

Geotechnical Testing For Earthwork In Railway Projects

November 2018

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FOREWORD

For carrying out Earthwork in Railway Formations, a number of Geotechnical tests are required to be carried out, which includes laboratory tests as well as field tests. To frame the terms of reference for these tests, to carry out the tests as per provisions of reference standard/code and to present the test results in requisite formats, proper understanding of these tests is necessary not only for the persons in the lab who carry out these tests, but also for the officials executing the earthwork in field.

This book on "Geotechnical Testing for Earthwork in Railway Projects" authored by Shri R. K. Shekhawat (Senior Professor) and Shri Sabyasachi Roy (Senior Instructor) of IRICEN, will serve as a ready reckoner regarding tests to be carried out for executing the Earthwork in Railway Projects.

It is expected that this publication will be useful for the engineers working in lab and also the field engineers executing the earthwork at site.

Suggestions for further improvements, including those related to addition/deletion of any topics, may be sent to IRICEN for consideration in next revision of the book.

November, 2018

Ajay Goyal
Director
IRICEN, Pune.
director@iricen.gov.in

PREFACE

For carrying out earthwork in Railway Projects, in embankment or cutting, various laboratory as well field tests have to be performed, as per the provisions of relevant codes, guidelines and specifications. These tests are required to ascertain the suitability of fill material to be used, need for strengthening or otherwise of the sub-soil and quality control of the work executed at site. These tests are carried out as per relevant Indian Standard and some other standards, when no Indian Standard is available.

Proper understanding or appreciation of these tests is essential for the field engineer for ascertaining as to which tests are required to be carried out, arranging these tests in lab or at site and interpreting the test results. For the engineers performing these test in lab also, proper appreciation of these test procedures is necessary for obtaining reliable test results.

The required test procedures are available in the concerned Indian Standard or other Codes/Standards/Guidelines. But it is very difficult, rather impractical, for every field engineer or lab engineer to possess all these Standards/Codes and use them whenever needed. A compilation of Lab Test Procedures was published by RDSO about 10-12 years back, but this is very old and it does not cover many important tests (e.g. Determination of E_{v2} and Field CBR test etc.). Similarly an IRICEN publication also covers some tests but it does not cover all the tests with requisite details and it is more about Geotechnical Investigation than Geotechnical Testing. To serve as a “ready reckoner” for field engineers and to serve as “laboratory manual”, this book is being published which covers all the tests required to be performed for earthwork in Railway Projects, in embankments as well as cuttings. Objective of Test, Reference Standard, Equipment used for performing the Test, Observations to be recorded, Results to be presented

and General Remarks (if any) have been brought out for each of the tests.

The support and help rendered by Shri B. Ravi Kumar (Senior Instructor/ IRICEN), in proof reading of the book and offering valuable suggestions, is appreciated.

It is felt that this book will be useful to engineers of Indian Railways. However, there is always a scope for improvement in any publication. Therefore, the suggestions for improvement are welcome from all the readers and the same may please be forwarded for incorporation in the future editions.

November, 2018

Sabyasachi Roy
Senior Instructor
IRICEN, Pune
sit10iricen@gmail.com

R. K. Shekhawat
Senior Professor
IRICEN, Pune
shekhawat.rajesh@iricen.gov.in

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Chapter-1

LABORATORY TEST AND PREPARATION OF REPORT

1. **Introduction:** Proper laboratory geotechnical testing is essential part in not only design and construction of railway formation, for embankments as well as for cuttings, but for quality control of the work executed. Hence, proper understanding of requisite laboratory tests is necessary for the laboratory personnel as well as for field officials.
2. **Use of Equipment:** For accurate and reliable test results, the equipment should be properly maintained, including timely calibration of various parts. It is necessary to see that all equipment are cleaned, both before and after use, to obtain proper results.
3. **Recording the Data:** In any test, it is always a good habit to record all data in the proper table/ format immediately after the test has been performed. Oftentimes, scribbles on scratch paper may be illegible later on or even get misplaced, which requires either conducting the test again or getting inaccurate results.
4. **Report Preparation:** The Cover Page of report should include the Title of the Test, Name of Lab and Date on which the test was performed. Following the cover page, the items listed below should be included in the body of the report:
   - Purpose of the experiment
   - Equipment used, with schematic diagram of the main equipment used
   - Brief description of the test procedure
   - Results: This should include the data sheet(s), sample calculations(s) and the required graph(s).
   - Remarks, if any.

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PART – A

DETERMINATION OF INDEX PROPERTIES OF SOIL

(1) Water Content Determination (Chapter – 2)
   (A) Oven Drying Method
   (B) Sand Bath Method
   (C) Rapid Moisture Meter Method

(2) Particle Size Analysis (Chapter – 3)
   (A) Dry and Wet Sieve Analysis
   (B) Hydrometer Test

(3) Determination of Specific Gravity (Chapter – 4)

(4) Determination of Consistency Limits (Chapter – 5)
   (A) Liquid Limit Test
   (B) Plastic Limit Test

(5) Determination of Compaction Characteristics of soil (Chapter - 6)
   (A) Heavy Proctor Test
   (B) Determination of Maximum and Minimum Density

(6) Permeability Test (Chapter - 7)
   (A) Constant Head Test
   (B) Falling Head Test

(7) Determination of Organic Content (Chapter - 8)

(8) Determination of Free Swell Index of Soil (Chapter - 9)

(9) Determination of Swell Pressure of Soil
(Chapter - 10)
(A) Consolidometer Method
(B) Constant Volume Method

Chapter-2

WATER CONTENT DETERMINATION

The water content also called the moisture content is the ratio of the weight of water to the weight of the solids in a given mass of soil. This ratio is usually expressed as percentage.

Water content determination of soil is required for calculating the "Dry Density" of soil, which is used to determine the suitability of the soil for earthwork and also for quality control of the earthwork being executed in the field. There are three common methods to determine the water content of soil, in lab and field, and they are being elaborated in this chapter.

(A) Oven Drying Method

1. **Introduction:** Oven Drying Method is most commonly used laboratory method for determining water/moisture content of the soil mass.
2. **Reference:** IS-2720 (Part-2):1973 (Reaffirmed-2015) "Methods of test for soils: Determination of water content"
3. **Apparatus required:**
   (i) Non-corrodible air-tight container (Fig. 2.01).
   (ii) Electric oven, to maintain the temperature between 105^0 to 115^0 C (Fig. 2.02).



Fig. 2.01



Fig. 2.02

(iii) Desiccator (Fig. 2.03).

(iv) Weighing Balance, of accuracy of 0.04% of the weight of the test sample (Fig. 2.04).



Fig. 2.03



Fig. 2.04

4. Procedure

(i) Clean the container with lid dry it and weigh it (W1).

(ii) Take a specimen of the sample in the container and weigh with lid (W2). The soil specimen shall be representative of the soil mass. The weight of specimen depends on the gradation of the soil to be tested, as given below:

Table 2.01

Size of particles more than 90% passing	Minimum weight of Sample (gm)
425 micron IS Sieve	25
2 mm IS Sieve	50
4.75 mm IS Sieve	200
10 mm IS Sieve	300
20 mm IS Sieve	500
40 mm IS Sieve	1000

(iii) Keep the container in the oven with lid removed. Dry the specimen to constant weight maintaining the temperature 105^0 to 110^0C for a period varying with the type of soil but usually 16 to 24 hours.

(iv) Record the final constant weight (W3) of the container with dried soil sample. Peat and other organic soils are to be dried at lower temperature (say 60^0 to 80^0) possibly for a longer period.

(v) Certain soils contain gypsum which on heating loses its water, if crystallized. If it is suspected that gypsum is present in the soil sample used for moisture content determination, it shall be dried at not more than 80^0 C and possibly for a longer time.

5. Observations and Recording:

Data and observation sheet is as given in Table 2.02 below.

Table 2.02 : Water Content Determination

S.No.	Sample No.	1	2	3
1	Weight of container with lid, W1 gm			
2	Weight of container with lid +wet soil, W2 gm			
3	Weight of container with lid +dry soil, W3 gm			
4	Water/Moisture content % w=[(W2-W3)/(W3-W1)] X 100			

6. Interpretation and Reporting of Result :

The natural moisture content of the soil sample is _____ %. The result should be reported to two significant figures.

7. General Remarks :

(i) A container without lid can also be used, when moist sample is weighed immediately after placing the container and oven dried sample is weighed immediately after cooling in desiccator.

(ii) As dry soil absorbs moisture from wet soil, dried samples should be removed before placing wet samples in the oven.

(iii) Most natural soils, which are sandy and gravelly in nature, may have water contents up to about 15 to 20%. In natural fine-grained (silty or clayey) soils, water contents up to about 50 to 80% can be found. However, peat and highly organic soils with water contents up to about 500% are not uncommon.

(B) Sand Bath Method

1. Introduction: This method is intended as a rapid alternative to the "Oven Drying Method". But it is less accurate and more suitable as a field test.

2. Reference: IS-2720 (Part-2):1973 (Reaffirmed-2015) "Methods of test for soils: Determination of water content".

3. Apparatus required:

(i) Container: any suitable non-corrodible air tight container.

(ii) Heat resistant tray: of suitable metal and about 5 to 7cm deep.

(iii) Weigh balance: of sufficient sensitivity to weigh the soil samples to an accuracy of 0.04% of the weight of the sample.

(iv) Sand bath: of suitable size and containing clean sand to a depth of at least 3 cm.

(v) Equipment for heating the sand bath: Kerosene stove or Spirit lamp.

(vi) Palette Knife or Steel Spatula: having a blade 10cm long and 2cm wide.

(vii) Scoop: about 20cm long and 10cm wide.

4. Test Procedure

(i) Take soil sample of mass as given in Table-2.01.

(ii) Clean the container with lid or the tray, as the case may be, dry and weigh (W1).

(iii) Take the required quantity of the soil specimen in the container crumbled and placed loosely and weigh (W2). Add a few pieces of white paper if necessary. Overheating of sample should be avoided and a convenient method of detecting overheating of the soil is by the use of small pieces of white paper mixed with the soil. Overheating is indicated if the paper turns brown.

(iv) Place the container with the lid removed on the sand bath and heat the sand bath. Care shall be taken not to get the sand bath too hot. During heating, the specimen shall be turned frequently and thoroughly with the

palette knife to assist the evaporation of water, care being taken to see that no soil is lost in the process. Dry the specimen to constant mass indicated by the difference between two consecutive masses of the container with lid with the dried specimen taken at suitable intervals after initial drying, being a maximum of 0.1 percent of the original mass of the soil specimen.

(v) When drying is complete, remove the container or the tray from the sand bath, cool and weigh (W3). The container should be weighed with lid.

5. **Observations and Recording:** Data and observation sheet is same as given in Table 2.02 above.

6. **Interpretation and Reporting Result:** The natural moisture content of the soil sample is _____ %. The result should be reported to two significant figures.

7. **General Remarks:** This method shall not be used if it is suspected that the soil contains a large proportion of gypsum calcareous matter or organic matter.

(C) Rapid Moisture Meter Method

1. **Introduction:** Water content determination generally takes 16 to 24 hours by oven drying method. To quickly determine the water content at site, use of Rapid Moisture Meter is very advantageous. In this method, water content is determined from the gas pressure developed by the reaction of calcium carbide with free water of the soil. From the calibrated scale of the pressure gauge, the percentage of water on total (wet) mass of soil is obtained and the same is converted to water content on dry mass of soil.

2. **Reference:** IS-2720 (Part-2):1973 (Reaf-firmed-2015) "Methods of test for soils: Determination of water content.

3. **Apparatus required (Fig. 2.05)**



Fig. 2.05: Apparatus for Rapid Moisture Determination

(i) Metallic pressure vessel, with a clamp for sealing the cup, along with a gauge calibrated in percentage water content; as per IS-12175:1987 (Reaffirmed-Dec'2011) "Specification for rapid moisture meter for rapid determination of water content for soil".

(ii) Counterpoised balance, for weighing the sample

(iii) Scoop, for measuring the absorbent (Calcium Carbide)

(iv) Steel balls: 3 steel balls of about 12.5mm diameter and 1 steel ball of 25mm diameter

(v) One bottle of the absorbent (Calcium Carbide)

(vi) Cleaning Brush

4. Procedure:

(i) Preparation of Soil Sample: Sand requires no special preparation needed. Coarse powders may be ground and pulverized. Cohesive and plastic soil is tested with addition of steel ball in the pressure vessels. The test requires about 6g of soil sample.

(ii) Set up the balance, place the soil sample in the pan till the mark matches with the index mark.

(iii) Hold the body of the moisture meter horizontally and gently deposit the levelled, scoop-full of the absorbent (Calcium Carbide) inside the chamber. Transfer the weighed soil to the cup.

(iv) Hold cup and chamber horizontally, bringing them together without disturbing the sample and the absorbent.

(v) Clamp the cup tightly into place. If the sample is bulky, reverse the above placement, that is, put the sample in the chamber and the absorbent in the cup.

(vi) In case of clayey soils, place all the 4 steel balls (3 smaller balls in the cup along with the soil and 1 bigger ball in the body along with the absorbent) in the body along with the absorbent.

(vii) Shake the unit up and down vigorously in this position for about 5 seconds.

(viii) Hold the unit horizontally, rotating it for 10 seconds, so that the balls roll around the inner circumference of the body.

(ix) Rest for 20 seconds. Repeat the above cycle until the pressure gauge reading is constant and note the reading. Usually it takes 4 to 8 minutes to achieve constant reading. This is the water content (m) obtained on wet mass basis.

(x) Finally, release the pressure slowly by opening the clamp screw and taking the cup out, empty the contents and clean the instrument with a brush.

5. Observations and Recording : From the water content (m) obtained on the wet mass basis as the reading on the rapid moisture meter, the water content (w) on the dry mass basis shall be calculated as follows:

$$w = [m / (100-m)] \times 100\ \%$$

6. Interpretation and Reporting of Result: The natural moisture content of the soil sample is _____ %. The result should be reported to two significant figures.

7. General Remarks: The absorbent is highly susceptible to absorption of moisture and so shall not be exposed to atmosphere; as a result the absorbent suffers deterioration and will give results to the lower side. Replace the lid of the absorbent container firmly at soon as the required amount of the absorbent for a test is taken from the bottle. The absorbent suffers deterioration with time.

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Chapter-3

PARTICLE SIZE ANALYSIS

Particle Size Analysis or Grain Size Analysis is done to find out the proportion of various standard sizes in the soil mass being tested. This information is used to find out whether the soil mass is “well graded” or “poorly graded” and for the purpose of “Soil Classification” also. This information is in-turn used to determine suitability of fill material soil for embankment or need of strengthening or otherwise in case of sub-soils. Particle Size Analysis for sizes of 75 micron and above is determined by “Sieve Analysis” and for finer size particles “Hydrometer Test” is conducted.

(A) Dry and Wet Sieve Analysis

1. **Introduction :** Dry and Wet Sieve Analysis is carried out to quantitatively determine the Particle/Grain Size Distribution for soil particles of size 75 micron and bigger. For soil particles of size 4.75mm and bigger, dry sieve analysis is done and for soil particles of size above 75 micron and below 4.75mm, wet sieve analysis is also needed if the soil particles are coated by clay/silt.

2. **Reference :** IS-2720 (Part-4):1985 (Reaffirmed-2015) “Methods of test for soils: Grain size analysis”.

3. **Apparatus required:**

   (i) Weigh Balance: sensitive to 0.1% weight of sample to be weighted (Fig. 3.01).

   (ii) Sieves – IS sieves of Size 100mm, 75mm, 19mm, 2mm, 425 micron and 75 micron IS Sieve); conforming to IS:460-1978 (Fig. 3.02).

(iii) Mechanical Sieve Shaker (Fig. 3.03).

(iv) Oven: Thermostatically controlled to maintain the temperature between 105 and 110^0 C.

(v) Trays or Bucket: two or more large metal or plastic water tight trays or a bucket about 30 cm in diameter and 30 cm deep (sizes of the trays are in the range of 45 to 90 cm^2 and 8 to 15cm deep).



Fig. 3.01



Fig. 3.02



Fig. 3.03

4. Procedure:

(i) The soil fractions retained on and passing 4.75mm Sieve shall be taken separately for the analysis.

(ii) Weigh the portion of the soil sample retained on 4.75mm Sieve and record the mass. The quantity of the soil sample taken shall be as under:

Table 3.01: Weight of Soil Sample

Maximum Size of Material Present in Substantial Quantities	Weight of Soil Sample (in kg)
75 mm	60
40 mm	25
25 mm	13

19 mm	6.5
12.5 mm	3.5
10 mm	1.5
6.5 mm	0.75
4.75 mm	0.4

Table 3.02: Maximum Weight Retained on Sieves

IS Sieve Designation	450mm Dia Sieves (in kg)	300m Dia Sieves (in kg)
80 mm	15	6
20 mm	4	2
4.75 mm	1.0	0.5
2 mm	-	0.20
425 micron	-	0.050
75 micron	-	0.025

(iii) Sieve the sample through the sieve net of size 4.75mm and higher. Other sieves may be introduced in between the sieves, depending upon the additional information desired to be obtained. The sieves shall be agitated and any particles may be tested to see if they will fall through, but they shall not be pushed through. The quantity taken each time for sieving shall be such that the maximum weight retained on each sieve does not exceed the values given in Table 3.02.

(iv) Record the mass retained on each sieve. If the sample appears to contain over 5% moisture, the water content shall be determined and the masses corrected accordingly. If the soil contains more

than about 20% gravel particles and the fines are very cohesive with considerable amounts adhering to the gravel after separation, the gravel shall be washed on 4.75mm Sieve using sodium hexametaphosphate solution.

(v) The portion of soil passing 4.75mm Sieve shall be oven-dried at 105 to 115°C. Weigh this soil mass to accuracy of 0.1% of its total mass and record the mass. Spread out this material in large tray or bucket and cover with water.

(vi) Wash the soaked soil specimen on 75 micron sieve until the water passing is substantially clean. The fraction retained on the sieve should be tipped in a try, dried in the oven and sieved through the nest of 2mm, 425 micron and 75 micron sieves. The fraction retained on each sieve should be weighed separately and the masses recorded.

5. Observations and Recording:

I. S. Sieve Size	Weight Retained on each sieve (gm)	Percentage weight Retained (gm)	Cumulative % Retained	% Finer
100 mm				
75 mm				
19 mm				
4.75 mm				
2 mm				
425 micron				
75 micron				

6. Calculation:

(i) The percentage of soil retained on each sieve shall be calculated on the basis of total weight of soil sample taken. Then, cumulative percentage of soil retained (which will give % finer when deducted from 100) on successive sieve is found.

(ii) Plot the sieve size on log scale v/s % finer. The graph is known as grading curve (Fig. 3.04). Corresponding to 10%, 30% and 60% finer, obtain diameters from graph, which are designated as D_{10}, D_{30} and D_{60}.

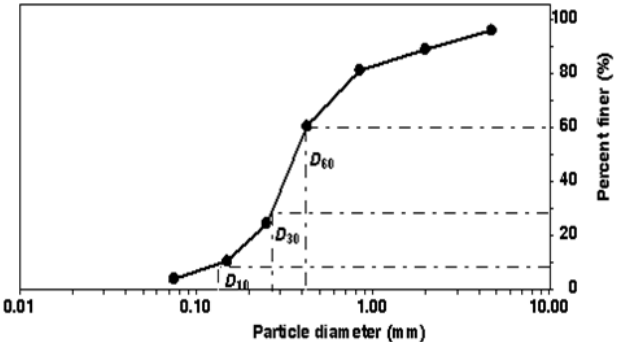


Fig. 3.04: Grading Curve

(iii) Calculate the Coefficient of uniformity coefficient (C_u) and the Coefficient of Curvature (C_c) using the following equations:

$$C_u = D_{60}/ D_{10} \ , \ C_c = D_{30}^2/ (D_{60} \ X \ D_{10})$$

7. General Remarks : In the case of clayey soils, dry sieving shall not be used and wet sieving shall be used.

(B) Hydrometer Test

1. **Introduction:** Hydrometer test is carried out to quantitatively determine the Particle/Grain Size Distribution for soil particles of size smaller than 75 micron.
2. **Reference:** IS-2720 (Part-4):1985 (Reaffirmed - 2015) "Methods of test for soils: Grain size analysis".
3. **Apparatus:**

   (i) Hydrometer: calibrated at 27°C, range of 0.995 to 1.030 g/cc, graduation lines at interval of 0.0005; in conformity with IS-3104: 1965 (Fig. 3.05).

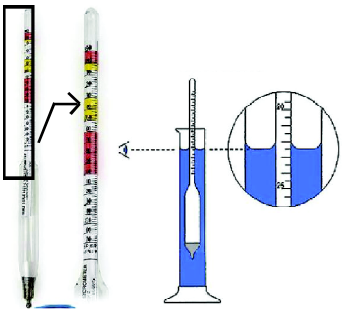


Fig. 3.05: Hydrometer



Fig. 3.06: Graduated Glass Cylinders



Fig. 3.07



Fig. 3.08

(ii) Dispersing agent solution containing sodium hexa- metaphosphate.

(iii) Evaporating dish.

(iv) Thermometer- with accuracy of 0.5^0 C (Fig. 3.07).

(v) Stopwatch.

(vi) Mechanical stirrer (Fig. 3.08).

4. Procedure

4.1 Calibration of Hydrometer

(i) *Determination of volume of the hydrometer bulb (V_h):* Pour about 800 ml of water in the 1000 ml measuring cylinder and note the reading at the water level. Immerse the hydrometer in water and note the water reading. The difference between the two readings is recorded as the volume of the Hydrometer bulb plus the volume of that part of the stem which is submerged. For practical purpose the error due to the inclusion of this stem volume may be neglected.

Alternatively, weigh the hydrometer the nearest 0.2g. This mass in grams is re-

corded as the volume of the hydrometer in ml. This includes the volume of the bulb plus the volume of the stem. For practical purposes the error due to the inclusion of the stem may be neglected.

(ii) In order to find the area of cross-section (A) of the measuring cylinder in which the hydrometer is be used, measure the distance, in cm, between two graduations of the cylinder. The cross-sectional area (A) is then equal to the volume included between the two graduations divided by the distance between them.

(iii) Measure the distance (h) from the neck to the bottom of the bulb, and record it as the height of the bulb.

(iv) With the help of an accurate scale, measure the height between the neck of the hydrometer to each of the other major calibration marks (R_h).

(v) Calculate the effective depth (H_e) corresponding to each of the calibration marks (or hydrometer readings, R_h) by the following expressions:

$$H_e = H + 0.5 (h - V_h/A)$$

The readings may be recorded as illustrated in Table 3.04:

Table 3.04: Calibration of Hydrometer

Hydrometer Reading, R_h	H (cm)	Effective depth, H_e (cm)

(vi) Draw a calibration curve between H_e and R_h (Fig. 3.09) which may be used for finding the effective depth (H_e) corresponding to hydrometer readings (R_h) during test.

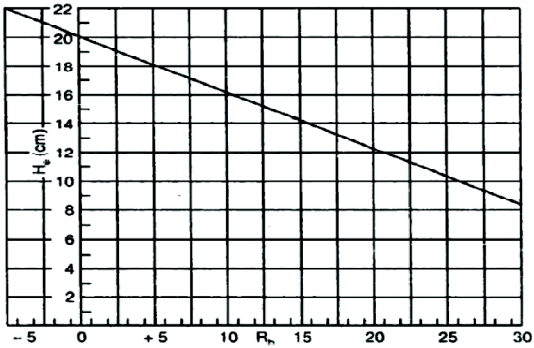


Fig. 3.09: Calibration Curve for Hydrometer

(vii) *Meniscus correction:* Insert the hydrometer in the measuring cylinder containing about 700 ml of water. Take the readings of the hydrometer at the top and bottom of the meniscus. The difference between two readings is taken as meniscus correction (C_M) which is a constant for a hydrometer.During the actual sedimentation test, the readings should be taken at the bottom of the meniscus but since the soil suspension is opaque, readings are taken at the top of meniscus, It is clear that readings decrease in the upward direction. Thus, the observed hydrometer readings are always less than the true one. Hence the meniscus correction is always positive.

4.2 Dispersion of soil

(i) Weigh 50 to 100g of oven-dried soil sample (M_d) passing the 2 mm IS Sieve (50g for clay soil and 100g if

it is a sandy soil). Add 150 ml of hydrogen peroxide to the soil sample placed in a wide mouth conical flask and stir it gently for few minutes with a glass rod. Cover the flask with glass and level it to stand overnight.

(ii) Next morning, the mixture in the conical flask is gently heated in an evaporating dish, stirring the contents periodically. Reduce the volume to 50 ml by boiling. Place the dish and its contents to the oven.

(iii) To the oven-dried soil in the evaporating dish, add 100 ml of sodium hexametaphosphate solution and warm the mixture gently for about 10 minutes. Transfer the mixture to the cup of the mechanical mixer using a jet of distilled water and stir it well for about 15 minutes. Transfer the soil suspension to the 75 micron IS sieve placed on a receiver and wash the soil on this sieve using jet of distilled water from a wash bottle. The amount of distilled water used during this operation may be about 500 ml.

(iv) Transfer the soil suspension passing the 75 micron IS Sieve to the 1000 ml measuring cylinder, and add distilled water to make the volume to exactly 1000 ml in the cylinder

(v) Collect the material retained 75 micron sieve and put it in the Oven for drying. Determine the dry mass of soil retained on 75 micron sieve.

4.3 Sedimentation test with hydrometer

(i) Weigh 50 to 100g of oven-dried soil samp

(i) Insert a rubber ball or any other suitable cover on the top of the 1000 ml measuring cylinder containing the soil suspension and shake it vigorously end over end. Stop shaking and allow it to stand. Immediately, start the stop watch, and remove the top cover from the cylinder.

(ii) Immerse the hydrometer gently to a depth slightly below its floating position and then allow it to float freely. Take the hydrometer readings after periods of 1/2, 1, 2 and 4 minutes. Take out the hydrometer, rinse it with distilled water and allow it to stand in a jar containing distilled water at the same temperature as that of the test cylinder.

(iii) The hydrometer is re-inserted in the suspension and readings are taken after periods of 8, 15 and 30 minutes; 1, 2 and 4 hours after shaking. The hydrometer should be removed, rinsed and placed in the distilled water after each reading. After end of 4 hours, readings should be taken once or twice within 24 hours.

(iv) *Composite correction:* In order to determine the composite correction, put 100 ml of dispersing agent solution in another 1000 ml measuring cylinder and make it to 1000 ml by adding distilled water. The cylinder should be maintained at the same temperature as that of the test cylinder containing soil specimen. Insert the hydrometer

in this comparison cylinder containing distilled water and the dispersing agent and take the reading corresponding to the top of the meniscus. The negative of the hydrometer reading so obtained gives the composite correction (C). The composite correction is found before the start of the test, and also at every time intervals of 30 minutes, 1 hour, 2 hours and 4 hours after the beginning of the test, and afterwards, just after each hydrometer reading is taken in test cylinder.

(v) The temperature of the suspension should be observed and recorded once during the first 15 minutes and then after every subsequent reading.

5. Observations

5.1 Tabulation of observations: The test observations and results are recorded as illustrated in Table 3.05.

5.2 Particle size distribution curve: The results of the Table 3.05 are plotted to get a particle size distribution curve with "percentage finer" (N) as the ordinate and the "particle diameter" (D) on logarithmic scale as abscissa.

6. Calculations :

(i) The loss in mass in pre-treatment of the soil in percentage is calculated from the following expression:

$$P = (1 - M_b/M_d) * 100$$

Where,

P = loss in mass in percentage

M_d = mass of dry soil sample taken from the soil passing 2 mm sieve

M_b = mass of the soil after pre-treatment

(ii) The diameter of the particles in suspension at any sampling time 't' is calculated from equation below:

$$D = 10^{-5} * F * (H_e/t)^{0.5}$$

Where, F is taken from Table 3.06.

(iii) The percentage finer N' based on the mass M_d is calculated from the equation given below:

$$N' = [(100*G) / M_d*(G-1)]*100$$

(iv) The percentage finer "N" based on total mass of dry soil sample (M) is obtained from the relation:

$$N = N' *(M'/M)$$

Where,

M' = cumulative mass passing 2mm sieve (out of which the soil weighing M_d was taken for the hydrometer analysis).

7. General Remarks

(i) Sodium hexametaphosphate has been found to ineffective when dealing with certain highly flocculated soils. In such cases dispersion may be carried out by adding N-sodium hydroxide solution at the rate of 4 ml per 10 g of soil.

(ii) The suspension should be kept out of direct sunlight and away from any local source of heat. Evaporation should be retarded by keeping a cover on the measuring cylinder between the readings.

(iii) The specific gravity should he determined for the fraction of the sample passing 75 micron sieve.

(iv) This method shall not applicable if less than 10% of the material passes the 75 micron IS Sieve.

