Module7: Laboratory testing for site characterization

Topics:

- Introduction
- Purpose of soil testing
- Available tests
    - Soil classification tests
        - Particle size distribution tests (Sieve analysis, Hydrometer analysis)
        - Plasticity tests (Liquid limit, Cone penetrometer test, Plastic limit)
        - Compaction tests (Proctor compaction test)
        - Particle density(Specific gravity) determination
    - Tests for Geotechnical parameters
        - Strength tests (CBR test, Franklin point load test, Lab vane test, Direct shear test, Triaxial test)
        - Stiffness tests
        - Consolidation tests (Casagrandeoedometer test, Triaxial dissipation test, Hydraulic oedometer test)
        - Seepage and permeability tests
    - Chemical tests

Keywords: *Soil classification test, Strength tests, Chemical tests*

7.1 Introduction:

Laboratory testing is part of the physical survey. As an integral part of site investigation, the need for laboratory tests will often dictate the type and frequency of sample to be taken and will therefore control the method of forming boreholes.

Two factors affect the quality of soil testing data required for a satisfactory prediction of soil behaviour.

1. The tests carried out must be appropriate for the acquisition of the required data, or their results must be empirically linked to the required soil parameters with sufficient precision for the required calculation.

2. In addition, sampling and testing must be carried out using techniques and accuracy which will yield parameters that are representative of the bulk of the soil in situ.

7.2 Purpose of soil testing:

In general, soil is tested in order to assess its variability and in order to obtain parameters for particular geotechnical calculations. These two distinct reasons for testing lead to very different testing programmes.

Routine tests carried out to allow the soil on a site to be divided into groups should ideally be scheduled for an initial phase of testing.

More expensive and complex tests are normally carried out on soil which is thought to be representative of each group; the samples to be tested cannot be so well selected before the results of classification tests are known. More complex tests require a longer test period.

7.2.1Available tests:

Soil tests are loosely brought into two groups.

1. The first provides information to allow the classification of soil into arbitrary groups.

2. The second includes all tests which provide parameters which may be used in geotechnical calculation and design.

Soil classification tests	Tests for geo-technical parameters
Sample description	Strength tests
Particle size distribution tests	Stiffness tests
Plasticity tests	Consolidation tests
Compaction tests	Seepage and permeability tests
Specific gravity tests	

Normally plasticity tests, particle size distribution and specific gravity tests are known as soil classification tests.

7.3 Soil classification tests:

Soil classification, although introducing a further stage of data acquisition into site investigation, has an important role to play in reducing the costs and increasing the cost-effectiveness of laboratory testing.

Together with detailed sample description, classification tests allow the soils on a site to be divided into a limited number of arbitrary groups, each of which is estimated to contain materials of similar geotechnical properties.

Subsequently more expensive and time-consuming tests carried out to determine geotechnical parameters for design purposes.

7.3.1 Particle size distribution tests:

Purpose:

This test is performed to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis is performed to determine the distribution of the coarser, larger-sized particles, and the hydrometer method is used to determine the distribution of the finer particles.

Significance:

The distribution of different grain sizes affects the engineering properties of soil. Grain size analysis provides the grain size distribution, and it is required in classifying the soil.

Equipment:

Balance, Set of sieves, Cleaning brush, Sieve shaker, Mixer (blender), 152H Hydrometer, Sedimentation cylinder, Control cylinder, Thermometer, Beaker, Timing device. Figure 7.1 shows, sieve set and sieve shaker.

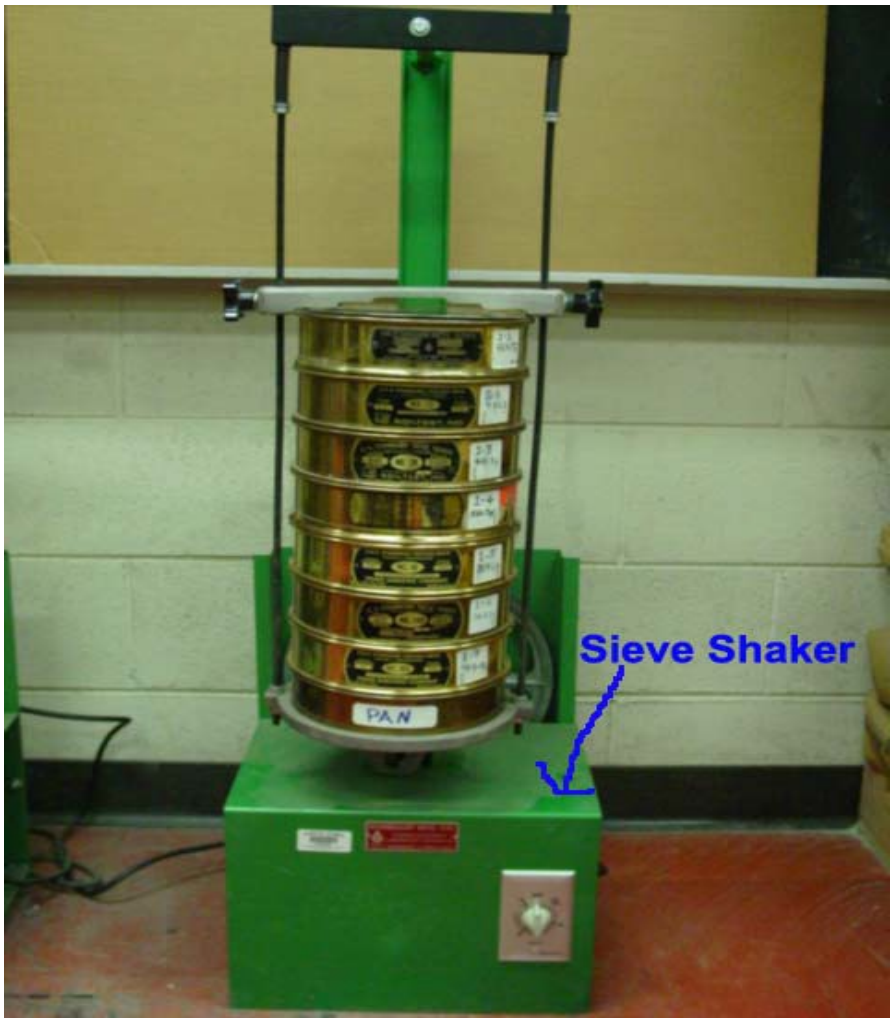

Figure 7.1: Particle size distribution apparatus

Test Procedure:

Sieve Analysis:

(1) Weigh each sieve as well as the bottom pan to be used in the analysis.

(2) Record the weight of the given dry soil sample.

(3) Sieves must be kept clean and assembled in the ascending order of sieve numbers. Place the pan below. Carefully pour the soil sample into the top sieve and place the cap over it.

(4) Place the sieve stack in the mechanical shaker and shake for 10 minutes.

(5) Remove the stack from the shaker and carefully weigh and record the weight of each sieve with its retained soil. In addition, remember to weigh and record the weight of the bottom pan with its retained fine soil.

The particle size distribution is obtained from records of the weight of soil particles retained on each sieve and is usually shown as a graph of 'percentage passing by weight' as a function of particle size.

Two methods of sieving are defined in code. Dry sieving is only suitable for sands and gravels which do not contain any clay.

Wet sieving requires a complex procedure to separate the fine clayey particles from the coarse fraction of the soil which is suitable for sieving, as summarized below.

1. Select representative test specimen by quartering and riffling.

2. Oven dry specimen at 105—110°C, and weigh.

3. Place on 20mm sieve.

4. Wire-brush each particle retained on the 20mm sieve to remove fines.

5. Sieve particles coarser than 20 mm. Record weights retained on each sieve.

6. Riffle particles finer than 20mm to reduce specimen mass to 2kg (approx.).

7. Spread soil in a tray and cover with water and sodium hexametaphosphate (2 g/l).

8. Stir frequently for 1 hour, to break down and separate clay particles**.**

9. Place soil in small batches on a 2mm sieve resting on a 63 m sieve and wash gently to remove fines.

10. When clean, place the material retained in an oven and dry at 105—110°C.

11. Sieve through standard mesh sizes between 20mm and 6.3 mm using the dry sieving procedure. Note weights retained on each sieve.

12. If more than 150 g passes the 6.3mm mesh, split the sample by riffling to give 100—150g.

13. Sieve through standard mesh sizes between 5mm and 6.3 mm sieve.

It is important that this procedure is closely adhered to. Inadequate dispersal of the clay particles, poor washing, overloading of sieves and insufficient sieving time can all lead to inaccurate results.

In particular, extra time and care may be required to ensure full dispersion of clay lumps within the test specimen.

