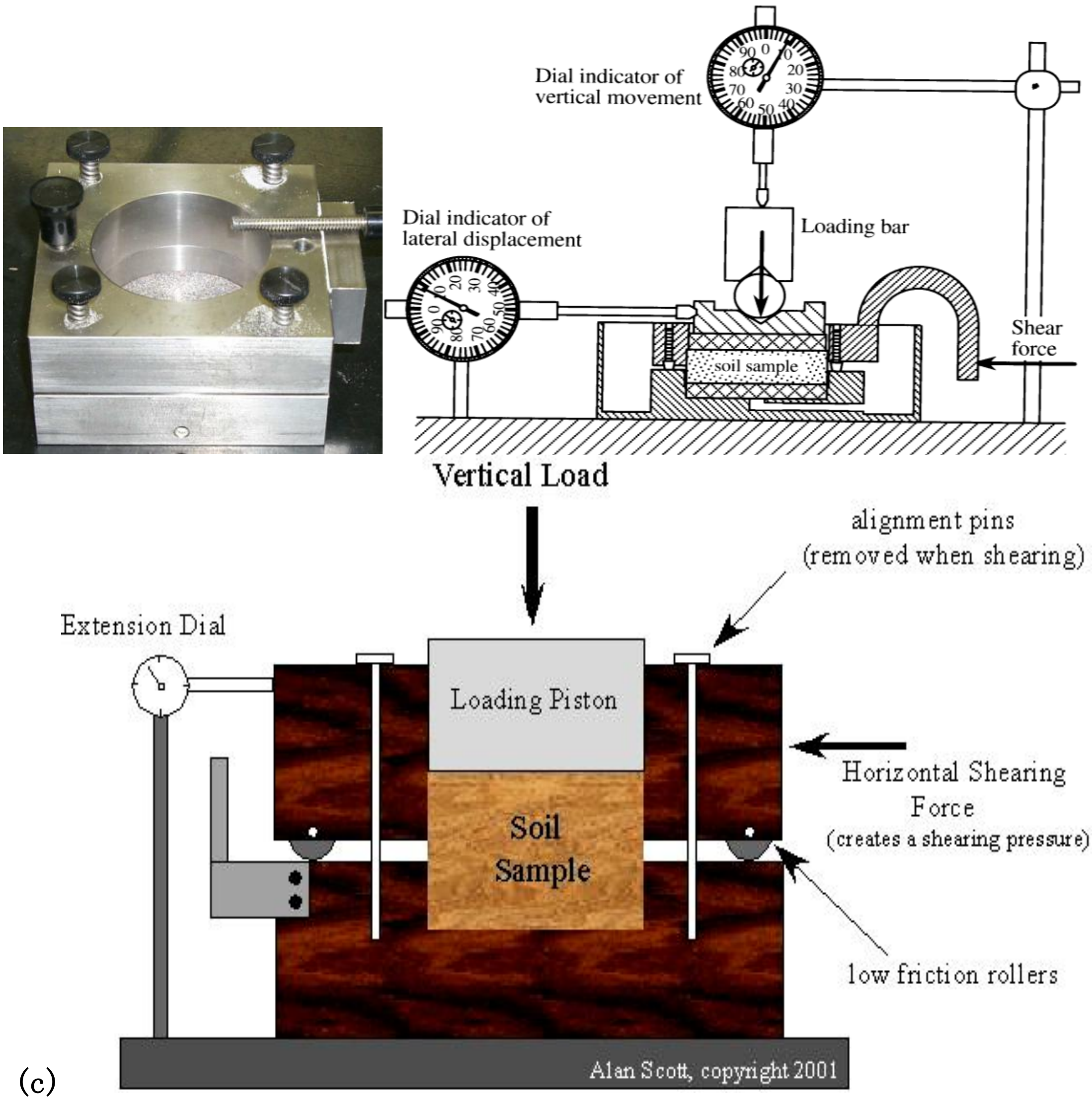
The direct-shear test imposes stress conditions on the soil that force the failure plane to occur at a predetermined location (on the plane that separates the two halves of the box). On this plane there are two forces (or stresses) acting - a normal stress, σ_n, due to an applied vertical load P_v and a shearing stress, τ, due to the applied horizontal load P_h. These stresses are simply computed as:

$$\sigma_n = P_v/A,$$
$$\tau = P_h/A,$$

where A is the nominal area of the specimen (or of the shear box). It is usually not corrected for the change in sample area caused by the lateral displacement of the sample under the shear load P_h. These stresses should satisfy Coulomb's equation.

The direct shear test is performed by placing a specimen into a cylindrical or square-shaped shear metal box (figure. 5.14 a), a, which is split in the horizontal plane. Schematic diagram of direct shear test and DS devices are shown in figure 5.14 b, c.

The shear box is made of two separate halves, an upper and a lower. A specimen of soil will be placed into a *shear box*, and consolidated under an applied normal load. Consider the following situation: a normal stress is applied vertically and held constant; a shear stress is then applied until failure. Vertical force (normal stress) is applied through a metal platen. Shear force is applied by moving one half of the box relative to the other to cause failure in the soil specimen. To conduct the test, one block remains fixed



(c)

Figure 5.14: a) the shear box; b) schematic diagram of direct shear test devices, c) schematic diagram of direct shear test.

while the other block is moved parallel to it in a horizontal direction. The soil fails by shearing along a plane that is forced to be horizontal.

After the application of the normal load, these two halves of the be moved relative to one another, shearing the soil specimen on the plane that is the separation of the two halves. Usually the test is carried out past failure to determine the residual strength which may be used in design. At this point the test is completed and the machine is dismantled. The shear loading is stopped, put into reverse, and reduced until no shear stress is on the sample.

As there are two unknown quantitie c and φ in the above equation, two values, as a minimum, of normal stress and shear stress will be required to obtain a solution.

The Back Pressured Shearbox (figure 5.15) is used for direct shear testing on soil specimens with varying degrees of saturation by controlling the pore water and pore air pressures of the specimen. The GDSBPS is based on a standard direct shear device, modified to allow the measurement and