Sample No. :
Mass of Dry Sample (M): **Mass of Fraction passing 75 micron sieve (M'):**
Specific Gravity of Soil particles of 75 micron sieve (G):
Hydrometer Number: **Meniscus Correction (Cm): +0.5**

Table: 3.05: Data and Observation sheet for Hydrometer analysis

Elapsed Time	Hydrometer Reading R_h	Temp, °C	Correction, C	$R_h = R_h' + C$	Eff. Depth H_e (cm)	Factor M	Particle size, D (mm)	$R_h = R_h' + C$	% finer (N') based on M_d	% finer based on whole N
0.5	20.25	26.5	-0.5	20.75	12.2	1254	0.061	19.75	63.0	58.0
1	19.00	26.5	-0.5	19.50	12.7	1254	0.045	18.50	59.0	54.3
2	17.50	26.5	-0.5	18.00	13.3	1254	0.033	17.00	54.4	50.0
4	15.50	26.5	-0.5	16.00	14.0	1254	0.023	15.00	48.0	44.1
8	15.50	26.5	-0.5	16.00	14.0	1254	0.165	15.00	48.0	44.1
15	11.50	27.0	-0.5	12.00	15.5	1252	0.0125	11.00	35.0	32.2
30	9.50	27.5	-0.5	10.00	16.3	1245	0.0092	9.00	29.0	26.7
60	7.25	29.0	0.0	7.75	17.1	1224	0.0062	7.25	23.0	21.2
120	6.00	30.0	+0.5	6.50	17.6	1210	0.0047	6.50	21.0	19.3
240	4.00	31.5	+1.0	4.50	18.3	1190	0.0033	5.00	16.0	14.7
480	2.25	31.5	+2.0	2.75	19.0	1190	0.0023	3.25	10.4	9.6
960	2.25	29.0	0.0	2.75	19.0	1224	0.0015	2.25	6.5	5.8

Table 3.06: Values of "F"

Temp ^0C	Factor "F"					
	G=2.50	G=2.60	G=2.65	G=2.70	G=2.75	G=2.80
15	1528	1479	1458	1435	1414	1395
16	1508	1460	1438	1417	1396	1377
17	1490	1442	1420	1399	1379	1360
18	1470	1423	1402	1381	1361	1342
19	1452	1406	1384	1364	1344	1325
20	1434	1389	1367	1347	1328	1309
21	1417	1372	1351	1331	1311	1294
22	1400	1355	1335	1315	1296	1278
23	1383	1339	1318	1291	1280	1262
24	1367	1323	1305	1284	1265	1248
25	1351	1308	1288	1269	1251	1233
26	1335	1293	1273	1254	1236	1219
27	1321	1279	1259	1241	1224	1206
28	1305	1264	1244	1226	1208	1191
29	1291	1250	1231	1213	1195	1178
30	1277	1236	1217	1199	1182	1165
31	1264	1224	1205	1187	1170	1154
32	1251	1211	1193	1175	1158	1142
33	1238	1199	1180	1163	1146	1130
34	1225	1186	1168	1151	1134	1118
35	1212	1174	1156	1139	1120	1107
36	1199	1161	1144	1127	1110	1095
37	1188	1150	1133	1116	1100	1084
38	1176	1139	1122	1105	1089	1074
39	1165	1128	1110	1094	1079	1064
40	1155	1111	1101	1084	1069	1055

NOTE: For intermediate values interpolation may be done.

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Chapter-4

DETERMINATION OF SPECIFIC GRAVITY

Specific Gravity is defined as the ratio of the weight of soil to the weight of equal volume of distilled water, at that temperature, both weights taken in air. The specific gravity of the soil particles lie within the range of 2.65 to 2.85. Soils containing organic matter and porous particles may have specific gravity values below 2.0. Soils having heavy substances may have values above 3.0.

1. **Introduction:** The knowledge of specific gravity is needed in calculation of soil properties like void ratio, degree of saturation, particle size analysis by hydrometer test etc. The value of specific gravity is sometimes used to determine the suitability of the soil as fill material.
2. **Reference:** IS-2720 (Part-3):1980 (Reaffirmed-2016) "Methods of test for soils: Determination of Specific Gravity, Section-1, Fine Grained Soils".
3. **Apparatus required:**



Fig. 4.01



Fig. 4.02

(i) Two Density bottles (pycnometers) of 50ml with stopper, having capillary hole (Fig. 4.01).

(ii) A water-bath maintained at a constant temperature of 27 ± 0.2^0 C.

(iii) Balance to weigh the materials (accuracy 0.001 g).

(iv) Wash bottle with distilled water (Fig. 4.02).

(v) Vacuum desiccator.

(vi) A thermostatically controlled drying oven, capable of maintaining temperature of 105 to 110°C.

4. Procedure

(i) Clean and dry the density bottle. Wash the bottle with water and allow it to drain. Wash it with alcohol and drain it to remove water. Wash it with ether, to remove alcohol and drain ether. Weigh the empty bottle with stopper (W_1)

(ii) Take about 5 to 10 gm of oven dried soil sample which is cooled in a desiccator. Transfer it to the bottle. Find the weight of the bottle and soil (W_2).

(iii) Put 10ml of distilled water in the bottle to allow the soil to soak completely. Leave it for about 2 hours.

(iv) Again fill the bottle completely with distilled water put the stopper and keep the bottle under constant temperature water baths (T_x^0).Take the bottle outside and wipe it clean and dry note. Now determine the weight of the bottle and the contents (W_3).

(v) Now empty the bottle and thoroughly clean it. Fill the bottle with only distilled

water and weigh it. Let it be W_4 at temperature (T_x^0 C).

(vi) Repeat the same process for 2 to 3 times.

5. Observations

Table 4.01

S.No.		1	2	3
1	Weight of density bottle (W_1, g)			
2	Weight of density bottle + dry soil (W_2, g)			
3	Weight of bottle + dry soil + water at temperature T_x^0C (W_3, g)			
4	Weight of bottle + water (W_4, g) at temperature T_x^0C			
	Specific Gravity G at T_x^0C			
	Average Specific Gravity at T_x^0C			

6. Calculations:

Specific gravity of Soil =

$$\frac{\text{Weight of soil}}{\text{Weight of water of equal volume at } 27^0 \text{ C}}$$

$= (W_2 - W_1)/[(W_4 - W_1) - (W_3 - W_2)]$

$= (W_2 - W_1)/[(W_2 - W_1) - (W_3 - W_4)]$

7. **Reporting:** Unless or otherwise specified, specific gravity values reported shall be based on water at 27^{0}C. The average of the values obtained shall be as the specific gravity to the nearest 0.01. If the two results differ by more than 0.03 the tests shall be repeated.

Specific Gravity at 27^0C = K x Specific Gravity at $T_x{}^0$C

Where, K = (Density of water at $T_x{}^0$ C)/(Density of water at 27^0 C)

Table 4.02 : Values of "K"

Temperature $T_x{}^0$C	K_{27}	Temperature $T_x{}^0$C	K_{27}
15	1.0026	28	0.9997
16	1.0024	29	0.9994
17	1.0023	30	0.9991
18	1.0021	31	0.9988
19	1.0019	32	0.9985
20	1.0017	33	0.9982
21	1.0015	34	0.9979
22	1.0013	35	0.9975
23	1.0010	36	0.9972
24	1.0008	37	0.9968
25	1.0005	38	0.9964
26	1.0003	39	0.9961
27	1.0000	40	0.9957

8. General Remarks: The method may also be used for medium and coarse grained soils if the coarse particles are grained to pass 4.75mm IS sieve before using.

Chapter-5

DETERMINATION OF CONSISTENCY LIMITS

The Consistency limits for soil (also known as Atterberg limits) are a basic measure of the critical water contents of a fine-grained soil. Depending on its water content, a soil may appear in one of four states i.e. solid, semi-solid, plastic and liquid. In each state, the consistency and behavior of a soil is different and consequently so are its engineering properties. Thus, the boundary between each state can be defined based on a change in the soil's behavior.

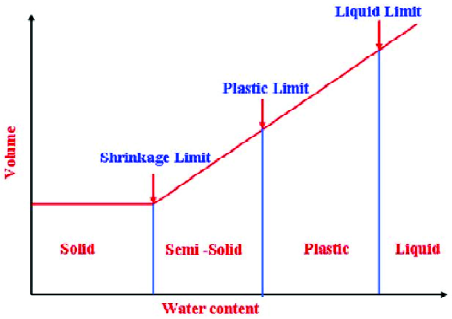


Fig. 5.01: Consistency Limits

(A) Liquid Limit Test: The liquid limit is the moisture content at which the groove, formed by a standard tool into the sample of soil taken in the standard cup, closes for 10mm on being given 25 blows in a standard manner. At this limit the soil possess low shear strength.

1. **Introduction:** From liquid limit, the compression index may be estimated, which is used in settlement analysis. If the

natural moisture content of soil is lesser than liquids limit, the soil can be considered as soft and if the moisture content is lesser than liquid limit, the soil is brittle and stiffer. The value of liquid limit is used in classification of the soil and it gives an idea about plasticity of the soil.

2. **Reference:** IS-2720 (Part-5):1985 (Reaffirmed-2015) "Methods of test for soils: Determination of Liquid and Plastic Limit".

3. **Apparatus required:**

(i) Grooving Tool (Fig. 5.02): As per IS:9259-1979.

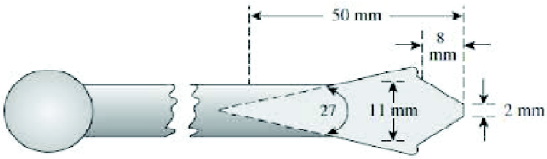


Fig. 5.02



Fig. 5.03



Fig. 5.04

(ii) Balance: sensitivity 0.01g (Fig. 5.03)

(iii) Mechanical Liquid limit device (Fig. 5.04): As per IS:9259-1979.

(iv) Porcelain evaporating dish: 12 to 15 cm in diameter

(v) Spatula

(vi) Electrical Oven

4. Procedure

(i) Take 120g sample of soil from thoroughly mixed portion of material passing 425 micron I.S. sieve. Mix distilled water to the soil, in a mixing disc, to form uniform paste. The paste shall have a consistency that would require 30 to 35 drops of cup to cause closer of standard groove for sufficient length.

(ii) Place a portion of the paste in the cup of Liquid Limit Device and spread into position, with few strokes of spatula. Trim it to a depth of 1cm at the point of maximum thickness and return excess of soil to the dish.

(iii) Divide the soil in the cup by firm strokes of the grooving tool, along the diameter through the centre line of the cam follower, so that clean sharp groove of proper dimension is formed.

(iv) Lift and drop the cup by turning crank at the rate of two revolutions per second, until the two halves of soil cake come in contact with each other for a length of about 12 mm by flow only. Record the number of blows required to cause the groove close for about 12 mm.

(v) Take a representative portion of soil from the cup, for water content determination.

(vi) Repeat the test with different moisture contents at least three more times for blows between 15 and 35.

5. Observations

Test Number	1	2	3	4
Container Number				
Weight of container				
Weight of container + wet soil				
Weight of container + dry soil				
Weight of water				
Weight of dry soil				
Moisture content (%)				
No. of blows				

6. Computation / Calculation

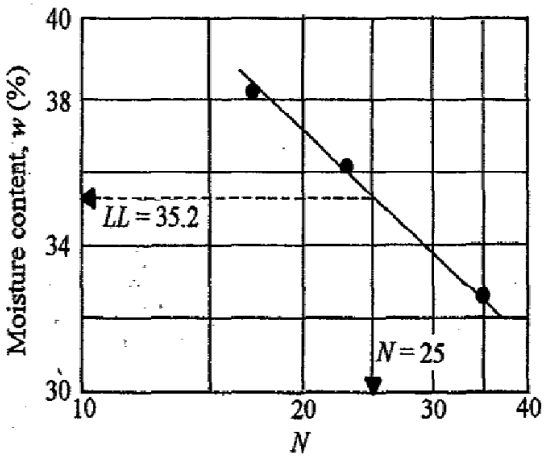


Fig. 5.05: Flow Curve

Draw a graph (Fig. 5.05) between moisture content (on y-axis) and number of blows "N" (on x-axis) on semi-log graph. The curve obtained is called flow curve. The moisture content corresponding to 25 drops (blows) represents "liquid limit". It is usually expressed to the nearest whole number.

Flow Index I_f = $(W_2-W_1)/ (\log N_1/N_2)$ = Slope of Flow Curve.

Where,

W_1 = moisture content in percent corresponding to N_1 drops, and

W_2 = moisture content in percent corresponding to N_2 drops.

7. **General Remarks :** The history of the soil sample, that is, natural state, air-dried, oven dried or unknown, the method used for the test reported and the period of soaking allowed after mixing of water to the soil shall also be reported.

(B) Plastic Limit Test : Plastic Limit of soil is the moisture content, expressed as percentage of the weight of the oven-dry soil, at the boundary between the plastic and semisolid states of consistency. It is the moisture content at which a soil will just begin to crumble when rolled into a thread 1/8 inch (3mm) diameter.

1. **Introduction:** Determination of Plastic Limit is as important as Liquid Limit, to ascertain Plasticity Index of the soil. The value of plastic limit is used in classification of the soil and it gives an idea about plasticity of the soil.
2. **Reference:** IS-2720 (Part-5) : 1985 (Reaffirmed-2015) “Methods of test for soils: Determination of Liquid and Plastic limit”.
3. **Apparatus required:**



Fig. 5.06



Fig. 5.07

(i) Porcelain dish: about 12cm in diameter (Fig. 5.06)

(ii) Glass plate: 10mm thick and 45cm square or larger (Fig. 5.06)

(iii) Air tight containers: to determine moisture content

(iv) Balance: Sensitive to 0.01gm

(v) Oven: thermostatically controlled with interior of non-corroding material to maintain the temperature around 105^{0}C and 110^{0}C (Fig. 5.07)

(vi) Rod: 3 mm in diameter and about 10 cm long. (Fig. 5.06)

4. Procedure:

(i) Take about 20gm of thoroughly mixed portion of the material passing through 425 micron I.S. sieve. Mix it thoroughly with distilled water in the evaporating dish till the soil mass becomes plastic enough to be easily molded with fingers. In case of clayey soils, allow it to season for sufficient time (for 24 hours) to allow water to permeate throughout the soil mass.

(ii) Take about 8gm of this plastic soil mass and roll it between fingers and glass plate with just sufficient pressure to roll the mass into a threaded of uniform diameter throughout its length. The rate of rolling shall be between 80 and 90 strokes per minute. Continue rolling till you get a threaded of 3mm diameter. Compare the diameter of thread at intervals with the rod. When the diameter reduces to 3mm, note the surface of the thread for cracks.

(iii) Kneed the soil together to a uniform mass and re-roll.

(iv) Continue the process until the thread crumbles when the diameter is 3mm. Collect the pieces of the crumbled thread in air tight container for moisture content determination.

(v) Repeat the test to at least 3 times and take the average of the results calculated to the nearest whole number.

5. Observation and Reporting:

Container No.			
Wt. of container + lid, W_1			
Wt. of container + lid + wet sample, W_2			
Wt. of container + lid dry sample, W_3			
Wt. of dry sample = $W_3 - W_1$			
Wt. of water in the soil = $W_2 - W_3$			
Water content (%) = $[(W_2 - W_3) / (W_3 - W_1)] \times 100$			

Average Plastic Limit=.....

6. Interpretation and Recording:

Plasticity Index, I_p = Liquid Limit – Plastic Limit =

Toughness Index = I_p/I_f =

7. General Remarks: In the case of sandy soils plastic limit should be determined first. When plastic limit cannot be determined, the plasticity index should be reported as NP (non-plastic).

When the plastic limit is equal to or greater than the liquid limit, the plasticity index shall be reported as zero.

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Chapter-6

DETERMINATION OF COMPACTION CHARACTERISTICS OF SOIL

For verifying the quality of compaction carried out in the field, it is necessary to determine the compaction characteristics of the soil in the laboratory, which involves maximum achievable dry density in the field and the moisture content at which this will be achieved.

(A) Modified Proctor Test: The Proctor compaction test is a laboratory method of experimentally determining the Optimum Moisture Content (OMC) at which a given soil type will become most dense and achieve its Maximum Dry Density (MDD).

1. **Introduction:** This test is performed to obtain the graphical relationship of the "dry density' to "moisture content" in the form of "compaction curve', for determining the values of Optimum Moisture Content (OMC) and Maximum Dry Density (MDD).
2. **Reference:** IS-2720(Part-8):1983 (Reaffirmed -'2015) "Methods of test for soils: Determination of water content-dry density relation using heavy compaction".
3. **Apparatus required:**



Fig. 6.01



Fig. 6.02

(i) Cylindrical Metal Mould: either of 100mm diameter and 1000 cm^3 volume or 150mm diameter and 2250 cm^3 volume, as per IS:10074-1982 (Fig. 6.01)

(ii) Balance: one of capacity 10 kg and least count 1g and other of 200 g capacity and sensitivity 0.01 g.

(iii) Oven: thermostatically controlled to maintain temperature between 105^0 to 110^0C (Fig. 6.02)

(iv) Metal Rammer: weight 4.89 kg (Fig. 6.01)

(v) Sieve: 19 mm, 4.75mm and 37.5 mm I.S. Sieves conforming to IS: 460 (Part 1) – 1985.

(vi) Steel Straightedge - about 30 cm in length and having one beveled edge.

4. Procedure

(i) Take a representative portion of air-dried soil large enough to provide about 6 kg of material passing 19mm IS sieve (for soils not susceptible to crushing during compaction) or about 15 kg of material passing 19mm IS sieve (for soils susceptible to crushing during compaction. Sieve this on a 19mm IS sieve and the reject the coarse fraction after its proportion of the total sample has been recorded.

(ii) Add suitable amount of water with the soil and mix it thoroughly. For sandy and gravelly soil add 3% to 5% of water. For cohesive soil the amount of water to be added should be 12% to 16% below the plastic limit.

(iii) Weigh the mould with base plate attached, to the nearest 1g and record the weight as W_1. Attach the extension collar with the mould. Compact the moist soil into the mould in five layers of approximately equal mass, each layer being given 25 blows, with the help of 4.89 kg rammer, dropped from a height of 450mm above the soil. The blows must be distributed uniformly over the surface of each layer.

(iv) After completion of the compaction operation, remove the extension collar and level carefully the top of the mould by means of straight edge. Weigh the mould with the compacted soil to the nearest 1g and record this weight as W_2.

(v) Remove the compacted soil from the mould and place it on the mixing tray. Determine the water content of a representative sample of the specimen. Record the moisture content as 'M' as per IS: 2720 (Part 2)-1973.

(vi) The remainder of the soil shall be broken up and repeat Steps (iii) to (v) above, by adding suitable increment of water to the soil. For sandy and gravelly soils the increment is generally 1% to 2% and for cohesive soils the increment is generally 2% to 4%. The total number of determinations made shall be at least five, and the moisture contents should be such that the optimum moisture content, at which the

maximum dry density occurs, is within that range.

(vii) For compacting soil containing coarse material up to 37.5mm size, the 2250 cm^3 mould should be used. A sample weighing about 30 kg and passing the 37.5 mm IS sieve is used for the test. Soil is compacted in five layers, each layer being given 55 blows of the 4.9 kg rammer.

5. Calculation : Bulk density, y_m, in g/cm^3 of each compacted specimen is calculated from the following equation:

$$y_m = (W_2 - W_1)/V_m$$

Where,

W_1 = Weight in g of mould + base plate

W_2 = Weight in g of mould + base plate + soil

V_m = Volume of mould in cm^3.

Dry density, y_d, in g/cm^3, of each compacted specimen is calculated from the following equation:

$$y_d = 100\, y_m / (100+M)$$

Where,

y_m = Bulk density of soil in g/cm^3.

M = Moisture content of soil

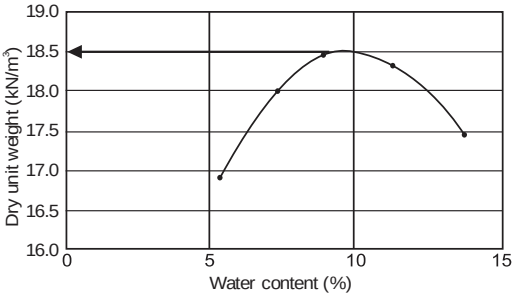


Fig. 6.03: Compaction Curve

The dry densities, y_d, obtained in a series of determinations is plotted against the corresponding moisture content 'M'. A smooth curve is then drawn through the resulting points, which is known as "compaction curve" (Fig. 6.03). The position of the maximum on this curve is determined, which is called Maximum Dry Density (MDD) and the corresponding moisture content is called Optimum Moisture Content (OMC).

6. Presentation of Results:

(i) The experimental points and the smooth curve drawn through them, showing the relationship between moisture content and dry density, shall be reported.

(ii) The dry density in g/cm^3 corresponding to the maximum point on the moisture content/dry density curve shall be reported as the maximum dry density to the nearest 0.01.

(iii) The percentage moisture content correspond-ing to the maximum dry density, on the moisture content dry density curve, shall be reported as the optimum moisture content and quoted to the nearest 0.2 for values below 5% to the nearest 0.5 for values from 5 to 10%, and to the nearest whole number for value exceeding 10%.

(iv) The amount of material retained on the 19mm IS sieve shall be reported to the nearest 1%.

(v) The method of obtaining the result shall be stated (4.9kg rammer method). The procedure used shall also be stated that is single sample or separate sample and the size of the mould used.

7. General Remarks:

(i) This procedure is satisfactory for cohesive soils but does not lend itself well to the study of the compaction characteristics of clean sands or gravels which displace easily when struck with rammer. Some nearly cohesionless soils compact satisfactorily in the standard test although in many cases the compaction curve is not well defined. Frequently, too in these cases indicated, maximum density is not as great as can be achieved readily in the field under available compaction methods. With a knowledge of the water density relation as determined by this test, better control of the field compaction of soil fill is possible because the optimum moisture content and the density which should be obtained are known by using this test procedure and these can be checked by field control tests.

(ii) It is important that the water is mixed thoroughly and adequately with the soil, since inadequate mixing gives rise to variable test results. This is particularly important with cohesive souls when adding a substantial quantity of water to the air-dried soil. With clays of high plasticity, or where hand mixing is employed, it may be difficult to distribute the water uniformly through the air-dried soil by mixing alone, and it may be necessary to store the mixed sample in a sealed container for a minimum period of about 16 hours before continuing with the test.

(iii) For some highly permeable soils such as clean gravels, uniformly graded and coarse clean sands the results of the laboratory compaction test may provide only a poor guide for specifications on field compaction. The laboratory test often indicates higher values of optimum moisture content than would be desirable for field compaction and the maximum dry density is often much lower than the state of compaction, that can readily be obtained in the field.

(B) Determination of Maximum and Minimum Density : For coarse grained soils, the compaction characteristics are determined by testing for the "maximum density" and "minimum density" in the lab. These values, used in conjunction with "in-situ density" of the earthwork obtained in the field are used to calculate "Relative Density/Density Index" which indicates the quality of the compaction.

1. **Introduction:** This test is performed to determine the "maximum density" and "minimum density" of coarse granular soils in the lab.

2. **Reference:** IS-2720 (Part-14) : 1983 (Reaffirmed-2015) "Methods of test for soils: Determination of density index (relative density) of cohesion-less soils".

3. **Apparatus required:**

   (i) Mould Assembly; consisting of standard mould, guide sleeves, surcharge baseplate, surcharge weights, surcharge baseplate handle, and dial indicator gauge – of 3000 cm^3 and 15000 cm^3 capacity conforming to IS: 10837-1984 (Fig. 6.04)

(ii) Vibrating Table: 75cmx75cm size, frequency of 3600 vibrations per minute, net mass of over 45 kg and suitable for use with 415 V three phase supply (Fig. 6.05)

(iii) Weighing Scale: 100 kg capacity with sensitivity of 20g conforming to IS: 1435-1991.

(iv) Metal Hand Scoop

(v) Metal Straightedge: about 40 cm long

(vi) Calibration bar: of metal 75mm x 300mm x 3mm in size

(vii) Surcharge Masses: one surcharge mass for each size mould conforming to requirements given in IS : 10837-1984.

(viii) Pouring Device: consisting of funnels 12 mm and 25 mm in diameter and 15 cm long, with cylindrical spouts and lipped brims for attaching to 15 cm diameter and 30 cm thigh metal cans.

(ix) Mixing Pans: suitable size are 60 X 90 cm and 10 cm deep and 40 x 40 cm and 5 cm deep.



Fig. 6.04



Fig. 6.05

4. Procedure

4.1 Calibration of Mould

(i) Measure inside diameter of mould at different depths using a bore gauge and take the average. Keep the mould on a flat surface or flat plate. Measure the height at different positions and take the average (accuracy 0.025 mm).Calculate the volume. Volume of the 3000 cm^3 mould should be calculated to the nearest 3 cm^3 and that of 15000 cm^3 mould to the nearest 30 cm^3.

(ii) Alternatively, fill the mould with distilled water till over flowing takes place. Slide thick glass plate over the top surface of mould. Weigh the water filling the mould.

(iii) Note the temperature of water. Obtain density of water for the above temperature from physical tables.

(iv) Calculate the volume of the mould which is weight of water filling the mould /density of water.

4.2 Preparation of the Sample

(i) The mass of soil sample to be taken depends upon the maximum size of particle in the soil as given in Table 6.01.

(ii) Dry the soil sample in a thermostatically controlled electric oven and Cool in the sample in a desiccator. Segregate soil lumps without breaking individual particles by sieving it through the required sieve size.

Table 6.01: Weight of Sample, Pouring Device and Size of Mould

Maximum size of soil particle	Mass of soil sample be used	Pouring Device to be used	Size of Mould to
75 mm	45 kg	Shovel or Extra Large Scoop	15000 cm^3
37.5 mm	12 kg	Scoop	3000 cm^3
19 mm	12 kg	Scoop	3000 cm^3
9.5 mm	12 kg	Pouring Device (25 mm dia. Spout)	3000 cm^3
4.75 mm	12 kg	Pouring Device (12 mm dia. Spout)	3000 cm^3

4.3 Minimum Density : Soil containing particles smaller than 9.50 mm should be placed as loosely as possible in the mould (weighing, W) by pouring the soil through the spout in a steady stream. The spout should be adjusted so that the height of free fall of the soil is always 25 mm. While pouring the soil the pouring device should be moved in a spiral motion from the outside towards the centre to form a soil layer of uniform thickness without segregation. The mould should be filled approximately 25 mm above the top and levelled with top by making one continuous pass with the steel straightedge. If all excess matter is not removed, an additional continuous pass should be made. Great care shall be exercised to avoid jarring the mould during the entire pouring and trimming operation. The mould and the soil should be weighed and the mass, W_1, recorded.

Soil containing particles larger than 9.50 mm should be placed by means of a large scoop (or shovel) held as close as possible to and just above the soil surface to cause the material to slide rather than fall into the previously placed soil. If necessary, large particles may be held by hand to prevent them from rolling off the scoop. The mould (weighing, W) should be filled to overflowing but not more than 25 mm above the top. The surface of the soil should be levelled with the top of the mould using the steel straightedge (and the fingers, if necessary) in such a way that any slight projections of the larger particles above the top of the mould shall approximately balance the larger voids in the surface below the top of the mould. The mould and the soil should be weighed and the mass, W_1, recorded.

Volume of mould = V, cm^3

Mass of dry soil $M_s = (W_1 - W)$, gm

$(Y_d)_{min} = M_s / V$, g/cm^3

4.4 Maximum Density : The guide sleeve should be assembled on top of the mould and the clamp assemblies tightened so that the inner surfaces of the walls of the mould and the sleeve are in line. The lock nuts on the two set screws equipped with them should be tightened. The third clamp should be loosened, the guide sleeve removed, the empty mould weighed and its mass recorded.

The mould filled for the determination of minimum density may also be used for this test.

The guide sleeves should be attached to the mould and the surcharge base plate should be placed on the soil surface. The surcharge weight should then be lowered on the base-plate using the hoist in the case of the 15000 cm^3 mould.

The mould should be fixed to the vibrator deck. The vibrator control should be set at maximum amplitude and the loaded soil specimen should be vibrated for 8 minutes.

The surcharge weight and guide sleeves should be removed from the mould. The dial gauge readings on two opposite sides of the surcharge base plate should be obtained and the average recorded. The mould with the soil should be weighed and its mass recorded.

Volume of mould = V, cm^3

Mass of dry soil $M_s = (W_2 - W)$, gm

$(Y_d)\ max = M_s / V$, g/cm^3

Chapter-7

PERMEABILITY TEST

The rate of flow of water, under laminar flow conditions, through a unit cross sectional are of soil mass, under unit hydraulic gradient, is defined as coefficient of permeability. Permeability of the soil governs the magnitude of excess pore water pressure built-up in the embankment or cuttings, during consolidation process or when the embankment is ponded by water. The excess pore water pressure in-turn significantly influences the stability of the embankments and indicate the need, or otherwise, of need for special measures (e.g. sandwich construction) to prevent/quickly dissipate excess pore water pressure. Coefficient of permeability is used to assess drainage characteristics of soil, rate of consolidation and to predict rate of settlement of soil bed. The coefficient of permeability is generally determined by two procedures.

(A) Constant Head Test

1. **Introduction:** This test is performed to determine the "coefficient of permeability" under constant water head conditions. This test is relevant for coarse-grained soils with a reasonable discharge.

2. **Reference:** IS-2720(Part 36):1987 (Reaffirmed- Dec'2016) "Methods of test for soils: Laboratory determination of permeability of granular soils (constant head)".

3. **Apparatus Required** (Fig. 7.01)

   (i) Permeameter: Cylindrical mould of non-corrodible material and having dimension based on the maximum particle size of the soil, as detailed in Table 7.01.

Table 7.01: Dimensions of Permeameter Mould

Max. Particle Size between IS Sieve Openings (mm)	Min. Cylinder Diameter (mm)			
	Not more than 35% of total Soil retained on Sieve Opening		More than 35% of total Soil retained on Sieve Opening	
	2.0 mm	10.0 mm	2.0 mm	10.0 mm
2.00 and 10.00	80	-	120	-
10.00 and 20.00	-	160	-	230
The diameter to length ratio may be about 1:2				

The Permeameter shall be fitted with:

(a) A porous disc or suitable reinforced screen at the bottom with a permeability greater than that of the soil specimen, but with openings sufficiently small (not larger than 10% of finer size of the soil to be tested) to prevent the movement of particles.

(b) Manometer outlets for measuring the loss of head.

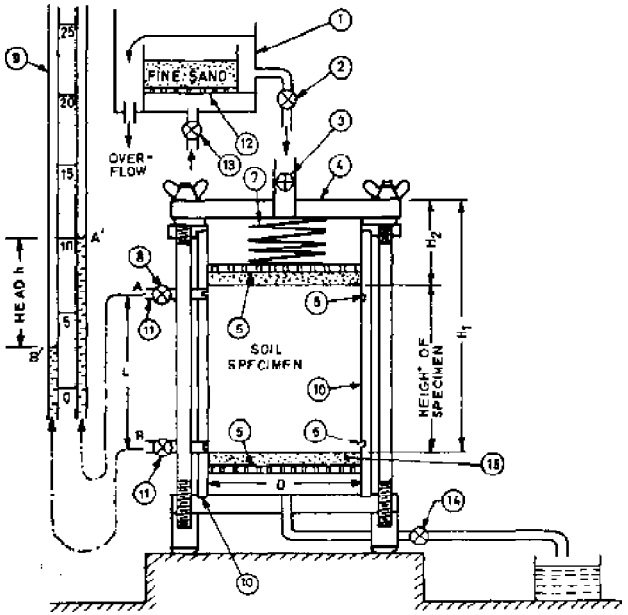
(c) A porous disc or suitable reinforced screen with spring attached to the top, or any other device for applying a light spring pressure of 2 to 4 kg total load when the top plate is attached in place. This will satisfy the requirement that there should be no soil volume change during a test.