The particle size distribution of the fine soil fraction, between about 0.1 mm and 1 μm may be determined by one of two British Standard sedimentation tests. Soil is sedimented through water, and Stokes' law, which relates the terminal velocity of a spherical particle falling through a liquid of known viscosity to its diameter and specific gravity, is used to deduce the particle size distribution.

Sedimentation tests make a number of important assumptions. Since Stokes' law is used, the following assumptions are implied:

1. The drag force on each particle is entirely due to viscous forces within the fluid. The particles must be spherical, smooth and rigid, and there must be no slippage between them and the fluid.

2. Each particle must move as if it were a single particle in a fluid of infinite extent.

3. The terminal velocity must be reached very shortly after the test starts.

4. The settling velocity must be slow enough so that inertia effects are negligible.

5. The fluid must be homogeneous compared with the size of the particle.

Hydrometer Analysis:

(1) Take the fine soil from the bottom pan of the sieve set, place it into a beaker, and add 125 mL of the dispersing agent (sodium hexametaphosphate (40 g/L)) solution. Stir the mixture until the soil is thoroughly wet. Let the soil soak for at least ten minutes.

(2) While the soil is soaking, add 125mL of dispersing agent into the control cylinder and fill it with distilled water to the mark.

(3) Transfer the soil slurry into a mixer by adding more distilled water, if necessary, until mixing cup is at least half full. Then mix the solution for a period of two minutes.

(4) Immediately transfer the soil slurry into the empty sedimentation cylinder. Add distilled water up to the mark.

(5) Cover the open end of the cylinder with a stopper and secure it with the palm of your hand. Then turn the cylinder upside down and back upright for a period of one minute. (The cylinder should be inverted approximately 30 times during that minute.)

(6) Set the cylinder down and record the time. Remove the stopper from the cylinder. After an elapsed time of one minute and forty seconds, very slowly and carefully insert the hydrometer for the first reading.

(7) The reading is taken by observing the top of the meniscus formed by the suspension and the hydrometer stem. The hydrometer is removed slowly and placed back into the control

cylinder. Very gently spin it in control cylinder to remove any particles that may have adhered.

(8) Take hydrometer readings after elapsed time of 2 and 5, 8, 15, 30, 60 minutes and 24 hours.

Typical Hydrometer and soaking soil specimen is shown in Figure 7.2.

This method is less accurate in principle because the hydrometer does not measure density at a fixed point below the surface of the fluid but determines an average value over the depth of its bulb.

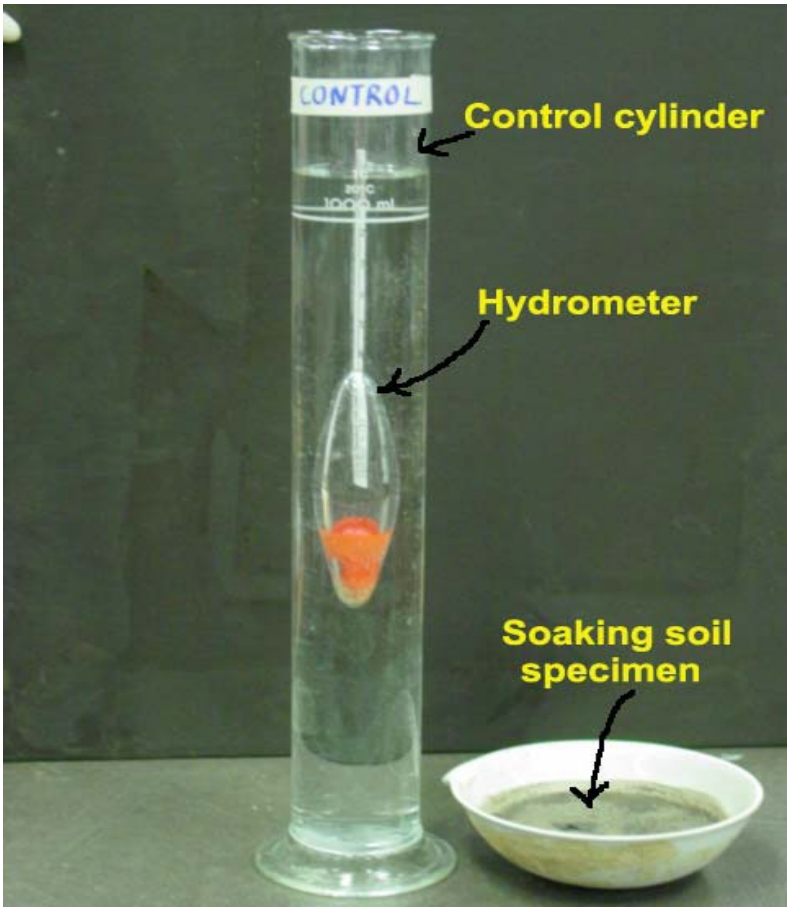

Figure 7.2: Hydrometer and Soaking soil specimen

7.3.2 Plasticity tests:

The plasticity of soils is determined by using relatively simple remoulded strength tests.

The liquid limit (LL) is arbitrarily defined as the water content, in percent, at which a part of soil in a standard cup and cut by a groove of standard dimensions will flow together at the base of the groove for a distance of 13 mm (1/2 in.) when subjected to 25 shocks from the

cup being dropped 10 mm in a standard liquid limit apparatus operated at a rate of two shocks per second.

The plastic limit is the moisture content of the soil under test when remoulded and rolled between the tips of the fingers and a glass plate such that longitudinal and transverse cracks appear at a rolled diameter of 3 mm.

The liquid limit of a soil can be determined using the cone penetrometer or the Casagrande apparatus.

The cone penetrometer is considered a more satisfactory method than the alternatives because it is essentially a static test which relies on the shear strength of the soil, whereas the alternative Casagrande cup method introduces dynamic effects.

Test Procedure for Liquid Limit:

Casagrande apparatus is shown in Figure 7.3

(1) Take roughly 3/4 of the soil and place it into the porcelain dish. Assume that the soil was previously passed through 475 micron sieve, air-dried, and then pulverized. Thoroughly mix the soil with a small amount of distilled water until it appears as a smooth uniform paste.

Cover the dish with cellophane to prevent moisture from escaping.

(2) Weigh four of the empty moisture cans with their lids, and record the respective weights and can numbers on the data sheet.

(3) Adjust the liquid limit apparatus by checking the height of drop of the cup. The point on the cup that comes in contact with the base should rise to a height of 10 mm. The block on the end of the grooving tool is 10 mm high and should be used as a gage. Determine the correct rate to rotate the crank so that the cupdrops approximately two times per second. (4) Place a portion of the previously mixed soil into the cup of the liquid limit apparatus at the point where the cup rests on the base. Squeeze the soil down to eliminate air pockets and spread it into the cup to a depth of about 10 mm at its deepest point. The soil pat should form an approximately horizontal surface.

(5) Use the grooving tool carefully cut a clean straight groove down the centre of the cup. The tool should remain perpendicular to the surface of the cup as groove is being made. Use extreme care to prevent sliding the soil relative to the surface of the cup.

(6) Make sure that the base of the apparatus below the cup and the underside of the cup is clean of soil. Turn the crank of the apparatus at a rate of approximately two drops per second and count the number of drops, N; it takes to make the two halves of the soil pat come into contact at the bottom of the groove along a distance of 13 mm (1/2 in.). If the number of drops exceeds 50, then go directly to step eight and do not record the number of drops, otherwise, record the number of drops.

(7) Take a sample, using the spatula, from edge to edge of the soil pat. The sample should include the soil on both sides of where the groove came into contact. Place the soil into a moisture can cover it.

Immediately weigh the moisture can containing the soil, record its mass, remove the lid, and place the can into the oven. Leave the moisture can in the oven for at least 16 hours. Place the soil remaining in the cup into the porcelain dish. Clean and dry the cup on the apparatus and the grooving tool.

(8) Remix the entire soil specimen in the porcelain dish. Add a small amount of distilled water to increase the water content so that the number of drops required closing the groove decreases.

(9) Repeat steps six, seven, and eight for at least two additional trials producing successively lower numbers of drops to close the groove.

One of the trials shall be for a closure requiring 25 to 35 drops, one for closure between 20 and 30 drops, and one trial for a closure requiring 15 to 25 drops. Determine the water content from each trial.