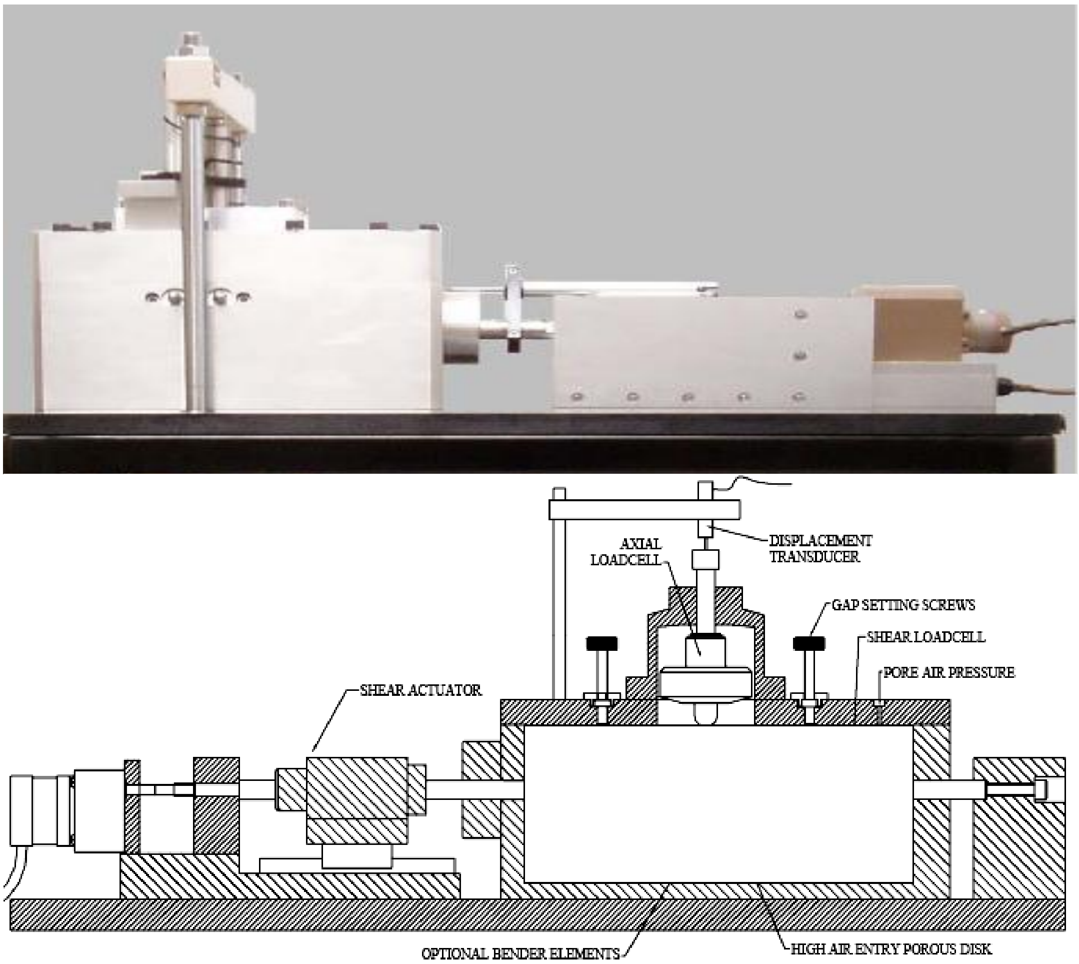


Figure 5.15. The back pressured shearbox and schematic of the back pressured shearbox with full unsaturated options

control of matric suction (the difference between the pore air and water pressures). The complete system runs using data acquisition software. This allows standard direct shear tests to be carried out as well as advanced unsaturated tests under computer control. Control parameters include: shear force and displacement; effective stress control; total stress control; pore air and water pressures; axial (normal) force and displacement (with optional axial actuator). The order of loading shearbox is showed on figure 5.16.

Consolidation of the Test Specimen. The frictional component of the shear strength of a soil depends directly upon the effective stress (which can be thought of as *intergranular stress* for granular soils). In order to determine the shear strength of a soil, therefore, the stage of consolidation of the soil must be known. From the boring logs and the initial moisture-density determinations, the in-situ effective overburden pressure at the sample is computed. To establish the appropriate Mohr failure envelope it is desirable to have at least three points on the failure envelope. *These are usually established at normal pressure of at least the overburden pressure, the overburden pressure plus the footing load, and finally, the overburden pressure plus at least two times the footing load.*

Consolidation data is taken at the usual time increments. The data will either be plotted on a square root of time graph or on a logarithm of time graph. The purpose of recording the time deformation characteristics of the soil sample is to establish the rate of shear testing of the soil. For clean sands and soils with high permeability, it is not necessary to go through this procedure. For all other types of soils it is necessary to establish the time deformation characteristics and estimate t_{50} or t_{90}. If the square root of time graph is used, t_{90} will be established, while if a logarithm base, t_{50} will be established. An estimate of the failure displacement is taken to be 0.2 of an inch, in the horizontal direction. Once we have established the rate of testing we are now ready to perform the test.

As in the triaxial testing, it is possible to run multiphase direct shear tests on one sample. This would be accomplished by stressing a soil until failure, at which time the normal pressure would be immediately increased and the soil would be allowed to consolidate; another direct shear test similar to the first would be run on the sample. After failure was reached in the second phase, the normal force would again be increased to the desired level and the sample would be left to consolidate. The process would be repeated for the third stage and the resulting Mohr envelope would be made from three points of shear stress at three different normal pressures.

The direct shear test can be used to measure the effective stress parameters of any type of soil as long as the pore pressure induced by the normal force and the shear force can dissipate with time. In the case of clean sands, this is no problem as the pore pressure dissipates readily; however, in the case of highly plastic clays, it is merely necessary to have a suitable strain rate so that the pore pressure can dissipate with time.

If an undisturbed sample has been obtained under the ground water table,

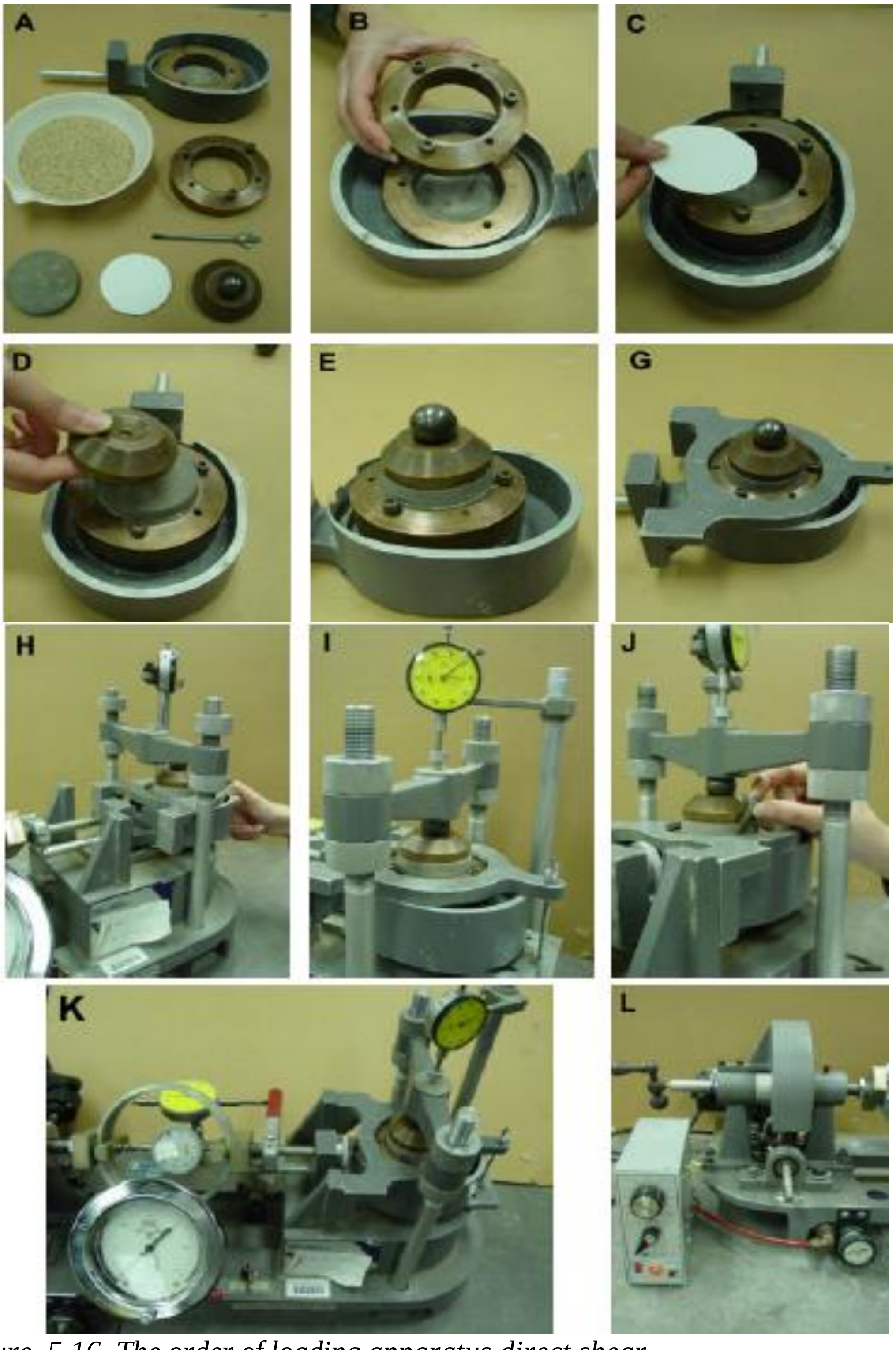


Figure. 5.16. The order of loading apparatus direct shear

it is absolutely necessary to submerge the sample during the test so as to simulate the field conditions. It is necessary to inundate the sample in order to relieve any surface tension that may be present in the sample.