(ii) Constant-head Filter Tank: A suitable water reservoir capable of supplying water to the Permeameter under constant head.

(iii) Large Funnels: These shall be fitted with special cylindrical spout, 25 mm in diameter for 10.00 mm maximum size particles, and 13 mm in diameter for 2.00 mm maximum size particles. The length of the spout should be greater than the full length of the permeability chamber at least by160 mm.

(iv) Specimen Compaction Equipment: Compaction equipment as deemed desirable may be used. The following are suggested:

(a) A vibrating tamper fitted with a tamping foot 50 mm in diameter;



1.	Constant head filter tank	2.	Filter tank valve
3.	Inlet valve	4.	Top plate
5.	Porous disc or screen	6.	Screened manometer groove
7.	Spring	8.	Manometer valve
9.	Manometer tubes	10.	Metal or transparent acrylic plastic cylinder
11.	Manometer outlet	12.	Screen
13.	Tap water valve	14.	Outlet valve
15.	Gravel filter		

Fig. 7.01: Constant Head Permeameter

(b) A sliding tamper with a tamping foot 50 mm in diameter and a rod for sliding weights of 100 g (for sands) to 1 kg (for soils with a large gravel content), having an adjustable height of drop to 100mm for sands and 200mm for soils with large gravel contents.

(v) Vacuum Pump or Water Faucet Aspirator: for evacuating and for saturating soil specimen under full vacuum.

(vi) Balance: Of 2 kg capacity and sensitive to 1 g.

(vii) Scoop: Capacity of 100 g of soil.

(viii) Miscellaneous Apparatus : Clock, Thermometers, 250ml graduated cylinder and mixing pan.

4. Procedure

4.1 Preparation of specimen for testing

(i) Make the initial measurements and record on the data sheet, the inside diameter of the permeameter (D), the length between the manometer outlets (L) and the depth measured at four symmetrically spaced points from the upper surface of the top plate of the permeability cylinder to the top of the upper porous stone or screen temporarily placed on the lower porous plate or screen (H_1). A duplicate top plate containing four large symmetrically spaced openings through which the necessary measurement scan be made, shall be employed to determine the average value for H_3. Calculate the cross sectional area of the specimen (A).

(ii) A small portion of the sample selected shall be taken for water content determinations. Record the weight of the remaining air-dried sample (W_1) for unit weight determination.

(iii) Place the prepared soil by one of the following procedures in uniform thin layers approximately 15 to 20 mm. For fine sands, water pump grease should be applied to the cylinder wall to prevent flow of water between the specimen and the wall. For coarse sand, a 7mm thickness of sponge rubber cemented to the cylinder wall is found to be satisfactory.

(iv) For soils having a maximum size of 10 mm or less, place the appropriate size of funnel in the permeability device with the spout in contact with the lower porous plate or screen or previously formed layer, and fill the funnel with sufficient soil to form a layer, taking soil from different areas of the sample in the pan. Lift the funnel by 15mm or approximately the unconsolidated layer thickness to be formed, and spread the soil with a slow spiral motion, working from the perimeter of the device towards the centre, so that a uniform layer is formed. Remix the soil in the pan for each successive layer to reduce segregation caused by taking soil from the pan.

(v) For soils with a maximum size greater than 10 mm, spread the

soil from a scoop. Turn the permeability cylinder sufficiently for the next scoopful, thus progressing around the inside perimeter to form a uniform compacted layer of a thickness equal to the maximum particle size.

(vi) Compact successive layers of soil to the desired relative density by appropriate procedure, to a height of about 20mm above the upper manometer outlet.

(vii) Level the upper surface of the soil by placing the upper porous plate or screen in position and by rotating it gently clockwise and anti-clockwise.

(viii) Measure and record the final height of specimen (H_1 - H_2), by measuring the depth from the Upper Surface of the perforated top plate employed to measure H_1 to the top of the upper porous plate or screen at four symmetrically spaced points after compressing the spring lightly to seat the porous plate or screen during the measurements; the final weight of air-dried soil used in the test (W_1 - W_2) by weighing the remainder of soil W_2 left in the pan. Compute and record the unit weights, void ratio and relative density of the test specimen as under:

Diameter of specimen = D, cm

Spacing between manometer outlets = L, cm

Length of specimen = $(H_1 - H_2)$, cm
Area of specimen= A = $\pi/4 \times D^2$, cm^2
Volume of specimen = V = A x $(H_1 - H_2)$, cm^3
Water content =W , %

Dry weight of soil specimen = W_s , g
Dry unit weight = $y = W_s/V$, g/cm^3

Specific Gravity = G_s

Void ratio =e = $[(G_s \times Yw)/Y] - 1$

(ix) With its gasket in place, press down the top plate against the spring and attach it securely to the top of the permeameter cylinder, making an airtight seal.

(x) Connect the inlet tube of the top plate of the permeameter to a vacuum pump or suitable aspirator capable of evacuating the air content from the specimen and the outlet tube in the base plate to the water container as shown in Fig. 7.02.

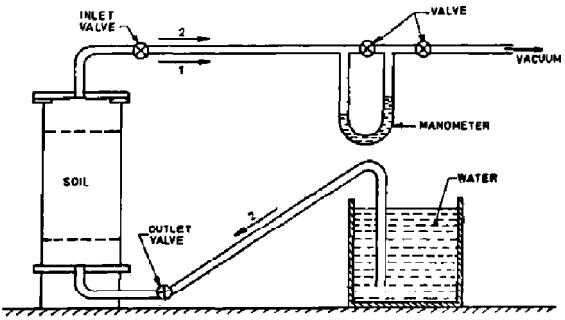


Fig. 7.02: Evacuating and Saturating the Specimen

(xi) Close the manometer outlets and the outlet valve at the base plate of the permeameter. Using a

vacuum pump or aspirator, evacuate the specimen under 500 mmHg, minimum for 15 minutes to remove air adhering to soil particles and from the voids. Follow the evacuation by a slow saturation of the specimen from the bottom upward under full vacuum in order to force any remaining air in the specimen. Continued saturation of the specimen can be maintained more adequately by the use of de-aired water, or water maintained at an in-flow temperature sufficiently high to cause a decreasing temperature gradient in the specimen during the test. Native water or water of low mineral content should be used for the test, but in any case the fluid should be described on the report form.

NOTE: Native water is water occurring in the rock or soil in-situ. It should be used if possible, but it (as well as de-aired water) may need a refinement not ordinarily feasible for large scale production testing in which case available water may be used and so stated in the report.

(xii) After the specimen has been saturated and the permeameter is full of water, close the bottom valve on the outlet tube and disconnect the vaccum. Fill the inlet tube with water from the constant-head tank by slightly opening the filter tank valve. Then connect the inlet tube to the top of the permeameter, open the inlet

valve slightly and open the manometer outlet cocks slightly, to allow water to flow, thus freeing them of air. Connect the water manometer tubes to the manometer outlets and fill with water to remove the air. Close the inlet valve and open the outlet valve to allow the water in the manometer tubes to reach their stable water level under zero head.

(xiii) Open the inlet valve from the filter tank slightly for the first run, delay measurements of quantity of flow and head until a stable head condition without appreciable drift in water manometer level is attained. Measure and record the time t, head h (the difference in level in the manometers), quantity of flow Q, and water temperature T.

(xiv) Repeat the test runs at heads, increasing by 5 mm in order to establish accurately the region of laminar flow with velocity v (where v = Q/At), directly proportional to hydraulic gradient i (where i = h/L). When departures from the linear relation become apparent, indicating the initiation of turbulent flow conditions, 10 mm intervals of head may be used to carry the test run sufficiently along in the region of turbulent flow to define this region if it is significant for field conditions.

NOTE: Much lower values of hydraulic gradient, h/L are required than generally recognized, in

order to ensure laminar flow conditions. The following values are suggested: loose compactness ratings h/L from 0.2 to 0.3; and dense compactness ratings h/L from 0.3 to 0.5; the lower values of h/L apply to coarser soils and the higher values to finer soils.

(xv) At the completion of the permeability test, drain the specimen and inspect it to establish whether it was essentially homogenous and isotropic in character. Any light and dark alternating horizontal streaks or layers are evidence of segregation of fines.

5. Observation and Recording

(i) The inside diameter of the permeameter (D), the length between manometer outlets (L) and depth (H_1) are measured.

(ii) For the given soil, water content is determined and recorded. The weight of air dried soil used in preparing soil specimen (W_s) is also recorded. The final height of specimen after compression by spring ($H_1 - H_2$) is measured and recorded. Dry unit weight and void ratio are calculated. The temperature of water (T) is measured and recorded,

(iii) During the test, observations are made of manometer readings h_1 and h_2, quantity of flow (Q) collected in a graduated jar in the time t are recorded. Head h (= h_1-h_2) is calculated and gradient i (= h/L) is calculated and recorded.

6. Calculations : Permeability at temperature k_T is calculated by :

$$k_T = Q / (A \times I \times t)$$

and permeability at 27^0C by using the expression

$$k_{27} = k_T \times (\mu_T / \mu_{27})$$

Where,

μ_T = Coefficient of viscosity at T^0 C

μ_{27} = Coefficient of viscosity at 27^0 C

7. Presentation of data: The coefficient of permeability is reported in cm/sec at T^0 C and 27^0 C. The dry density, the void ratio and the degree of saturation shall be reported. The test results should be tabulated as below:

Diameter of specimen (D) = ... cm

Spacing between manometer outlets (L) = ... cm

Length of specimen ($H_1 - H_2$) = ... cm

Area of specimen (A) = μ /4 x D^2 = ... cm^2

Volume of specimen (V) = A x ($H_1 - H_2$) = ... cm^3

Water content (W) = ... %

Dry weight of soil specimen (W_s) = ... g

Dry unit weight (Y) = W_s/V = ... g/cm^3

Specific Gravity (G_s) = ...

Void ratio (e) = $[(G_s \times Y_w)/Y] - 1$ = ...

Experiment No.	1	2	3
Manometer readings: h_1 (cm) h_2 (cm)			
Quantity of Flow: Q (cm^3)			
Time: t (Seconds)			
Head: $h = h_1 - h_2$ (cm)			
$i = h / L$			
$K_T = Q/(A \times i \times T)$ (cm/sec)			
Remarks			

(B) Falling Head Test

1. **Introduction:** This test is performed to determine the "coefficient of permeability" under variable water head conditions. This test method is relevant for soil with low water discharge.
2. **Reference:** IS-2720 (Part-17):1986 (Reaffirmed -2016) "Methods of test for soils: Laboratory determination of permeability".
3. **Apparatus Required:** Same as for "Constant Head Test".
4. **Procedure:** Prepare the specimen in the same way as for "Constant Head Test". The specimen shall be connected through the top inlet to selected stand-pipe. The bottom outlet shall be opened and the time interval required for the water level to fall from a known initial head to a known final head as measured above the centre of the outlet shall be recorded. The stand-pipe shall be refilled with water and the test repeated till three successive observations give nearly same time interval; the time intervals being recorded for the drop in head from the same

initial to final values, as in the first determination. Alternatively, after selecting the suitable initial and final heads, h_1 and h_2 respectively, time intervals shall be noted for the head to fall from h_1 to $\sqrt{h_1 . h_2}$ and similarly from $\sqrt{h_1 . h_2}$ to h_2. The time intervals should be the same; otherwise the observation shall be repeated after refilling the stand-pipe.

5. Observation and Recording

(i) The dimensions of specimen, length Land diameter D, are measured and recorded. Area 'a' of stand-pipe is recorded. The temperature T, of water is also measured and recorded.

(ii) During the test, observations are made of initial time t_1, final time t_f, initial head h_1 and final head h_2 in stand-pipe are recorded. h_1/h_2 and $\log_{10} (h_1/h_2)$ are calculated and recorded. The permeability k_T is calculated and recorded.

(iii) At the end of the test, the weight of wet soil specimen W_t is measured and recorded. Then the sample is dried in the oven for 24 hours and the dry weight W_s is measured and recorded. The water content, W is computed and noted. Void ratio, e and degree of saturation S are calculated using specific gravity G, of the specimen and water content, W.

6. Calculations: Permeability at temperature (k_T) is calculated by:

$$k_T = 2.303 \times [aL/A(t_f - t_i)] \times \log 10 (h_1/h_2)$$

and permeability at 27°C by using the expression

$$k_{27} = k_T \times (\mu_T / \mu_{27})$$

Where,

μ_T = Coefficient of viscosity at T^0 C

μ_{27} = Coefficient of viscosity at 27^0 C

7. **Presentation of data:** The coefficient of permeability is reported in cm/sec at T^0 C and 27^0C. The dry density, the void ratio and the degree of saturation shall be reported. The test results should be tabulated as below:

Diameter of specimen (D) ..... cm

Length of Specimen (L) ..... cm

Area of specimen (A) = $\pi/4 \times D^2$ = ... cm^2

Volume of specimen (V) = A x L ..... cm^3

Area of Stand-pipe (a) = ... cm^2

C = 2.303 x (aL/A) ..... cm

Temperature of Water = ... ^{0} C

Experiment No.	**1**	**2**	**3**
Initial Time: t_i (Seconds)			
Final Time: t_f (Seconds)			
Initial Head: h_1 (cm)			
Final Head: h_2 (cm)			
h_1/h_2			
$\log_{10}(h_1/h_2)$			
k_T (cm/sec)			
Remarks			

Weight of wet soil specimen after the test, W_t=..... g

Weight of dry soil specimen, W_s = ... g

Water content, $W = (W_t - W_s)/W_s \times 100$ = ... %

Specific gravity of specimen, G_g = ...

Void ratio, $e = (VG_g - W_s)/W_s$ = ...

Degree of saturation, $S = G_g W/e$= .....%

Permeability at 27^0C, $k_{27} = k_T x(\mu_T/\mu_{27})$ = ... cm/s

8. General Remarks: During test, there should be no volume change in the soil and soil should be completely saturated. The flow should be laminar and in a steady state condition.

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Chapter-8

DETERMINATION OF ORGANIC CONTENT

1. **Introduction:** The organic content is the ratio, expressed as a percentage, of the mass of organic matter in a given mass of soil to the mass of the dry soil solids. Organic matter influences many of the physical and chemical properties of soils. Some of the properties influenced by organic matter include soil structure, water holding capacity, compressibility and shear strength. Organic matter is an undesirable constituent of the soil from the engineering point of view since it causes swelling or shrinkage of the soil when the moisture content or the applied load changes. The estimation of organic matter, therefore, forms an important part of soil examination.Organic soils are generally not considered as "suitable" for use as embankment fill material.

2. **Reference:** IS-2720 (Part-22):1972 (Reaffirmed-2015) "Methods of test for soils: Determination of organic matter".

3. **Apparatus required**

   (i) Oven: thermostatically controlled to maintain the temperature between 105° and 110°C, with interior of non-corroding material (Fig. 8.01).

   (ii) Weigh Balance: sensitive to 0.001 g.

   (iii) Volumetric Flask: two, of one-litre capacity.

   (iv) Burettes: two, of 25 ml, graduated in 0.1 ml.

   (v) Pipettes: 10 ml and 1 ml pipette fitted with a rubber teat.

(vi) Conical Flasks: two, of 500 ml capacity.

(vii) Graduated Measuring Cylinders: 200 ml and 25 ml.

(viii) Desiccator: with desiccating agent other than sulphuric acid.

(ix) Glass Weighing Bottle: approximately 25mm in diameter and of 50 mm height fitted with a ground glass stopper.

(x) Sieves: 10mm and 425micron IS Sieve.

(xi) Wash Bottle

(xii) Reagents: The reagents used shall be of analytical quality.

(a) Potassium Dichromate Normal Solution- Dissolve 49.035g of potassium dichromate in distilled water to make one litre of solution.

(b) Ferrous Sulphate, Approx. 0.5N Solution- Dissolve approx. 140g of ferrous sulphate in 0.5 N sulphuric acid to make one litre of solution (add 14ml of concentrated sulphuric acid to distilled water to make one litre of solution for 0.5 N sulphuric acid).

NOTE- This solution is unstable in air and should be kept tightly stoppered. It should be standardized against the potassium dichromate solution at least once in a week.

(c) Sulphuric Acid, Concentrated- of specific gravity 1.836.

(d) Orthophosphoric Acid, 85 Percent- of specific gravity 1.70 to 1.75.

(e) Indicator Solution- 0.25g of sodium diphenylamine-sulphonate dissolved in 100ml of distilled water.



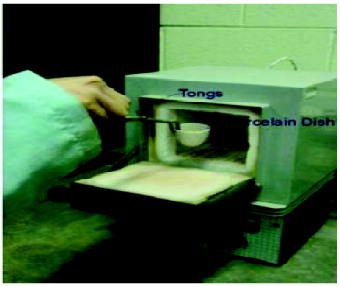


Fig. 8.01

4. Procedure

(i) Standardization of Ferrous Sulphate Solution: Run 10ml of normal potassium dichromate solution from a burette into a 500ml conical flask. Add 20 ml concentrated sulphuric acid, swirl the mixture and allow cooling for some minutes. Then add 200 ml of distilled water followed by 10 ml of phosphoric acid and 1 ml of the indicator, and shake the mixture thoroughly. Then add ferrous sulphate solution from the second burette in 0.5 ml increments, with contents of the flask being swirled, until the colour of the solution changes from blue to green. Add further 0.5 ml of potassium dichromate changing the colour back to blue. Add ferrous sulphate solution drop by drop with continued swirling until the colour of the solution changes from blue to green after addition of a single drop. The total volume of ferrous sulphate solution used (X) shall be noted to the nearest 0.05 ml (1ml ferrous sulphate solution is equivalent to 10.5/X ml potassium dichromate).

(ii) Preparation of Soil Sample: The portion of the air dried sample shall be weighed, its moisture content determined using separate sample and the equivalent oven-dried

weight (W_1) recorded. It shall then be sieved on a 10mm IS Sieve and weight of the material passing shall be recorded (W_2) to the nearest 0.1 percent of total weight. A sample weighing approximately 100g shall be obtained from the material passing the 10mm Sieve and pulverized so that it passes the 425 micron Sieve. A 5g soil sample shall be taken from the thoroughly mixed portion of the material passing the 425micron Sieve and used for the test.

(iii) The sample shall be placed in a glass weighing bottle and weighed to 0.001g. A small quantity, from 5g to 0.2g shall be transferred to a dry 500ml conical flask, the weighing bottle reweighed and the equivalent weight on oven-dry basis of soil specimen removed (W_3) calculated by difference and allowing for the moisture content of the soil.

(iv) Run about 10ml of N potassium dichromate solution into the conical flask from a burette and add 20 ml of concentrated sulphuric acid from a measuring cylinder. Thoroughly swirl the mixture for about one minute and allow it to stand on a heat insulating surface, such as asbestos or wood, for 30 min to allow oxidation of organic matter to proceed. Then add 200 ml of distilled water, with 10 ml of orthophosphoric acid and 1 ml of the indicator. The mixture shall be shaken vigorously. If the, indicator is absorbed by the soil, a further 1 ml of the solution shall be added. Ferrous sulphate solution shall then be added from the second burette in 0.5 ml increments, the contents of the flask being swirled, until the colour of the solution changes from blue to green. A further 0.5 ml of potassium dichromate shall then be added, changing the colour of the

solution back to blue. Ferrous sulphate solution shall then be added drop by drop with continued swirling until the colour of the solution changes from blue to green after the addition of a single drop. The total volume of the ferrous sulphate solution used (Y) shall be noted to the nearest 0.05 ml.

5. Calculations

(i) The total volume (V ml) of potassium dichromate used to oxidize the organic matter in the soil is given by the following formula:

$$V = 10.5 (1 - Y/X)$$

Where:

Y = total volume of ferrous sulphate used in test, and

X = total volume of ferrous sulphate used in standardization test.

(ii) The percentage of organic matter (OM) present in the oven-dried sample shall be calculated from the following formula:

$$OM \text{ (percent by weight)} = (0.67 W_2 V)/(W_1 W_3)$$

Where:

W_2 = weight on oven-dry basis of the soil sample passing 10mm IS Sieve,

V = total volume of potassium dichromate used to oxidize the organic matter,

W_1 = weight on oven-dry basis of the total soil sample taken for the test before sieving, and

W_3 = weight on oven-dry basis of the soil specimen used in the test.

6. Reporting : The organic matter content shall be reported to the nearest 0.1% of the original oven-dry soil.

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Chapter-9

DETERMINATION OF FREE SWELL INDEX OF SOIL

1. **Introduction:** Free swell or differential free swell, also termed as “free swell index”, is the increase in volume of soil without any external constraint when subjected to submergence in water. A soil with high free swell index is considered “unsuitable” for use as embankment fill material and in case the sub-soil is having high free swell index then suitable “ground improvement measures” may be needed before constructing embankment on such soil. Actual magnitude of swelling pressures developed depends upon the dry density, initial water content, surcharge loading and several other environmental factors.

2. **Reference:** IS-2720 (Part-40):1977 (Reaffirmed-2016) “Methods of test for soils: Determination of free swell index of soils”.

3. **Apparatus required**

   (i) Oven: thermostatically controlled to maintain the temperature between 105 and 110°C, with interior of non-corroding material

   (ii) Weigh Balance: sensitive to 0.01 g.

   (iii) Sieve: 425 micron IS Sieve.

   (iv) Glass Graduated Cylinders: two, 100 ml capacity conforming to IS: 878-2008.

4. **Procedure**

   (i) Take two 10g oven dried soil specimens, passing through 425 micron IS sieve. Pour each soil specimen in each of the two glass graduated cylinders of 100 ml capacity.

(ii) Fill one cylinder with kerosene oil and the other with distilled water up to the 100 ml mark. Stir both the cylinders with a glass rod to remove entrapped air and allow them to settle for 24 hours, to attain equilibrium without any further change of volume of soils. Read out the final volume of soils in each of the cylinders.

5. Calculations: The level of soil in the kerosene graduated cylinder shall be read as the original volume of the soil samples, kerosene being a non-polar liquid does not cause swelling of the soil. The level of the soil in the distilled water cylinder shall be read as the free swell level.

Free swell index, percent = $((V_d - V_k) / V_k) * 100$

Where,

V_d = the volume of soil specimen read from the graduated cylinder containing distilled water, and

V_k = the volume of soil specimen read from the graduated cylinder containing kerosene.

The free swell index is reported to the nearest whole number.

6. General Remarks : In the case of highly expansive soils such as Sodium Bentonite, the sample size may be 5 g or alternatively a cylinder of 250ml capacity for 10 g of sample may be used.

Chapter-10

DETERMINATION OF SWELL PRESSURE OF SOIL

The expansive clays increase in their volume when they come in contact with water owing to surface properties of these clay types. The pressure which the expansive soil exerts, if it is not allowed to swell or the volume change of the soil is arrested, is known as Swelling Pressure of Soil. The swelling pressure is dependent upon several factors namely (a) the type and amount of clay in the soil and the nature of the clay mineral, (b) the initial water content and dry density, (c) the nature of pore fluid, (d) the stress history of the soil including the confining pressure and (e) drying and wetting cycles to which the soils have been subjected to.

A soil with high swell pressure is considered "unsuitable" for use as embankment fill material and in case the sub-soil is having high swell pressure then suitable "ground improvement measures" may be needed before constructing embankment on such soil. One of the methods for constructing embankments on expansive soils is to lay a CNS (non-swelling clay) layer at the base of the embankment, to isolate the sub-soil from the moisture (mainly rain water) coming in vertically downwards direction. For any clay to be classified as "CNS (non-swelling clay)", it should not have swell pressure more than the specified value (normally 0.5 kg/m^2).

In laboratory, one dimensional swelling pressure test is conducted using either fixed or the floating rings on both undisturbed and re-moulded soils in the partially saturated condition. Two methods are employed, namely, "consolidometer method" in which the volume change of the soil is permitted and the corresponding pressure required to bring back the soil to its original volume is measured and "constant volume method" in which the

volume change is prevented and the consequent pressure is measured.

(A) By Consolidometer Method

1. **Introduction:** This test is performed to determine the "Swell Pressure" of soil using "Consolidometer Method", also known as "constant pressure method".
2. **Reference:** IS-2720 (Part-41):1977 (Reaffirmed-2016) "Methods of test for soil: Measurement of swelling pressure of soils".
3. **Apparatus required**

   (i) Consolidometer (Fig. 10.01): A device to hold the sample in a ring either fixed or floating with porous stones on each face of the sample. A consolidometer shall also provide means for submerging the sample, for applying a vertical load and for measuring the change in the thickness of the specimen. The provision for fixing of the dial gauge shall be rigid and in no case shall the dial gauge be fixed to a cantilevered arm. The dial gauges shall be fixed in such a way to record the vertical expansion of the specimen.



Fig. 10.01

(a) Specimen Diameter: 60mm.

(b) Specimen Thickness: Shall be at least 20mm in all cases. However, the thickness shall not be less than 10 times the maximum diameter of the grain in the soil specimen. The diameter to thickness ratio shall be a minimum of 3.

(c) Ring: The ring shall be made of non-corrosive material. The inner surface shall be highly polished or coated with a thin coating of silicon grease or with a low friction material. The thickness of the ring shall be such that under assumed hydrostatic stress conditions in the sample the change in diameter of the ring will not exceed 0.03% under the maximum load applied during the test. The ring shall have one edge bevelled suitably so that the sample is pressed into the ring with least disturbance. The ring shall be placed with its cutting edge upwards and clamped with a special clamp which should not damage the sharp edge. The clamp should be made circular with central hole equal in diameter of the porous stone and should be perfectly concentric with the sample. The ring shall be provided with a collar of internal diameter same as that of the ring and of effective height 20mm. The collar shall rest securely on the specimen ring.

(d) Porous Stones: The stones shall be of silicon carbide or aluminium oxide and of medium grade. It shall have a high permeability compared to that of the soil being tested. The diameter of the top stone shall be 0.2 to 0.5mm less than the internal diameter of the ring.

The thickness of the stone shall be minimum 15mm. The top stone shall be loaded through a corrosion resistant plate of sufficient rigidity and of minimum thickness 10mm to prevent breakage of the stone. The loading plate shall have suitable holes for free drainage of water.

(ii) Dial Gauge: Accurate to 0.01mm with a traverse of at least 20 mm.

(iii) Moisture Room: For storing samples and for preparing samples in climates where there is likelihood of excessive moisture loss during preparation (optional).

(iv) Soil Trimming Tools: Fine wire-saw, knife, spatula, etc. for trimming sample to fit into the inside diameter of the consolidometer ring with minimum disturbances.

(v) Oven: Thermostatically controlled with interior of non-corroding material to maintain the temperature between 105 and 110°C.

(vi) Desiccator: With any desiccating agent other than sulphuric acid.

(vii) Balance: Sensitive to 0.01 g.

(viii) Containers: For water content determination.

4. Procedure

(i) *Preparation of Specimen from Undisturbed Soil Samples:* Clean and weigh the empty container. Cut-off the specimen either from undisturbed tube sample or from block sample, the latter generally being more representative of the field conditions.

Remove about 30mm height of soil from one end of the soil sample. Gradually insert the consolidation ring in the sample by pressing with hands and carefully removing the material around the ring. The soil specimen so cut shall project as far as 10mm on either side of the ring. Then trim the specimen smooth and flush with the top and bottom of the ring. Wipe the container ring clear of any soil sticking to the outside and weigh again with the soil. The whole process should be quick to ensure minimum loss of moisture and if possible shall be carried out in the moisture room. The representative specimens from the soil trimming shall be taken in moisture content cans and their moisture content determined in accordance with IS:2720 (Part-XI).

(ii) *Preparation of Specimen from Disturbed Soil Sample:* The soil sample shall be compacted to the desired (field) density and water content in a standard compaction proctor mould. Samples of suitable sizes are cut from it as given in Para (i) above.

NOTE-1: Since swelling pressure of the soil is very much influenced by its initial water content and dry density, it shall be ensured that in the case of undisturbed soil samples, the specimen shall be collected from the field for test during the driest season of the year, so that the swelling pressure recorded shall be maximum. In case of remoulded soil samples, the initial water content shall be at the shrinkage limit or field water content, so that the swelling pressure recorded shall be maximum.

NOTE-2: The desiccated soil obtained from the field coupled with smaller thickness of consolida-

tion ring make the undisturbed soil specimen always in danger of being disturbed during trimming; hence great care shall be taken to handle the specimen delicately with the least pressure applied to the soil.

(iii) Saturate the porous stones by boiling in distilled water for at least 15 minutes and moisten all surfaces of the consolidometer which are to be enclosed. Assemble the consolidometer with the soil specimen (in the ring) and porous stones at top and bottom of the specimen, providing a wet filter paper between the soil specimen and the porous stone. Then position the loading block centrally on the top porous stone.

(iv) Mount this assembly on the loading frame such that load applied is transmitted to the soil specimen through the loading cap.

(v) In the case of the lever loading system, the apparatus shall be properly counter-balanced. If a jack with load measurements by platform scales is used as the loading systems the tare weight with the empty consolidation apparatus, excluding those parts which will be on top of the soil specimen, which rest on the platform shall be determined before filling the ring with the soil and this tare weight shall be added to the computed scale loads required to give the desired pressures at the time of loading the soil specimen.

(vi) Screw the holder with the dial gauge to record the progressive vertical heave of the specimen under no load.

(vii) Place initial setting load of 50 gf/cm^2 (this includes weight of the porous stone and the loading pad) on the loading hanger and note initial reading of the dial gauge.

(viii) Connect the system to a water reservoir with the level of water in the reservoir being at about the same level as the soil specimen and allow water to flow in the sample. Then allow the soil to swell.

(ix) Record the free swell readings shown by the dial gauge under the seating load at different time intervals. The dial gauge readings shall be taken till equilibrium is reached. This is ensured by making a plot of swelling dial reading versus time in hours, which plot becomes asymptotic with abscissa (time scale). The equilibrium swelling is normally reached over a period of 6 to 7 days in general for all expansive soils.

(x) Then subject the swollen sample to consolidation under different pressures. Record the compression dial readings till the dial readings attain a steady state for each load applied over the specimen. The consolidation loads shall be applied till the specimen attains its original volume.