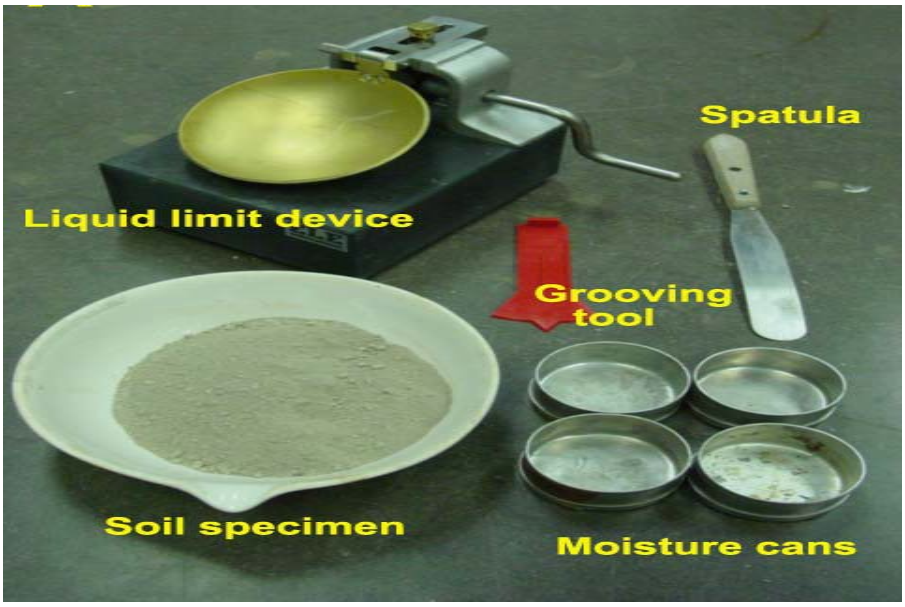

Figure 7.3: Casagrande apparatus used to find out liquid limit

Test Procedure for Cone-penetrometer test:

The cone penetrometer is considered a more satisfactory method than the alternative because it is essentially a static test which relies on the shear strength of the soil, whereas the alternative Casagrande cup method introduces dynamic effects.

In the penetrometer test, the liquid limit of the soil is the moisture content at which an 80 g, 300 cone sinks exactly 20 mm into a cup of remoulded soil in a 5s period. At this moisture content, the soil will be very soft.

Test Procedure for Plastic limit:

(1) Weigh the empty moisture cans with their lids, and record the respective weights.

(2) Take the remaining 1/4 of the original soil sample and add distilled water until the soil is at a consistency where it can be rolled without sticking to the hands.

(3) Form the soil into an ellipsoidal mass. Roll the mass between the palm or the fingers and the glass plate.

Use sufficient pressure to roll the mass into a thread of uniform diameter by using about 90 strokes per minute. The thread shall be deformed so that its diameter reaches 3.2 mm (See Figure 7.4).

(4) When the diameter of the thread reaches the correct diameter, break the thread into several pieces. Knead and reform the pieces into ellipsoidal masses and re-roll them. Continue this alternate rolling, gathering together, kneading and re-rolling until the thread crumbles under the pressure required for rolling and can no longer be rolled into a 3.2 mm diameter thread.

(5) Gather the portions of the crumbled thread together and place the soil into moisture can and cover it. If the can does not contain at least 6 grams of soil, add soil to the can from the next trial.

Figure 7.4: Rolled soil sample to determine plastic limit

Immediately weigh the moisture can containing the soil, record it's mass, remove the lid, and place the can into the oven. Leave the moisture can in the oven for at least 16 hours.

(6) Repeat steps three, four, and five at least two more times. Water content from each trial is determined.

Plasticity tests are widely used for classification of soils into groups on the basis of their position on the Casagrande chart, but in addition they are used to determine the suitability of wet cohesive fill for use in earthworks, and to determine the thickness of sub-base required beneath highway pavements.

7.4 Compaction tests:

Standard provides three specifications for laboratory compaction tests:

1. 2.5 kg rammer method;

2. 4.5 kg rammer method; and

3. Vibrating hammer method for granular soils.

Compaction has been defined as 'the process whereby soil particles are constrained to pack more closely together through a reduction in the air voids, generally by mechanical means'. Compaction is therefore a rapid process which does not normally involve a significant change in moisture content.

Proctor compaction test:

The 2.5 kg rammer method is derived from the work of Proctor. The test subsequently became adopted by the American Association of State Highway Officials (AASHO), and was known as the Proctor or AASHO compaction test.

In the original test, a mould of capacity 1/30 ft^3 with an internal diameter of 4 in was filled with soil at fixed moisture content in three approximately equal layers. Each layer was compacted by 25 blows of a 2 in. dia. 5.5 lb rammer dropping through a height of 12 in.

After compaction, the soil was trimmed to the level of the top of the mould, and the wet weight of soil and its moisture content were determined. The process was repeated for several increasing moisture contents, and a compaction curve (i.e. dry density as a function of moisture content) was obtained.

Figure 7.5 shows typical compaction curve

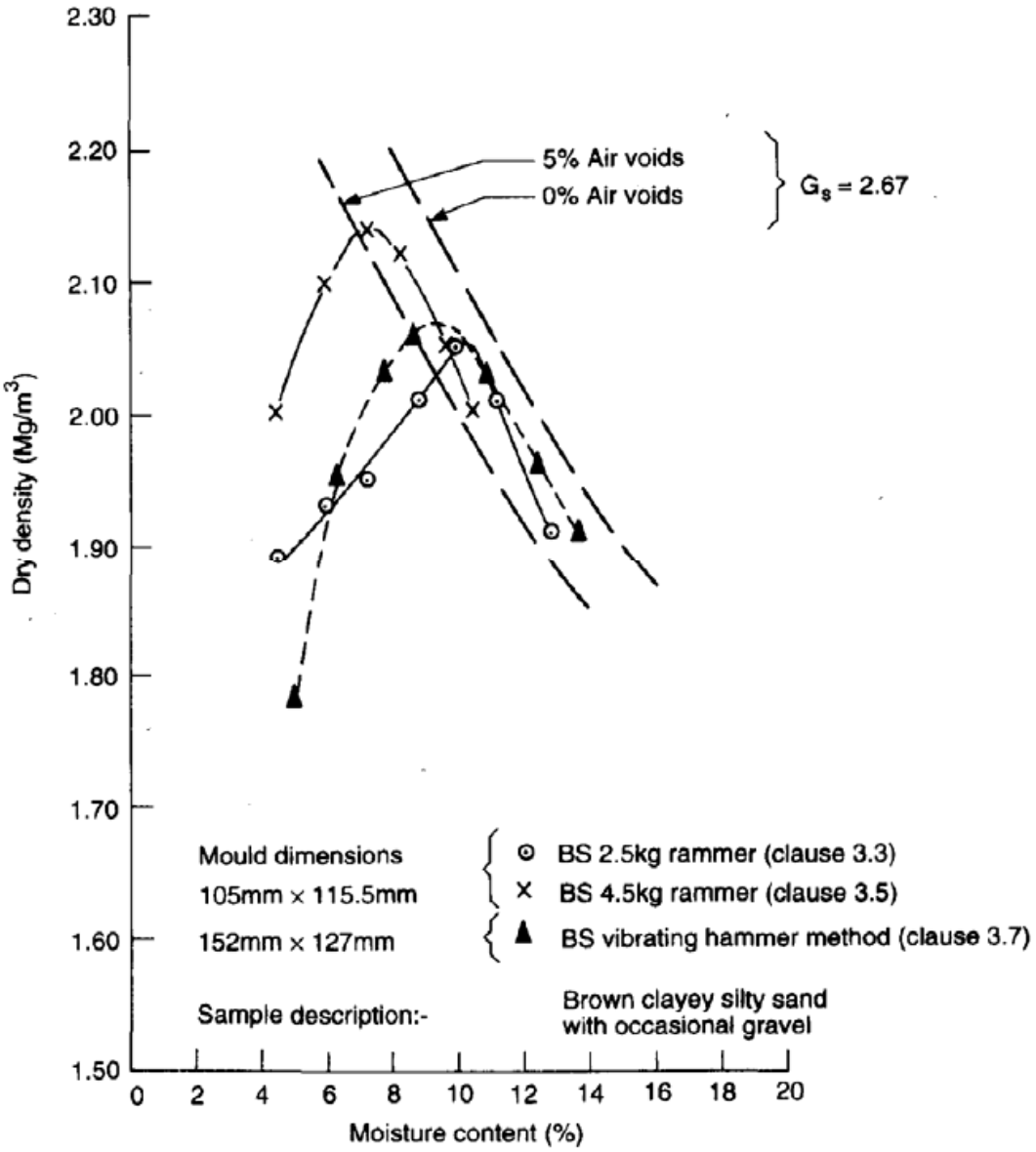

Figure 7.5: Compaction curves

Different types of soil react in very different ways to each type of roller. It is commonly known that increasing levels of compactive effort tend to produce higher maximum dry density values in conjunction with progressively lower optimum moisture content, the 'lines of optimums' developed in field compaction trials with different plant are not coincident.

The objects of field compaction are to obtain sufficient strength, eliminate collapse, and reduce compressibility of fill to an acceptable level.