The direct shear test can also be used to determine the undrained shear strength of saturated plastic clays, with qualification. The test must be run rapidly within 5 to 10 minutes to avoid consolidation under normal force during testing, and hence, an increase in shear strength.

Similarly, it is desirable to maintain the levels of induced pore pressure within the sample and not to allow it to dissipate, giving an increase in strength. Although the top and bottom face of the soil sample are in contact with filter paper and porous stones providing drainage access, the low permeability of the plastic clay reduces the rate of pore pressure decrease with time. As a reasonable approximation, it is assumed that the pore pressure across the shear plane is very close to the original pore pressure induced by the normal load. Therefore, as a reasonable approximation, the test measures the undrained shear strength of the soil.

In cases where the soil sample is saturated insitu regardless of grain size, soil type, the sample should be inundated to relieve any surface tension that may develop between the soil particles, inducing tensile forces within the sample. These tensile forces would tend to consolidate the sample three dimensionally.

The DS provides reasonably reliable values for the effective strength parameters, c’ and φ', provided that slow rates of testing are utilized (figure 5.17). The following graph illustrates the results of the equation above.

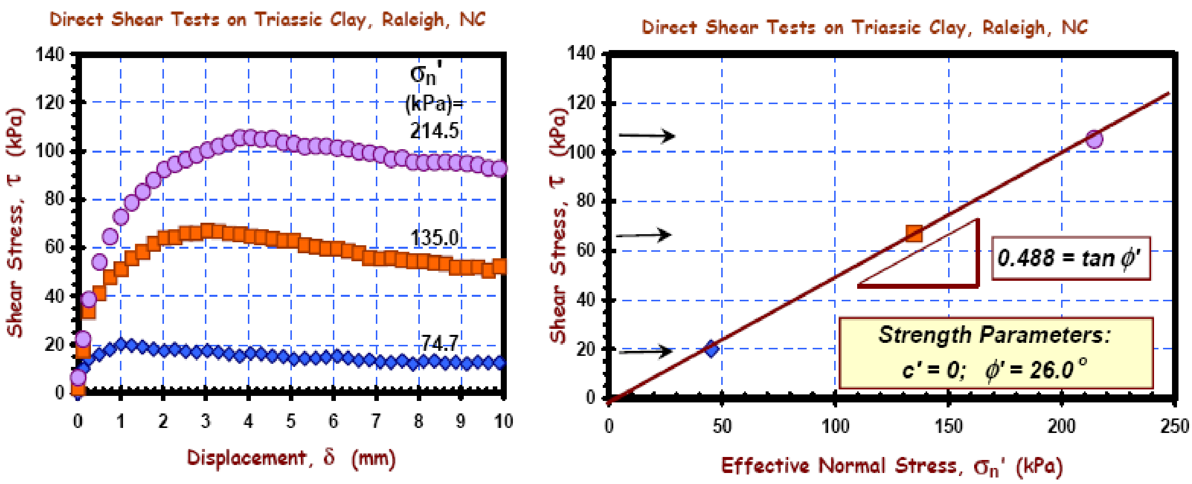


Figure 5.17. Illustrative Results from DS Tests on Clay Involved in Route 1 Slope Stability Study, Raleigh, NC.

Repeated cycles of shearing along the same direction provide an evaluation of the residual strength parameters (c_r' and φ_r').

The Direct Simple Shear DSS test was developed in an attempt to refine the direct shear test by providing shear strain distortion, rather than horizontal displacement. The Direct Simple Shear Apparatus (figure 5.18) is designed for testing soil specimens with conditions of simple shear and plane strain throughout out the specimen. These conditions are representative for a number of field problems (e.g., horizontal portions of failure planes), that cannot be obtained with other standard laboratory testing methods. The apparatus is designed for drained or undrained (constant volume) shear tests on undisturbed or reconstituted specimens of clay, silt or sand. Deformation-controlled monotonic or load-controlled cyclic shear tests can be performed.

Figure 5.18. The direct simple shear apparatus

The original Geonor h-12 DSS apparatus was developed by Landva and Bjerrum in the mid 1960's, and has been used extensively at The Norwegian Geotechnical Institute (NGI) and by others through the world. Earlier test devices used a cylindrical specimen confined in rubber membrane reinforced with a series of evenly spaced rigid rings. Later versions developed by the Norwegian Geotechnical Institute (NGI) used square specimens with hinged end plates that could tilt to maintain fixed specimen length during shearing. As before, piezoelectric bender elements for determining G_{max} of the soil specimen can be supplied as an option.

The NGI version is used by a number of European geotechnical agencies. Some of the studies performed show that this device provides a means of studying plane strain (i.e., embankment loads). Studies at NGI, Swedish

Geotechnical Institute, and Politecnico di Torino have concluded that the DSS provides the most representative mode for the mobilized undrained strength in stability analyses involving embankments, footings, and excavations in soft ground.

The standard DSS apparatus consists of a direct simple shear device with vertical and horizontal motors, load cell and a control unit for: consolidation load, constant load or height during shear for the vertical motor, shear direction rate and direction for the horizontal motor. The soil specimen is mounted inside a rubber membrane of circular cross section and reinforced by a spiral wire winding. This membrane prevents radial deformation of the specimen, but allows vertical deformation during consolidation and shear deformation during simple shear with very little resistance. With this membrane, constant volume conditions during shear are attained by simply keeping the height of the specimen constant. The change in stress (load/area) required to keep the specimen volume constant is equal to the pore pressure measured in an undrained test.

Apparatus: shear box, divided into two halves by a horizontal plane and fitted with locking and spacing screw; box container to hold the shear box; base plate having cross grooves on its top surface; grid plates perforated (2 nos.); porous stones 6 mm thick (2 nos.); proving ring; dial gauge accuracy 0.01 mm – 2 mm; static compaction device, spatula; loading yoke, loading frame, loading pad.

Procedure

1. Preparation of sample

A) undisturbed sample : Specimen is prepared by pushing a cutting ring of size 10 cm dia and 3 cm high , in the undisturbed soil sample. The square specimen of size 6 cm x 6 cm x 2.4 cm is then cut from circular specimen.

B) disturbed sample :

(a) cohesive soil :- the soil may be compacted to required density and moisture content directly into the shear box after fixing the two halves of the shear box together by mean of the fixing screw.

(b) cohesion less soil :- soil may be tamped in the shear box itself with base plate and grid plate or porous stone as required in place at the bottom of the box.

1. Measure the internal dimension of the shear box and average thickness of the grid plates

2. Fix the upper part of the box to the lower part using the locking screw. Attach the base to the lower part .

3. Place the grid plate in the shear box keeping the serration's of the grid at right angle to the direction of shear.

Place a porous stone over the grid plate.

4. Weight the shear box with base plate, grid plate and porous stone.

5. Place soil specimen in the box and weight the box.

6. Place inside the box container and the loading pad on the box. Mount the box container on the loading pad.

7. Bring the upper half of the box in contact with the proving ring. Check the contact by giving slight movement.

8. Fill the container with water and mount the loading yoke on the ball placed on loading pad.

9. Mount one dial gauge on the loading yoke to record the vertical displacement and another dial gauge on the container to record the horizontal displacement.