5. Observations: The observations shall be recorded as per the format given in Table: 10.1.

Table: 10.1 Swelling Pressure Test by Consolidometer Method

Form-I

Project: Dated:

Tested By:

1. Details of soils sample
   (i) Location
   (ii) Boring No.
   (iii) Depth
   (iv) Visual description of soil
   (v) Liquid limit
   (vi) Plasticity index
   (vii) Percentage of soil fraction below 0.002mm
2. Details of soil specimen
   (i) Undisturbed or remoulded
   (ii) Specific gravity of the soil

Natural Density			Moisture Content		
Description	Test-1	Test-2	Description	Test-1	Test-2
Weight of container ring+ wet specimen			Weight of container + wet soil		
Weight of container			Weight of container + dry soil		
Diameter of container			Weight of container		
Initial thickness of soil sample			Weight of water		
Wet density in g/ml			Weight of dry soil		
Dry density in g/ml			Moisture content in %		

Date:

Time of Starting:

Elapsed Time in Hours	Swelling dial reading
0	
0.5	
1	
2	
4	
8	
12	
16	
20	
24	
36	
48	
60	
72	
96	
120	
144	

Form-2

Data Sheet for Swell - Compression Test

Pressure Increment		Compression	Change in thickness of expanded specimen
In kgf/cm^2	In kN/m^2		
(0.0-0.05)	0-5		
(0.05-0.10)	5-10		
(0.10-0.25)	10-25		

(0.25-0.50)	25-50		
(0.50-1.00)	50-100		
(1.00-2.00)	100-200		
(2.00-4.00)	200-400		
(4.00-8.00)	400-800		
(8.00-16.00)	800-1600		

6. **Calculations and Report:** The observed swelling dial reading recorded in Form-1 shall be plotted with elapsed time as abscissa and swelling dial reading as ordinates on natural scale. A smooth curve shall be drawn joining these points. If the curve so drawn becomes asymptotic with the abscissa, the swelling has reached its maximum and swelling phase shall be stopped, and the consolidation phase shall be started. The compression readings shall be tabulated as in Form-2 and a plot of change in thickness of expanded specimen as ordinates and consolidation pressure applied as abscissa in semi-logarithmic scale shall be made. The swelling pressure exerted by the soil specimen under zero swelling condition shall be obtained by interpolation and expressed in kN/m^2 (kgf/cm^2).

(B) By Constant Volume Method

1. **Introduction:** This test is performed to determine the "Swell Pressure" of soil using "Constant Volume Method".
2. **Reference:** IS-2720 (Part-41):1977 (Reaffirmed-2016) "Methods of test for soil: Measurement of swelling pressure of soils".
3. **Apparatus required** (Fig. 10.02)

   (i) Consolidometer, including Specimen Diameter, Specimen Thickness, Ring, Porous Stones etc.: Same as for "Consolidometer Method".

(ii) Dial Gauge, Moisture Room, Soil Trimming Tools, Oven, Desiccator, Balance and Containers: Same as for "Consolidometer Method".



Fig. 10.2

(iii) Loading Unit of 5000 kg Capacity - Strain controlled type.

(iv) High Sensitive Proving Ring of 200 kg Capacity

4. Procedure

(i) Preparation of Specimen from Undisturbed Soil Samples as well as Preparation of Specimen from Disturbed Soil Sample: Same as for "Consolidometer Method".

(ii) Keep the consolidation specimen ring with the specimen between two porous stones saturated in boiling water providing a filter paper between the soil specimen and the porous stone. The loading block shall then be positioned centrally on the top of the porous stone.

(iii) Then place this assembly on the platen of the loading unit. The load measuring proving ring tip attached to the load frame shall be placed in contact with the consolidation cell without any eccentricity. A direct strain measuring dial gauge shall be fitted to the cell. Inundate the specimen with distilled water and allow it to swell.

(iv) Note down the initial reading of the proving ring. The swelling of the specimen with increasing volume shall be obtained in the strain measuring load gauge. To keep the specimen at constant volume, the platen shall be so adjusted that the dial gauge always show the original reading. This adjustment shall be done at every 01 mm of swell or earlier. The duration of test shall conform to the requirements given in "Consolidometer method". Then dismantle the assembly and extract the soil specimen from the consolidation ring to determine final moisture content in accordance with IS: 2720(Part-IX).

5. Observations: The observations shall be recorded as per the format given in Table: 10.2.

Table: 10.2 Swelling Pressure Test by Constant Volume Method

Form-I

Project : Dated:

Tested By:

1. Details of soils sample
   - (i) Location
   - (ii) Boring No.
   - (iii) Depth
   - (iv) Visual description of soil
   - (v) Liquid limit
   - (vi) Plasticity index

(vii) Percentage of soil fraction below 0.002mm

2. Details of soil specimen
   (i) Undisturbed or remoulded
   (ii) Specific gravity of the soil

Natural Density			Moisture Content		
Description	Test-1	Test-2	Description	Test-1	Test-2
Weight of container ring+ wet specimen			Weight of container + wet soil		
Weight of container			Weight of container + dry soil		
Diameter of container			Weight of container		
Initial thickness of soil sample			Weight of water		
Wet density in g/ml			Weight of dry soil		
Dry density in g/ml			Moisture content in %		

Swell Pressure Data

Date	Time	Strain Dial Gauge Reading Before Adjustment	Proving Ring Reading	Differences	Load in kg	Swell Pressure in kg/cm^2	Remarks

6. Calculations and Report: The difference between the final and initial dial readings of the proving ring gives total load in terms of division which when multiplied by the calibration factor gives the total load. This when divided by the cross-sectional area of the soil specimen gives the swell pressure expressed in kN/m^2 (kgf/cm^2).

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PART – B

DETERMINATION OF ENGINEERING PROPERTIES OF SOIL

(1) Direct Shear Test (Chapter – 11)

(2) Tri-axial Shear Test (Chapter – 12)

(3) Lab Vane Shear Test (Chapter – 13)

(4) California Bearing Ratio (CBR) Test (Chapter – 14)

(5) Consolidation Test (Oedometer Test) (Chapter – 15)

(6) Los Angeles Abrasion Value Test (Chapter – 16)

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Chapter-11

DIRECT SHEAR TEST

1. **Introduction:**For any soil, Cohesion (C) and angle of Internal Friction(Ø)are two important engineering properties, which indicate the shear strength of soil. These two parameters are required for design of slopes, calculation bearing capacity of any strata, calculation of consolidation parameters and in many other analyses. Direct shear test is used to predict these parameters quickly, especially in cohesionless soils.

2. **Reference:**IS-2720(Part-13):1986(Reaffirmed-2016) "Methods of test for soils: Direct shear test".

3. **Apparatus required:**

   (i) Direct shear box, as per IS:11229-1985 (Reaffirmed-2015).

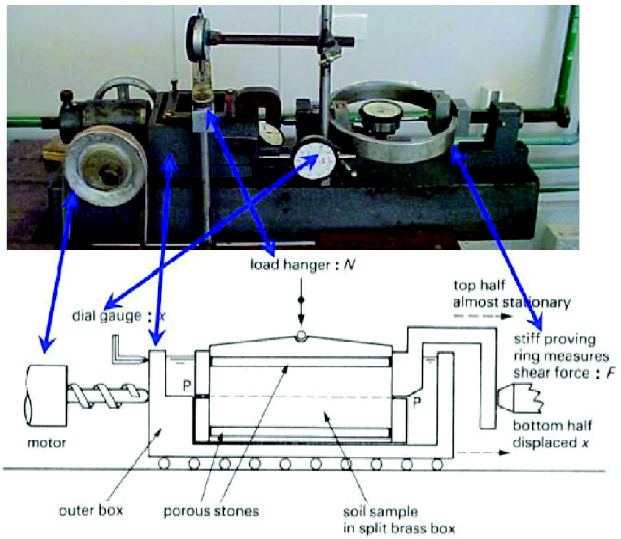


Fig. 11.01: Direct Shear Box

The direct shear box shall consist of following:

(a) Upper and lower parts of shear box coupled together with two pins,

(b) Grid plates - 2 pairs,

(c) Stone plates,

(d) 4 Base plate,

(e) Top plate,

(f) Loading pad, and

(g) Water jacket.

(ii) Loading Frame- It shall satisfy the following requirements:

(a) The vertical stress on the sample shall remain vertical and constant during the test and there shall be arrangement to measure compression.

(b) The shear stress or strain can be applied in the dividing plane of thetwo parts of the shear box.

(c) It shall be possible to maintain a constant rate of increase in stress during the test (irrespective of the strain rate) with arrangement to get different rates of stress increase.

(d) In case of a strain-controlled apparatus, the strain rate should remain constant irrespective of the stress. Suitable arrangement shall be provided to obtain different strain rates.

(e) No vibrations should be transmitted to the sample during the testand there should not be any loss of shear force due to friction between the loading frame and the shear box-container assembly.

(iii) Weights- for providing the required normal loads, if necessary.

(iv) Proving Ring- force measuring of suitable capacity, fitted with adial gauge accurate to 0.002mm to measure the shear force.

NOTE:For normal testing, proving rings of 100 to 250kg capacity, depending on the type of soil and the normal load on the sample during test, may be needed.

(v) Micrometer Dial Gauges- accurate to 0.01mm; one suitably mounted to measure horizontal movement and the other suitably mounted to measure the vertical compression of the specimen.

(vi) Sample Trimmer or Core Cutter

(vii) Stop Clock

(viii) Balance-of 1 kg capacity, sensitive to 0.1 g.

(ix) Spatula and a Straight Edge

4. Procedure:

4.1 Preparation of Specimen

(A) Undisturbed Specimens - Specimens of required size shall be prepared from the undisturbed specimen collected from field.

(B) Remoulded Specimens

(i) Cohesive soils may be compacted to the required density and moisture content, the sample extracted and then trimmed to the required size. Alternatively, the soil may be compacted to therequired density and moisture content directly into the shear boxafter fixing the two-halves of the shear box together by means of the fixing screws.

(ii) Cohesionless soils may be tamped in the shear box itself with the base

plate and grid plate or porous stone as required in place at the bottom of the box.

The cut specimen shall be weighed and trimmings obtained during cutting shall be used to obtain the moisture content. Using this information,the bulk dry density of the specimen in the shear box shall be determined.

4.2 Undrained Test: The shear box with the specimen, plain grid plateover the base plate at the bottom of the specimen, and plain grid plate atthe top of the specimen should be fitted into position in the load frame.The serrations of the grid plates should be at right angles to the direction of shear *(see Note)*. The loading pad should be placed on the top grid plate. The water jacket should be provided so that the sample does not get dried during the test. The required normal stress should be applied and the rate of longitudinal displacement/shear stress application so adjusted that no drainage can occur in the sample during the test. The upper part of the shear box should be raised such that a gap of about 1mm is left between the two parts of the box. The test may now be conducted by applying horizontal shear load to failure or to 20 percent longitudinal displacement,whichever occurs first. The shear load readings indicated by the proving ring assembly and the corresponding longitudinal displacements should be noted at regular intervals. If necessary, the vertical compression, if any, of the soil specimen may be measured to serve as a check to ensure that drainage has not taken place from the soil

specimen. At the end of thetest, the specimen should be removed from the box and the final moisture content measured. A minimum of three (preferably four) tests shall be made on separate specimens of the same density.

Note: As porous stones are not used for the undrained tests, plain plates of equal thickness should be substituted in their place so as to maintain the shear plane in the sample in the middle of its thickness.

4.3 Consolidated Undrained Test : The apparatus should be assembled in a similar way to that given in Para-4.2 except that instead of the plain gridplates, perforated grid plates and saturated porous stones should be used at the top and bottom of the specimen. The procedure is same as in Para-4.2 except that after the application of normal stress, the vertical compression of the soil with time should be recorded.The shear test should be conducted only after complete consolidation has occurred under the particular normal stress. The rate of shear should be such that water does not drain from the specimen at the time of application of the shear load. At the end of the test, the specimen should bere moved from the box and the final moisture content measured. A minimum of three (preferably four) tests should be made on separated specimens of the same density at different normal stresses.

4.4 Consolidated Drained Test-The shear box with sample and perforated grid plates and porous stones should be fitted into the load frame as in Para-4.3. After application

of normal stress which is done in increments, the sample should be allowed to consolidate. When the consolidation has completely occurred, the shear test should be done at such a slow rate that at least 95 percent pore pressure dissipation occurs during the test in this calculated time factor (see Appendix-A). At the end of the test, the specimen should be removed from the box and the final moisture content measured. A minimum of three (preferably four) tests should be made on separate specimens of the same density at different normal stresses.

4.5 The normal stresses to be selected for the test should correspond to the field conditions and design requirements.

5. Calculations and Report:

5.1 Results of tests shall be recorded suitably. A recommended proforma for recording the results is given in Appendix-B.

5.2 From the calibration chart of the proving ring, the loads corresponding to the load dial readings obtained during the test should be calculated. The loads so obtained divided by the corrected cross-sectional area of the specimen gives the shear stress in the sample. The corrected cross-sectional area shall be calculated from the following equation:

$$\text{Corrected area} = A_0 (1 - \delta/3)$$

Where:

A_0 = initial area of the specimen in cm^2, and

δ = displacement in cm.

5.3 The longitudinal displacement at a particular load may beeither noted directly from the strain dial readings or calculated as the product of the corresponding time reading and the strain rate, allowing for the compression of the provingring, where applicable. The stress-longitudinal displacement readings should be plotted and the maximum stress and corresponding longitudinal displacement together with the normal load applied during the test recorded (see Note).

Note: In general, failure in direct shear may be considered to take place either at maximum shear or at the maximum obliquity of the Mohr failure envelope.If the failure is assumed to take place at maximum shear and not at maximum obliquity, the angle of shearing resistance thus obtained will be smaller, giving anerror, if any, on the safe side. It should, however, be noted that differences in the values of the angle of shearing resistance obtained by using the two criteria mentioned above are more important for sands than for clays.

5.4 The maximum shear stress and the corresponding longitudinal displacement and applied normal stress should be recorded for each test and the results should be presented in the form of a graph in which the applied normal stress is plotted as abscissa and the maximum shearing stressis plotted as ordinate to the same scale. The angle which the resulting straight line makes with the horizontal axis and the intercept which the straight line makes with the vertical axis shall be reported as the angle of shearing resistance and cohesion intercept respectively (see Note).

Note: The normal stress-maximum shear stress relationship may not be astraight line in all cases. In such cases, the shear parameters may be ob-

tained by drawing a tangent to the normal stress expected in the field.

5.5 In case of the consolidated un drained and consolidated drained tests, the load at which the specimen is consolidated and the consolid-ation characteristics as determined during the consolidation part of the test should also be reported.

6. General Remarks:

6.1 In the shear box test, the specimen is not failing along its weakest plane but along a predetermined or induced failure plane i.e. horizontal plane separating the two halves of the shear box. This is the main draw back of this test. Moreover, during loading, the state of stress cannot be evaluated. It can be evaluated only at failure condition i.e. Mohr's circle can be drawn at the failure condition only. Also failure is progressive.

6.2 Direct shear test is simple and faster to operate. As thinner specimens are used in shear box, they facilitate drainage of pore water from a saturated sample in less time. This test is also useful to study friction between two materials, one material in lower half of box and another material in the upper half of box.

6.3 The angle of shearing resistance of sands depends on state of compaction, coarseness of grains, particle shape and roughness of grain surface and grading. It varies between 28^{o} (uniformly graded sands with round grains in very loose state) to 46^{o} (well graded sand with angular grains in dense state).

6.4 The volume change in sandy soil is a complex phenomenon depending on gradation, particle shape, state and type of packing, orientation of principal planes, principal stress ratio, stress history, magnitude of minor principal stress, type of apparatus, test procedure, method of preparing specimen etc. In general loose sands expand and dense sands contract in volume on shearing. There is a void ratio at which either expansion contraction in volume takes place. This void ratio is called critical void ratio. Expansion or contraction can be inferred from the movement of vertical dial gauge during shearing.

6.5 The friction between sand particles is due to sliding and rolling friction and interlocking action.The ultimate values of shear parameter for both loose sand and dense sand approximately attain the same value so, if angle of friction value is calculated at ultimate stage, slight disturbance in density during sampling and preparation of test specimens will not have much effect.

6.6 IS:2720 (Part-13) covers the methods for determination of shear strength of soil with a maximum particle size of 4.75mm in undrained,consolidated undrained and consolidated drained conditions.

Note: The undrained test can be performed only for highly impermeableclays. When silty clays and silts are involved, partial drainage is inevitable. This fact should be recognized while interpreting the results.

APPENDIX - A

(Clause 4.4)

RATE OF SHEAR FOR CONSOLIDATED DRAINED TEST

A-1 RATE OF STRAIN

A-1.1 For sandy soils, a rate of strain of 0.2 mm/min may be suitable.For clayey soils, a rate of strain of 0.01 mm/min or slower may be used but actual rate of strain suitable for the soil under test may be ascertained as in A-1.1.1.

A-1.1.1 From the consolidation data collected, the compression dial readings should be plotted against the logarithm of time and from this curve, the value of coefficient of consolidation, C_V, should be computed from the formula:

$$C_v = 0.197\ h^2 / t_{50}$$

Where:

2h = initial thickness of the specimen, and

t_{50} = time corresponding to 50 percent consolidation.

A-1.1.2 The requisite time to failure when theoretically 95 percentdissipation is ensured, may be obtained from the following equation:

$$t_f = h^2 / n\ C_v\ (1 - U_c) = 20\ h^2 / 3\ C_v$$

Where:

t_f = time to failure,

2h = initial thickness of the specimen,

n = a constant for drainage from both ends = 3, and

U_C = degree of pore pressure dissipation.

From a knowledge of approximate strain expected at failure, the rate of strain for the test may be calculated. In the case of cohesive soils, the failure may be assumed as taking place at 5 percent deformation.

APPENDIX - B

(Clause 5.1)

PROFORMA FOR RECORDING TEST RESULTS

Project _____ Location of samples _____
Bore hole No. ___ Sample No. ____
Rate of strain _____ Proving-ring/load cell No. _____
Calibration curve _____
Load-hanger lever ratio _____

Soil Specimen Measurements

Dimensions _____ Area of specimen _____
Thickness _____ Volume of specimen _____
Initial wet weight of specimen _____
Moisture content _____ (Average of _____ tests)
Bulk density _____
Final wet weight of the specimen _____
Moisture content at shear zone _____

Consolidation

Hanger load _____ Applied load _____
Normal stress _____

Date and Time	Vertical Dial Reading	Vertical Dial Difference	Thickness of Specimen

Shearing Stage

* Rate of shearing _____ mm/min

(* Should be decided after analyzing consolidation-time data in case of drained tests)

Date and Time	
Displacement Dial Reading	
Displacement d	
Area Correction	
Corrected Area	

Stress Dial reading	
Shear Force	
Shear Stress	
Vertical Dial Reading	
Vertical Dial Difference	
Thickness of Specimen	

Plot shear stress-shear displacement curve and find:

(a) Maximum shear stress, and

(b) Corresponding shear displacement.

Summary of Results

Test No.	
Normal Stress	
Shear Stress at Failure	
Shear Displacement at Failure	
Initial Water Content	
Final Water content	
Remarks	

Plot shear normal stress displacement curve and find:

(a) Cohesion intercept, and

(b) Angle or shearing resistance.

Chapter-12

TRIAXIAL SHEAR TEST

1. **Introduction:** The tri-axial shear test is most versatile of all the shear test testing methods for getting shear strength of soil i.e. Cohesion (C) and Angle of Internal Friction(Ø), though it is bit complicated. This test can measure the total as well as effective stress parameters both. These two parameters are required for design of slopes, calculation bearing capacity of any strata, calculation of consolidation parameters and in many other analyses. This test can be conducted on any type of soil, drainage conditions can be controlled, pore water pressure measurements can be made accurately and volume changes can be measured. In this test, the failure plane is not forced, the stress distribution of failure plane is fairly uniform and specimen can fail on any weak plane or can simply bulge.

2. **Reference:**

   (i) IS-2720 (Part-11): 1993 (Reaffirmed- 2016) "Determination of the shear strength parameters of a specimen tested in unconsolidated undrained triaxial compression without the measurement of pore water pressure".

   (ii) IS-2720 (Part-12):1981(Reaffirmed - 2016) "Determination of the shear strength parameters of soil from consolidated undrained triaxial compr-ession with the measurement of pore water pressure".

3. **Apparatus required :** Many variants of apparatuses, with different types of components, are used for conducting trial compression test on soil. Even the I.S. code also stipulates the minimum required

functionalities in the equipment to be used and permits use of variants. Therefore, the apparatus listed below are typical one and there can be many variants of this.

(i) Tri-axial test apparatus(Fig. 12.01):It consists of a circular base that has a central pedestal. The pedestal has one or two holes which are used for the drainage of the specimen in a drained test of for the pore pressure measurement in an undrained test. A triaxial cell is fitted to the top of the base plate with the help of 3 wing nuts after the specimen has been placed on the pedestal. The triaxial cell is a Perspex cylinder which is permanently fixed to the top cap and the bottom brass collar. There are three tie rods which support the cell. The top cap is a bronze casting with its central portion forming a bush through which a stainless steel ram can slide. The ram is so designed that it has minimum of friction and at the same time does not permit any leakage. There is an air-release valve in the top cap which is kept open when the cell is filled with water for applying the confining pressure. An oil valve is also provided in the top cap to fill light machine oil in the cell to reduce the leakage of water past the ram in long duration tests. The apparatus is mounted on a loading frame. The deviator stress is applied to the specimen from a strain controlled loading machine. The loading system consist of either a screw jack operated by an electric motor and gear box or a hydraulic ram operated by a pump.

The triaxial test apparatus has the following special attachments:

(a) Mercury Control System: The cell pressure in a triaxial test in maintained constant with a self-compensating mercury control system. It typically consists of two limbs of a water-mercury manometer. (Fig. 12.02). The pressure in the water of the triaxial cell develops due to the difference in levels of the mercury in the two pots. The water pressure at the centre of the specimen in the triaxial cell, at a height of h_3 above the datum, can be calculated using the theory of manometers. As the mercury surface in the upper pot is open to atmosphere, the (gauge) pressure there is zero. From the manometer equation:

$$0 + Y_m h_1 - Y_m h_2 - (h_3 - h_2)\, Y_w = \sigma_c$$

Where:

σ_c is the cell pressure at the centre of the specimen

Y_w is the unit weight of water, and

Y_m is the unit weight of mercury

The above equation can be simplified as:

$$\sigma_c = Y_m\, (h_1 - h_2) + (h_2 - h_3)\, Y_w$$

The upper pot is supported by a spring. When the volume of the specimen decreases due to consolidation or when the water leaks past the ram, water flows from the lower pot to the cell and the mercury level in the lower pot rises by a small amount Δh. The mercury level in the upper pot would also fall by the same amount if the two pots are of the same cross-sectional area. However, the difference of mercury levels in the two pots is maintained constant by the spring. The stiffness (k) of the spring is selected such that it reduces in length and causes

a rise of the upper pot as soon as its weight decreases due to flow of mercury. The stiffness of the spring is given by:

$$k = A\, y_m\, [1 / \{2 - (y_w/y_m)\}] - W$$

Where:

A = cross-sectional area of the mercury pot, and

W = Weight of unit length of tube filled with mercury which is also lifted above the floor.

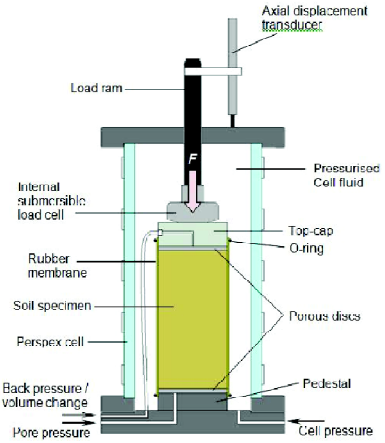


Fig. 12.01 : Triaxial Test Apparatus

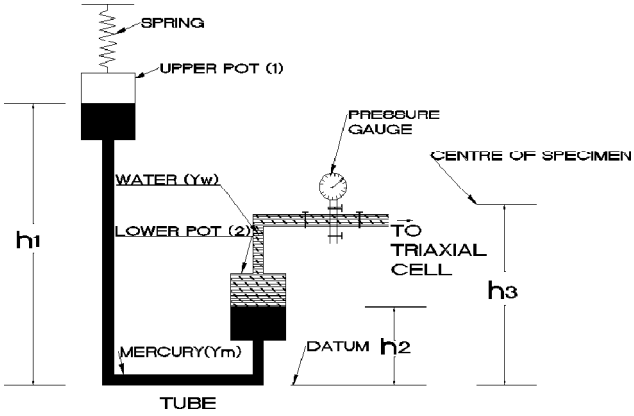


Fig. 12.02: Mercury Control System

(b) Pore water Pressure Measurement Device : The pore water pressure in the triaxial specimen is measured by attaching it to the device shown in Fig. 12.03. It consists of a null indicator in which no-flow condition is maintained. For accurate measurement, no-flow condition is essential because the flow of water from the sample to the gauge would modify the actual magnitude of the pore water pressure. Further, the flow of water leads to a time lag in the attainment of a steady state in samples of cohesive soils because of low permeability.

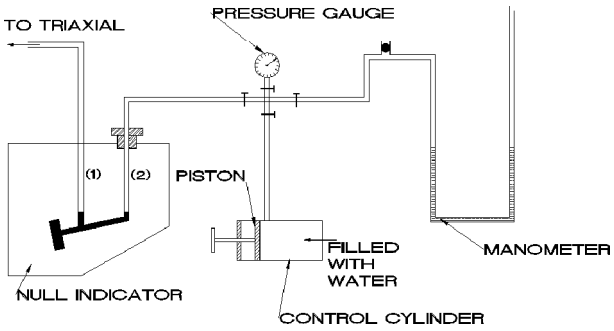


Fig. 12.03: Pore Water Pressure Measurement Device

The null indicator is essentially a U-tube partly filled with mercury. One limb of the null indicator is connected to the specimen in the triaxial cell and the other limb is connected to a pressure gauge. A control cylinder, which is filled with water, is attached to the system. The water can be displaced by a screw-controlled piston in the control cylinder. The whole system is filled with de-aired water. The tubes connecting the specimen and the null-

indicator should be such that these undergo negligible volume changes under pressure and are free from leakage.

Any change in the pore-water pressure in the specimen tends to cause a movement of the mercury level in the null-indicator. However, the no-flow condition is maintained by making a corresponding change in the other limbs by means of the control cylinder. Thus the mercury levels in the two limbs remain constant. The pressure applied by the control cylinder is recorded by pressure gauge or the manometer.

If the specimen is partially saturated, a special fine, porous ceramic disc is placed below the sample in the triaxial cell. The ceramic disc permits only pore water to flow, provided the difference between the pore air pressure and pore water pressure is below a certain value, known as the air-entry value of the ceramic disc. Under undrained conditions, the ceramic disc will remain fully saturated, provided the air-entry value is high. It may be mentioned that if the required ceramic disc is not used and instead the usual coarse, porous disc is used, the device would measure air pressure and not water pressure in a partially saturated soil.

In modern equipment, sometimes the pore water pressure is measured by means of a transducer and not by conventional null indicator.

(c) Volume Changes Measurement: Volume changes in a drained test and during consolidation stage of a consolidated undrained test are measured by means of a burette connected to the specimen in the triaxial cell.

For accurate measurements, the water level in the burette should be approximately at the level of the centre of the specimen (Fig. 12.04).

During consolidation stage, the volume of the specimen decreases and the water level in the burette rises. The change in the volume of the specimen is equal to the volume of the water increased in the burette. During shearing of specimens of dense sand when the volume of the sample increases, the water flows from the burette to the specimen. The increase in volume of the specimen is equal to the volume of water decreased in the burette.

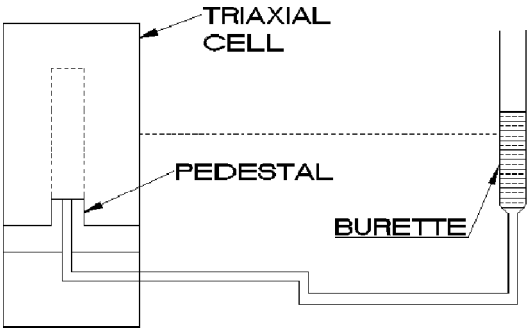


Fig. 12.04: Volume Change Measurement

4. **Triaxial test on Cohesive Soils:** The following procedure is used for conducting the triaxial tests on cohesive soils:

   4.1 Consolidated Undrained test : A de-aired, coarse porous disc or stone is placed on the top of the pedestal in the triaxial test apparatus. A filter paper disc is kept over the porous stone. The specimen of the cohesive soil is then placed over the filter paper disc. The usual size of the specimen is about

37.5mm diameter and 75.0mm height. A porous stone is also placed on the top of the specimen. De-aired vertical filter strips are placed at regular spacing around the entire periphery such that these touch both the porous stones. The sample in then enclosed in a rubber membrane, which is slided over the specimen with the help of a membrane stretcher. The membrane is sealed to the specimen with O-rings.

The triaxial cell is placed over the base and fixed to it by tightening the nuts. The cell is then filled with water by connecting it to the pressure supply. Some space in the top portion of the cell is filled by injecting oil through the oil valve. When excess oil begins to spill out through the air-vent valve, both the valves (oil valve and air-vent valve) are closed. Pressure is applied to the water fiiled in the cell by connecting it to the mercury-pot system. As soon as the pressure acts on the specimen, it starts consolidating. The specimen is connected to the burette through pressure connections for measurement of volume changes. The consolidation is complete when there is no more volume change.

When the consolidation is complete, the specimen is ready for being sheared. The drainage valve is closed. The pore water pressure measurement device is attached to the specimen through the pressure connections. The proving ring dial gauge is set to zero. Using the manual control provided in the loading frame, the ram is pushed into the cell but not allowed to touch the loading cap. The loading

machine is then run at the selected speed. The proving ring records the force due to friction and the upward thrust acting on the ram. The machine is stopped, and with the manual control, the ram is pushed further into the cell bringing it in contact with the loading cap. The dial gauge for the measuring axial deformation of the specimen is set to zero.

The sample is sheared by applying the deviator stress by the loading machine. The proving ring readings are generally taken corresponding to axial strains of 1/3%, 2/3%, 1%, 2%, 3%,4%, 5%, until failure or 20% axial strain.