7.5 Particle density (specific gravity) determination:

Specific gravity is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature.

The specific gravity tests are relatively simple and are based upon determination of the dry weight of a sample of the soil, and the weight of the same sample plus water in a container of known volume.

The volume of the container is obtained by weighing the container empty, and full of water. The main problems in conducting the test are: accurate weighing and complete removal of the air from the soil after the addition of water.

Test procedure for specific gravity:

(1) Determine the weight of the empty clean and dry pycnometer, W_p.

(2) Place 10g of a dry soil sample (passed through the sieve No. 10) in the pycnometer. Determine and record the weight of the pycnometer containing the dry soil, W_{ps}.

(3) Add distilled water to fill about half to three-fourth of the pycnometer. Soak the sample for 10 minutes.

(4) Apply a partial vacuum to the contents for 10 minutes, to remove the entrapped air.

(5) Stop the vacuum and carefully remove the vacuum line from pycnometer.

(6) Fill the pycnometer with distilled (water to the mark).Determine the weight of the pycnometer and contents, W_B.

(7) Empty the pycnometer and clean it. Then fill it with distilled water only (to the mark). Determine the weight of the pycnometer and distilled water, W_A.

(8) Empty the pycnometer and clean it.

Specific gravity is obtained from the equation:

$$G_s = W_0 / W_0 + (W_A - W_B)$$

Where:

W_0 = weight of sample of oven-dry soil, g = $W_{PS} - W_P$

W_A = weight of pycnometer filled with water

W_B = weight of pycnometer filled with water and soil

The major difficulty with this test is the provision of a satisfactory vacuum and measuring the length of time required to remove the air completely. These factors can clearly lead to errors in specific gravity determinations.

The major problem arising from an incorrect particle density determination is that of the credibility of compaction tests carried out on the same soil.

A low particle density value will push the zero air voids line on a dry density/moisture content plot down and to the left, and may show compaction test results to be apparently impossible (and therefore inaccurate) as they cross over the zero air voids line.

7.6 Tests for geotechnical parameters:

A wide range of tests has been used to determine the geotechnical parameters required in calculations for example, of bearing capacity, slope stability, earth pressure and settlement.

Strength tests:

The principal tools available for strength determination in a good UK geotechnical testing laboratory are the California Bearing Ratio (CBR) apparatus, the Franklin Point Load Test apparatus, the laboratory vane apparatus and various forms of direct shear and triaxial apparatus.

7.6.1 California bearing ratio (CBR) tests:

The CBR test is primarily used to assess the strength of materials used in or beneath flexible highway or airfield pavements.

The CBR test was specifically developed by the California State Highway Department for the evaluation of sub-grade strengths in the investigation of existing pavements of known performance in use.

The test is carried out by forcing a standard plunger (approximately 50mm dia.) into the soil at a more or less constant rate of 1.25 mm/min. Measurements of applied load and plunger penetration are made at regular intervals, and a curve is plotted for penetrations of up to 12.5 mm.

The California Bearing Ratio is obtained by dividing the plunger loads at penetrations (after bedding correction) of 2.5 and 5.0 mm by the loads given at the same penetrations on a standard crushed stone.

The loads given by the soil under test are expressed as percentages of the standard load, and the highest value is taken as the CBR value for design.

The CBR test primarily involves shear deformation of the soil beneath the plunger, but its results cannot be accurately related to any of the fundamental shear strength parameters. Its use is therefore restricted to the design of road and airfield pavements.

Typical CBR apparatus and test results are given in Figure 7.6

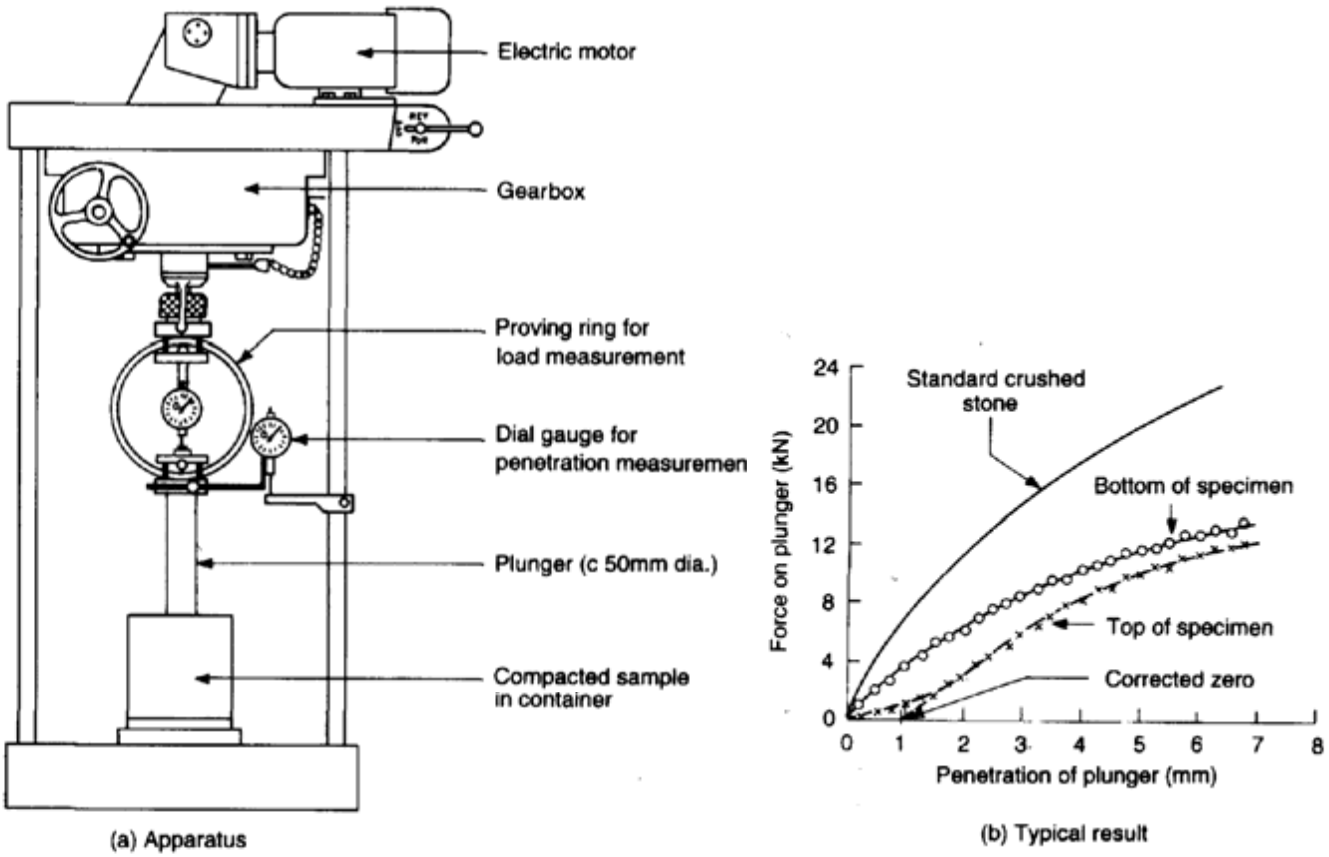

Figure 7.6: CBR apparatus and test results

7.6.2 Franklin point load test:

The Franklin point load test was developed at Imperial College, London to provide a quick and reliable measurement of the strength of unprepared rock core samples, both in the fieldand the laboratory.

The apparatus consists of a small loading frame which is activated by a hydraulic hand pump and ram. Figure 7.7 shows schematic representation of Franklin point load test.

Rock core is placed between pointed platens of standard dimensions and loaded until failure occurs. The point load strength index:

$$I_s = P/D^2$$

Where P = force required to break the specimen, and D = distance between the platen contact points.