10. Place the weight on loading yoke to apply a normal stress.

11. Allow the sample to consolidate under the applied normal stress. Note reading of vertical displacement dial gauge.

12. Remove the locking screws. Using the spacing screws, raise the upper part slightly above the lower part such as that gap is slightly larger than the maximum particle size. Remove the spacing screws.

13. Adjust all dial gauges to read zero. The proving ring also read zero.

14. Apply the horizontal shear load at constant rate of strain.

15. Record reading of the proving ring, the vertical displacement dial gauge.

16. Continue the test, till the specimen fails or till a strain of 20 % is reached.

17. At the end of the test, remove the specimen from the box.

18. Repeat the test on identical specimens under the normal stress.

Direct shear test on annular specimens is highly recommended for measuring residual shear strength on annular specimens. The sample is subjected to a normal stress on level faces and to a single torque parallel to same. This apparatus differs greatly from the traditional units in which shearing stress is measured by means of an arm acting on two proving rings or load cells (figure 5.19). Here a strain gauge type torque meter, directly keyed onto the rotational axis, is used. This solution, besides allowing greater precision, also enables the specimen to be positioned more easily. Stress is displayed in digital form on a special readout unit. The vertical loading device is also new: the traditional weight lever has been replaced by an axial jack with Bellofram rolling membranes operated by compressed air. A precision valve enables load to be selected and maintained constant, whilst a strain gauge type transducer and relevant digital readout unit continues displaying the applied load measurement.

A third strain gauge type transducer and respective digital readout unit also provide axial strain of specimen during consolidation and shearing.

Angle of rotation is measured, again in digital form, at stepper motor with precision Harmonic drive gear box.

Of great interest is the possibility of using with a PC interface in order to acquire test data automatically.

Figure 5.19. The Tecnotest's apparatus direct shear test on annular specimens

*The direct shear te*st is particularly applicable to those foundation design problems where it is necessary to determine the angle of friction between the soil and the material of which the foundation is constructed, e.g., the friction between the base of a concrete footing and underneath soil. In such cases, the lower box is filled with soil and the upper box contains the foundation material.

In addition to the direct shear test, other tests exist for the determination of shear strength of soils. These tests include: triaxial test and simple shear tests. The direct shear test was formerly quite popular, but with the development of the triaxial test which is much more flexible, it has become less popular in recent years.

The advantages of direct shear test are:

1. Cheap, fast and simple - especially for sands.

2. Failure occurs along a single surface, which approximates observed slips or shear type failures in natural soils.

Disadvantages of the test include:

1. Difficult or impossible to control drainage, especially for fine-grained soils.

2. Failure plane is forced-may not be the weakest or most critical plane in the field

3. Non-uniform stress conditions exist in the specimen.

4. The principal stresses rotate during shear, and the rotation cannot be controlled.

Principal stresses are not directly measured.

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

The distribution of normal stresses and shearing stresses over the sliding surface is not uniform; typically the edges experience more stress than the center. Due to this, there is progressive failure of the specimen, i.e., the entire strength of the soil is not mobilized simultaneously.

In spite of the above shortcomings, the direct shear test is commonly used as it is simple and easy to perform

5.2.8. Miniature vane tests

Miniature vane testers are used to determine the *undrained shear strength (s_u) and sensitivity (S_t)* of saturated clays and silts. The test is performed by inserting a four-bladed vane into the soil and applying rotation to shear a cylindrical surface (figure 5.20). The undrained shear strength is computed from the measured torque.

The test assumes that the stresses applied are limited to the cylindrical surface represented by the diameter and the height of the vane. This is hardly the case in reality. Depending on the strength and stiffness, the soils in an area radiating outward from the surface of the idealized cylindrical zone are also disturbed by the shearing action of the vane. A portion of the torque therefore is used to mobilize this zone. Thus the assumption that the only sheared zone is the one defined by the outline of the vane blades introduces varying degrees of error.

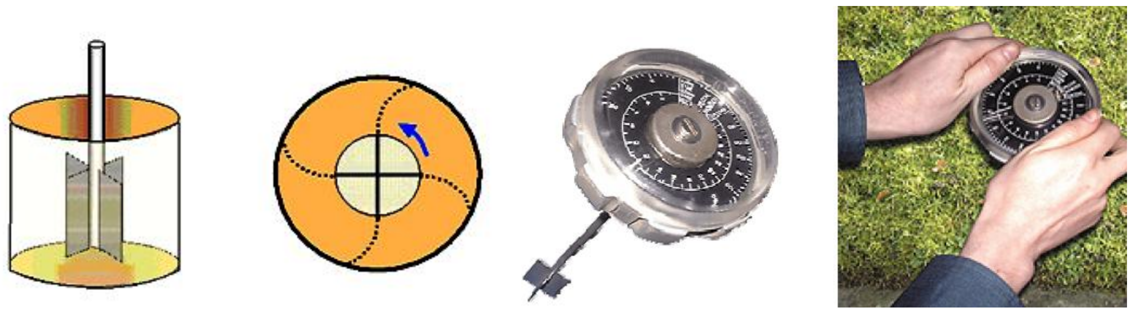


Figure 5.20. Hand shear vane tester

Pocket shearmeter is completed with sensitive vane, standard vane and high-capacity vane (figure 5.21). Range 0 to 1 x 0.05 kgf/cm^2. The shearmeter can be used on tube samples, on the sides of pits, cuttings etc. It is an invaluable tool for initial site investigation work.

Inspection vane is an essential tool for civil engineers involved in site investigation work. The unit is supplied in kit form with a carrying case incorporating the measuring head, extension rods, vanes, etc (figure 5.22).

The measuring head comprises a T-handle that is springloaded against the extension rod adaptor.

The laboratory vane apparatus is based on a design by the Transport Research Laboratory, England and is available in hand or motorized versions. Stress is applied through the 12.7 x 12.7 mm vane by means of any one of four calibrated springs. The motorized version produces a shearing rate of 10. per minute (figure 5.23). The miniature vane is similar to the field vane shear device, except that it is smaller (blade diameter 12.7 mm, blade height 25.4 mm).

The analysis of the tests assumes that strength of the soil being tested is isotropic, which is not true for all deposits. The test, however, can be a useful tool for measuring anisotropy and remolded strength of saturated clays and silts. The ratio of peak to remolded undrained strengths is the sensitivity (*St*). The laboratory vane shear test should be used as an index test.

Figure 5.21. Pocket shearmeter

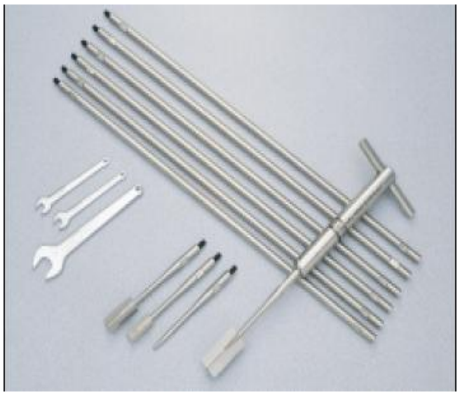


Figure 5.22. Field inspection vane tester

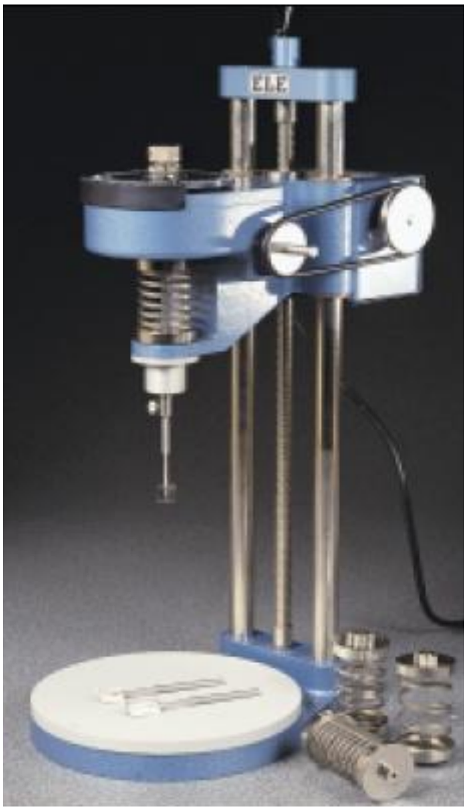


Figure 5.23. Laboratory vane apparatus and accessories

5.2.9. Triaxial strength tests

Triaxial tests provide controllable stresses and reliable measurements, which are necessary for critical analyses. Purpose triaxial strength test is to determine strength characteristics of soils including detailed information on the effects of lateral confinement, porewater pressure, drainage and consolidation.