Upon completion of the test, the loading is shut off. Using the manual control, all additional axial stress is removed. The cell pressure is then reduced to zero and the cell is emptied. The triaxial cell is unscrewed and removed from the base. O-rings are taken out, and the membrane is removed. The specimen is then recovered after removing the loading cap and the top porous stone. The filter paper strips are peeled off. The post-shear mass and length are determined. The water content of the specimen is also found.

4.2 Unconsolidated Undrained test: The procedure is similar to that for a consolidated-undrained test, with one basic difference that the specimen is not allowed to consolidate in the first stage. The drainage valve during the test is kept closed. However, the specimen can be connected to the pore-water pressure measurement device if required.

Shearing of the specimen is started just after the application of the cell pressure.

The second stage is exactly the same as in the consolidated-undrained test described above.

4.3 Consolidated Drained test: The procedure is similar to that for a consolidated-undrained test, with one basic difference that the specimen is sheared slowly in the second stage. After the consolidation of the specimen in the first stage, the drainage valve is not closed. It remains connected to the burette throughout the test. The volume changes during the shearing stage are measured with the help of the burette. As the permeability of cohesive soils is very low, it takes 4-5 days for the consolidated drained test.

5. Triaxial tests on Cohesionless Soils: Triaxial tests on specimens of cohesionless soils can be conducted using the procedure as described for cohesive soils. As the samples of cohesionless soils cannot stand of their own, a special procedure is used for preparation of the sample as described below.

(i) A metal former, which is a split mould of about 38.5mm internal diameter, is used for the preparation of the sample (Fig. 12.05). A coarse porous stone is placed on the top of the pedestal of the triaxial base and the pressure connection is attached to a burette (not shown). One end of a membrane is sealed to the pedestal by O-rings. The metal former is clamped to the base. The upper metal ring of the former is kept inside the top end of the rubber membrane and is held with the help of clamp before placing the funnel and the rubber bung in position as shown in figure.

(ii) The membrane and the funnel are filled with de-aired water. The cohesionless soil

which is to be tested is saturated by mixing it with enough water in a beaker. The mixture is boiled to remove the entrapped air. The saturated soil is deposited in the funnel, with a stopper in position, in the required quantity. The glass rod is then removed and the sample builds up by a continuous rapid flow of saturated soil in the former. The funnel is then removed. The funnel is then removed. The sample may be compacted if required. The surface of the sample is leveled and a porous stone is placed on its top. The loading cap is placed gently on the top porous stone. O-rings are fixed over the top of the rubber membrane.

(iii) A small negative pressure is applied to the sample by lowering the burette. The negative pressure gives rigidity to the sample and it can stand without any lateral support. For sample of 37.5mm diameter, a negative pressure of 20cm of water (or 2 kN/m^2) is sufficient. As soon as the negative pressure is applied, the consolidation of the sample occurs and it slightly shortens. The diameter of the upper porous stone should be slightly smaller than that of the specimen so that it can go inside when the sample shortens; otherwise, a neck is formed.

The split mould is then removed and the diameter and the height of the sample are measured. The thickness of the membrane is deducted from the total diameter to get the net diameter of the sample. The cell is then placed over the base and clamped to the base. It is then filled with water.

The rest of the procedure is the same as for cohesive soils.

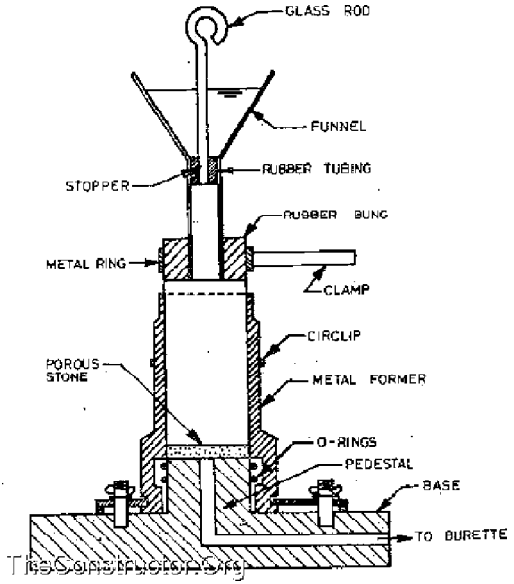


Fig. 12.05: Sample preparation for Cohesionless Soil

6. Merits and demerits of Triaxial Test : The triaxial test has the following merits and demerits.

6.1 Merits :

(i) There is complete control over the drainage conditions. Tests can be easily conducted for all three types of drainage conditions.

(ii) Pore pressure changes and the volumetric changes can be measured directly.

(iii) The stress distribution on the failure plane is uniform.

(iv) The specimen is free to fail on the weakest plane.

(v) The state of stress at all intermediate stages upto failure is known. The Mohr circle can be drawn at any stage of shear.

(vi) The test is suitable for accurate research work. The apparatus is adaptable to special requirements such as extension test and tests for different stress paths.

6.2 Demerits:

(i) The apparatus is elaborate, costly and bulky.

(ii) The drained test takes a longer period as compared with that in a direct shear test.

(iii) The stain condition in the specimen are not uniform due to frictional restraint produced by the loading cap and the pedestal disc. This leads to the formation of the dead zones at each end of the specimen. The non-uniform distribution of stresses can be largely eliminated by lubrication of end surfaces. However, non-uniform distribution of stresses has practically no effect on the measured strength if length/diameter ratio is equal to or more than 2.0

(iv) It is not possible to determine the cross-sectional area of the specimen accurately at large strains, as the assumption that the specimen remains cylindrical does not hold good.

(v) The test stimulates only axis-symmetrical problems. In the field, the problem is generally 3-dimensional. A general test in which all the three stresses are varied would be more useful.

(vi) The consolidation of the specimen in the test is isotropic; whereas in the field, the consolidation is generally anisotropic.

Despite the above mentioned demerits, the triaxial test is extremely useful. It is the only reliable test for accurate determination of the shear characteristics of all types of soils and under all the drainage conditions.

7. Computation of various Parameters:

7.1 Post-Consolidation Dimensions : In consolidated - drained and consolidated-undrained tests, the consolidation of the specimen takes place during the first stage. As the volume of the specimen decreases, its post-consolidation dimensions are different from the initial dimensions. The post consolidation dimensions can be determined approximately assuming that the sample remains cylindrical and it behaves isotropically. Let L_i, D_i and V_i be the length, diameter and the volume of the specimen before consolidation. Let L_0, D_0 and V_0 be the corresponding quantities after consolidation.

Volumetric change, $\Delta V_i = V_i - V_0$

Volumetric change is measured with the help of burette.

Volumetric strain, $\in v = \Delta V_i / V_i$

For isotropic consolidation, the volumetric strain is three times the linear strain ($\in_l$), thus:

$$\in_l = \in v / 3$$

$$L_0 = L_i - \Delta L_i = L_i - L_i \times \in_l$$

$$L_0 = L_i (1 - \in_l) = L_i (1 - \in_l/3)$$

Similarly, $$D_0 = D_i (1 - \in_v/3)$$

The post consolidation diameter D_0 can also be computed after L_0 has been determined from the relation:

$$(\pi/4 . D_0^2) \times L_0 = V_0$$

$$\text{Or } D_0 = \sqrt{V_0 / [(\pi/4) \times L_0]}$$

7.2 Cross-sectional area during Shear Stage: As the sample is sheared, its length decreases and the diameter increases. The cross-sectional area A at any stage during shear can be determined assuming that the sample remains cylindrical in shape. Let ΔL_0 be the change in length and ΔV_0 be the change in volume. The volume of the specimen at any stage is given by $V_0 \pm \Delta V_0$.

Therefore, $A (L_0 - \Delta L_0) = V_0 \pm \Delta V_0$

Or, $A = V_0 \pm \Delta V_0 / L_0 - \Delta L_0 = V_0 (1 \pm \Delta V_0 / V_0) / L_0 (1 - \Delta L_0/L_0)$

This is the general equation which gives the cross-sectional area of the specimen and it can be written as:

$$A = A_0 L_0 (1 \pm \Delta V_0/V_0) / L_0 (1 - \in_l)$$
$$= A_0 (1 \pm \Delta V_0/V_0) / (1 - \in_l)$$

Where, $\in_1$ is the axial strain in the sample.

For an undrained test, the volumetric change (ΔV_0) is zero and then the equation above becomes:

$$A = A_0 / (1 - \in_l)$$

The stresses in the specimen at various stages of shear should be calculated using the cross-sectional area A as found above.

7.3 Stresses

(i) Deviator Stress: The deviator stress (sd) acting on the specimen when the axial load applied by the loading machine is P can be obtained as:

$$\sigma_d = P/A$$

The deviator stress (σ_d) is equal to ($\sigma_1 - \sigma_3$).

It may be noted that the load indicated by the proving ring is slightly more than P because of friction on the ram and the upward thrust on the ram due to pressure of the water in the cell. The correction can be determined separately.

A more convenient procedure is to lift the ram above the specimen when the cell pressure has been applied. The machine is started keeping the strain rate the same as to be used in the actual test. The proving ring records the load. To account for correction, the dial gauge on the proving ring is set to zero to indicator zero load. This automatically compensates the ram friction and the upward thrust on the ram due to cell pressure. Thus the load indicated by the proving ring during shear would be equal to the load P applied to the specimen.

(ii) Principal stresses: The minor principal stress (σ_3) is equal to the cell pressure (σ_c). The major principal stress (σ_1) is equal to the sum of the cell pressure and the deviator stress.

Thus, $\sigma_1 = \sigma_3 + (\sigma_1 - \sigma_3) = \sigma_3 + \sigma_d$

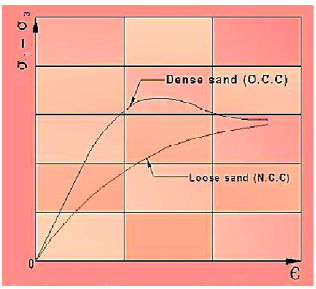
(iii) Compressive strength: The deviator stress at failure $(\sigma_1 - \sigma_3)_f$ is known as the compressive strength of the soil.

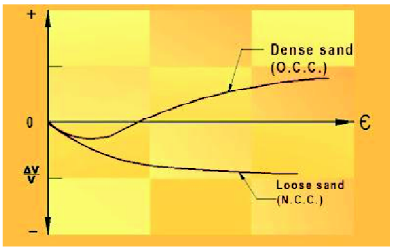
8. Presentation of results of Tri-axial tests

8.1 Stress-Strain Curves : Drained Test **:** Fig. 12.06 shows the stress-strain curve for a drained test. The y-axis shows the deviator stress ($\sigma_1 - \sigma_3$) and the x-axis, the axial strain ($\in_l$). For dense sand (and over-consolidated clay), the deviator

stress reaches a peak value and then it decreases and becomes almost constant, equal to the ultimate stress, at large stains. For loose sand (and normally consolidated clay), the deviator stress increases gradually till the ultimate stress is reached.

The volumetric strain is shown in Fig. 12.07. In dense sand (and over-consolidated clay), there is a decrease in the volume at low strains, but at large strains, there is an increase in the volume. In loose sand (and normally consolidated clay), the volume decreases at all strains. (For some loose sands, there is a slight tendency to increase in the volume at large strains).





Stress Strain Curves for Drained Test

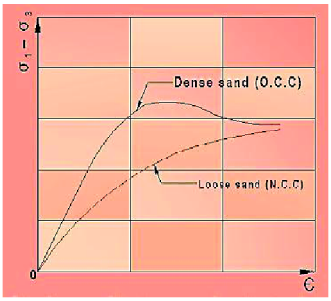
Fig. 12.06: Stress – Strain Curve

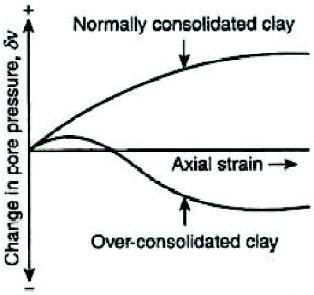
Fig. 12.07: Volumetric Strain

Consolidated-undrained test : Fig. 12.08 shows the stress-strain curve for a consolidated-undrained test. The shape of the curves is similar to that obtained in a consolidated-drained test. In a consolidated-undrained test, there is an increase in the pore water pressure

throughout for loose sand (and normally consolidated clay), as shown in Fig. 12.09. However, in the case of dense sands (and over-consolidated clay), the pore water pressure increases at low strains but at large strains it becomes negative (below atmospheric pressure).

8.2 Mohr Envelopes: For drawing the failure envelopes, it is necessary to test at least three samples at three different cell pressures in the stress range of interest. For dense sands and over-consolidated clays, the failure envelope can be drawn either for the peak stress or for the ultimate stress. For loose sands and normally consolidated clays, the failure envelope is drawn for the ultimate stress, which is usually taken at 20% strain. Further, the failure envelope can be drawn either in terms of effective stresses or in terms of total stress. Of course, the two envelopes will give different values of strength parameter(c and θ).





Stress – Strain Curves, CU Test

Fig. 12.08
Stress – Strain Curve

Fig. 12.09
Pore Pressure

Brittleness I_B is expressed as:

I_B = (Peak strength–Ultimate strength)/ Ultimate strength

(i) Effective Stresses: Fig. 12.10 shows the failure envelope for a normally consolidated clay in terms of effective stresses obtained from a consolidated drained test. The failure envelope has an angle of shearing resistance of θ and passes through origin. First the Mohr circles for the three tests are drawn in terms of effective stresses corresponding to failure conditions. Then the best common tangent is drawn to the three circles. The common tangent is the failure envelope. As each circle represents a failure, there must be at least one point on it which gives the stresses satisfying the failure criterion. Obviously, the common tangent joins all such points of the three circles.Thus for normally consolidated clays, shear strength is:

$$s = \sigma' \tan \theta'$$

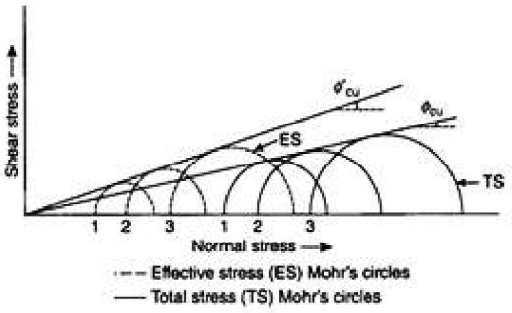


Fig. 12.10: Failure Envelopes for normally consolidated Clay

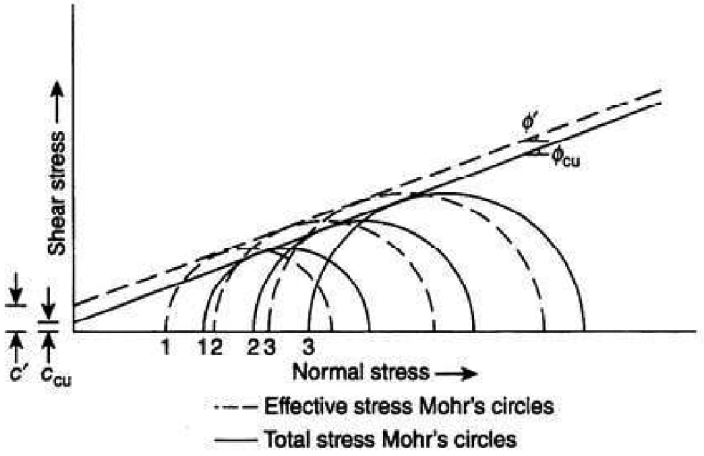


Fig. 12.11: Failure Envelopes for over consolidated Clay

Fig. 12.11 shows the failure envelope for over-consolidated clay in terms of effective stresses. The failure envelope is slightly curved in the initial portion, but, for convenience, it is approximated as a straight line. The failure envelope has an intercept c' on the Y-axis, the angle of shearing resistance is θ'. In the case of over-consolidated clays, shear strength is:

$$s = c' + \sigma' \tan \theta'$$

The failure envelopes in terms of effective stresses can also be drawn from the results of a consolidated-undrained test (CU test) when the pore water pressure measurements are also taken. The shear strength parameter c' and θ' obtained from the consolidated-undrained tests and that from consolidated-drained test are approximately equal. Drained tests on dense sands and over-consolidated clays

give slightly higher values of θ'due to extra work required during dilation (increase in volume), but the difference is small, and therefore, usually neglected.

(ii) Total stresses: The failure envelope in terms of total stresses can be drawn from the test results of a consolidated-undrained test.

*(**Note:** In consolidated-drained tests, the total stresses are also equal to the effective stresses, as the pore water pressure throughout is zero).*

The failure envelopes are similar in shape to that in terms of effective stresses but the values of the strength parameters are quite different. Fig. 12.10 shows the failure envelopes for effective stresses and also for total stresses for normally consolidated clay. The angle of shearing resistance in terms of total stresses (σ_{cu}) is much smaller than the angle (θ'). In the case of normally consolidated clays, shear strength is:

$$S = \sigma \tan \theta_{cu}$$

Fig. 12.11 shows the failure envelope for an over consolidated clay in terms of total stresses. The angle of shearing resistance (θ_{cu}) is much smaller than the angle θ' obtained in terms of effective stresses. In the case of over consolidated clays, shear strength is:

$$S = c_{cu} + \sigma \tan \theta_{cu}$$

The angle of shearing resistance θ_{cu} obtained from the total stress envelopes is also known as apparent angle of shearing resistance.

Fig. 12.12 shows the failure envelope in terms of total stress obtained from an unconsolidated-undrained test on normally consolidated clay. The failure envelope is horizontal (θ = 0) and has a cohesion intercept of c_u. In this case, shear strength is s = c_u. The failure envelope for over-consolidated clay is also horizontal, but the value of c_u will be more, depending upon the degree of over-consolidation.

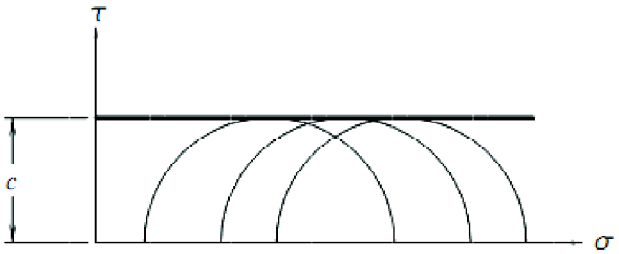


Fig. 12.12

For an unconsolidated-undrained test, the failure envelope cannot be drawn in terms of effective stresses. In all the tests conducted at different confining pressures, the effective stress remains the same. This is due to the fact that an increase in confining pressure results in an equal increase in pore water pressure for a saturated soil under undrained conditions. Thus only one Mohr circle in terms of effective stresses is obtained from all the three tests. It may be noted that the deviator stress at failure is the same for all specimens.

Chapter-13

LAB VANE SHEAR TEST

1. **Introduction:** The laboratory vane shear test for the measurement of shear strengthof cohesive soils is useful for soils of low shear strength of less than about 0.5 kgf/cm^2. This test gives the undrained strength of the soil, in undisturbed as well as remoulded conditions both.Vane shear test is a cheaper and quicker method of measuring the shear strength of clay, as compared to very elaborate tri-axial shear test or direct shear test.

2. **Reference:** IS-2720(Part-30):1980 (Reaffirmed-2016) “Methods of test for soils: Laboratory vane shear test”

3. **Apparatus required:**



Fig. 13.01: Lab Vane Shear Apparatus

(i) Vane: The vane shall consist of four blades each fixed at 90° to the adjacent blades.The width of vane, from edge of one

vane to edge of another vane, is 12±0.15 mm. The height of vane is 24±0.25 mm. The thickness of vane is 0.5±0.05 mm. The vane should not deform under the maximum torque for which it is designed. The penetrating edge of the vane blades shall be sharpened having an included angle of 90°. The vane blades shall be welded together suitably to a central rod, the maximum diameter of which should preferably not exceed 2.5 mm in the portion of the rod which goes into the specimen during the test. The height of rod, above the vanes, is 60mm. The vane should be properly treated to prevent rusting and corrosion.

It is important that the dimensions ofthe vane are checked periodically to ensure that the vane is not distorted or worn.

(ii) Vane Shear Apparatus: The apparatus may be either of the hand-operated type or motorized. Provisions should be made in the apparatus for the following:

(a) Fixing of vane and shaft to the apparatus in such a way thatthe vane can be lowered gradually and vertically into the soil specimen.

(b) Fixing the tube containing the soil specimen to the base of the equipment for which it should have suitable hole.

(c) Arrangement for lowering the vane into the soil specimen(contained in the tube fixed to the base) gradually and verticallyand for holding the vane properly and securely in the loweredposition.

(d) Arrangement for rotating the vane steadily at a rate of approximately 1/60 rev/min (0.1°/s) and for measuring the rotation of the vane.

(e) A torque applicator to rotate the vane in the soil and a device for measuring the torque applied to an accuracy of 0.05 cm.kgf.

(f) A set of springs capable of measuring shear strength of 0.5 kgf/cm^2.

4. Procedure:

4.1 The specimen in the tube should be at least 37.5mm in diameter and 75mm long. Mount the specimen container with the specimen on the base of the vane shear apparatus and fix it securely to the base. If the specimen container is closed at one end it should be provided at the bottom with a hole of about 1mm diameter. Lower the shear vanes into the specimen to their full length gradually with minimum disturbance of the soil specimen so that the top of the vane is at least 10mm below the top of the specimen. Note the readings of the strain and torque indicators. Rotate the vane at a uniform rate approximately 0.10^o/s by suitably operating the torque applicator handle until the specimen fails. Note the final reading of the torque indicator. Torque readings and the corresponding strain readings may also be noted at desired intervals of time as the test proceeds.

4.2 Just after the determination of the maximum torque rotate the vane rapidly through a minimum of ten revolutions. The remoulded strength should then be determined within 1 minute after completion of the revolution.

5. Calculations : For vane testing instruments that do not read the torque directly, a calibration curve to convert the readings to cm kgf of torque shall be provided. These calibration curves shall be

checked periodically. Calculate the shear strength of the soil using the following formula:

$$S = 3T/19$$

Where,

S = shear strength in kgf/cm^2, and

T = torque in cm.kgf.

6. General Remarks: The formula for shear strength is based on following assumptions:

(a) Shearing Strength in the Horizontal and Vertical directions are the same.

(b) At the peak value, Shear Strength is equally mobilized at the end surface as well as at the centre, and

(c) The shear surface is cylindrical and has a diameter equal to the diameter of the vane.

Chapter-14

CALIFORNIA BEARING RATIO (CBR) TEST

1. **Introduction:** The CBR value of a soil can be considered to be an index which in some fashion is related to its strength. The value is highly dependent on the condition of the material at the time of testing. Empirical correlations have been developed to correlate CBR values to parameters like modulus of subgrade reaction, modulus of resilience and plasticity index. For Railway Formation purpose, the test is performed on remoulded specimens which are compacted dynamically and the CBR value is used as a parameter to judge the suitability or otherwise of the any soil/blended material for use as embankment fill, sub-grade, prepared sub-grade and blanket. The methodology given here covers the laboratory method for the determination of CBR value of remoulded/compacted soil specimens in soaked state.

2. **Reference:**

   (i) IS-2720 (Part-16) : 1987 (Reaffirmed-2016) "Methods of test for soils: Laboratory determination of CBR"

   (ii) RDSO report No. RDSO/2009/GE:G-0014- Guidelines and Specification for Design of Formation for Heavy Axle Load.

3. **Apparatus required :**

   (i) CBR Test Apparatus: Consisting of Loading machine with capacity of atleast 5000 kg and equipped with a movable head or base which enables Plunger of 50 mm dia. to penetrate into the specimen at a rate of 1.25 mm/ minute.

(ii) Cylindrical mould: Inside dia. 150mm and height 175mm with a detachable perforated base plate of 235mm dia. and 10mm thickness. Net capacity-2250 ml; conforming to IS-9669:1980 (Reaffirmed-2016).

(iii) Collar: A detachable extension collar of 60mm height.

(iv) Spacer Disc: 148mm in diameter and 47.7mm in height along with handle.

(v) Metal rammers: Weight 4.89 kg with a drop 450mm.

(vi) Weights: One annular metal weight and several slotted weights weighing 2.5 kg each, 147mm in dia., with a central hole of 53mm in diameter.

(vii) Dial Gauge: Two dial gauges reading to 0.01mm.

(viii) Sieves: 4.75mm and 19mm size, as per IS:460 (Part-1)–1985.

(ix) Miscellaneous apparatus, such as a mixing bowl, straight edge, scales, soaking tank or pan, drying oven, filter paper and containers.



Fig. 14.01: CBR Test Apparatus

4. Procedure:

4.1 Preparation of Test Specimen:

(i) Remoulded specimen: The test material should pass 19mm IS sieve and retained on 4.75mm IS sieve. The dry density for a remoulding shall be either the field density or the value of the maximum dry density estimated by the compaction test, Heavy Compaction Test as per IS-2720 (Part-8):1983, for Railway Formation. The water content used for compaction shall be the optimum water content or the field moisture as the case may be.

(ii) Dynamic Compaction: A representative sample of the soil weighing approximately 4.5 kg or more for fine grained soil and 5.5 kg or more for granular soil shall be taken and mixed thoroughly with water. If the soil is to be compacted to the maximum dry density at the optimum moisture content, the exact mass of the soil required shall be taken and the necessary quantity of water added so that the water content of the soil sample is equal to the determined optimum moisture content.

(iii) Fix the extension collar and the base plate to the mould. Insert the spacer disc over the base. Place the filter paper on the top of the spacer disc.

(iv) Apply Lubricating Oil to the inner side of the mould. Compact the mix soil in the mould using heavy compaction i.e. compact the soil in 5 layers with 55 blows to each layer by the 4.89 kg rammer.

(v) Remove the extension collar and trim the compacted soil carefully at the level of top of mould, by means of a straight edge. Any holes developed on the surface of the compacted soil by removal of the coarse material, shall be

patched with the smaller size material. Remove the perforated base plate, Spacer disc and filter paper and record the mass of the mould and compacted soil specimen. Place a disc of coarse filter paper on the perforated base plate, invert the mould and compacted soil and clamp the perforated base plate to the mould with the compacted soil in contact with the filter paper.

(vi) Place a filter paper over the specimen and place perforated plate on the compacted soil specimen in the mould. Put annular weights to produce a surcharge equal to weight of base material and pavement, to the nearest 2.5 kg.

(vii) Immerse the mould assembly and weights in a tank of water and soak it for 96 hours. Mount the tripod for expansion measuring device on the edge of the mould and record initial dial gauge reading. Note down the readings every day against time of reading. A constant water level shall be maintained in the tank throughout the period.

(viii) At the end of soaking period, note down the final reading of the dial gauge and take the mould out of water tank.

(ix) Remove the free water collected in the mould and allow the specimen to drain for 15 minutes. Remove the perforated plate and the top filter paper. Weigh the soaked soil sample and record the weight.

4.2 Procedure for Penetration Test

(i) Place the mould assembly with test specimen on the lower plate of penetration testing machine. To prevent upheaval of soil into the hole of the surcharge weights, 2.5 kg annular weight shall be placed on the soil surface prior to seating the penetration plunger after which

the remainder of the surcharge weights shall be placed.

(ii) Seat the penetration piston at the center of the specimen with the smallest possible load, but in no case in excess of 4 kg so that full contact of the piston on the sample is established.

(iii) Set the load and deformation gauges to read zero. Apply the load on the piston so that the penetration rate is about 1.25 mm/min.

(iv) Record the load readings at penetrations of 0.5, 1.0, 1.5, 2.0, 2.5, 4.0, 5.0, 7.5, 10 and 12.5 mm.

(v) Raise the plunger and detach the mould from the loading equipment. Take about 20 to 50 g of soil from the top 30 mm layer and determine the moisture content.

5. Observation and Recording: Record the load readings corresponding to the specified penetration values:

Penetration (mm)	Applied Load (kgf)	Penetration (mm)	Applied Load (kgf)
0.5 mm		4.0 mm	
1.0 mm		5.0 mm	
1.5 mm		7.5 mm	
2.0 mm		10.0 mm	
2.5 mm		12.5 mm	

The following table gives the standard loads:

Penetration of Plunger (mm)	Standard Load (kgf)
2.5	1370
5.0	2055

6. Calculations: If the initial portion of the curve is concave upwards (Fig. 14.02), apply correction by drawing a tangent to the curve at the point of greatest slope and shift the origin. Find and record

the correct load reading corresponding to each penetration.

$$\text{CBR Value(in \%)} = (P_T/P_S) \times 100$$

Where,

P_T = Corrected test load corresponding to the chosen penetration from the load penetration curve.

P_S = Standard load for the same penetration taken from the table above.

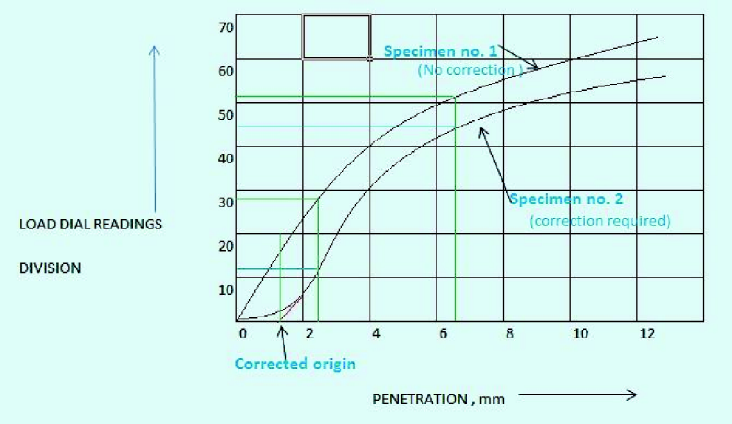


Fig. 14.02: Plot of Load versus Settlement

CBR of specimen at 2.5 mm penetration =

CBR of specimen at 5.0 mm penetration =

The CBR values are usually calculated for penetration of 2.5mm and 5mm. Generally the CBR value at 2.5mm will be greater than at 5mm and in such a case/the former shall be taken as CBR for design purpose. If CBR for 5mm exceeds that for 2.5mm, the test should be repeated. If identical results follow, the CBR corresponding to 5mm penetration should be taken for design.