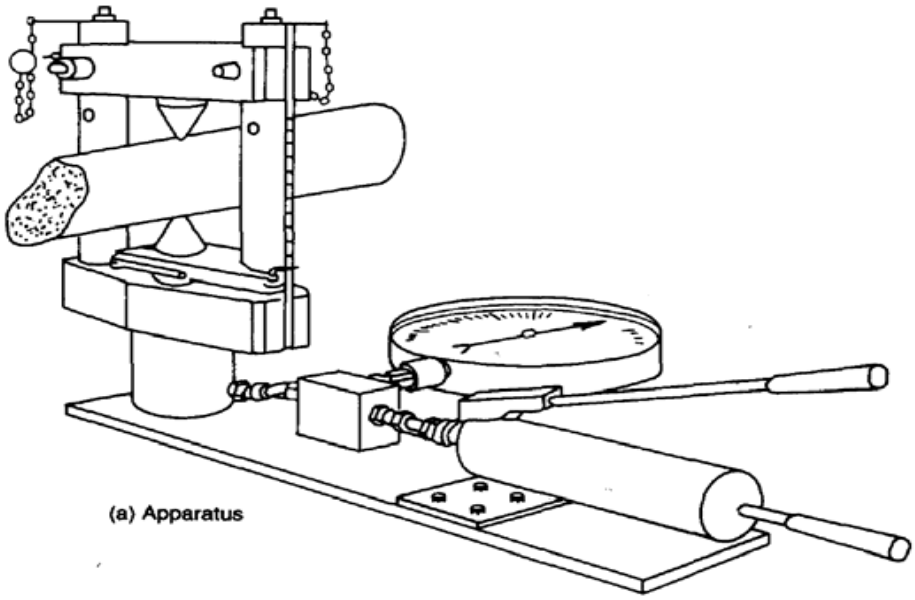

Figure 7.7: Franklin point load test apparatus

Tests may most reliably be carried out across the core diameter, but results can also be obtained when discs of core are loaded axially. Under this latter condition, corrections to the point load strength index will be required which will depend on the aspect ratio of the specimen.

Under extreme circumstances, when only irregular lumps of rock are available, the test can be carried out along the shortest axis of the lump, but results will be less reliable. It has therefore, been proposed that a standard classification be adopted by correcting all values to a reference diameter of 50mm. A correction chart for this purpose is prepared based on the results of tests on five rock types at diameters between 10 and 80 mm.

7.6.3 Laboratory vane test

The principle involved in the vane test is discussed under 'In situ testing'. Whilst the field vane typically uses a blade with a height of about 150 mm, the laboratory vane is a small-scale device with a blade height of about 12.7 mm and a width of about 12.7 mm.

The small size of the laboratory vane makes the device unsuitable for testing samples with fissuring or fabric, and therefore it is not very frequently used.

7.6.4 Direct shear test:

The direct shear test carried out in the shear box apparatus is similar to vane shear test. Figure7.8 shows, the basic components of the direct shear apparatus. Soil is cut to fit tightly

into a box which may be rectangular or circular in plan and is normally rectangular in elevation.

The box is constructed to allow displacement along its horizontal mid-plane, and the upper surface of the soil is confined by a loading plate through which normal stress may be applied.

Shear load is applied to the lower half of the box, the upper half being restrained by a proving ring or load cell which is used to record the shear load. The sample is not sealed in the shear box; it is free to drain from its top and bottom surfaces at all times.

The cross-sectional area over which the specimen is sheared is assumed to remain constant during the test.

Cut-plane shear box testing gives results which have been found to be satisfactory, based on back-analysis. A specimen of clay is placed in the shear box and allowed to swell for 24h under the weight of the load hanger.

Following this, the specimen is consolidated under the required normal pressure and measurements of vertical compression are made. The two halves of the box are then separated sufficiently to allow a cheese wire to cut smoothly through the specimen.

The two halves of the specimen are then separated, and the soil surfaces smoothed by rubbing a glass plate lubricated by distilled water over the surfaces. When smooth, the lower half of the soil specimen is raised by packing it with a few layers of filter paper: the box is reassembled, and after applying a normal stress, the specimen is subjected to large displacements on the preformed shear surface by repeatedly reversing the travel of the box.

The maximum shear stress obtained for each stage of shearing should be plotted against the logarithm of cumulative displacement, and shearing should continue until this curve levels out. The lowest maximum shear stress values (in the final shear stage of each test) are plotted against their imposed normal stresses to obtain the residual effective strength parameters (c'r (normally zero) and φ'r for a soil.

A better form of test to find residual parameters is carried out on an annular specimen in the ring shear apparatus, described by Bishop et al. Because of its cost and complexity this apparatus has failed to find a place in site investigation testing laboratories.

The simplified ring shear test is carried out on an annular specimen of remoulded clay 5mm thick, with internal and external diameters of 70mm and 100mm respectively. The specimen is confined radially between concentric rings and the vertical normal stress is applied via two porous bronze loading platens.

Relative rotary motion takes place between the confining rings (which are fixed to the lower loading platen) and the upper platen. This causes the sample to shear, the shear surfaces forming close to the upper platen.

The loading platens are roughened in order to prevent slip at the platen—soil interface. The upper platen reacts against two matched proving rings (or load cells) which provide a measurement of the torque transmitted through the soil specimen.

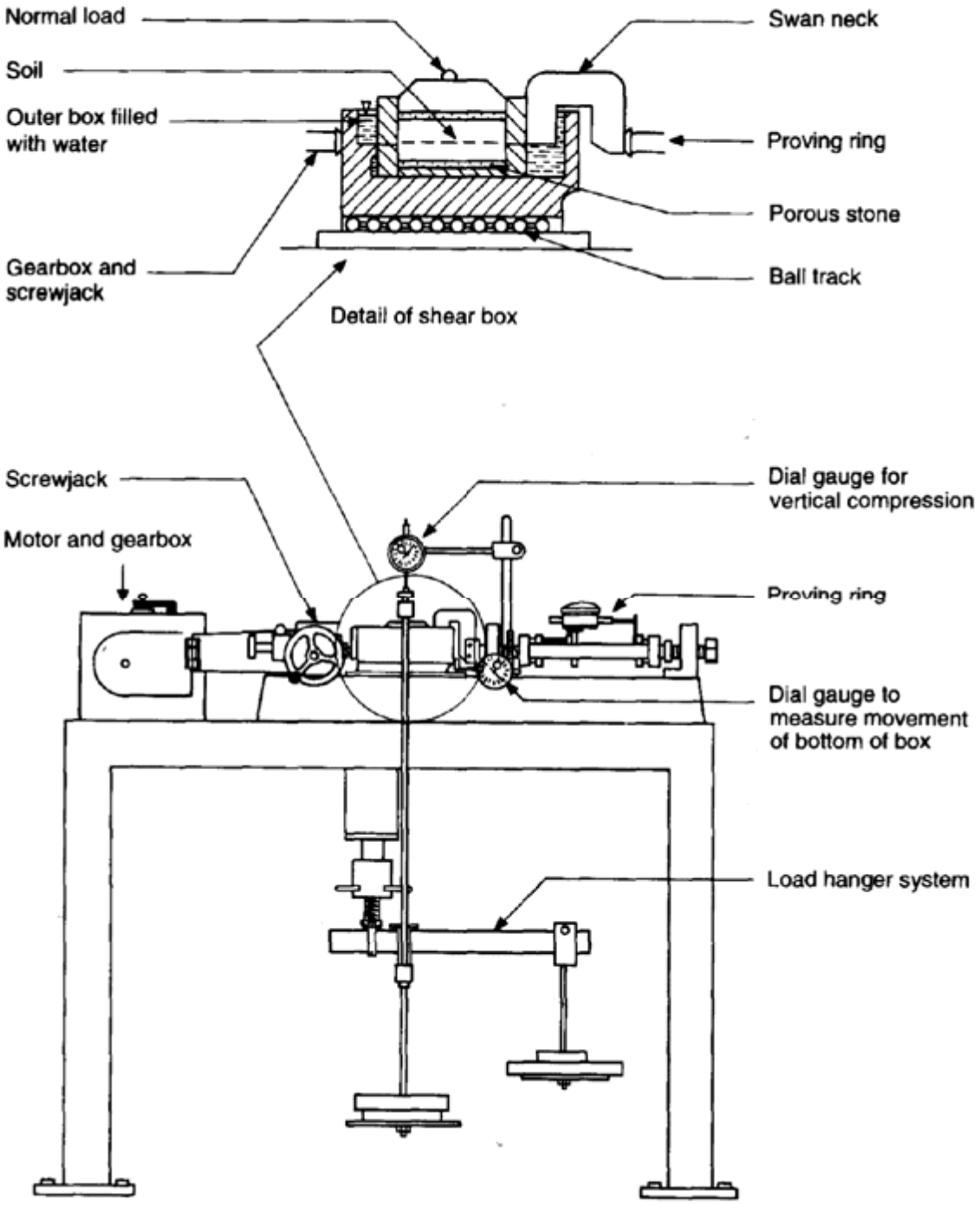

Figure 7.8: Bishop direct shear box

7.6.5 Triaxial test:

The triaxial apparatus has been described in great detail by Bishop and Henkel (1962). The test specimen is normally a cylinder with an aspect ratio of two, which is sealed on its sides by a rubber membrane attached by rubber 'O' rings to a base pedestal and top cap. Figure 7.9 shows typical triaxial apparatus

Water pressure inside the cell provides the horizontal principal total stresses, while the vertical pressure at the top cap is produced by the cell fluid pressure and the ram force.

The triaxial apparatus requires one or two self-compensating constant pressure systems, a volume change measuring device and several water pressure sensing devices.