The triaxial compression test is used to measure the shear strength of a soil under controlled drainage conditions.

The test is called "triaxial" because the three principal stresses are assumed to be known and are controlled. During shear, the major principal stress, σ_1 is equal to the applied axial stress ($Ds = P/A$) plus the chamber (confining) pressure, σ_3. The applied axial stress, $\sigma_1 - \sigma_3$ is termed the "principal stress difference" or sometimes the "deviator stress“. The intermediate principal stress, σ_2 and the minor principal stress, σ_3 are identical in the test, and are equal to the confining or chamber pressure.

Triaxial tests provide a reliable means to determine the friction angle of natural clays & silts, as well as reconstituted sands. The stiffness (modulus) at intermediate to large strains can also be evaluated.

Developed by Casagrande in an attempt to overcome some of the serious disadvantages of the direct shear test.

Advantages of triaxial tests over direct shear test
– more versatile;
– drainage can be well controlled;
– there is no rotation of the principal stresses like the direct shear test;
– also the failure plane can occur anywhere.

Triaxial compression test is the same as the unconfined compression test but with the addition of lateral pressure (figure 5.24, a). In this test, a cylindrical sample is subjected to an axial load until failure occurs while also being subjected to confining pressures approximating a range of in situ stress levels.

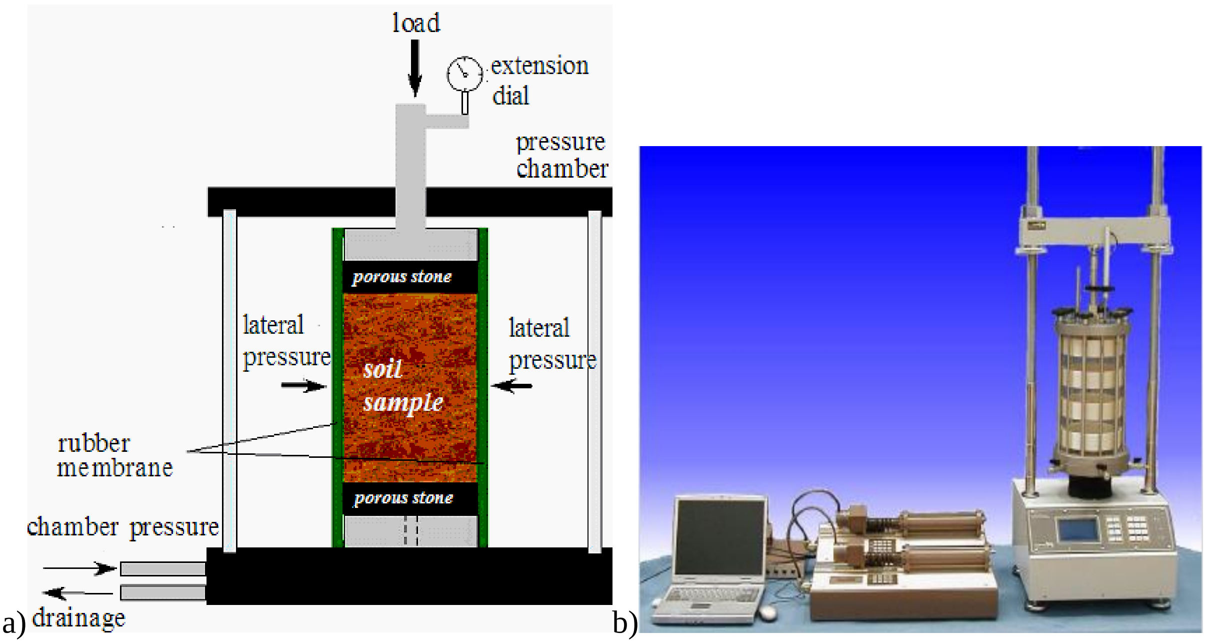


Figure 5.24. a) Schematic diagram of triaxial cell, b) the triaxial testing system

The sample is encased by a thin rubber membrane and placed inside a plastic cylindrical chamber that is usually filled with water or glycerine. The sample is subjected to a total confining pressure (σ_3) by compression of the fluid in the chamber acting on the membrane. A backpressure (u_o) is applied directly to the specimen through a port in the bottom pedestal. Thus, the sample is initially consolidated with an effective confining stress: $\sigma_3' = (\sigma_3 - u_o)$. (Note that air should not be used as a compression medium). To cause shear failure in the sample, axial stress is applied through a vertical loading ram (called deviator stress). Axial stress may be applied at a constant rate (strain controlled) or by means of a hydraulic press or dead weight increments or hydraulic pressure (stress controlled) until the sample fails.
The axial load applied by the loading ram corresponding to a given axial deformation is measured by a proving ring or electronic load cell attached to the ram. Connections to measure drainage into or out of the sample, or for porewater pressure are also provided. Deflections are monitored by either dial indicators. The triaxial testing system is shown in figure 5.24, b and section of a typical triaxial cell in figure 5.25.

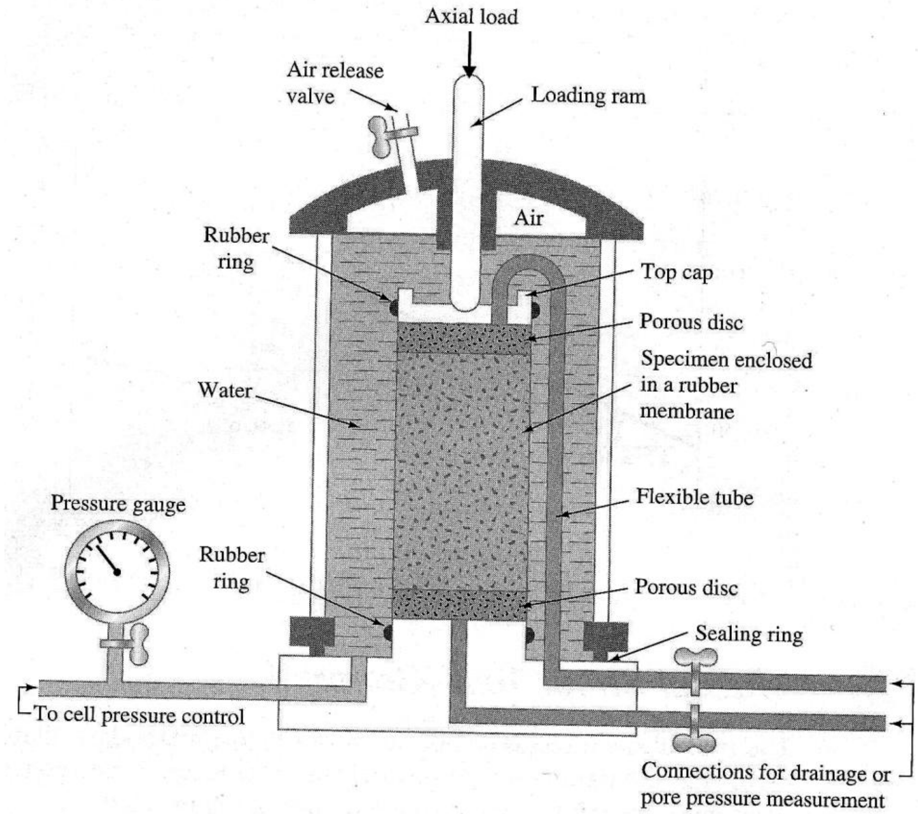


Figure 5.25. Section of a typical triaxial cell

Even in clays, long-term shear strength is estimated assuming drained conditions. In general, there are five types of triaxial tests:

1. consolidated undrained (CU test);

2. undrained unconsolidated (UU test);
3. consolidated drained (CD test);
4. consolidated undrained with pore pressure measurement (CU);
5. cyclic triaxial loading tests (CTX).

Consolidated-Undrained (CU) differs from the *UU* test by applying confining stresses to reconsolidate the specimen before testing. If the goal is to determine short-term undrained shear strengths, then no pore water pressure measurements are made, which simplifies test procedures. However, the advantage of using this test type is to determine effective stress parameters for both short-term and long-term design cases, which requires the measurement of pore water pressures and more effort in test set-up. The Geotechnical Engineer should determine the type of data needed for the design analyses and the locations and numbers of tests required. The *CU* test specimen is allowed to consolidate under the confining pressure prior to shearing, but no drainage is permitted during shearing. This test takes longer and is more expensive than a UU test because the sample must be backpressure saturated (to accurately measure pore water pressures and specimen response), which may take a few days. A minimum of three tests at different confining pressures is required to develop the Mohr envelope over the applicable stress range. Often a 3-stage test can be performed on one specimen, which saves time and reduces cost, as well as eliminates inconsistency between specimens.

A 3-stage test might not be possible where soft/compressible soil specimens experience large deformations in the first or second shearing stages.

Different types of soils show different characteristics on being subjected to loading. The test helps to determine load supporting capacity of a particular soil under fully saturated condition. This test is required for design of foundation for structure and analysis of slope stability. This test can simulates long term as well as short term shear strength for cohesive soils if pore water pressure is measured during the shearing phase.

Various Loading devices may be used to apply axial load to the specimen. These devices can be classified as either apparatus in which axial loads are measured outside the triaxial chamber or apparatus in which axial loads are measured inside the triaxial chamber by using a proving ring or frame, an electrical transducer, or a pressure capsule. Any- equipment used should be calibrated to permit determination of loads actually applied to the soil specimen.

Loading devices can be further grouped under controlled-strain or controlled-stress types. In controlled-strain tests, the specimen is strained axially at a predetermined rate; in controlled-stress tests, predetermined increments of load are applied to the specimen at fixed intervals of time. Controlled-strain loading devices, such as commercial testing machines, are preferred for short-duration tests using piston-type test apparatus. If available, an automatic stress-strain recorder may be used to measure and record applied axial loads and strains.

Procedure

1. Remove wax sealing from field sample tube.
2. Place sample cutter tube (38 mm inner dia) on field sample tube.
3. Insert sample cutter tube in the soil with the help of hydraulic jack.
4. Take out the sample cutter tube from field sample tube by pushing soil with hydraulic jack.
5. Transfer soil sample from sample cutter tube to split mould of proper length (76 mm).
6. Take out soil specimen from split mould.
7. Clean base of triaxial cell.
8. Put porous stone over bottom pedestal and a filter paper of 38 mm dia over this porous stone.
9. Place soil specimen over filter paper and put another filter paper then porous stone on the top of the soil specimen.
10. Place about 8 filter paper strips vertically around soil specimen extending from top porous stone to bottom porous stone to facilitate uniform and quick saturation.
11. Put rubber membrane around the soil specimen with the help of stretcher.
12. Place ring around top and bottom pedestal in the grooves.
13. Place the triaxial cell and tight the nut to the base plate.
14. Saturate the soil sample from 24 to 48 hours, by opening drainage valve, which is connected with burette filled with water. Water level in burette is kept little more than the top of specimen.
15. After saturation triaxial cell is filled with water and all around cell pressure (σ_3) is applied by mercury controlled device. The pore water pressure is measured the sample is saturated (not less than 90% of σ_3).
16. Four soil specimen of a sample are tested at 0.5, 1.0, 1.5, and 2.0 kg / cm2 of lateral pressure (σ_3).
17. For consolidated un-drained test (UC), the sample is to be placed for consolidation. The drainage reading during consolidation in the burette is to be recorded in time interval of 1, 4, 9, 16, 25, 36.....minutes up to 24 hrs.
18. On account of consolidation the length and diameter of specimen changed.
19. Changed length, cross sectional area and rate of strain on consolidated specimen has to be calculated.

20.Apply calculated rate of strain on consolidated specimen and note down the deformation and corresponding load on specimen un till the failure of specimen.

21.Four specimen has been tested at four confining pressure (0.5, 1, 1.5 and 2 kg/cm^2) as explained above.

22.Now from above reading plot Mohr's circle and get the shear parameter.

In a UU test, the samples are not allowed to drain or consolidate prior to or during the testing. The results of undrained tests depend on the degree of saturation (S) of the specimens. Where *S*=100%, the test results will provide a value of undrained shear strength, however, the test is affected by sample disturbance and rate effect. This test is not applicable for granular (S=100%) soils. The test is performed with the drain valve closed for all phases of the test. (Water is not allowed to drain).

Consolidated-Drained (CD) is not as common as the CU test because the test may require a potentially longer time to run and is primarily applicable for relatively cohesionless soils. This test is similar to the CU test except that drainage is allowed during shearing and the rate of shearing is very slow to prevent the buildup of excess pore water pressure. This test may take days to perform, making it expensive. These tests are not performed often because similar results can be obtained with the quicker CU tests. As with the CU test, a minimum of three tests (or stages) is required. Effective stress strength parameters are obtained without the need for pore water pressure measurement. Test results are used for calculating long-term stability of embankments where relatively cohesionless materials exist along potential failure surfaces.

You will conduct a CD test on sand. Soil specimens will be loaded to failure under 3 different confining pressures; 15, 30 and 45 psi. Failure will be defined as the peak or 3 maximum value of principal stress difference reached.

The CU test with porewater pressure measurements is the most useful as it provides a direct measure of the undrained shear strength (s_u), for triaxial compressive mode, as well as the important effective stress parameters (c' and φ').

Cyclic triaxial tests are used for projects with repeated and/or cyclic loading, resilient modulus determinations, and/or liquefaction analysis of soils.

In each of these tests, the specimen is initially consolidated to the effective vertical overburden stress prior to shear. If additional specimens from the same tube are tested, these may be tested at confining stress levels of 0.5 (σ_{vo}') to 1.5 (σ_{vo}'), in order to provide a range of operating values.

The results can be presented in terms of Mohr Circles of stress to obtain the strength parameters (figure 5.26).

If more than two or three tests are conducted, the results are more conveniently plotted on *q-p* space, where $q = 1/2(\sigma_1 + \sigma_3)$ and $p' = 1/2 (\sigma_1' + \sigma_3')$, as illustrated in figure 5.27.