Chapter-15

CONSOLIDATION TEST (OEDOMETER TEST)

1. **Introduction:** Consolidation test isused to determine the rate and magnitude of soil consolidation when the soil is restrained laterally and loaded axially. The Consolidation test is also referred to as Standard Oedometer test or One-dimensional compression test. This test is carried out on saturated soil specimens, especially in cohesive soils. The consolidation parameters obtained by this test are used to determine the consolidation settlement and time of consolidation for a given loading state (i.e. given height of embankment). These parameters are also used in design of "Ground Improvement measures", provided for construction of embankment on soft soils.
2. **Reference:** IS-2720(Part-15):1965(Reaffirmed-2016) "Methods of test for soils: Determination of Consolidation Properties".
3. **Apparatus Required:**



Fig. 15.01: Consolidation Test Apparatus

The "Consolidation Test Apparatus" (Fig. 15.01) consists of following parts:

(i) Consolidation Ring (Fig. 15.02): The ring shall be rigid and made of non-corrosive material. The inner surface shall be smooth and highly polished. Teflon coating is suitable for this purpose. Alternatively, silicon grease is good for stiff & medium soils and thick oil may be used for very soft soils. The ring shall be provided with a cutting edge to facilitate preparation of sample. The minimum inner dia. of ring shall be 60mm. If specimen are to be obtained by extruding and trimming, the inner dia. of ring shall be at least 10mm less than the inside dia. of the sampling tube. The height of ring shall not be less than 20mm, with diameter to height ratio of about 3. The specimen height shall not be less than 10 times the maximum particle size.

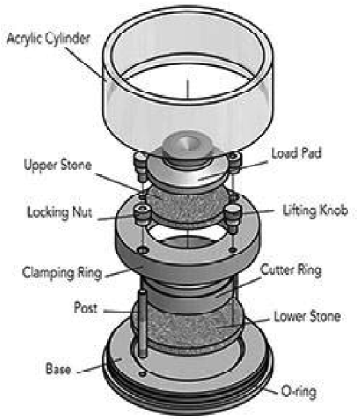


Fig. 15.02: Consolidation Ring

(ii) Porous stones: These stones shall be placed at top and bottom of the specimen and

shall be of silicon carbide, aluminium oxide or other porous material not attacked by the soil and not having any electro-chemical reaction with other parts of the equipment. The porosity of the stones shall be such that free drainage is assured throughout the test,but no intrusion of soil into the pores of thestones takes place. If necessary,a sheet of filter paperof diameter equal to that of the stone, may be placed between the stone and the soil surface, in order to prevent intrusion.

Note: Filter paper shall not be used when stiff claysare being tested since the paper has high compressibility.

(iii) Consolidation Cell: A container within which the consolidation ring is placed containing the specimen between the top and bottom porous stones. The cell shall be capable of being filled with water to a level higher than the top of the upper porous stone, of having an axial, vertical load applied to the top of the specimen and of allowing measurement of the change in height of the specimen on its central axis.

(iv) Dial Gauge: The gauge shall read to an accuracy of at least 0.01 percent of the specimen height, and have a travel of at least 50 percent of the specimen height.

(v) Loading Device:The device shall enable vertical force to be applied axially, in suitable increments, to the test specimen, through a suitable loading yoke. It shall be capable of maintaining specified loads for long periods of time while the specimen is deforming, with a variation of less than ±1 percent of the applied load.The force is applied to the loading cap of the specimen

centrally, through some form of spherical seating. The applied load is known to an accuracy of at least ± 1 percent.The loading device shall permit application of a load increment within a period of 2 seconds, without significant impact.

(vi) Equipment for Measuring Initial Height of Test Specimen to an accuracy of 0.1mm : Vernier reading Callipers, micrometer screw gauge or 25 mm travel dial-gauge mounted in comparator.

(vii) Moisture Content Containers, Drying Air-Oven Maintained at 110 ± 5^{0}C Desiccator

(viii) Balances Sensitive to 0.01g: for weighing the specimen and determining moisture content.

4. Procedure:

4.1 Preparation of Test Sample:

(i) Weigh the empty consolidation ring, designated W_1.

(ii) If the specimen is prepared from a tube sample, a representative sample shall be extruded and cut off, with two plane faces of the resulting soil disc being parallel to each other. The thickness of the disc of soil shall be somewhat greater than the height of the consolidation ring.

(iii) If the specimen is prepared from a block sample, a disc similar in size to that specified above shall be cut from the block with two parallel faces. The diameter of the disc shall be at least 10mm greater than the inside diameter of the consolidation ring. Care shall be taken to ensure that the soil stratum is oriented such that the laboratory test will load the soil in the same direction relative to the stratum as the applied force in the field.

(iv) Using the weighed consolidation ring as a template, the edges of the disc shall be trimmed carefully until the ring just slides over the soil. The last fraction of soil is pared away by the cutting edge of the ring as it is pushed down slowly and evenly over the sample with no unnatural voids against the inner face of the ring; this process is best done using a mechanical guide to prevent tilting or horizontal movement of the ring. The top and bottom surfaces shall project above and below the edges of the ring to enable final trimming.

(v) Should an occasional small inclusion interfere with the trimming operation, it shall be removed, and the cavity filled completely with material from the parings. Alternatively, if sufficient sample is available, it would be preferable to eventually extrude and discard the portion of the specimen containing the inclusion from the ring, leaving a specimen free of such disturbed zones. If inclusions are known to exist in a soil sample, a large diameter consolidation ring should be used, in order to minimize the relative effect of the disturbed zones. If excessive inclusions are encountered during trimming, the sample should be discarded. If no alternative exists, the tube sample shall be extruded directly into a consolidation ring of equal diameter.

(vi) The soil sample thus obtained shall be trimmed flush with the top and bottom edges of the ring. For soft to medium soils, excess soil should be removed using a wire saw, and final trimming may be done with a straight edge if necessary. For stiff soils a straight edge alone may be used for trimming. Excessive remoulding of the soil surface by the straight edge should be avoided. In the case of very soft soils, special care should be taken so that the specimen

may not fall out of, or slide inside the ring during trimming.

(vii) A sample of soil similar to that in the ring, taken from the trimmings, shall be used for determining moisture content.

(viii) The thickness of the specimen (H) shall be measured and it shall be weighed immediately (W_2). Should the nature of the soil make satisfactory thickness determination difficult, the ring height may be assumed as specimen height.

4.2 Assembly of Apparatus:

(i) The bottom porous stone shall be centered on the base of the consolidation cell. If soils sensitive to moisture increase (swelling or collapsing soils) are being tested, the stone should be placed dry. When testing softer clays, the stone should be wet, and it may be covered by a wet filter paper. No filter paper shall be used for the stiffer and moisture sensitive soils.

(ii) The ring and specimen shall be placed centrally on the bottom porous stone, and the upper porous stone and then the loading cap shall be placed on top. The top stone shall be placed dry or wet, and with or without filter paper.

(iii) The consolidometer shall be placed in position in the loading device and suitably adjusted. The dial gauge is then clamped into position for recording the relative movement between the base of the consolidation cell and the loading cap. A seating pressure of 0.05 kgf/ cm^2 shall be applied to the specimen.

(iv) The consolidation cell shall be filled with water, preferably with distilled water. The type of water used shall be noted in the data sheet

(v) The specimen shall then be allowed to reach equilibrium for 24 hours.

4.3 Loading:

(i) It is generally desirable that the applied pressure at any loading stage be double than that at the preceding stage. The test may, therefore, be continued using a loading sequence which would successively apply stress of 0.1, 0.2, 0.4, 0.8, 1.6, 3.2, etc.kgf/ cm^2 on the soil specimen.

(ii) For each loading increment, after application of load, readings of the dial gauge shall be taken using a time sequence such as 0, 0.25, 1, 2.25, 4,6.25, 9, 12.25, 16, 20.25, 25, 36, 49, 64, 81, 100, 121, 144, 169, 196, 225 etc., minimum up to 24 hour(s) or 0, 1/4, 1/2, 1, 2, 4, 8, 15, 30, 60 min, and 2, 4, 8, 24 hour(s). These time sequences facilitate plotting of thickness or change of thickness of specimen against square root of time or against logarithm of time.

The loading increment shall be left atleast until the slope of the characteristic linear secondary compression portion of the thickness versus log time plot is apparent, or until the end of primary consolidation is indicated on a square root of time plot. A period of 24 hours will usually be sufficient, but longer times may be required. If a period of 24 hours is seen to be sufficient, it is recommended that this commonly used load period be used for all load increments. In every case, the same load increment duration shall be used for all load increments during a consolidation test.

(iii) It is desirable that the final pressure be of the order of at least four times the pre-consolidation pressure, and be greater than the maximum effective vertical pressure which will occur in situ due to the overburden and the proposed construction.

(iv) On completion of the final loading stage, the specimen shall be unloaded by pressure decrements which decrease the load to one-fourth of the last load. Dial gauge readings may be taken as necessary during each stage of unloading. If desired, the time intervals used during the consolidation increments may be adopted; usually it is possible to proceed much more rapidly.

(v) In order to minimize swell during disassembly, the last unloading stage should be to 0.05 kgf/ cm^2 which should remain on the specimen for 24 hours. On completion of this decrement, the water shall be siphoned out of the cell and the consolidometer shall be rapidly dismantled after the release of the final load. The specimen, preferably within the ring, shall be wiped free of water, weighed (W_3), and there after placed in the oven for drying. If the ring is required for further testing, the specimen may carefully be removed from the ring in order to prevent loss of soil, and then weighed and dried.

(vi) Following drying, the specimen (plus ring) shall be reweighed (W_4).

(vii) The porous stones shall be boiled clean after the test, in order to prevent clay from drying on them and reducing their permeability.

5. Observation and Recording: The specimen data shall be recorded at the top of the data sheet shown in Appendix-A. This includes apart from soil identification etc., specific gravity of soil particles, the specimen measurements and water content determinations. The specimen preparationprocedure and the type of water used shall alsobespecified.

The data concerning dial readings with time for each pressure increment for both loading and unloading stages shall be recorded on the data sheet shown in Appendix-B.

The data obtained after specimen disassembly concerning the final wet weight of the specimen (W_3) and the dry weight (W_4) shall be recorded in space provided in Appendix-A.

6. Calculations:

6.1 Determination of Coefficient of Consolidation (C_v) : Plot dial gauge readings ver-sus square root of "t" (Fig. 15.03) for each load increment and draw smooth curve joining the points. Each curve should be identified by noting down the pressure acting on the specimen during the load increment and the duration of the load increment. The coefficient of consolidation, "C_v" determined from the curve, shall be recorded onthe curve as well.

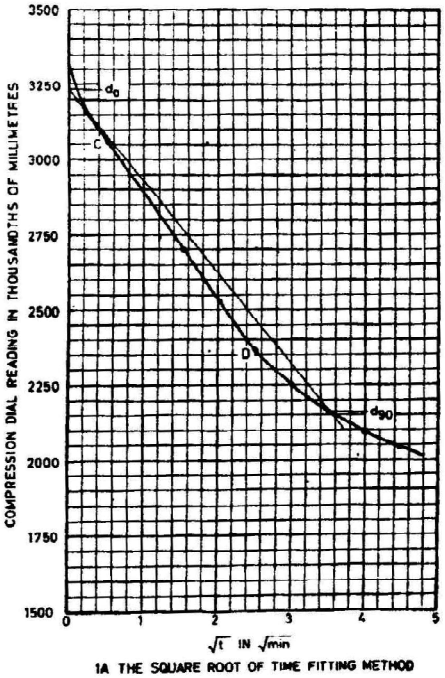


Fig. 15.03: Plot of dial gauge readings v/s square root of "t"

(i) The dial reading corresponding to zero primary consolidation, that is, do, is found by extrapolating the straight line portion of the curve, that is, DC back to t = 0.

(ii) Straight line is then drawn from d_0 such that the abscissae of this line are 1.15 times the abscissae of the straight line CD.

(iii) The point at which the drawn line intersects the experimentally obtained curve, that is, d_{90}, corresponds to 90 percent primary consolidation.

(iv) The time required for 90 percent consolidation is read off the curve as t_{90} and recorded in col 9 of Appendix - A.

(v) The coefficient of consolidation, C_V for the load increment under consideration may be calculated from the formula below :

$$C_V = [0.848 (H_{SV}/2)^2] / t_{90}$$

Where, H_{SV} is the average specimen thickness for the load increment given in Col. 10 of Appendix-A and C_V has units of length per unit time consistent with the units used and should be recorded in Col. 11 of Appendix-A.

6.2 Determination of Compressibility

6.2.1 Coefficient of Compressibility

(i) Transfer the final dial gauge reading for each pressure incrementfrom Appendix-B to Col. 2 of Appendix-A, recording it against thetotal applied pressure which is noted in Col.1 of Appendix-A.

(ii) From the dry weight of the specimen, W_s, the volume of soilvoids, V_s shall be obtained as :

$$V_s = W_s / G_s Y_w$$

Where :

G_s = specific gravity of the solid particles, and

Y_w = unit weight of water.

(iii) The equivalent height of soil solids can be determined as :

$$H_s = V_s / A$$

Where, A is area of specimen in cm^2

(iv) From Col. 2 of Appendix-A, determine ΔH for each pressureincrement and record it in Col. 3.

(v) The height of specimen at the end of each pressure increment, H, can be determined by subtracting ΔH of a particular increment from H of the specimen prior to application of that increment. This is to be recorded in Col. 4 of Appendix-A.

(vi) Void ratio, e, is obtained as

$$e = (H / H_s) - 1$$

and recorded in Col.5 of Appendix-A.

(vii) Values of d_e and d_σ obtained from Col.5 and 1 of Appendix-A respectively are recorded in Col. 6 and 7 of Appendix-A (d_e and d_σ being the differences between successive values of e and σ_1 respectively).

(viii) The coefficient of compressibility, a_v, with units of inverse of units for stress shall be calculated as

$$a_v = d_e / d_\sigma$$

and recorded in Col. 8 of Appendix-A.

6.2.2 Compression Index (Ce) : Plot the void ratio, e versus logs. The slope of the straightline portion, that is, for the soil in the normally consolidated state, isdesignated as C_e. This can be directly obtained from the plot or calculatedas

$$C_e = d_e / \log (\sigma_2 / \sigma_1)$$

where σ_2 and σ_1 are the successive values.

7. Presentation of Results :

The results of a consolidation test are presented in the form of a set of curves showing the relationship of e versus and log σ, a_v versus log σ and C_v versus log σ. The value of C_e is also reported separately.

Appendix - A

Consolidation Test: Pressure Void Ratio Data

Project _____
Sample No. _____
Soil Identification _____
Specific Gravity _____
Specimen Preparation _
Procedure _____
Type of Water Used __

Specimen Measurements
Diameter, D = _____ cm
Area, A = $D^2/4$ = ____ cm^2
Thickness, H_e = _____ cm
Wt of ring (W_1)= _____ gm
Wt of specimen + ring (W_2) __ g
Final wt of specimen (W_3) = __ g
Dry wt of specimen +ring (W_4) = __ g
Dry wt of specimen (W_s) = $W_4 - W_1$ = ___ g
Equivalent height of solids

$$H_s = W_s / (G_s \gamma_w A)$$

Water Content
Can No. _____
Wt. of can + wet soil = ____ g
Wt of can + dry soil = _____ g
Wt of can = _____ g
Wt of water = ____ g
Wt of dry soil = ___ g

Test No. _____
Date _____
Tested by ____

Applied Pressure (kgf/cm^2)	Final Dial Reading	Compression ΔH (cm)	Specimen Height H_s (cm)	e = (H/H_s)-1	d_e	d_σ	a_v = (d_e)/(d_σ) (cm^2/kg)	t_{90} min	H_{sv} cm	C_v (cm^2/min)
1	2	3	4	5	6	7	8	9	10	11

Appendix – B

Consolidation Test: Pressure Increment Data

Project _____ Loading Frame No. _____ Test No. _____
Sample _____ Least Count of Dial Gauge _____ Date _____
Soil Identification _____ Tested By _____
Page No.

Pressure Increment From _____ to _____ kgf/cm^2

Pressure Increment From _____ to _____ kgf/cm^2

Pressure Increment From _____ to _____ kgf/cm^2

Date and Time	Elapsed Time (minute)	Dial Reading	Date and Time	Elapsed Time (minute)	Dial Reading	Date and Time	Elapsed Time (minute)	Dial Reading
1	2	3	4	5	6	7	8	9

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Chapter-16

LOS ANGELES ABRASION VALUE TEST

1. **Introduction:** Los Angeles abrasion test finds the percentage wear of blanket material due to attrition action between the blanket material and steel balls used as abrasive charge. Los Angeles Abrasion Value is one of the specified properties for the blanket material, especially when it is produced by blending of quarry products.
2. **Reference:** IS-2386(Part-4):1963 (Reaffirmed-2016) "Methods of Test for Aggregates for Concrete-Mechanical Properties".
3. **Apparatus Required :**



Fig. 16.01: Abrasion Testing Machine



Fig. 16.02: Abrasive Charge

(i) Los Angeles machine (Fig. 16.01 & 16.03) : The Los Angeles abrasion testing machine shall consist of a hollowsteel cylinder, closed at both ends, having an inside diameter of 700mm and an inside length of

500mm. The cylinder shall be mounted on stub shafts attached to the ends of the cylinders in such a manner that itmay be rotated about its axis in a horizontal position. An opening in the cylinder shall be provided, which shall be closed dust-tight with a removable cover bolted in place. A removable steel shelf, projecting radially 88mm into the cylinder and extending its full length, shall be mounted along one element of the interior surface of the cylinder.The position ofthe shelf shall be such that the distance from the shelf to the opening,measured along the circumference of the cylinder in the direction of rotation, shall be not less than 1250mm.

(ii) I.S. Sieves: 80mm, 63mm, 50mm, 40mm, 25mm, 20mm, 12.5mm, 10mm, 6.3mm, 4.75mm and 2.36 mm (as per gradation of aggregate) and 1.7 mm.

(iii) Abrasive Charges: Cast iron or steel spheres of approximately 48mm in diameter and each weighing 390 to 445 g (Fig. 16.02).

4. Procedure:

(i) The test sample shall consist of clean aggregates, dried in oven at 105^0 to 110^0 C. Gradation of the Aggregate should be checked to assess the Grade of the Aggregate, using 10000 g of samples for coarse aggregates and 5000 g of sample for fine aggregates, as given in Table 16.01. The grading used shall be the most nearly representing the aggregate furnished for the work.

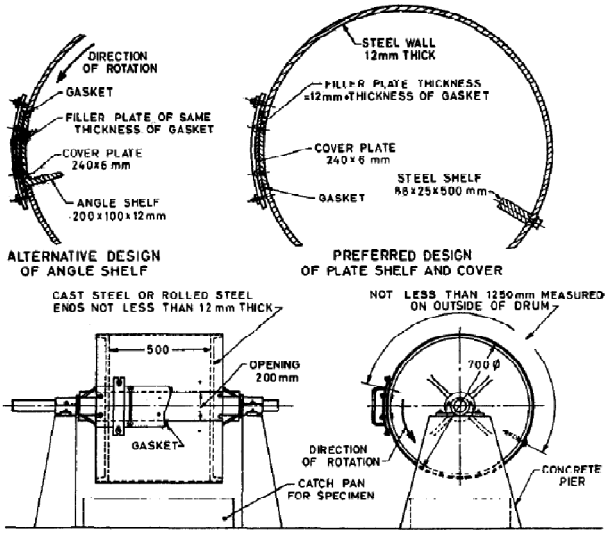


Fig. 16.03: Los Angele Abrasion Testing Machine

Table 16.01: Gradation of Aggregate

Sieve Size		Weight in g of Test Sample for Grade						
Passing (mm)	Retained on (mm)	A	B	C	D	E	F	G
80	63					2500		
63	50					2500		
50	40					5000	5000	
40	25	1250					5000	5000
25	20	1250						5000
20	12.5	1250	2500					
12.5	10	1250	2500					
10	6.3			2500				
6.3	4.75			2500				
4.75	2.36				5000			

(ii) The test sample and the abrasive charge shall be placed in the Los Angeles abrasion testing machine. The amount of charge

shall be as given in Table 16.02. The machine rotated at a speed of 20 to 33 rev/min. For Grading A, B, C and D, the machine shall be rotated for 500 revolutions; for Grading E, F and G, it shall be rotated for 1000 revolutions.

Table 16.02: Charge to be used

Grading	Number of spheres	Weight of charge (gm)
A	12	5000 ± 25
B	11	4584 ± 25
C	8	3330 ± 20
D	6	2500 ± 15
E	12	5000 ± 25
F	12	5000 ± 25
G	12	5000 ± 25

(iii) The machine shall be so driven and so counter-balanced as to maintain a substantially uniform peripheral speed. If an angle is used as the shelf, the machine shall be rotated in such a direction that the charge is caught on outside surface of the angle.

(iv) At the completion of the test, the material shall be discharged from the machine and a preliminary separation of the sample made on a sieve coarser than the 1.70 mm IS Sieve.

(v) The material coarser than the 1.70 mm IS Sieve shall be washed and dried in an oven at 105 to 110°C to a substantially constant weight, and accurately weighed to the nearest gram.

5. Calculation : The difference between the original weight and the final weight of the test sample is expressed as a percentage of the original weight of the test sample. This value is reported as the Abrasion Value.

$$\text{Abrasion Value} = (A - B/A) \times 100$$

Where,

A = weight in g of dry sample,

B = weight in material coarser than 1.70mm IS Sieve, after washing and oven-dried upto constant weight, in g.

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PART – C

FIELD TESTS

(1) Standard Penetration Test (Chapter - 17)

(2) Tests for Checking Quality of Compaction (Chapter - 18)
   (A) By Sand Replacement Method
   (B) By Core Cutter Method
   (C) By Nuclear Gauges

(3) Determination of E_{V2} (Chapter - 19)

(4) Field Vane Shear Test (Chapter - 20)

(5) Field California Bearing Ratio (CBR) Test (Chapter - 21)

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Chapter-17

STANDARD PENETRATION TEST

1. **Introduction:** Standard Penetration Test (SPT) involves measurement of resistance to penetration of a standard sampling spoon under dynamic loading. The resistance is empirically correlated with many engineering properties like density index, consistency, bearing capacity etc. This test is useful for general exploration of erratic soil profiles, for finding depth of bed rock or hard stratum and to have an approximate indication of the strength and other properties of soils, particularly the cohesionless soils, from which it is difficult to obtain undisturbed samples. In Railway formation, the SPT value is determined as part of geotechnical exploration to get an idea about the ground profile (i.e. type of soil) with depth and also to decide about the need or otherwise for "ground improvement" in case of embankment construction over soft soils.

2. **Reference:** IS-2131:1981 (Reaffirmed-2016) "Method for Standard Penetration Test for Soil".

3. **Apparatus Required:**

3.1 Drilling Equipment

(i) The equipment used shall provide a clean borehole, 100 to 150mmin diameter, for insertion of the sampler to ensure that the penetration test is performed on undisturbed soil and shall permit driving of the split spoon sampler to obtain penetration record and the sample.

Note: The stiffness of the drill rod used for testing influences the N value obtained by means of the test. A light rod 'whips' under the blows of the hammer. The drill rod shall preferably have a stiffness equal to A-rod (41.3 mm outer diameter).

For depths of exploration more than 10m, special precautions shall be taken to keep the rod vertical by using centering spacers and/or by using stiffer rods to minimize the whipping effect. Spacers may be provided at every 10m, or more frequently, if necessary.

(ii) Casing or Drilling Mud: It shall be used when drilling in sand, softclay or other soils in which the sides of borehole are likely to cave in. Insandy and other non-cohesive soils, below water table it is often preferable to use drilling mud rather than a casing. If drilling mud alone is not successful, casing may be used along with the drilling mud.

3.2 Split Spoon Sampler: The split spoon sampler shall conform to IS: 9640-1980 (Reaffirmed-2016). (Fig. 17.01)

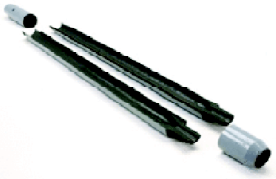


Fig. 17.01: Split Spoon Sampler

3.3 Drive Weight Assembly

(i) The drive weight assembly shall consist of a driving head and a 63.5 kg weight with 75cm free fall. It shall be ensured that the energy of the falling weight is not reduced by friction between the drive weight and the guides or between rope and winch drum.

(ii) The rods to which the sampler is attached for driving should be straight, tightly coupled and straight in alignment. For driving the casing, a hammer heavier than 63.5 kg may be used.

3.4 Lifting bail, Tongs, Rope, Screw jack etc.

4. Procedure

4.1 Driving the Casing: Where casing is used, it shall not be driven below the level at which the test is made or soil sample is taken. In the case of cohesionless soils which cannot stand without casing, the advancement of the casing pipe should be such that it does not disturb the soil to be tested or sampled; the casing shall preferably be advanced by slowly turning the casing rather than by driving, as the vibration caused by driving may alter the density of such deposits immediately below the bottom of the borehole.

4.2 Cleaning the Borehole:

(i) In case wash boring is adopted for cleaning the borehole, side-discharge bits are permissible, but in no case shall a bottom-discharge bit be permitted. The process of jotting through an open tube sampler, and then testing and sampling when the desired depth is reached shall not be permitted.

(ii) While boring through soils, such as sands that may be disturbed by the flow of water into the drill hole, no water shall be added to the borehole while boring above the water table. While boring below water table, the water in the borehole shall be maintained at least 1.5m above the level of the water table. Bentonite slurry of appropriate consistency-may be required to help the water level to be maintained above the water table. The raised level of the water in the borehole should be maintained even if casing is used to stabilize the borehole.

(iii) While boring through sand using casing to stabilize the sides of the borehole, the outer

diameter of the shell shall be at least 25mm smaller than the inner diameter of the casing. The distance between the end of the casing and the bottom of the borehole should be as close as possible and in any case not exceed 150mm, if only water is used to stabilize the borehole; in case bentonite is used, this distance may be up to 300mm.

(iv) The borehole shall be cleaned up to testing or, sampling elevation, using suitable tools, such as augers, that will ensure that there is minimum mixing up of the soil from the bottom of the borehole. In cohesive soils, the borehole may be cleaned with bailer with a flap valve. This should not be used in sands.

4.3 Obtaining the Samples:

(i) Tests shall be made at every change in stratum or at intervals of not more than 1.5m whichever is less. Tests may be made at lesser intervals if specified or considered necessary. The intervals be increased to 3m if in between vane shear test is performed.

(ii) The sampler shall be lowered to the bottom of the borehole. The following information shall be noted and recorded:

(a) Depth of bottom of borehole below ground level,

(b) Penetration of the sampler into the soil under the combined weight of sampler and rods (to be noted from readings of the scale over the drill rod at the top),

(c) Water level in the borehole or casing, and

(d) Depth of bottom of casing below ground level.

(iii) The split spoon sampler resting on the bottom of borehole should be allowed to sink under its own weight; then the split spoon sampler shall be seated 15cm with the blows of the hammer falling through 75cm. Thereafter, the split spoon sampler shall be further driven by 30cm or 50 blows (except that driving shall cease before the split spoon sampler is full). The number of blows required to effect each 15cm of penetration shall be recorded. The first 15cm of drive may be considered to be seating drive. The total blows required for the second and third 15cm of penetration shall be termed the penetration resistance N; if the split spoon sampler is driven less than 45cm (total), then the penetration resistance shall be for the last 30cm of penetration (if less than 30cm is penetrated, the logs should state the number of blows and the depth penetrated).

(iv) Under the following conditions the penetration is referred as refusal and test is halted. Care shall be taken to ascertain if refusal is due to presence of boulders locally:

(a) 50 blows are required for any 150mm penetration,

(b) 100 blows are required for 300mm penetration, and

(c) 10 successive blows produce no advancement.

(v) The entire sampler may sometimes sink under its own weight when very soft sub-soil stratum is encountered. Under

such conditions, it may not be necessary to give any blow to the split spoon sampler and SPT value should be indicated as zero.

(vi) If on lowering the sampler by means of a string of rods it is found to rest at a level above the bottom of the casing, the penetration test and sampling should not be carried out at that stratum.

4.4 Removal of Sampler and Labelling:

(i) The sampler shall be raised to the surface and opened. A typical sample or samples of soil from the opened split spoon shall be put into jars without ramming. The jars shall have a self-sealing top, or shall be sealed with wax to prevent evaporation of the soil moisture. Jars shall be of such a size that they can be filled without deforming the sample.

(ii) Typical samples shall be cut to such a size as to fill the jars and thereby reduce the water loss to the air in the jars. If packing as specified is not available, liner may be used in the sampling spoon. In such a case, the internal diameter of the sampling spoon should be soadjusted that the total internal diameter after incorporating the liner is 35mm. The sample in the liner shall be waxed properly at both the ends to keep up the natural moisture content during transit.

(iii) Labels shall be fixed to the jar or notations shall be written on the covers (or both) with the following information:

(a) Origin of sample,

(b) Job designation,

(c) Boring number,

(d) Sample number,

(e) Depth of sampling,

(f) Penetration record,

(g) Length of recovery, and

(h) Date of sampling.

The jars containing samples shall be stored in suitable containers for shipment. Samples shall not be placed in the sun.

4.5 Field Observations : Information with regard to water table, elevations at which the drilling water was lost or elevations at which water under excess pressure was encountered shall be recorded on the field logs. Water levels before and after putting the casing, where used, shall be measured. In sands, the level shall be determined as the casing is pulled and then measured at least 30 minutes after the casing is pulled; in silts, at least 24 hours after the casing is pulled; in clays, no accurate water level determination is possible unless pervious seams are present. However, the 24 hours level shall be recorded for clays. When drilling mud is used and the water level is desired, casing perforated at the lower end shall be lowered into the borehole and the borehole bailed down. Ground water levels shall be determined after bailing at time intervals of 30 minutes and 24 hours until all traces of drilling mud are removed from inside the casing.

4.6. Corrections in SPT value:

(i) Correction for Overburden: As depth increases, the overburden pressure also increases.Hence lateral earth pressure increases and more energy is required to drive the sampler hence this correction is applied.N value for cohesionless soil shall be corrected for overburden as per Fig. 17.02 (N').