The ram force may be measured outside the cell using a proving ring, but most modern systems now use an internal electrical load cell mounted on the bottom of the ram.

The ram is driven into the triaxial cell by an electrical loading frame which will typically have a capacity of 5000 or 10000 kgf and is capable of running at a wide range of constant speeds; triaxial tests are normally carried out at a controlled rate of strain increase.

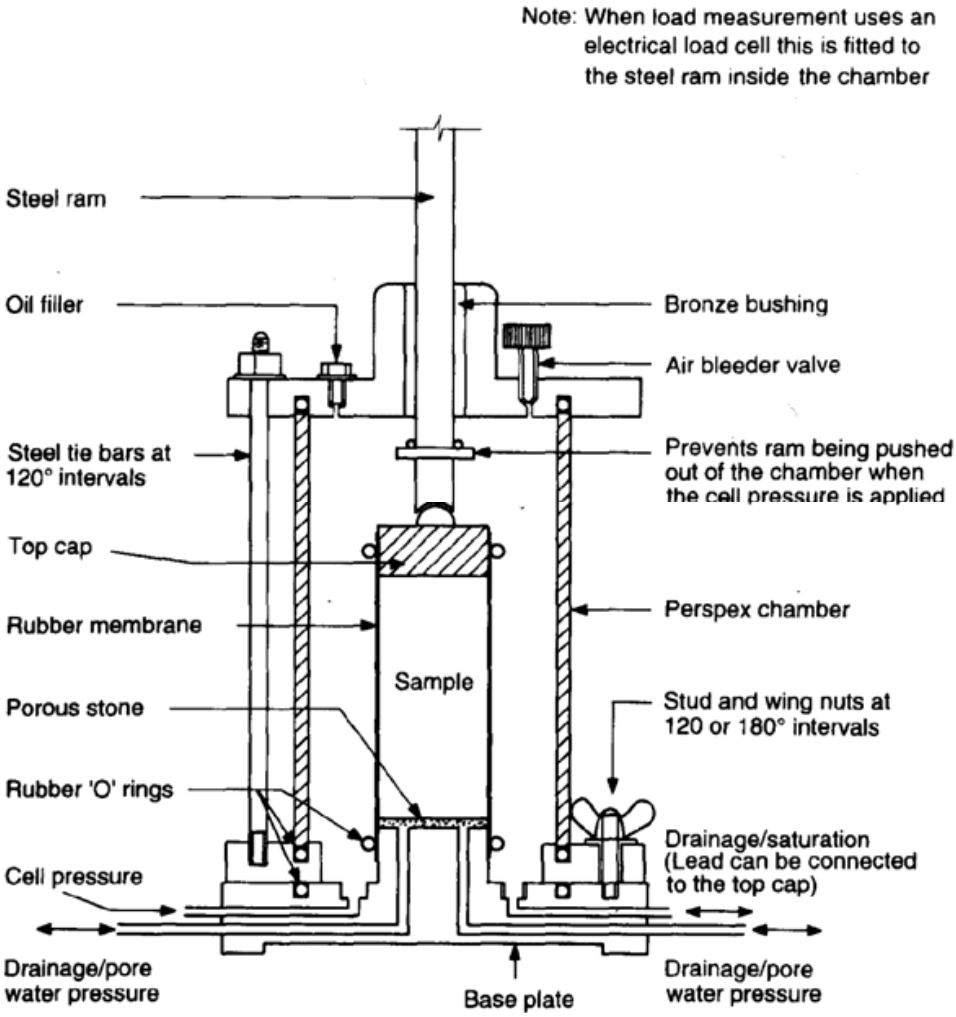

Figure 7.9: Triaxial cell

The three most common forms of test are:

1. The un-consolidated undrained triaxial compression test, without pore water pressure, measurement;

2. The consolidated undrained triaxial compression test, with pore water pressure measurement; and

3. The consolidated drained triaxial compression test, with volume change measurement.

7.6.6 Stiffness tests:

During the last decade, two important parallel developments have taken place which has resulted in the measurement of stiffness being considered more important than that of strength in geotechnical design, particularly for sensitive structures. These developments are:

1. Methods of measuring strain locally on laboratory test specimens have shown that the stress—strain behaviour of many soils and weak rocks is significantly non-linear with very high stiffness at the small strains operational around most engineering structures; and

2. Certain features of field measurements of ground deformation around full-scale structures, which could not be modelled, using linear elastic theory, are resolved if non-linear formulations are used incorporating very high initial stiffness.

These developments have resulted in the application of finite element models in geotechnical design becoming commonplace, with stiffness parameters derived both from special laboratory stiffness tests and from field geophysics.

In most soils, any discontinuities such as fissures will generally have a stiffness that is similar to that of the intact soil such that the intact soil stiffness may be used to predict with reasonable accuracy ground deformations and stress distributions.

This means that laboratory triaxial tests on good quality ‘undisturbed’ specimens may yield adequate stiffness parameters for design purposes.

The errors in the computation of strain that arise from this method of measurement result from the fact that apparatus is compliant; the load cell, porous stones, lubricated end platens and filter papers will all compress under increasing axial load.

Further, errors are associated with bedding caused by lack of fit or surface irregularities at the interfaces between the specimen and loading surfaces. Although the errors due to apparatus compliance can be evaluated with reasonable certainty by careful calibration, the bedding error can be very difficult to assess since its magnitude depends on the way in which the ends of the specimen are prepared.

Thus, the only way to obtain accurate determinations of axial strain is to carry out the measurement remotely from the ends of the specimen, and preferably on its middle third. This type of measurement is referred to as 'local strain' measurement.

The instrumentation for local strain measurements includes:

1. Miniature displacement transducers;

2. Proximity transducers;

3. Electro level gauges;

4. Hall Effect semiconductor and

5. Strain gauged metal strips (LDT).

Of these, the electro-level gauges and the Hall Effect semiconductors are in use in commercial laboratories and local strain gauges (axial and radial) based on the Hall Effect semiconductor are available commercially. Figure 7.10 shows Electro level Gauge and Hall effect Gauge.

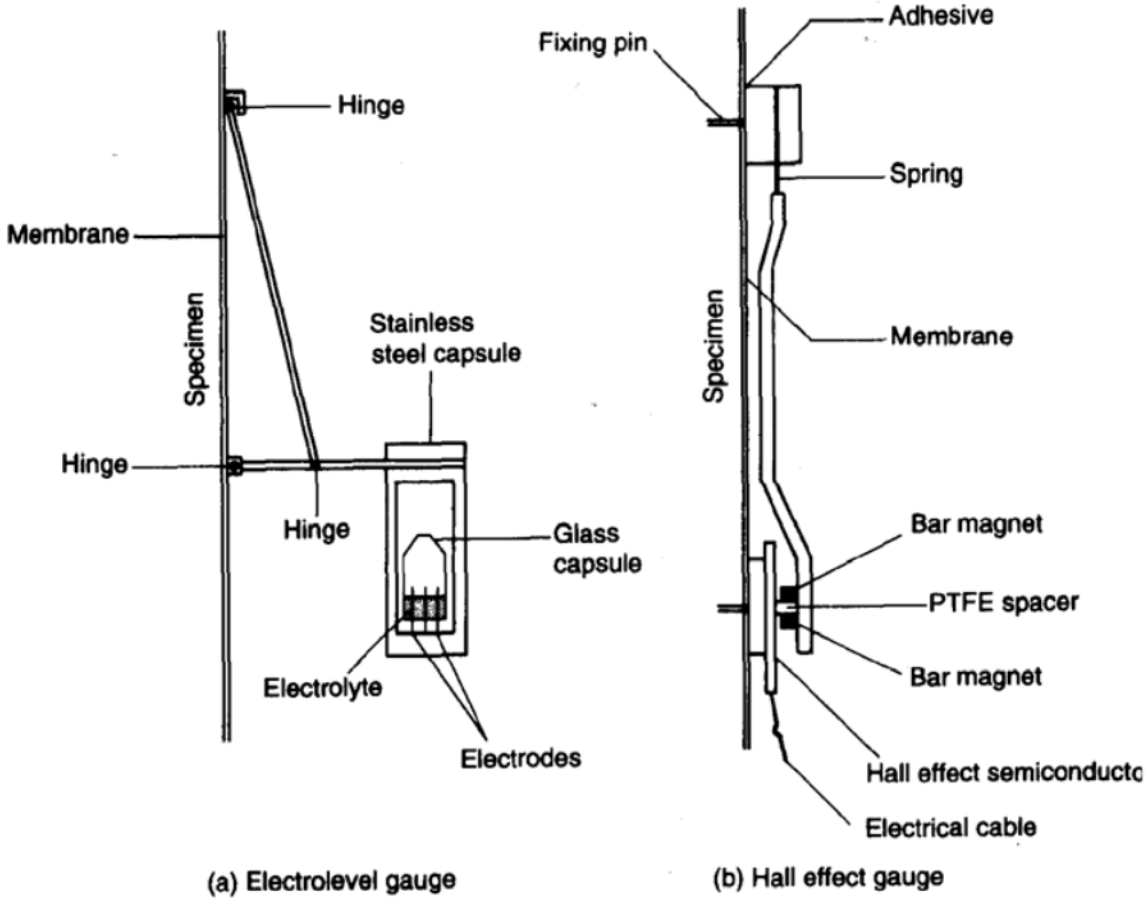

Figure 7.10: Electro level Gauge and Hall effect Gauge

In soil mechanics, it has become traditional to emphasize the non-linear stress— strain behaviour by plotting secant modulus against log axial strain.