The range of precision made triaxial cells has been designed to meet the requirements of the modern soils laboratory. The cells have been treated to minimize corrosion. Particular attention has been paid to the quality of finish between the piston and the head. Final assembly includes the fitting of an O-ring seal and the use of special lubricant to reduce friction to a minimum and eliminate water leakage. The piston load capacity is designed to accept high horizontal forces which may be present during the final stages of a test.

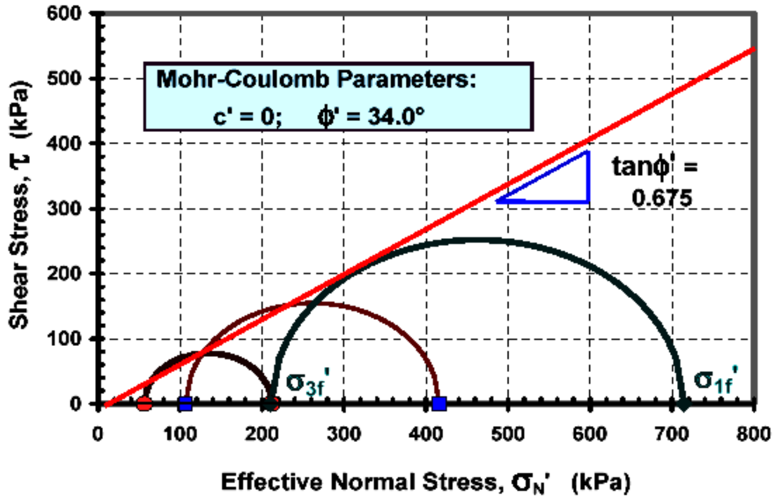


Figure 5.26. Effective stress Mohr circles for consolidated undrained triaxial tests

Drained conditions occur when rate at which loads are applied are slow compared to rates at which soil material can drain *(k - dependent).* Sands drain fast; therefore under most loading conditions drained conditions exist in sands. Exceptions: pile driving, earthquake loading in fine sands.

In clays, drainage does not occur quickly; therefore excess pore water pressure does not dissipate quickly. Therefore, in clays the short-term shear strength may correspond to undrained conditions.

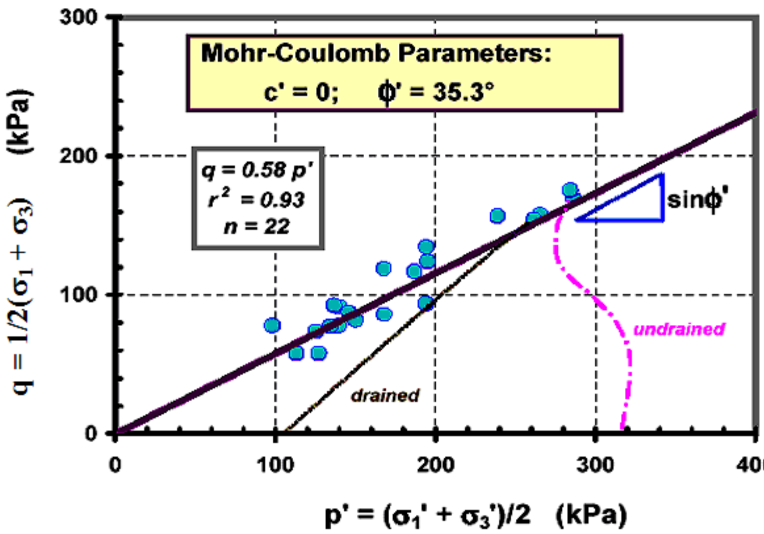


Figure 5.27. Effective q-p. strength envelopes for consolidated undrained triaxial

5.2.10. Unconfined compression test

The unconfined compression test (UC) is an important method of determining the undrained shear strength of cohesive and semi-cohesive soils (c_u). The unconfined compression test determines approximate undrained shear strengths due to the slightly relaxed in situ pressures of the sample. The lack of confinement introduces a relatively large error range. This test is a fast and economical means of approximating the shear strength at shallow depths, but the reliability decreases with increasing depth. Triaxial testing is recommended when more reliable values of cohesive shear strength are desired.

Every building or structure which is founded in or on the earth imposes loads on the soil that supports the foundations. The stresses set up in the soil cause deformation of the soil. Stress failure is caused by slippage of soil particles, which may lead to sliding of one body of soil relative to the surrounding mass. The process is known as shear failure and occurs when shear stresses set up in the soil mass exceed the maximum shear resistance which the soil can offer, i.e. its shear strength.

The test is performed on a cylindrical sample without any confining pressure, subjected to an axial load until failure occurs. A simple hand- or motor-operated compressive load test frame is commonly used, although use of triaxial apparatus can achieve the same results. This test is typically performed on cohesive soils. Total stress parameters are obtained, which are only applicable for the sampled depths. The cohesion is taken as one-half the unconfined compressive strength, q_u.

An axial load is placed onto a sample, the load is increased until (a) the soil fails, or (b) 15% strain has occurred. This load is known as the unconfined compressive strength. There is no lateral support on the soil sample for this measurement figure 5.28, a. An unconfined compression test machine in which strain-controlled tests can be performed is shown in figure 5.28, b. This lightweight, hand-operated machine is self-contained and requires no pressure or electrical connections. The machine is supplied complete and ready to operate with a load ring, strain dial, platens and calibration charts. The machine essentially consists of a top and a bottom loading plate. The bottom of a proving ring is attached to the top loading plate. The top of the proving ring is attached to a cross-bar which, in turn, is fixed.

The soil specimens are tested without any confinement or lateral support (σ_3=0). Axial load is rapidly applied to the sample to cause failure. At failure the total minor principal stress is zero (σ_3= 0) and the total major principal stress is σ_1 (figure 5.28. a).

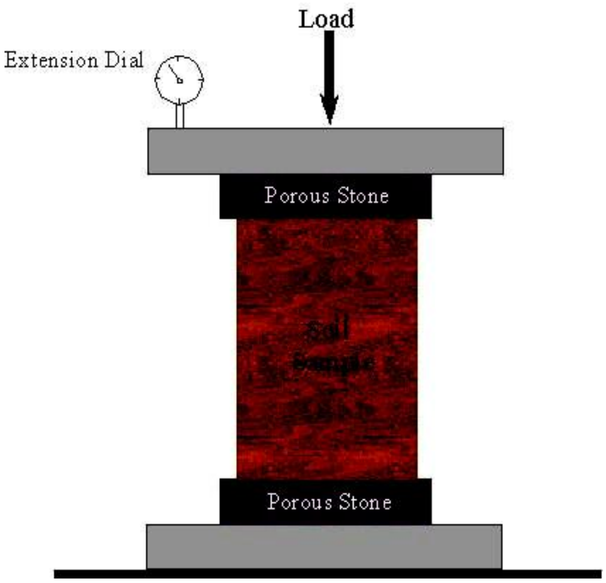


Figure 5.28. a) Schematic diagram of unconfined compression test, b) unconfined compression tester

The maximum measured force over the sample area is q_u and referred to as the unconfined compression strength (figure 5.29).

The undrained shear strength of the soil is equal to one half of the unconfined compressive strength, $c_u = q_u/2$.

Apparatus: unconfined compression apparatus comprising hydraulic loading device with proving ring and deformation dial gauge; vernier caliper; sample extractor; coning tool; sampling tube; spatula; split mould.