(ii) Correction for Dilatancy: The value N' shall be corrected for dilatancy if the stratum consists of fine sand and silt below water table for values of N' greater than 15, as under:

$$N'' = 15 + \tfrac{1}{2}(N' - 15)$$

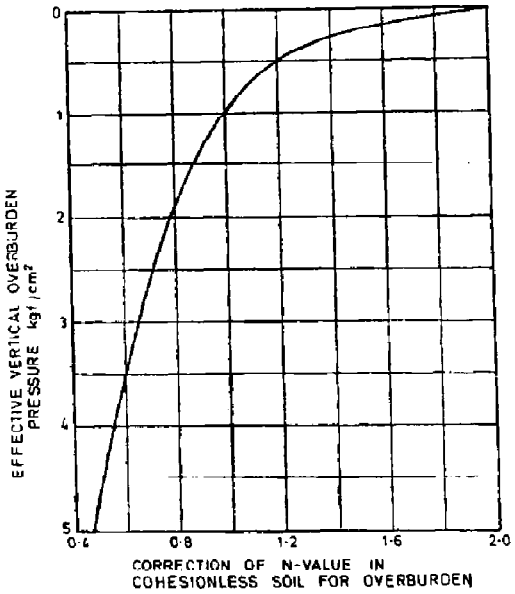


Fig. 17.02: Correction for Overburden

5. **Observation:** Data obtained in borings shall be recorded with following details:

   (a) Date of boring,
   (b) Reference datum,
   (c) Job identification,
   (d) Boring number,
   (e) Sample number,
   (f) Type ofsampler,
   (g) Drilling method,
   (h) Sample elevation and recovery ratio,
   (j) Limits of stratum,
   (k) Water table information,
   (m) Soil identification, including condition of samples,
   (n) Penetration records,
   (p) Casing used, and
   (q) Weather data.

   The data obtained shall be prepared in a final form as a soil profile to show the nature and extent of the soil strata over the area under consideration.

6. **General Remarks:** Generally in SPT disturbed samples are collected through split spoon sampler, which can be used for determining index properties. If Shelby tubes are used with suitable adaptor then undisturbed samples can also be collected which can be used for determining engineering properties i.e. shear strength and consolidation properties. Shelby tubes are used inbetween the SPT test locations along the depth of bore hole.

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Chapter-18

TESTS FOR CHECKING QUALITY OF COMPACTION

Achieving stipulated quality of compaction of the fill material, for the earthwork carried out in field, is important, for ensuring that the compacted soil mass is having the designed or requisite engineering properties. Various methods, commonly employed for railway formations, are being covered in this chapter. In most of these methods, the "in-place/bulk density" of soil mass achieved in the field and the corresponding "water content" are determined. Using these two parameters, the "dry density" of the compacted soil mass is calculated.

The "degree of compaction", which is ratio of "dry density achieved in field" divided by "maximum dry density achieved in lab by Heavy Proctor Test", indicates the quality of compaction and it should be more than the stipulated value. In case of coarse grained soils,having fines passing 75 micron sieve upto 5 percent, the quality of compaction is checked by calculating "relative density", which should be more than the stipulated value. The relative density is calculated by following formula:

$$RD = [(Y_d - Y_{dmin})/(Y_{dmax} - Y_{dmin})] \times (Y_{dmax}/Y_d)$$

Where:

Y_d = Dry density of compacted earthwork, as checked in the field

Y_{dmax} & Y_{dmin} = Maximum and minimum dry density for the soil, as determined in the lab

(A) By Sand Replacement Method

1. **Introduction:** This method can be used for checking "in-place/bulk density" of any type of soil.

2. **Reference:**IS-2720(Part-28):1974 (Reaffirmed -2015) "Methods of test for soils: Determination of dry Density of soil in-place, by the Sand Replacement Method".

3. **Apparatus Required:**

(i) **Sand Pouring Cylinder** (Fig. 18.01) - mounted over a pouring cone and separated by a valve or shutter.

(ii) **Cylindrical Calibrating Container** (Fig. 18.01) - with internal diameter of 200mm and internal depth of 250mm, fitted with a flange approximately 75mm wide and about 5mm thick surrounding the open end. The volume of the container should be given to an accuracy of 0.2 percent.

(iii) **Tools for excavating holes** - suitable tools, such as bent spoon,dibber, large screw driver, pointed steel rod about 300mm long and 5 to 10mm dia. with a wooden handle.

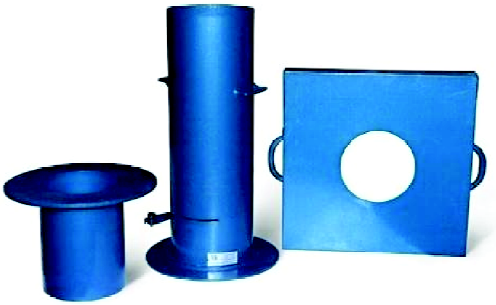


Fig. 18.01: Sand Replacement Method Apparatus

(iv) Balance - accurate to 1 g.

(v) Plane Surface - a glass plate or other plane surface about 600mm square and 10mm thick or larger.

(vi) Metal Container - to collect the excavated soil and to take the supply of sand to fill the pouring cylinder. This may be provided witha suitable cover.

(vii) Metal Tray with Central Hole (Fig. 18.01) - a metal tray 450mm square and 50mm deep with a 200mm dia. hole in the centre.

(viii) Clean, uniformly graded natural sand passing 1.0mm IS Sieve andretained on 600 micron IS Sieve. It shall be free fromorganic matter and shall be oven dried and stored for a suitable period to allow its water content to reach equilibrium with atmospheric humidity.

(ix) Suitable test set-up for checking moisture content of the soil

4. Procedure:

4.1 Calibration of Apparatus

(i) Fill the sand pouring cylinder with clean sand so that the level of the sand in the cylinder is within about 10mm from the top. Find out the initial weight of the cylinder plus sand (W_1) and this weight should be maintained constant throughout the test for which the calibration is used.

(ii) Keep the cylinder on glass plate. Allow the sand of volume equal to that of the calibrating container to run out of the cylinder by opening the shutter, open the shutter and when no further movement of sand takes place in the cylinder, close the shutter and remove the cylinder carefully. Weigh the sand collected on the glass plate to nearest 0.1

percent of its total weight. Repeat these measurements at least three times and take the mean weight (W_2). This is the weight of sand filling the cone portion of the sand pouring cylinder.

(iii) The internal volume (V) in ml of the calibrating container shall be determined by the weight of water contained in the container when filled to the brim.

(iv) Keep the calibrating container on a large tray during the procedure to collect the sand overflowing from the cone when the cylinder is removed. Place the pouring cylinder concentrically on the top of the calibrating container and fill with the constant weight of sand (W_1), keeping the shutter on pouring cylinder closed during this operation. Open the shutter and allow the sand to run out. When no further movement of the sand takes place in the cylinder, close the shutter. Remove the pouring cylinder and weigh the sand remaining in it to the nearest 0.1 percent of its initial weight. Repeat these measurements at least three times and take the mean weight (W_3).

4.2 Determination in-place density of soil:

(i) Expose approximately 60 cm^2 flat area, at the place where the soil is to be tested and trim it down to a level surface.

(ii) Keep the metal tray with central hole on the prepared surface of the soil

with the hole over the portion of the soil to be tested. Excavate the hole in the soil using the hole in the tray as apattern, to the depth of the layer to be tested up to a maximum of 250mm. Collect the excavated soil carefully, leaving no loose material in the hole and weigh it to the nearest gram (W_w). Remove the metal tray

(iii) Keep a representative sample of the excavated soil in anair-tight container and determine its water content (w).

(iv) Place the pouring cylinder filled with the constant weight of sand (W_1) in such a way that base of the cylinder covers the hole concentrically. Open the shutter of pouring cylinder allow the sand to run out. When no further movement of the sand takes place, close the shutter. Remove the cylinder and weigh the sand remaining in it to the nearest 0.1 percent of its initial weight (W_4).

5. Observations and Calculations:

5.1 Calibration

1.	Mean weight of sand in cone (of pouring cylinder) (W_2) in g =
2.	Volume of calibrating container (V) in ml =
3.	Weight of sand (+ cylinder) before pouring (W_1) in g =
4.	Weight of sand (+ cylinder) after pouring (W_3) in g =
5.	Weight of sand to fill calibrating container ($W_a=W_1-W_3-W_2$) in g =
6.	Bulk density of sand $Y_s = W_a / V$ (g/cc) =

5.2 Measurement of Soil Density

1.	Determination no. =
2.	Weight of wet soil from hole W_w in g =
3.	Weight of sand (+ cylinder) before pouring W_1 in g =
4.	Weight of sand (+ cylinder) after pouring W_4 in in g =
5.	Weight of sand in hole ($W_b = W_1-W_2-W_4$) in g =
6.	Bulk density $Y_b = (W_w / W_b) \times Y_s$ in gm/cc =
7.	Water content container no. =
8.	Weight of soil for water content determination =
9.	Weight of over dried soil in g =
10.	Water content (w), in percent =
11.	Dry density Yd = $100Y_b / (100+w)$ in g/cc =

6. **General Remarks :** The hole excavated must be of volume approximately equal to the volume of the calibrating container.

(B) By Core Cutter Method

1. **Introduction:** this method is relatively simpler and quicker as compared to sand replacement method, but in some of the coarse grained soils (with little fines) sometimes it is not possible to take core cutter samples.

2. **Reference:** IS-2720 (Part-29) : 1975 (Reaffirmed-2015) “Methods of test for soils: Determination of dry density of soils in-place by the core-cutter method”.

3. Apparatus required:



Fig. 18.02: Core Cutter Method Apparatus

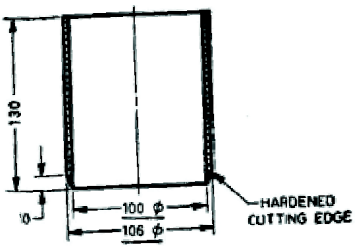


Fig. 18.03: Core Cutter

3.1 Cylindrical Core Cutter (Fig. 18.02 & 18.03): of seamless steel tube, 130mm long and 10cm internal diameter, with a wall thickness of 3mm, beveled at one end. The cutter shall be kept properly greased or oiled.

3.2 Steel Dolly (Fig. 18.02 & 18.04)**:**2.5cm high and 10cm internal diameter with a wall thickness of 7.5mm with a lip to enable it to be fitted on top of the core-cutter.

3.3 Steel Rammer (Fig. 18.02 & 18.05): with solid mild steel foot 140mm diameter and 75mm height with a concentrically screwed 25mm diameter solid mild steel staff. The overall length of the rammer including the foot as well as the staff should be approximately 900mm. The rammer (foot and staff together) should weigh approximately 9kg.

3.4 Balance: Accurate to 1 g.

3.5 Palette Knife: A convenient size is one having a blade approximately 20cm long and 3cm wide.

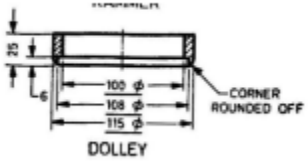


Fig. 18.04
Steel Dolly

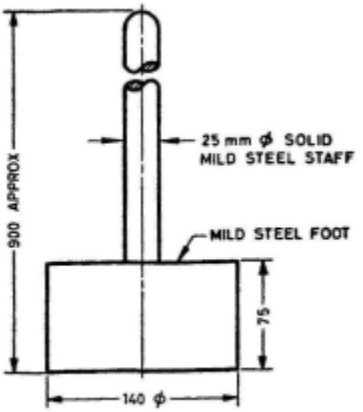


Fig. 18.05
Steel Rammer

3.6 Straight Edge: A steel strip about 30cm long, 2.5cm wide and 3 to 5mm thick, with one beveled edge.

4. Procedure:

(i) Measure the inner dimension of the cutter nearest to 0.25mm and calculate its volume (V_c).

(ii) Weigh the cutter to the nearest gram (W_c).

(iii) Expose and level a small area, approximately 30 cm^2, of the soil layer to be tested. Place the steel dolly on top of the cutter and ram the cutter vertically down into the soil layer until only about 15mm of the dolly protrudes above the surface, care being taken not to rock the cutter. Dug out the cutter from the surrounding soil, care being taken to allow some soil to project from the lower end of the cutter. Trim the soil core flat at the ends of the cutter by means of the straight edge.

(iv) Weigh the cutter with the soil core to the nearest gram (W_s).

(v) Remove the soil core from the cutter and place a representative sample in an air-tight container and determine its water content (w).

(vi) It is necessary to make a number of repeat determinations (at least three) and to average the results, since the dry density of the soil varies appreciably from point to point. The number of determinations should be such that an additional one would not alter the average significantly.

5. Observations and Calculations:

5.1 The bulk density (Y_b) that is the weight of the wet soil per cubic centimeter shall be calculated from the following formula:

$$Y_b = (W_s - W_c) / V_c \text{ g/cm}^3$$

Where :

W_s = weight of soil and core-cutter in g,

W_c = weight of corecutter in g, and

V_c = volume of corecutter in cm^3

5.2 The dry density (Y_d) that is the weight of the dry soil per cubic centimetershall be calculated from the following formula:

$$Y_d = 100\ Y_b / (100 + w), \text{ g/cm}^3$$

Where

Y_b = bulk density, and

w = water content of soil (percent) to two significantfigures.

5.3 Results of the test should be recorded in the format as under:

1. Determination No.	1	2	3
2. Weight of core cutter + wet soil (Ws) in g			
3. Weight of core-cutter (W_c) in g			
4. Weight of wet soil ($W_s - W_c$) in g			
5. Volume of core cutter (V_c) in cm^3			
6. Bulk density $\gamma_b = (W_s - W_c)/V_c$ in g/cm^3			
7. Water content container No.			
8. Weight of container with lid (W_1) in g			
9. Weight of container with lid and wet soil (W_2) in g			
10. Weight of container with lid and dry soil (W_3) in g			
11. Water content (w), in percent $w = [(W_2-W_3)/(W_3-W_1)] \times 100$			
12. Dry density $\gamma_d = 100\ \gamma_b/(100+w)$ in g/cm^3			

(C) By Nuclear Gauges

1. Introduction: Checking of in-place density and moisture content, for quality control of earthwork done, by conventional methods is time consuming and the test cannot be repeated at the same location again, in case of any doubt about any particular test result. An answer to this is use of nuclear gauges, simple equipment, which use low level radiation to measure the in-place density and moisture content of any granular material. This equipment gives results very fast and number of tests can be performed at the same location.

There is no Indian Standard/RDSO Guidelines for global use of this test method. However, RDSO had issued test procedure for use of nuclear gauges in some project locations of DFCCIL. The procedure given herein is based on this and it covers determination of in-situ density and water content of soil by means of a Nuclear Gauge designed to operate on the

ground surface. The quality of the result produced is dependent on the competence of the personnel using the gauge and the suitability of the equipment and facilities used.

2. Reference:

(i) ASTM D6938-10: Standard Test Method for "in-place density and water content of soil and soil aggregate by Nuclear Methods (Shallow Depth)".

(ii) BS:1377: Part-9: 1990 – British Standard Method of test for soils for Civil Engineering Purpose: In-situ Tests.

(iii) AASHTO: 310-02:Standard Specification for "in-place density and water content of soil and soil aggregate by Nuclear Methods (Shallow Depth)".

(iv) National Association of Testing Authorities (NATA) Australia: Technical Note 32, March'2013: Use of Nuclear gauges for Testing Soils & Asphalts.

3. Apparatus Required:

3.1 Nuclear Gauge (Fig.18.06) shall consist of:

(a) Gamma Source: A sealed source of high-energy gamma radiation, such as cesium or radium.

(b) Gamma Detector: Any type of gamma detector such as a Geiger-Muellertube(s).

(c) Fast Neutron Source: A sealed mixture of a radioactive material such as a mericium, radium and a target material such as beryllium, or a neutron emitter such as californium-252.

(d) Slow Neutron Detector: Any type of slow neutron detector such as boron tri - fluoride or helium-3 proportional counter.

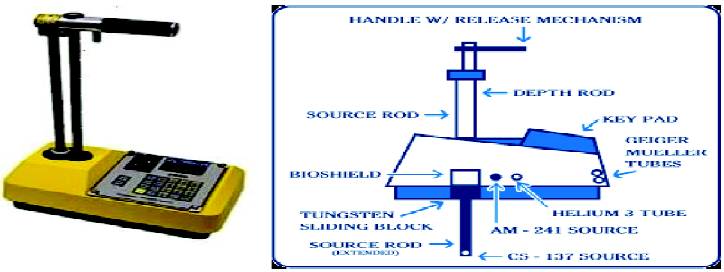


Fig. 18.06: Nuclear Gauge

3.2 Reference Standard: A block of material used for checking in strumen to peration, correction of source decay, and to establish conditions for are producible reference countrate.

3.3 Site Preparation Device (Fig. 18.07): A plate or straight edge or other suitable leveling tool used for levelling the test site to the required smoothness, and in the Direct Transmission Method, guiding the drive pin to prepare a perpendicular hole.



Fig. 18.07: Site Preparation Device

3.4 Drive Pin: A pin of slightly larger diameter than the probe, used to prepare a hole in the test site for inserting the probe.

(a) Drive Pin Guide: A fixture that keeps the drive pin perpendicular to the test site. Generally part of the site preparation device.

(b) Drive Pin Extractor: A tool that is used to remove the drive pin in avertical direction so that the pin will not distort the hole in the extraction process.

3.5 Hammer : Heavy enough to drive the pin to the required depth without undue distortion of the hole.

4. Procedure : Two alternative modes are provided in the gauges:

(a) Direct Transmission Mode in which the gamma source rod extends through the base of the gauge in to a pre-formed hole to a desired depth. This is preferred mode for Density Measurement and should be used where possible because of its deeper zone of influence.

(b) Backscatter Mode in which the gamma and neutron sources and the detectors are kept at the surface. Moisture Density can be determined only by using the backscatter mode.

4.1 Calibration of the Gauge

(i) Calibration of the gauge should be done by an Accredited Agency or by the Manufacturer of the gauge, in accordance with procedure given in ASTM: D-6938.

(ii) Gauges shall be calibrated initially and after any repairs that can affect the gauge geometry or the existing calibration. To be within specified tolerances, calibration curves, tables,

or equivalent coefficients shall be verified, at periods not to exceed 12 months. If the tolerances cannot be met at any time, the gauge shall be calibrated to establish new calibration curves, tables or equivalent coefficients. If the owner does not establish a verification procedure, the gauge shall be calibrated at a period not to exceed 12 months.

(iii) Record of calibration shall be maintained, in conformity with procedure given in Annex-A1 and Annex-A2 of ASTM:D-6938 and before use of any gauge it should be ensured that the gauge is having valid calibration certificate issued in conformity with stipulated standard.

4.2 Standardization of the Gauge: Nuclear gauges are subject to long-term aging of the radioactive sources, which may change the relationship between count rates and the material density and water content. To correct for this aging effect, Standardization of the gauge shall be performed at the start of each day's work and a record of this data should be retained. This procedure shall also be repeated after 8 Hours of continuous use. Standardization should be done with the gauge located at least 9m away from other nuclear moisture density gauges and clear of large masses of water or other items which can affect the reference count rates. The standardization process is as under:

(i) Turn on the gauge and allow for stabilization according to the manufacturer's recommendations.

(ii) Using the reference standard block, whose serial number matches with the

serial number on the gauge, take a reading that is at least four times the duration of a normal measurement period (typically one minute). Use the procedure recommended by the gauge manufacturer to establish the compliance of the standard measurement to the accepted range. Without specific recommendations from the gauge manufacturer, use the procedure given in Para (iii) below.

(iii) If the values of the current standardization counts are outside the limits set by Eq. 1 and Eq. 2, repeat the standardization check. If the second standardization check satisfies Eq. 1 and Eq. 2, the gauge is considered in satisfactory operating condition.

$$0.99(N_{dc})e^{(-\ln(2)/T_{d(1/2)})} \leq N_{d0} \leq 1.01\ (N_{dc})e^{(-\log_n(2)/T_{d(1/2)})} \quad (1)$$

$$0.98(N_{mc})e^{(-\ln(2)/T_{m(1/2)})} \leq N_{m0} \leq 1.01\ (N_{mc})e^{(-\log_n(2)/T_{d(1/2)})} \quad (2)$$

Where:

$T_{d(1/2)}$ = the half-life of the isotope that is used for the density determination in the gauge.

$T_{m(1/2)}$ = the half-life of the isotope that is used for the water content determination in the gauge.

N_{dc} = the density system standardization count acquired at the time of the last calibration or verification.

N_{mc} = the moisture system standardization count acquired at the time of the last calibration or verification,

N_{d0} = the current density system standardization count,

N_{m0} = the current moisture system standardization count,

t = the time that has elapsed between the current standardization test and the date of the last calibration or verification,

ln(2) = the natural logarithm of 2, which has a value of approximately 0.69315,

e = the inverse of the natural logarithm function, which has a value of approximately 2.71828.

If for any reason, the measured density or moisture becomes suspect during the day's use, another standardization check should be performed.

4.3 Procedure for Measurement:

(i) Select a test location where the gauge will be placed at least 600mm away from any object sitting on or projecting above the surface of the test location. If measurement is to be made at a specific location and the aforementioned clearance cannot be achieved, such as in a trench, follow the gauge manufacturer's correction procedure(s). Keep all other radioactive sources at least 9m away from the gauge to avoid any effect on the measurement.

(ii) Prepare the test site in the following manner:

(a) Remove all loose and disturbed material and additional material as necessary to expose the true surface of the material to be tested.

(b) Prepare an area to accommodate the gauge by grading or scraping the area to a smooth condition so as to obtain maximum contact between the gauge and material being tested.

(c) The depth of the maximum void beneath the gauge shall not exceed 3mm. Use native fines or fine sand to fill the voids and smooth the surface with a rigid straight edge or other suitable tool. The depth of the filler should not exceed approximately 3mm.

(d) The placement of the gauge on the surface of the material to be tested is critical to accurate density measurements. The optimum condition is total contact between the bottom surface of the gauge and the surface of the material being tested. The total area filled should not exceed approximately 10% of the bottom area of the gauge.

(iii) Turn on and allow the gauge to stabilize (warm up) according to the manufacturer's recommendations.

(A) In Direct Transmission Mode

(a) Make a hole perpendicular to the prepared surface using the rod guide and drive pin. The hole should be a minimum of 50mm deeper than the desired measurement depth and of an alignment that insertion of the probe will not cause the gauge to tilt from the plane of the prepared area. Care must be taken in the preparation of the access hole in uniform cohesionless granular soils. Measurements can be affected by damage to the density of surrounding materials when forming the hole.

(b) Remove the hole-forming device carefully to prevent the distortion of the hole, damage to the surface or loose material to fall into the hole.

(c) Place the gauge on the material to be tested. Lower the probe into the hole to desired test depth. As a safety measure, it is recommended that the probe not be extended out of its shielded position prior to placing it into the test site.

(d) Secure and record one or more one-minute bulk density and water content readings. Read the in-situ bulk density and water content directly.

(e) Retract the probe into the housing and check that the radioactive source is safely housed.

(B) In Backscatter Mode

(a) Seat the gauge firmly. Set the gauge into the Backscatter position.

(b) Secure and record one or more set(s) of one-minute density and water content readings.

(c) Read the in-situ bulk density and water content.

5. Water Content Correction and Oversize Particle Correction: For getting accurate values of water content and bulk density, both of these corrections need to be made when applicable.

(a) Prior to using the gauge-derived water content on any new material, the value should be verified. As part of a user developed procedure, occasional samples should be taken from beneath the gauge and comparison testing done to confirm gauge-derived water content values. All gauge manufacturers have a procedure for correcting the gauge-derived water content values.

(b) When oversize particles are present, the gauge can be rotated about the

axis of the probe to obtain additional readings as a check. When there is any uncertainty as to the presence of these particles it is advisable to sample the material beneath the gauge to verify the presence and the relative proportion of the oversize particles.

(c) When sampling for water content correction or oversize particle correction, the sample should be taken from a zone directly under the gauge. The size of the zone is approximately 200mm in diameter and a depth equal to the depth setting of the probe when using the direct transmission mode; or approximately 75mm in depth when using the backscatter mode.

6. Calculation: In most of the gauges, the Bulk Density, Water Content and Dry Density are calculated and displayed directly. Otherwise, calculate the Dry Density (Y_d) as under:

$$Y_d = (100 \times Y_b)/ (100 + w)$$

Where,

Y_b = Bulk Density of the soil determined by Nuclear Gauge

w = Moisture Content of the soil (in %).

If the Nuclear Gauge determines the Water Content of the soil per unit volume of the soil, then the Dry Density is calculated as under:

$$Y_d = Y_b – W$$

Where,

Y_b = Wet density

W = Moisture Density i.e. water mass per unit volume of soil

The Moisture Content (in %) "w" can be calculated as:

$$w = (W \times 100)/ (Y_b - W)$$

7. **Verification of Gauge Working:** At the start of each day's work, the Degree of Compaction (% compaction) should be measured at minimum 3 locations by the Nuclear Gauge and compared with the results given by the conventional methods like Sand Replacement Method or Core Cutter Method at the same locations.

   (i) If the difference in Degree of Compaction (%) at any of the location is more than 6%, the reason for the difference should be examined in detail.

   (ii) If the difference in average Degree of compaction (%) of all the locations, ignoring its' sign, is more than 4%, the standardization of the gauge shall be repeated.

   (iii) After standardization, the procedure given above shall be repeated again. If the difference in average degree of compaction (%) of all the locations, ignoring its' sign, is still more than 4%, the nuclear gauge shall be re-calibrated, to bring the difference in average Degree of compaction (%) of all the locations within 4%.

8. **Reporting of results:** The Field Data Records shall include, as a minimum, the following:

   (i) Make, model and serial number of the Nuclear Gauge.

   (ii) Validity date of Gauge calibration.

   (iii) Data for Standardization of the gauge.

   (iv) Data/details about daily Verification of gauge Results.

(v) Location of test (e.g. Chainage and Lift/ Layer No.).

(vi) Visual description of material tested.

(vii) Name of the operator(s).

(viii) Test mode (i.e. Direct transmission or Backscatter) and depth of test.

(ix) Any corrections made in the reported values and reasons for these corrections (i.e. over-sized particles, water content).

(x) Maximum Laboratory Density value.

(xi) Bulk Density measured.

(xii) Water Content in percent.

(xiii) Dry Density Calculated/measured.

(xiv) Degree of Compaction / Percent Compaction.

9. Safety Precautions:

(i) These gauges utilize radioactive materials that may be hazardous to the health of the users unless proper precautions are taken. Users of these gauges must become familiar with applicable safety procedures and government regulations.

(ii) Effective user instructions, together with routine safety procedures and knowledge of and compliance with Regulatory Requirements, are a mandatory part of the operation and storage of these gauges.

(iii) This procedure does not purport to address all of the safety concerns, if any, associated with its use. The user of this standard should establish appropriate safety and health practices and ensure compliance to all regulatory limitations.

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Chapter-19

DETERMINATION OF E_{V2}

1. **Introduction:** E_{V2} denotes deformation modulus at any finished layer of the formation and it is determined from second cycle in the cyclic plate load test. For Railway formations, E_{V2} is stipulated as one of the acceptance criteria on top of various finished layers in the formation.

2. **Reference:** German Standard DIN:18134:2012 "Soil - Testing procedures and testing equipment - Plate load test".

3. **Apparatus required:**

   3.1 Reaction loading system: The reaction loading system shall produce a reaction load which is at least 10 kN greater than the maximum test load required. It may be a loaded truck or roller or any other object of sufficient mass.

   3.2 Loading plate: The 300mm dia. loading plate shall have two handles (Fig. 19.01) and minimum thickness of 25mm.

   3.3 Loading system: The loading system consists of a hydraulic jack, capable of applying and releasing the load in stages. The hydraulic jack shall be hinged on both sides and secured against tilting. The pressure piston shall act through at least 150mm.

   The height of the plate loading apparatus during operation should not exceed 600mm. In order to compensate for differences in the heights of the vehicles used as reaction loads, elements shall be provided that allow the initial length of the hydraulic

jack to be increased to at least 1000mm. Suitable means shall be provided to prevent buckling of these elements.

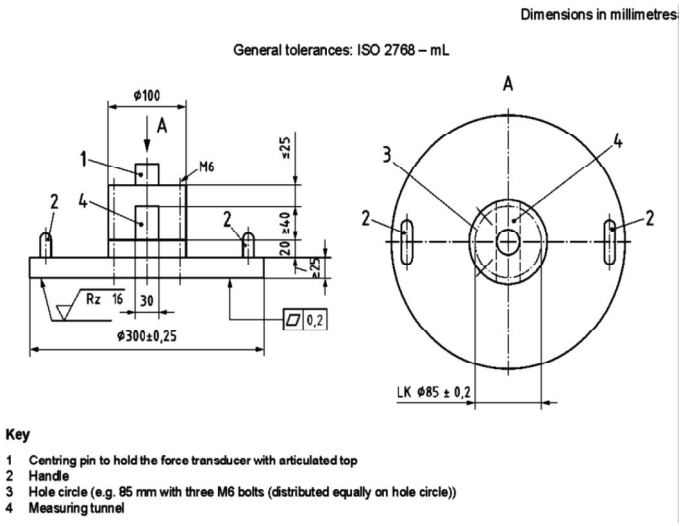


Fig. 19.01: Loading Plate

3.4 Force-measuring apparatus: A mechanical or electrical force transducer shall be fitted between the loading plate and the hydraulic jack. It shall measure the load on the plate with a maximum permissible error of 1 % of the maximum test load. The stress shall be indicated at a resolution of at least 0.001 MN/m^2.

3.5 Settlement-measuring device: The settlement-measuring device can be with a rotatable contact arm or with a contact arm capable of being moved horizontally in axial direction (i.e. with a slide bearing, see Fig. 19.02).The settlement-measuring device consists of:

(i) A frame supported at three points (see "2" in Fig. 19.02),

(ii) A vertically adjustable, torsion-proof, rigid contact arm (see "4" in Fig. 19.02),

(iii) A displacement transducer or dial gauge (see "1" in Fig. 19.02).

The distance from the centre of the loading plate to the centre line of the support shall be at least 1.5m and shall not be greater than 1.6m (see Fig. 19.02).