In soil mechanics practice, for materials considered to be relatively unaffected by cementing, it is common to normalize secant modulus(E or G) with respect to the mean effective stress (p'_0) immediately before shearing, but for materials which are considered to be cemented to a significant degree it is thought better to normalize with respect to the undrained strength (c_u). Examples of such normalized curves are shown in Figure 7.11.

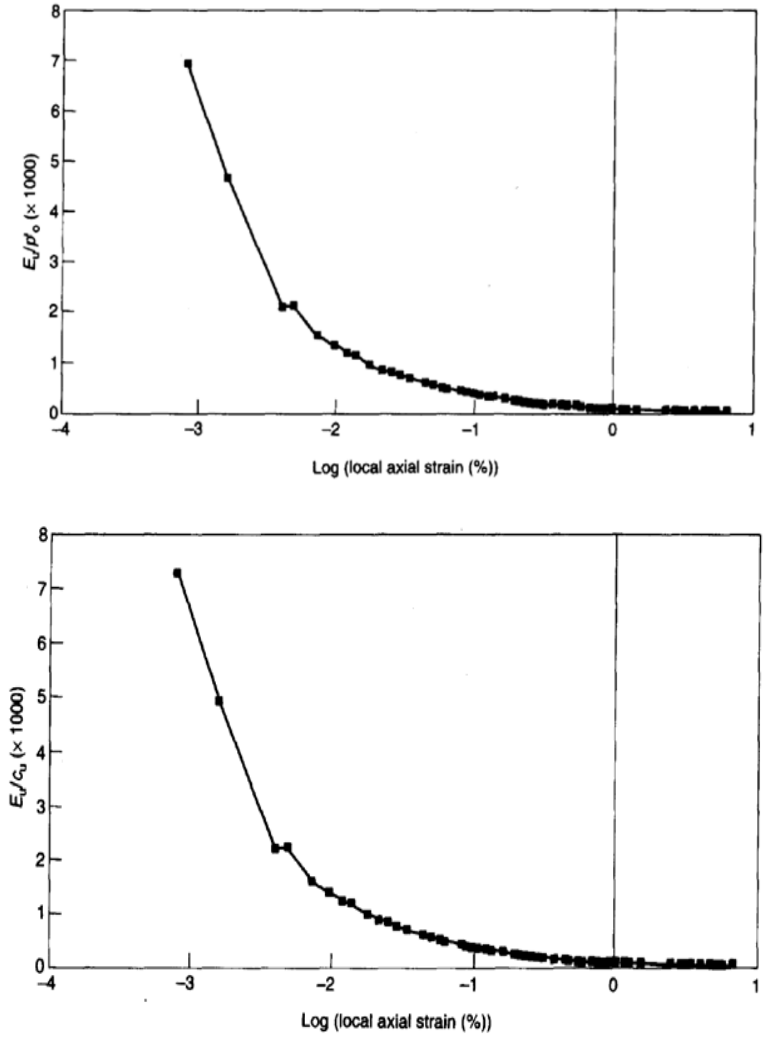

Figure 7.11: Normalized stiffness—log strain curves

Recent research has shown that stiffness measured in the triaxial apparatus is affected by the following factors (in descending order of importance):

1. Cementing (bonding or structuring);

2. Effective stress (in less cemented materials, such as most lightly and moderately over-consolidated clays);

3. Sample disturbance (see Chapter 6) which results both in changes in effective stress and in destructuring;

4. History (i.e. over-consolidation);

5. Stress path and stress-path rotation

The stress path followed will have a significant influence on the measured stiffness. In particular, changes in direction of the stress path such as a loading path followed by an unloading path will result in an increase in stiffness

6. Ageing (i.e. creep and rest period)

Rest period refers to the period during which the soil remains at a constant stress between the end of the most recent stress path and the start of the current path. The duration of the rest period can have a significant effect on the measured stiffness.

7.6.7 Consolidation tests:

Consolidation tests are frequently required either to assess the amount of volume change to be expected of a soil under load, for example beneath a foundation, or to allow prediction of the time that consolidation will take.

Three pieces of apparatus are in common use for consolidation testing. These are:

1. Theoedometer

2. The triaxial apparatus and

3. The hydraulic consolidation cell.

Casagrandeoedometer test:

The Casagrandeoedometer test is most widely used. The apparatus consists of a cell which can be placed in a loading frame and loaded vertically. In the cell the soil sample is laterally restrained by a steel ring, which incorporates a cutting shoe used during specimen preparation. Figure 7.12 shows the Casagrandeoedometer apparatus.

The top and bottom of the specimen are placed in contact with porous discs, so that drainage of the specimen takes place in the vertical direction when vertical stress is applied; consolidation is then one-dimensional.

In the standard procedure the specimen is subjected to a series of pre-selected vertical stresses (e.g. 6, 12, 25, 50, 100, 200, 400, 800, 1600, 3200 kN/m^2) each of which is held constant while dial gauge measurements of vertical deformation of the top of the specimen are made, and until movements cease (normally 24 h).

Dial gauge readings are taken at standard intervals of time after the start of the test (i.e. 0, 15 and 30s, 1, 2, 4, 8, 15, 30 and 60mm, 2, 4, 8 and 24h).

At the same time that the first load is applied, the oedometer cell is flooded with water, and if the specimen swells the load is immediately increased through the standard increments until swelling ceases.

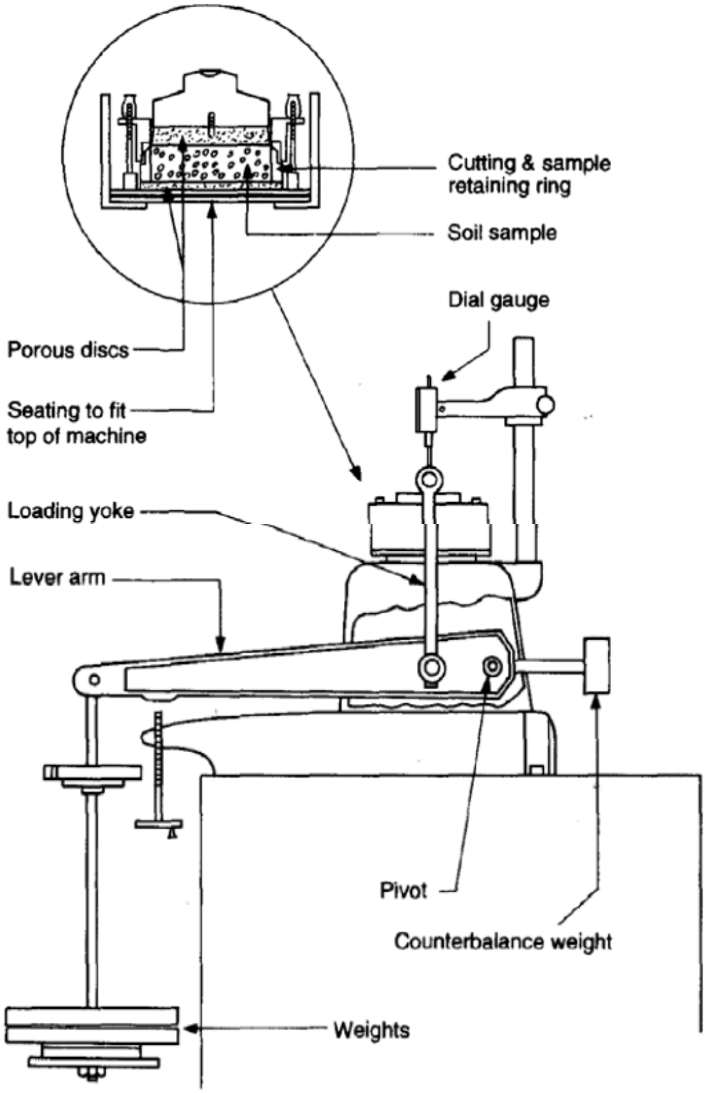

Figure 7.12: Casagrandeoedometer apparatus

The results of each loading stage of an oedometer test are normally plotted as the dial gauge readings either as a function of square root of elapsed time, or as a function of the logarithm of elapsed time.