Procedure

1. Coat the split mould lightly with a thin layer of grease.

2. Push the sample out of the sampling tube into the split mould using the sample extractor with

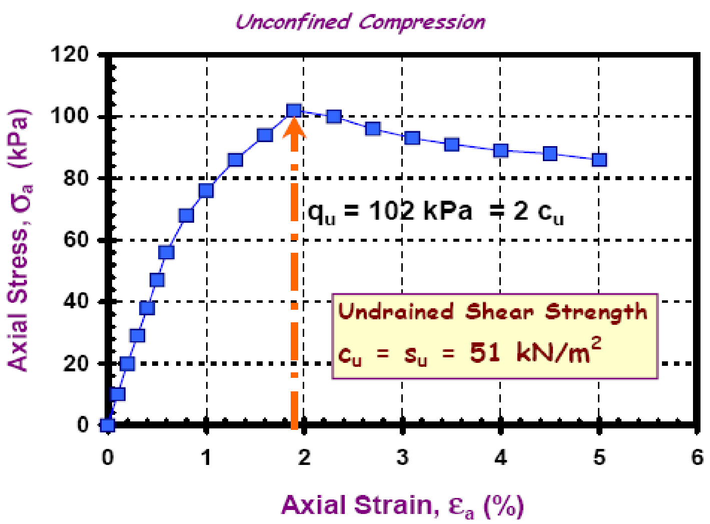


Figure 5.29. Measured stress-strain for unconfined compressive test

negligible disturbance of the specimen.

3. Remove the specimen from the split mould by splitting the mould into two part and use the coning tool to form cones on two ends of the specimen.

4. Measure the length and diameter of the specimen and weigh it.

5. Place the specimen on the bottom plate of the compression machine. Adjust the upper plate to make contact with the specimen.

6. Adjust dial gauge and proving ring gauge to zero.

7. The rate strain of 1.5 mm / minute is applied to soil specimen.

8. Continue the test until failure surfaces have clearly developed or until an axial strain of 20 % is reached.

9. Take the sample from the failure zone of the specimen for water content determination.

The determination of unconfined compressive strength of undisturbed, remolded or compacted soils is limited to cohesive or naturally or artificially cemented soils. Application of this test to non-cohesive soils may result in underestimation of the shear strength. The test is inexpensive and requires a relatively short period of time to complete. However, due to the absence of lateral pressures and lack of control over pore pressures, it has major inaccuracies.

The stress-strain curves and failure modes observed during testing provide an index value of the soil properties in addition to strength. For example, an ill-defined failure or yielding of the sample signifies relatively soft, fat clay, while a sudden brittle failure indicates that of a desiccated clay or cemented material. The stress-strain curves developed from these tests should be used with caution when determining soil modulus for input to numerical analyses, such as finite element analysis, which are very sensitive to minor variations of the modulus.

Figure 5.30 shows a soil specimen after failure. Soils with inclined *fissures, sand & silt lenses and slickensides have a tendency to fail prematurely along these weaker planes in unconfined compression tests. It is essential that such failure modes be reported to the engineer, who may request further more sophisticated testing such as triaxial tests to obtain more realistic determination of the in situ strength.*

Figure 5.30. A soil specimen after failure

5.2.11. California Bearing Ratio test

Purpose of California Bearing Ratio test is used to determine the bearing capacity of a compacted soil under controlled moisture and density conditions.

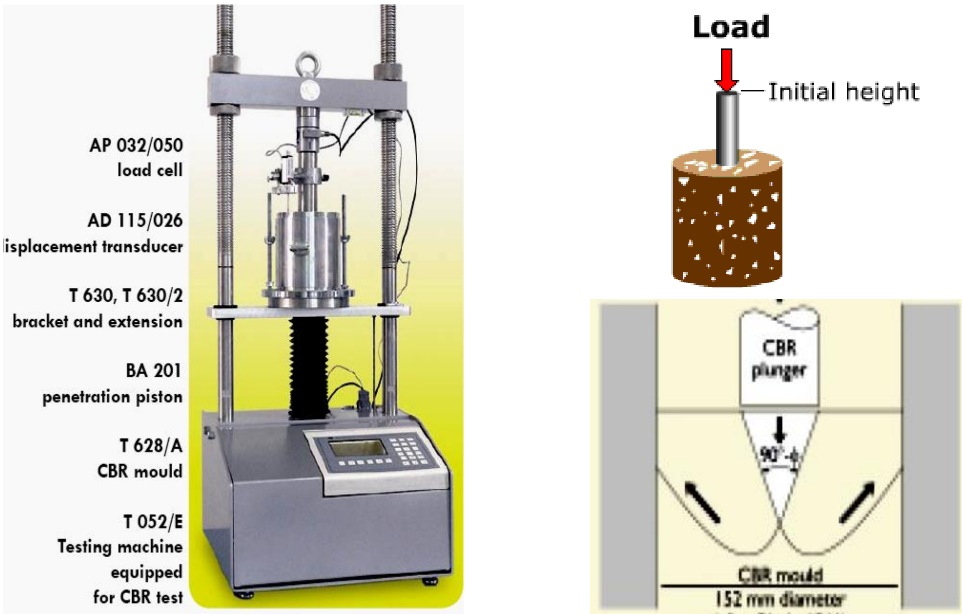
The California Bearing Ratio test, or CBR test as it is usually termed, is an empirical test first developed in California, USA (1930), for estimating the bearing value of highway sub-bases and sub grades. The test follows a standardized procedure and there are numerous ways of preparing samples and in this respect American practice differs in detail from British practice. This test can be performed in the laboratory on prepared samples or in-situ on location. It is important to appreciate that this test, being of an empirical nature, is valid only for the application for which it was developed, i.e. the design of highway base thicknesses.

The test results are expressed in terms of a bearing ratio which is commonly known as the California Bearing Ratio (CBR):

$$CBR = 100\left(\frac{x}{y}\right)$$

where x - material resistance or the unit load on the piston (pressure) for 0.1 or 0.2 inches of penetration, y - standard unit load (pressure) for well graded crushed stone (for 0.1 inches of penetration = 1000 psi, for 0.2 inches of penetration = 1500 psi.

The CBR is obtained as the ratio of the unit load required to cause a certain depth of penetration of a piston into a compacted specimen of soil at some water content and density (figure 5.31, a), to the *standard unit load* required to obtain the same depth of penetration on a standard sample of crushed stone (usually limestone).

a)


b)

Figure 5.31. The CBR test: a) apparatus series CBR-Test; b) mechanism of failure of soil specimen

Mechanism of failure is shown on figure 5.31, b. Typically soaked conditions should be used to simulate anticipated long-term conditions in the field. The CBR test is run on three identically compacted samples. Each series of the CBR test is run for a given relative density and moisture content. The geotechnical engineer must specify the conditions (dry, at optimum moisture, after soaking, 95% relative density, etc.) under which each test should be performed.

CBR is a practical bearing capacity test, yet provides only discrete point test data for evaluation. Most CBR testing is laboratory-based, thus the results will be highly dependent on the representativeness of the samples tested. The test results are used for highway, airport, parking lot and other pavement designs using empirical local or agency-specific methods. More often than not, pavement failures are due to poor drainage, overloaded truck traffic, increased overall road traffic, and wear.

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Учебное издание

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КРАМАРЕНКО Виолетта Валентиновна

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«Геология и разведка полезных ископаемых».

Научный редактор
к.г.-м.н., доцент Т.Я. Емельянова

Отпечатано в Издательстве ТПУ в полном соответствии с качеством предоставленного оригинал-макета

Подписано к печати 05.11.2010. Формат 60x84/16. Бумага «Снегурочка».
Печать XEROX. Усл.печ.л. 9,01. Уч.-изд.л. 8,16.
Заказ . Тираж 30 экз.

Национальный исследовательский Томский политехнический университет
Система менеджмента качества
Томского политехнического университета сертифицирована
NATIONAL QUALITY ASSURANCE по стандарту ISO 9001:2008

ИЗДАТЕЛЬСТВО ТПУ. 634050, г. Томск, пр. Ленина, 30
Тел./факс: 8(3822)56-35-35, www.tpu.ru