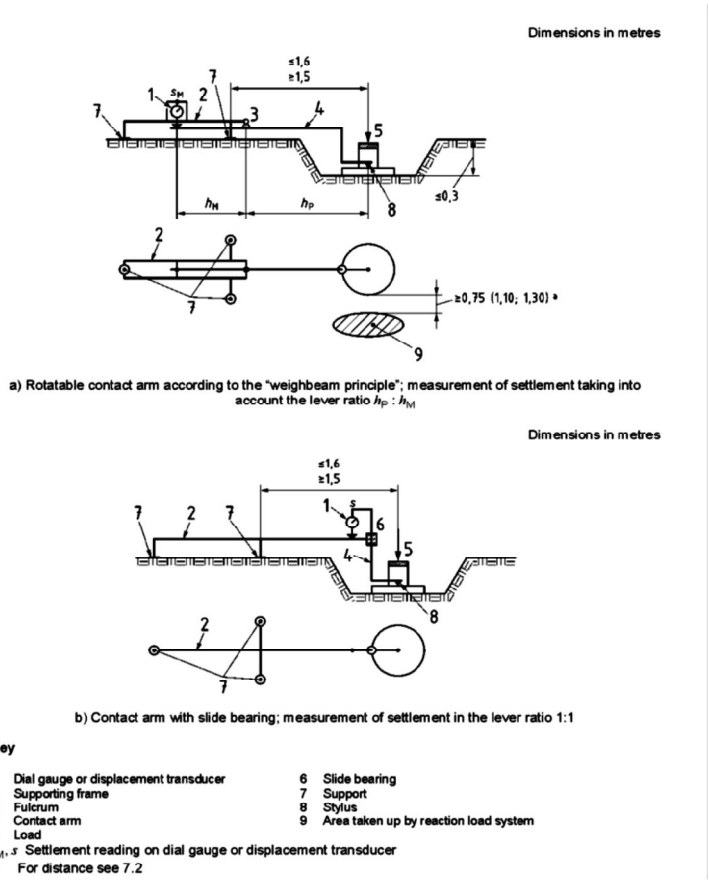
The h_P:h_M ratio (Fig. 19.02) shall not exceed 2.0. The setting of the assembly shall be capable of being locked so that the h_P/h_M ratio does not change during measurement.

The settlement-measuring device shall be capable of measuring the settlement of the loading plate with a maximum permissible error of 0.04 mm in the measuring range up to 10mm. The indication shall have a resolution of at least 0.01mm.

Measurement of settlement with alternative measuring devices is permitted if these have at least the same resolution and measure to the same accuracy and are recognized as remaining unaffected by soil deformation occurring as a result of testing.

4. Test conditions : In the case of soil which has formed a surface crust, has been softened or has been otherwise disturbed in its upper zone, this disturbed soil shall be removed before the plate load test is carried out. The density of the soil under test shall remain as unchanged as possible.

For fine-grained soil (e.g. silt, clay), the plate load test can only be carried out and evaluated satisfactorily if the soil is stiff to firm in consistency. In case of doubt, the consistency of the soil under test shall be determined at various depths up to a depth “d” below ground level (d= diameter of loading plate).



a) Rotatable contact arm according to the “weighbeam principle”; measurement of settlement taking into account the lever ratio $h_P : h_M$

b) Contact arm with slide bearing; measurement of settlement in the lever ratio 1:1

Fig. 19.02: Settlement Measuring Device

5. Procedure for test:

5.1 Test area preparation: An area sufficiently large to receive the loading plate shall be levelled using suitable tools (e.g. steel straight edge or trowel) or by turning or working the loading plate back and forth. Any loose material shall be removed.

5.2 Setting up the plate loading apparatus: The loading plate shall lie on, and be in full contact with, the test surface. If necessary, a thin bed (i.e. only a few millimeters in thickness) of dry medium-grained sand or gypsum plaster paste shall be prepared to obtain a level surface. The plate shall be bedded on this surface by turning and slightly tapping on its upper face. When using gypsum plaster as bedding material, the plate shall be greased on its underside. Any excess plaster shall be removed with the spatula before it sets. Testing shall not begin until the plaster has set.

The hydraulic jack shall be placed onto the middle of, and at right angles to, the loading plate beneath the reaction loading system and secured against tilting. The minimum clearance between loading plate and contact area of the reaction load shall be 0.75 m. The reaction load shall be secured against displacement at right angles to the direction of loading.

5.3 Arrangement of settlement-measuring device: In order to measure settlement, the stylus shall be placed in the center of the loading plate. The distance between the support for the supporting frame and the area taken up by the reaction load shall be at least 1.25m. The dial gauge or

transducer shall be set up so as to be vertical.

When placing the loading plate, care shall be taken to ensure that the stylus of the contact arm can be passed without hindrance into the measuring tunnel in the plinth of the loading plate and positioned centrally on the plate.

The settlement - measuring device shall be prot-ected from sunlight and wind. Care shall be taken to ensure that the device and the reaction loading system are not subjected to vibration during the test.

5.4 Preloading: Prior to starting the test, the force transducer and dial gauge or displacement transducer shall be set to zero, after which a load shall be applied corresponding to a stress of 0.01 MN/m^2. The reading of the gauge or transducer shall not be reset to zero until at least 30 second after the preload has been applied.

5.5 Loading and unloading: To determine the strain modulus, Ev, the load shall be applied in not less than six stages, in approximately equal increments, until the required maximum stress is reached. Each change in load (from stage to stage) shall be completed within one minute. The load shall be released in 3 stages, to 50%, 25% and approximately 2 % of the maximum load. Following unloading, a further (2^{nd}) loading cycle shall be carried out, in which; however, the load is to be increased only to the penultimate stage of the first cycle (so that the full load is not reached).

When increasing and decreasing the load, 60 sec after the previous loading stage has been reached shall elapse before beginning

the next stage. The load shall be held constant during this period. The reading shall be recorded at the termination of each loading stage (see Table-19.1 & 19.2).

If any local inhomogeneity is encountered (e.g. stones, or soil of varying consistency), this shall be recorded.

Table 19.1: Measured values for first cycle

Loading stage no.	Load F (kN)	Normal Stress σ_0 (MN/m^2)	Dial gauge reading S_m (mm)	Settlement of loading plate S (mm)
0	0.71	0.01	0	1
1	5.65	0.080	0.86	1.15
2	11.31	0.160	1.57	2.09
3	17.67	0.250	2.15	2.87
4	23.33	0.330	2.44	3.25
5	29.69	0.420	2.85	3.80
6	35.34	0.500	3.16	4.21
7	17.67	0.250	2.97	3.96
8	8.84	0.125	2.78	3.71
9	0.71	0.01	1.94	2.59

If, during the loading cycle, a higher load than intended is inadvertently applied, this load shall be maintained and a note made in the test report.

Table 19.2: Measured values for second loading cycle

Loading stage no.	Load F (kN)	Normal Stress σ_0 (MN/m^2)	Dial gauge reading S_m (mm)	Settlement of loading plate S (mm)
9	0.71	0.01	1.94	2.59
10	0.71	0.080	2.42	3.23
11	5.65	0.160	2.65	3.53
12	11.31	0.250	2.84	3.79
13	17.67	0.330	2.99	3.99
14	23.33	0.420	3.10	4.13

6. Evaluation and Representation of Results:

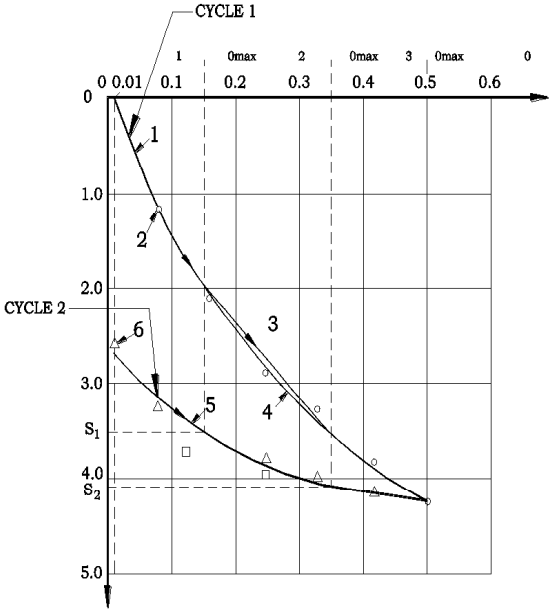


Fig. 19.03: Load (Stress) – Settlement Curve

6.1 Load-settlement curve: For each load increment, the average normal stress (60)and the associated settlement reading (M) shall be recorded on the dial gauge or displacement transducer. For the assembly shown in Fig. 19.2(b), M shall be taken as the settlement (s) of the plate. For the assembly shown in Fig. 19.2(a), S is to be obtained by multiplying the settlement reading (S_M) by the lever ratio h_P:h_M, in accordance with Equation (1):

$$S = S_M \frac{h_P}{h_M} \qquad (1)$$

A load (mean stress below the plate)-settlement fitting curve shall be drawn for the first loading cycle and second loading cycle as shown in Fig. 19.03

6.2 Calculation of strain modulus, Ev: Calculation of the strain modulus (Ev) from the first and of the second loading cycle shall be based on load-settlement fitting curves, by using following equation:

$$E_v = 1.5 * r * (\Delta \sigma)/(\Delta s)$$

Where,

E_v is the strain modulus, in MN/m^2;

r is the radius of loading plate, in mm;

$\Delta \sigma$ is the change in stress between $0.7\sigma_{max}$ and $0.3\sigma_{max}$; and

Δs is the change in settlement corresp-onding to stress values of $0.7\sigma_{max}$ and $0.3\sigma_{max}$, from the load settlement curve, in mm.

The subscript 1 shall be used after E_v to denote the first loading cycle and the subscript 2 to denote the second loading cycle. σ_{max} from the first loading cycle shall also be used to determine the parameters of the second loading cycle also.

7. Examples for determination of E_v: A typical load (stress) – settlement curve is plotted in Fig. 19.03 and using this curve, values of Ev are calculated as under:

For First loading cycle:

E_{v1} = 1.5*150*(0.7 σ_{0max} – 0.3 σ_{max})/
(S at 0.7 σ_{0max} – S at 0.3 σ_{max})

= 1.5 * 150 * (0.4 * 0.500)/(3.5 – 1.95) = 29.03 MN/m^2

For Second loading cycle:

E_{v2} = 1.5*150*(0.7 σ_{0max} - 0.3 σ_{max})/
(S at 0.7 σ_{0max} – S at 0.3 σ_{max})

= 1.5 * 150 * (0.4 * 0.500)/(4.08 – 3.50)= 77.58 MN/m^2

Chapter-20

FIELD VANE SHEAR TEST

1. **Introduction:** The vane shear test is an in-situ test to estimate the undrained shear strength of fully saturated clays without disturbance. The test is relatively simple, quick, and provides a cost-effective way of estimating the soil shear strength. Therefore, it is widely used in geotechnical investigations. The results of the test are not reliable if clay contains silt or sand.
2. **Reference:** IS 4434:1978 (Reaffirmed-2016) "Code of Practice for in-situ Vane Shear test for soils".
3. **Apparatus required:**



Fig. 20.01: Vane Shear Test

3.1 Vane: shall consist of four mutually perpendicular blades. The height of the vane shall be twice the overall diameter. It is recommended that the overall diameter of the vane should be

37.5,50,65,75 or 100mm. The design of the vane shall be such that it causes as little remoulding and disturbance as possible to the soil when inserted into the ground for a test. The blades shall be as thin as possible,consistent with the strength requirements. The vane should not deform under the maximum torque for which it is designed. The penetrating edge of the vane blades shall be sharpened having an included angle of 90^0. The vane blades shall be welded together suitably either directly or to a central rod, the maximum diameter of which should preferably not exceed 12.5mm. The area ratio of the vane shall be kept as low as possible and shall not exceed 18 percent for the 37.5mm vane and 12 percent for the 50, 65, 75 and 100mm diameter vanes. The area ratio may be calculated using the following formula:

$$Ar = [\{8t (D-d) + xd^2\}/xD^2] \times 100 (\%)$$

Where,

Ar = area ratio in percent,

t = thickness of vane blades in mm,

D = overall diameter of vane in mm, and

d = diameter of central vane rod including any enlargementdue to welding in mm.

The vane rod (the rod to which the vane blades are fixed) may be enclosed in a suitably designed sleeve from just above the blades and throughout the length it penetrates the soil to exclude the soil particles and the effects of soil adhesion. This sleeve shall commence above the blades at a distance equivalent to about two diameters of the vane rod.

3.2 Torque Applicator: The torque applicator shall have a clamping device to rigidly secure it to the anchor casing and shall have an attachment to securely hold the string of rods connecting the vane.The instrument shall be capable of applying a torque to thevane through the string of rods and to measure the same. It should also have a device to read the angular rotation of the upper end of the extension rod. The torque applicator shall be provided with speed control sothat the rate of rotation may be maintained at 0.10/s. Friction exerted by the torque applicator should be of negligible magnitude and shall be checked periodically.

The capacity and accuracy of the instrument shall be one of the following as may be specified by the purchaser:

(a) Measure torque up to 60 N.m to an accuracy of 1N.m, or

(b) Measure torque up to 200 N.m to an accuracy of 2.5 N.m.

3.3 Rod System: The string of torque rods connecting the vane to the torque applicator called the rod system may be of the quick coupling typeor of the threaded type. The length of the rods shall preferably be 1 m with a few of smaller lengths. These rods shall have sufficient diameter such that their elastic limit is not exceeded when the vane is stressed to its capacity. The threaded rods shall be so coupled that the shoulders of the male and female ends shall meet to prevent any possibility of the coupling tightening when the torque is applied during the test. If a

vane housing is used, the torque rods shall be equipped with well-lubricated bearings where they pass through the housing. These bearings shall be provided with seals to prevent soil from entering them. The torquerods shall be guided so as to prevent friction from developing between the torque rods and the walls of casing or boring.

3.4 Dummy Rod: of dimensions equal to that of the vane rod of the vanes used.

3.5 Guides for Rod: of suitable type provided with ball bearing arrangement so as to enable the rod to rotate freely.

3.6 Drilling Equipment : The equipment used shall provide a cleanhole of the required diameter for insertion of the vane to ensure that thevane test is performed on undisturbed soil.

3.7 Jacking Arrangement: for pushing the shoe and vane (where required).

4. Procedure: The test can be conducted either from the ground surface or from the bottom of a borehole or a test pit.

4.1 Test from bottom of Bore-Hole :

(i) Sink the bore-hole up to the depth required and extend the casing up to the full depth. Fix the torque applicator anchor plate to the casing.

(ii) Connect the vane of suitable size to the rods and lower it to the bottom of the bore-hole, putting guides at suitable intervals but not more than about 5m as the rods are extended. Push the vane with a moderate steady force up to a depth of five times the diameter ofthe bore-hole below the bottom of the bore-

hole or shoe. Take precautions to make sure that no torque is applied to the torque rods during the thrust.No hammering shall be permitted. Fix the torque applicator with frame to the anchor plate and connect the rods to it. Tighten the torque applicatorto the frame properly.

(iii) Allow a minimum period of five minutes after insertion of the vane. Turn the gear handle so that the vane is rotated at the rate of 0.1°/s.Note the maximum torque reading attained. If necessary, note the torque indicator dial gauge readings at half-minute intervals and continue rotating the vane until the reading drops appreciably from the maximum.

(iv) Just after the determination of the maximum torque, rotate the vane rapidly through a minimum of ten revolutions. The remoulded strength should then be determined within one minute after completion of the revolutions.

(v) Remove the vane testing assembly, continue boring, and collect soil sample from the level of the vane testing for laboratory analysis to ascertain whether the deposit will behave as a purely cohesive soil.

(vi) In case where a sleeve is not provided for the vane rod and the soilis in contact with the rod, determine the friction between the soil and the vane rod by conducting tests at appropriate depths using the dummy rod corresponding to that of the vane used in the test. The test should be conducted as with the vane except that the vane is replaced

by dummy rod. The test should be conducted in an adjacent bore-hole at the same depth at which the vane tests were conducted. The dummy rod shouldbe pushed into the ground to the same distance as the vane rod at that depth.

4.2 Test by Direct penetration from Ground Surface:

(i) Lock the vane in-place inside the protecting shoe and jack or driveit to the required depth. Care shall be taken to see that the rods remain tight while the vane is lowered. Place guides about every 3m to centralize and reduce friction between the rods and extension pipes.

(ii) When the vane and protecting shoe have penetrated to therequired depth, push the vane steadily, without twisting, a distance of 5 times the diameter of the shoe, into the undisturbed soil below the protecting shoe. Rotate the vane till the soil fails.

(iii) Remove the torque measuring instrument and pull back the vane fully into its protecting shoe before advancing for another test or before being removed from the ground taking precautions that the vane is not damaged by the shoe.

(iv) In the case where soil is in contact with the torque rods, determine the friction between the soil and the rod by means of torque tests conducted on similar rods at similar depths with no vane attached. Conduct the rod friction test at least once on each site; this shall consist of aseries of torque tests at varying depths. A dummy should be used instead of the vane if the vane rod is not provided with a sleeve.

5. **Records:** Records of vane shear test shall be maintained in an Appendix-A, which gives a recommended proforma for the record of results.

6. **Calculations:** For vane testing instruments that do not read the torque directly a calibration curve to convert the readings to Newton Meter (centimeter-kilogram) of torque shall be provided. These calibration curves shall be checked periodically.

   For a rectangular vane, calculate the shear strength of the soil usingthe following formula:

$$S = \frac{M}{\pi D^2 (H/2+D/6)\pi D^2} \times 10^6 \quad \text{OR } S = \frac{M}{(H/2+D/6)} \times 10^3$$

   Where,

   S = Shear Strength in kN/m^2 (kgf/cm^2),

   M = torque, to shear the soil in N.m (kgf. cm) {corrected for vane rod and torque rod resistance, if any},

   D = Overall diameter of vane in mm and

   H = height of vane in mm.

   If H= 2 D, then the formula reduces to:

$$S = 3M/\ 11D^3\ X\ 10^6 \quad (S \text{ in } kN/m^2)$$

$$S = 3M/\ 11D^3\ X\ 10^3 \quad (S \text{ in } kgf/m^2)$$

Depth

	D	U	R	D	U	R	D	U	R
Depth of Vane Tip below bottom of Bore Hole or Vane Shoe									
Time to failure									
Maximum reading on Torque Measuring Equipment									
Maximum Torque									
Number of revolutions for Remoulding									
Shear Strength of Undisturbed Soil, kN/m^2 (kgf/cm^2)									
Shear Strength of Remoulded Soil, kN/m^2 (kgf/cm^2)									
Sensitivity									

Table 19.2: Time-Rotation Readings

Time in Minutes	Depth								
	D	U	R	D	U	R	D	U	R
0.0									
0.5									
1.0									
1.5									
2.0									
2.5									
3.0									
3.5									
4.0									
4.5									
5.0									
5.5									

6.0									
6.5									
7.0									
7.5									
8.0									
8.5									
9.0									
9.5									
10.0									

D = test with dummy if used or any other test for the determination of friction of vane rod and/or torque rods.
U = test with vane in undisturbed soil.
R = test with vane in remoulded soil.

Record of deviation from standard procedure, if any, with reasons:

Appendix – A

(Clause – 5)

GENERAL

Project: Drilling or testing foreman:

Bore-hole No.(ifany): Supervising engineer:

Date of test:

DETAILS OF BORING (IF ANY)

Location: Log of soil conditions:

Reference elevation:

or

Ground elevation:

Cased/uncased:

Method of making the hole:

Level of water in the bore-hole/

level of ground water at the timeof test:

Notes on driving resistance:

DETAILS OF VANE TEST

Test from bottom of bore-hole: Test by direct

Vane test apparatus No.: penetration from

Van Size : ground surface :

Vane constant :

Diameter of dummy rod

(if used):

Conversion factor for torque measuring equipment:

Chapter-21

FIELD CALIFORNIA BEARING RATIO (CBR) TEST

1. **Introduction:**This test method describes the procedure for checking the CBR value on top of any finished layer of earthwork in field. This test serves as a quality control measure in construction of Railway formation.
2. **Reference:** IS-2720(Part-31):1990 (Reaffirmed-2015) "Methods of test for soils: Field determination of California Bearing Ratio".
3. **Apparatus required:**



Fig. 21.01: Setup for Field CBR Test

3.1 Loading Device(Fig. 21.02): A mechanical screw loading jack with swivel head for applying load to the penetration piston. The

devise should have an arrangement for attachment to truck, tractor, truss or any other equipment used to provide load reaction. The jack should be such that a uniform penetration rate of 2.5 mm/min can be achieved. The capacity of the jack should not be less than 50 kN.

3.2 Equipment for providing Reaction for Loading(Fig. 21.01): Truck, tractor, truss or any other suitable equipment. If truck or tractor is used they should be loaded suitably to give the necessary reaction. If truss is used it should be suitably anchored.

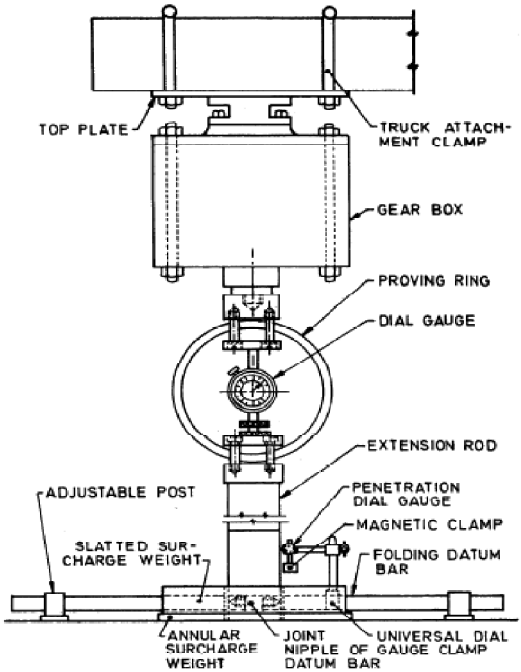


Fig.21.02: Field CBR Apparatus

3.3 Jacks: Two track-type jacks of 50 to 120 kN capacity, having double acting combination trip and automatic lowering in cases where loaded truck or tractor is use for providing necessary reaction.

3.4 Proving Ring: One calibrated proving ring of suitable capacity having an accuracy of not more than one percent of the anticipated load shall be used. The calibration of the proving ring shall be checked periodically at least once a year.

3.5 Metal Penetration Piston: 50 ± 0.1mm in diameter and not less than 100mm long.

3.6 Extension: Internally threaded pipe or rod extensions not less than 200cm long furnished in the following quantities and length:

Length of Extension (cm)	No. of Extensions	Length of Extension (cm)	No. of Extensions
5	2	50	1
10	2	100	1
30	2		

3.7 Connectors: For coupling the penetration piston and proving ring assembly either directly or through extension pieces.

3.8 Dial Gauge: Reading to 0.01mm having a travel of 25mm, for measuring the penetration of the piston.

3.9 Dial Gauge Support: Rigid and of steel, angle welded construction or light alloy pipe construction about 2m long, of overall height 30cm and 45cm wide at the feet with universal or ordinary dial gauge holder adjustable anywhere along the length of the support.

3.10 Surcharge Weight: One annular metal weight of mass 5 kg and of 250mm diameter with a central hole 53mm in diameter. Two circular

slotted weights of mass 5 kg and of diameter 215 to 250mm, with a central hole and slot width of 53mm. Two circular slotted weights of mass 10 kg and of diameter 215 to 250mm, with a central hole and slot width of 53mm.

3.11 Miscellaneous Apparatus: Other general apparatus, such as spirit level, pick, spade, scoop and brush, apparatus for moisture determination and density determination

4. Procedure:

4.1 The general surface area to be tested should be exposed, cleaned of all loose and dried material and levelled. Extreme care shall be taken not to disturb the test surface. The spacing of the tests should be such that operations in one area do not disturb the soil in the other area. For testing operations this spacing may be 50cm for the penetration piston used in the test.

4.2 If actual service conditions in the field warrant, the surface to be tested may be soaked to the desired degree. During the process of soaking the required surcharge weights should be kept in place. The test surface should be drained of all free water, levelled and allowed to stand for at least 15minutes before starting further operations.

4.3 The equipment used to provide load reaction (truck, tractor, truss etc.), should be so located that the centre of the beam against which the loading jack will work is over the centre of the surface to be tested. If loaded truck or tractor is used for providing the necessary reaction, the rear wheels of the truck or tractor should be completely raised by means of the track

type jacks placed below the frame of the body near the wheels in order to avoid the loss of loading effort which would otherwise be spent on the flexing of the axial springs of the vehicle at the time of testing. In order to avoid accidents due to the failure of jacks near the wheels and the lifting of the vehicle at higher loads, the rear side of the body of the vehicle should be placed over two rigid supports. The screw jack with swivel should be installed to the underside of the equipment providing reaction, at the correct position for the test. The proving ring should be connected to the bottom end of the jack and the piston connector to the bottom of the proving ring. The piston should then, be connected using, if necessary, lengths of extension pipes or rods. It should be ensured that the entire assembly is plumb and the loading jack should be clamped in position.

4.4 The surcharge annular weight of mass 5 kg should be kept in position on the surface to be tested so that when the piston is lowered, it will pass through the hole in the annular weight. The penetration piston should be seated with the smallest possible load not exceeding a total load of 40 N (or unit load of 0.02 MPa) so that full contact is established between the piston and the surface to be tested. For materials with irregular surface the piston may be seated on a thinnest practical layer of fine limestone screening or plaster of Paris spread over the surface.

4.5 While the seating load is on the piston, a 3 to 6mm layer of clean sand should be spread over the surface to be covered by

the surcharge annular weight. This helps in distributing the surcharge load over the surface uniformly.

4.6 Surcharge weights, sufficient to produce an intensity of loading, equal to the weight of the base material and pavement, except that the minimum weight applied should be 150 N including that of the annular weight [this weight gives an intensity of loading approximately equal to that in the laboratory bearing ratio test] should be applied. The penetration indicating dial should be suitably fixed for reading the penetration and the dial set to zero.

4.7 Load shall be applied on the penetration piston so that the penetration is approximately 1.25 mm/min. The load readings shall be recorded at penetration of 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0, 7.5, 10.0 and 12.5mm. The maximum load and penetration shall be recorded if it occurs for a penetration less than 12.5mm.

4.8 After the completion of the test, a sample shall be collected from the point of penetration, for moisture content determination. Besides the moisture content, the in-place density shall be determined about 15 cm away from the point of penetration.

5. Calculations:

5.1 Load Penetration Curve:

The load penetration curve shall be plotted (Fig. 21.03). This curve may be convex upwards although the initial portion of the curve may be concave upwards due to surface irregularities. A correction shall then be applied by drawing a tangent to the

curve at the point of maximum slope. The corrected curve shall be taken to be this tangent, together with the convex portion of the original curve, with the origin of strains shifted to the point where the tangent cuts the horizontal axis for penetration, as illustrated in Fig. 21.03.

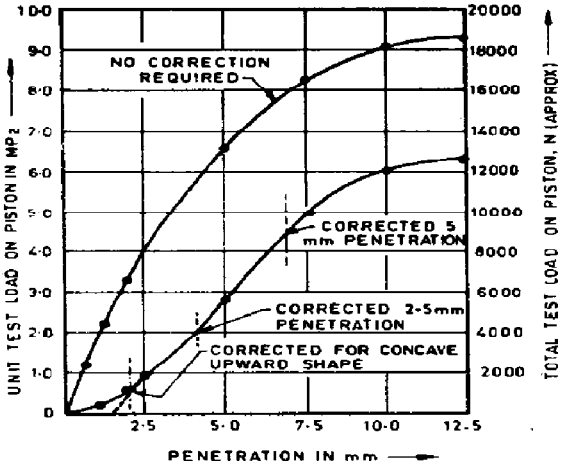


Fig.21.03: Typical Load Penetration Curve

5.2 Bearing Ratio: Corresponding to the penetration value at which the bearing ration is desired, corrected load values shall be taken from the load penetration curve and the bearing ratio calculated as follows:

Bearing Ratio = $(P_t / P_s) \times 100$

Where,

P_t = Corrected load corresponding to the chosen penetration from the load penetration curve,

P_s = Standard load for the same depth of penetration as for Pt, taken from Table 21.1.

The bearing ratios are usually calculated for penetration of 2.5mm and 5mm. Generally, the bearing ratio at 2.5mm penetration will be greater than that at 5mm penetration and in such a case the former shall be taken as the CBR value for design purpose. If the bearing ratio corresponding to a penetration of 5 mm exceeds that for 2.5mm, the test shall be repeated. If identical results follow, the bearing ratio corresponding to 5mm penetration shall be taken for design.

Table 21.1: Standard Loads, Ps

Penetration Depth, mm	Unit Standard Load, MPa	Total Standard Load, N
2.5	6.86	13430
5.0	10.30	20150
7.5	13.10	25790
10.0	15.90	31180
12.5	17.90	35300

6. **Report:** The bearing ratio shall be reported correct to the first decimal place. The details in the recommended Proforma (Para-7.1) for the record of test results shall be given.
7. **Number of Field of Tests:** Three in-place bearing ratio tests shall be performed at each location to be tested. However, if the results of the three tests in any group do not show reasonable agreement, three additional test shall be performed at the same location and numerical average of the six tests shall be used as the bearing ratio at that location. A reasonable agreement between the minimum and maximum values of the three tests where the bearing ratio is less than 10% permits a tolerance of 3%, from 10% to 30% a tolerance of 5%, from 30% to 60% a tolerance of 10%, and greater than 60%, a tolerance of 25%. If it is known that single value is erratic for any reason, that value should be discarded and another test performed.

7.1 Proforma for Test Report:

Location .....

Tested by .....

Material at the test point .....

Date .....

Depth of tests point .....

Condition of test: Soaked /nsoaked.....

Period of soaking, if any .....

Surcharge weight used during soaking .....

Moisture content .....

Density .....

Method used for determination of density .....

Penetration test Surcharge weight used .....

Penetration, mm	Proving Ring Dial Gauge Reading	Load N	Corrected Load N (Ref 5.1)
0.5			
1.0			
1.5			
2.0			
2.5			
3.0			
4.0			
5.0			
7.5			
10.0			
12.5			

Bearing Ratio at 2.5 mm Penetration:

1.}

2.}

3.}

Average: (Average of 1, 2 & 3)

Bearing Ratio at 5.0 mm Penetration:

1.}

2.}

3.}

Average: (Average of 1, 2 & 3)

Reason, if test is rejected: ……

……..

Result of repeat test, if conducted ……

……..

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NOTES

NOTES

For any suggestions, errors etc, please contact
Shri R. K. Shekhawat, Senior Professor, IRICEN, Pune
on Email : shekhawat.rajesh@iricen.gov.in &
Shri Sabyasachi S. Roy, Sr. Instructor, IRICEN, Pune
on Email : sit10iricen@gmail.com

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