The coefficient of consolidation (c_v) used in calculations of settlement can be obtained from the curves, using Taylor and Merchant's method, or Casagrande's method respectively.

In soft clays, the effects of sample disturbance are to reduce the compressibility values measured in the oedometer.

In hard clays and soft rocks, the mass compressibility is much affected by the compressibility of the joints and bedding planes, which traverse it.

7.6.8 Triaxial dissipation test:

The measurement of consolidation characteristics can be carried out in the triaxial dissipation test. The most common size of specimen is 102mm high x 102mm dia., and the test is carried out in a triaxial chamber such as might be used for a consolidated undrained triaxial compression test with pore pressure measurement.

The specimen is compressed under the isotropic effective stress produced by the difference between the cell pressure and the back pressure, and volume change is recorded as a function of time, as in the consolidation stage of an effective strength triaxial compression test, but in addition pore pressure is measured at the base of the specimen.

Drainage occurs upwards in the vertical direction but soil compression is three-dimensional, and for this reason the results of this test are not strictly comparable with those of an oedometer test.

The compressibility determined from volume changes during the triaxial dissipation is greater than that measured under conditions of zero lateral strain, and the difference is most pronounced for over-consolidated clays and compacted soils.

When testing compacted soils the initial stages of the test involve undrained application of cell pressure, to allow an assessment of the pore pressure parameter B. Most natural soils will be saturated in situ, and these are normally subjected to re-saturation by the application of back pressure.

Consolidation is started by either increasing the cell pressure or decreasing the back pressure to give the required effective stress, and then applying the back pressure to the top of the specimen.

Volume changes and pore pressure measurements are plotted as a function of the logarithm of elapsed time.

Owing to the expansibility of leads in the system, and the compressibility of air in the specimen, the initial volume gauge reading must be corrected by assuming that volume changes in the first few minutes of the test are proportional to the square root of time.

The coefficient of consolidation (c_v) can be determined by matching the theoretical relationships between pore pressure dissipation, volume change and the logarithm of time at the 50% point.

$$t_{50} = 0.197H^2 / c_v \text{at 50\% volume change}$$

$$t_{50} = 0.38H^2 / c_v \text{at 50\% pore pressure dissipation}$$

Where H = specimen height.

The triaxial dissipation test is time consuming (no filter drains can be used), but relatively straightforward. Pore pressure measurement should be carried out by electrical pressure transducer and de-airing of the triaxial system should be thorough.

Because of the relatively long period of testing, even minor leaks will obscure the soil behaviour and may not become obvious until sometime after the start of the test. Because small pressure and volume changes are significant in the later stages of this test, it is important that it is carried out in a temperature-controlled environment so that expansion of the measuring system and cell relative to the specimen do not obscure the consolidation process.

7.6.9 Hydraulic oedometer test:

The consolidation of large specimens can be carried out in the hydraulic oedometer. This apparatus prevents lateral strain by confining the specimen in a bronze cast ring, and provides vertical stress through a pleated bellows-like rubber membrane (the rubber 'jack'), and is restrained at the top and bottom by thick metal plates bolted to the bronze ring.

Hydraulic oedometers are available for specimens of 76mm, 152mm and 254mm dia. A 254mm dia. specimen may have a thickness of 75—100mm.

The hydraulic oedometer test has most of the advantages of the triaxial consolidation test in that pore water pressure can be controlled by a constant pressure source through the top drain, and pore water pressure measurements can be made at small ceramics flush mounted in the base plate and connected to pressure transducers.

In addition, volume change measurements may be made by monitoring the movements of the settlement rod which brings the back pressure line through the top plate, or by measuring the movement of the water in the back pressure line with a volume gauge.

Because of the use of a rubber jack, high vertical stress levels can be applied to the specimen without the need for a loading frame.

The hydraulic oedometer cell can be used with at least four types of specimen drainage: drainage may be vertical to a single porous sintered bronze plate beneath the rubber jack, or porous plates may be provided at the top and bottom of the specimen in which case mid-plane pore pressure dissipation cannot be measured.

Horizontal drainage may be used either by augering a sand drain in the centre of the specimen (inward drainage) or by placing a 1.5mm thick porous plastic material at the periphery of the specimen.

Because of the tendency of soils to be layered, their horizontal coefficients of permeability and consolidation will often be many times greater than in the vertical direction. The ability of the hydraulic oedometer to test with vertical compression and horizontal drainage is a major advantage.

7.7 Permeability tests:

Laboratory determinations of the permeability of granular soils can be made using the constant head and falling head permeability tests.

Cohesive soils can be tested for coefficient of permeability in the laboratory, and indeed it was for this purpose that Terzaghi produced the one-dimensional consolidation theory.

Terzaghi noted that smear on the specimen boundaries greatly affected the measured soil permeability in his permeameter tests, and used an oedometer test in order that all water flow would occur out of the sample. Thus, the coefficient of permeability can be obtained from triaxial or hydraulic consolidation tests since

$$K = C_v m_v \gamma_w$$

Where k = coefficient of permeability, c_v = coefficient of consolidation, m_v = the coefficient of compressibility, and γ_w = density of water.

Where the coefficient of permeability is required with greater accuracy, determinations for clays can be made under constant head gradient either in the triaxial apparatus, or in the hydraulic oedometer.

The specimen can be subjected to a total stress level approximating to that in the ground, and the pore pressures applied at each end of the specimen can be arranged to give an average equal to the field pore pressure.

In this situation the accuracy of the test is very much affected by the differences in effective stress across the specimen. The applied pressure difference should be kept to less than 10% of the average effective stress on the specimen.

Inevitably some changes of effective stress are introduced by these tests, because even if the pressure difference driving the water could be kept very small, the horizontal in situ stress on the specimen could not normally be accurately predicted.

Changes of effective stress at the start of the test introduce consolidation or swelling, or both, and the test must therefore be run until steady flow is achieved.

7.8 Chemical tests:

During site investigation, it is often necessary to carry out laboratory tests to determine the effects of the sub-soil or groundwater on concrete to be placed as foundations.

Chemical tests may also be used to check the soundness of aggregates for concrete or soil cement, to determine if electrolytic corrosion of metals will take place, or simply to act as index tests.

The available chemical tests are:

1. Organic matter content

2. Loss of ignition or ash content

3. Sulphate content of soil and groundwater

4. Carbonate content

5. Chloride content

6. Total dissolved solids

7. pH value

8. Resistivity

9. Redox potential

Aqueous solutions of sulphates will attack the hardened cement in concrete, leading to chemical changes which are associated with a large volume increase. This increase of volume causes cracking and spalling.

If fresh sulphates can readily move to the concrete, the speed at which deterioration takes place will be accelerated. Insoluble sulphates in the ground are not a problem.

The rate at which sulphate attack can occur is a function of the type and concentration of the sulphates, the amount of groundwater movement, the permeability of the concrete, the type of cement and the type of structure in which it is to be used.

Where groundwater will be encouraged to travel along the face of a structure the concrete will be at a higher risk than where groundwater is static. The normal method of avoiding problems due to sulphates is to ensure that the concrete is dense and impermeable, with sufficiently high cement content.

The effect of acid attack on concrete is discussed many researchers . They point out that it is not reasonable to make recommendations for the type of cement or concrete based solely on the knowledge of the pH value of a particular soil.

The risk of acid attack should be assessed from pH data, depth of water table, the likelihood of water movement, the thickness of concrete, and whether it is subject to any hydrostatic head.

Examples of low and high risk conditions are given below.

1. Low risk. pH 5.5—7.0, stiff un-fissured clay soil with water table below foundation level.

2. High risk. pH< 3.5, permeable soil with water table above foundation level and risk of groundwater movement.

In high risk conditions super-sulphated cements or protective impermeable membranes are used.

Sulphate content, chloride content, and organic matter content in aggregate or materials intended for use as soil cement can seriously affect the behaviour of the finished product. A part from any damaging chemical effects these materials have very low strengths in their solid form.

When dispersed throughout the mix, high organic contents in material used for soil cement can interfere with hydration, while chlorides may lead to unsightly efflorescence and in large quantities will attack the steel in reinforced concrete and cause rapid deterioration owing to the spalling of the cover.

In this module, a summary of the laboratory tests is presented. In order to perform each test, respective standards needs to be referred. It can be also be noted that the sample size and testing methods are globally similar; some specific changes based on region data can be found in each country standards. American Society for Testing and Materials (ASTM), is a well accepted standard for many geotechnical field and laboratory experiments. It is suggested that ASTM International can be referred for further reading and testing.